Ninth Canadian Conference on
Earthquake Engineering
Ottawa, Ontario, Canada
26-29 June 2007

PROBABILISTIC ASSESSMENT OF CAPACITY CURVES OF DUCTILE STEEL-MOMENT-RESISTING-FRAMES DESIGNED ACCORDING TO 1995 AND 2005 NBCC

W. Wang[1] and H.P. Hong[2]

ABSTRACT

Six steel moment resisting frames are designed according to the 1995 and 2005 editions of the National Building Code of Canada. A probabilistic assessment of capacity curves of the designed frames is carried out. The assessment is focused on the investigation of the influence of the uncertain structural properties on probabilistic characterization of the capacity of structures whose designs could be governed by strength or drift requirements. The capacity (in relation to the code provisions) is defined using the capacity curve obtained from the nonlinear static pushover analysis. Statistical characterizations of the variables defining capacity at yield, drift at yield, and stiffness before and after yield are provided. This investigation not only is important for comparing the changes in structural capacity caused by the recent changes in the seismic provisions, but also provides statistics on the overstrength level of the frames designed according to these codes, which are relevant for any quantitative evaluation of structural safety.

Introduction

The current edition of the National Building Code of Canada (NBCC) (NBCC 2005) differs from the 1995 edition of the NBCC (NBCC 1995) in several ways. For example, the former uses the uniform hazard spectrum defined based on the 2% in 50 year return period value and the adoption of the overstrength and ductility related force modification factors, whereas the latter uses the 10% in 50 years return period value and a standard design spectra, and considers a force modification factor and calibration factor. They use different drift requirements as well.

Comparison of the minimum strength requirements dictated by the 1995 edition of the NBCC (1995-NBCC) and the 2005 edition of the NBCC (2005-NBCC) for structural design is highlighted by Heidebrecht (2003), Humar and Mahgoub (2003) and Mitchell et al. (2003). It shows that differences of the design base shear coefficients for structures whose designs are governed by strength requirements do exist, although not extremely large. However, designs may often be governed by the code recommended drift requirement rather than the strength requirement. In such cases, comparison of the structural capacity must also be carried out to assess the capacity of structures designed according to the codes rather than simply using strength requirements. It should be noted that the use of the equivalent static load procedure for the design does not provide a direct indication on the performance of the designed structure after yielding, and/or near collapse. Such performance could be approximately assessed using the nonlinear static pushover analyses (NSPA). The NSPA provides information on the seismic capacity of the structure

1 Dept. of Civil Engineering, University of Western, London, Ontario, N6A 5B9

which is defined by the force-deformation curve or capacity curve of the structure developed by a series of static analysis. The force may be represented by the base shear and the deformation could be represented by the global drift or the interstorey drift. The NSPA account approximately the redistribution of internal forces and its pros and cons has been presented extensively by Krawinkler and Seneviratna (1998).

Since material properties and geometrical variables are uncertain, the impact of this uncertainty on the structural response and seismic demand has been investigated by Sues et al. (1985), Wen (1993), Song and Ellingwood (1999), and Hong and Jiang (2004). These studies suggested that the uncertainty in material properties and geometrical variables and the uncertainty in the damping ratio do not influence significantly the variability of the responses. However, it appears that the systematic assessment of the impact of the uncertain structural properties on the obtained force-deformation characteristics of the structures designed according to the NBCC codes is not available. This is especially so if one is interested in the statistics of the possible overstrengthening for designs governed by drift requirement.

The main objective of this study is to compare the capacity of steel moment resisting frames (SMRF) designed using the 1995-NBCC to those that are designed based on the 2005-NBCC. The capacity is defined using the capacity curve obtained from the NSPA. Statistical characterizations of the variables defining capacity at yield, drift at yield, and stiffness before and after yield will be assessed. Also, probabilistic assessments of the ratio of the expected capacity at yield to the factored design lateral load capacity at yield and, of the ratio of the expected capacity at yield to the “nominal” lateral load capacity at yield will be carried out. For the assessments, uncertainties in structural properties are considered.

Frames Designed According to the 1995 and 2005 Editions of the NBCC

Six ductile SMRFs, which are designed in accordance with the 1995-NBCC and the 2005-NBCC, are considered. The frames are part of 3-, 7- or 10-storey office buildings to be located in Victoria, B.C. The general design considerations are shown in Fig. 1, and the designed frames are depicted in Figs. 2 and 3. All frames with an interstorey height of 4 m are designed to resist the lateral earthquake load. For easy reference, the designed frames are denoted as *i*SF-*j* where *i* refers to the number of storey, and *j* equal to 95 and 05 refers to the 1995-NBCC and the 2005-NBCC employed for the design, respectively. Details leading to these designed frames are given in (Wang 2006).

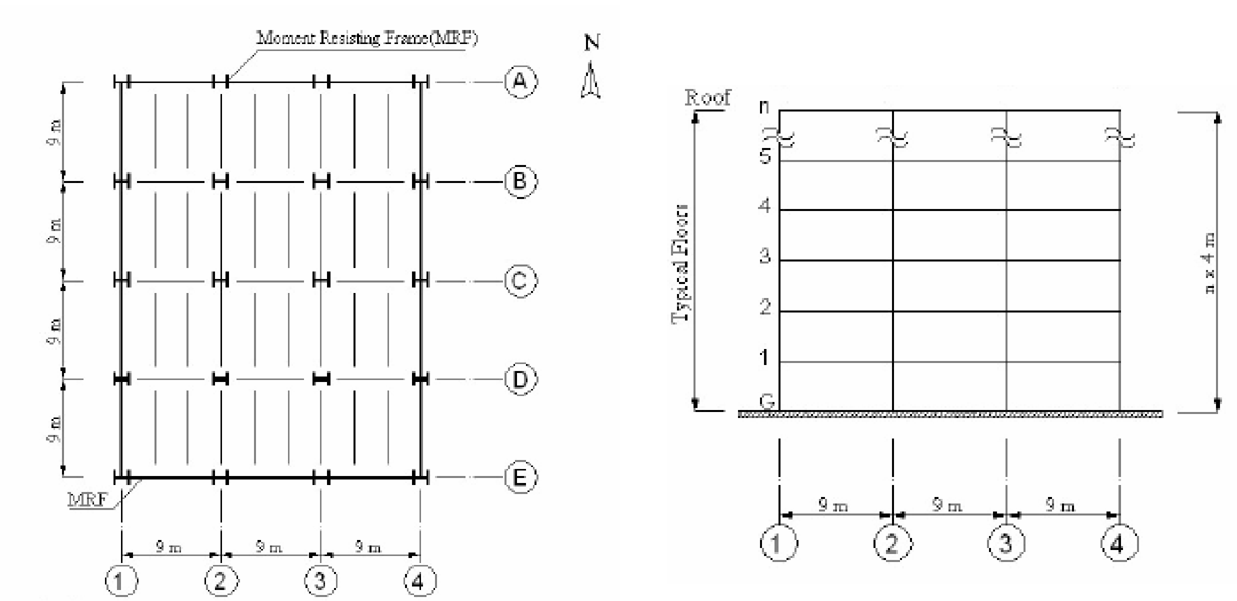
Designs satisfy both the strength and drift requirements. The designs are governed by the drift requirement rather than the minimum design strength requirement. The fundamental natural vibration period T_n, the minimum required design base shear force V_d, and the interstorey drift limit for the frames are shown in Table 1.

Table 1. Comparison of the base shear and the drift requirement.

Frame	3SF-05	7SF-05	10SF-05	3SF-95	7SF-95	10SF-95
T_n (s)	1.08	2.28	3.19	1.21	2.23	2.93
V_d (kN)	527	683	674	531	980	1258
Interstorey drift limit[a]	$0.025h_s/(R_dR_o/I_E) = 0.0033h_s$			$0.02h_s/R = 0.005h_s$		

Note a) R in the 1995-NBCC for the considered structure equals 4.0; R_dR_o in the 2005-NBCC for the considered structure equals 7.5, I_E equals 1.0; h_s is the interstory height.

It can be observed from Table 1 that the minimum required design base shear force according to the 2005-NBCC is almost the same as that according to the 1995-NBCC for the 3-storey SMRF. However, the differences in the minimum required base shear force by the two versions of the code for the 7-storey and 10-storey SMRFs are much more significant. V_d obtained by using the 1995-NBCC for the 7-storey and the 10-storey SMRFs are about 30% and 90% greater than those obtained based on the 2005-NBCC.



a) PLAN b) ELEVATION

Figure 1. General design consideration (plane and elevation).

NSPA Capacity Curve by Ignoring Uncertainty in Material and Geometrical Properties

The capacity of a structure under seismic excitations could be characterized using results from the NSPA (FEMA-450 2004). The NSPA considers that the structural capacity or load-deformation curve can be obtained by monotonically increasing lateral forces with an invariant height-wise load distribution. Although the NSPA is inherently inadequate for representing the detailed dynamic behaviour of a structure, it is hoped that, at least on average, it could lead to results that approximate structural dynamic behaviour. The NSPA that is implemented in DRAIN-2DX (Prakash, et al. 1993) is used in the following.

For the analysis, the height-wise lateral load or force distribution pattern as suggested in the codes is adopted for the pushover analysis. A plot of the load-deformation curve (i.e., capacity curve) obtained from the NSPA is illustrated in Fig. 4 for the strain hardening ratio ε of the structural steel equal to zero. Also illustrated in the figure is the capacity curve approximated by a bilinear relation using an iterative procedure described by Krawinkler (1996) (see also Kim and Choi 2004; FEMA-356 2000). The procedure leads to that the area under the original capacity is equal to that under the approximated bilinear curve. In Fig. 4, F_{cy}, D_{cy}, K_e, α, D_{uo}, and F_{uo} represent the shear capacity at yield, the global drift ratio at yield, the effective elastic stiffness, and the ratio of the post-yield stiffness to the effective elastic stiffness. These quantities are employed to characterize the capacity curve. The point with coordinate (D_{uo}, F_{uo}) shown in the figure represent the global drift ratio and the total lateral load (shear force) at which the last converged analysis result is obtained. Note that F_{uo} can be considered as collapse load capacity; while D_{uo} could only be viewed as the lower bound of the maximum inelastic displacement capacity of the steel frame since the structural steel is considered to be elastoplastic. If ε greater than zero is considered, the drift ratio increases monotonically as the total lateral load increases. Therefore, in such a case, one must adopt an assumption about the maximum drift ratio or force to provide a more realistic representation of the capacity curve. As a lowest limit on the maximum drift ratio for if ε is not equal to zero, one may consider that the maximum drift ratio is equal to D_{uo} that is obtained for ε = 0%. This will be referred as Procedure A. Alternatively, one may consider that the maximum drift ratio for if ε is not equal to zero, equals R or R_d times the drift ratio at yield for the 1995-NBCC or the 2005-NBCC, respectively. This choice, which will be referred as Procedure B, is based on the consideration that R or R_d actually represents the ductility-related force modification factor.

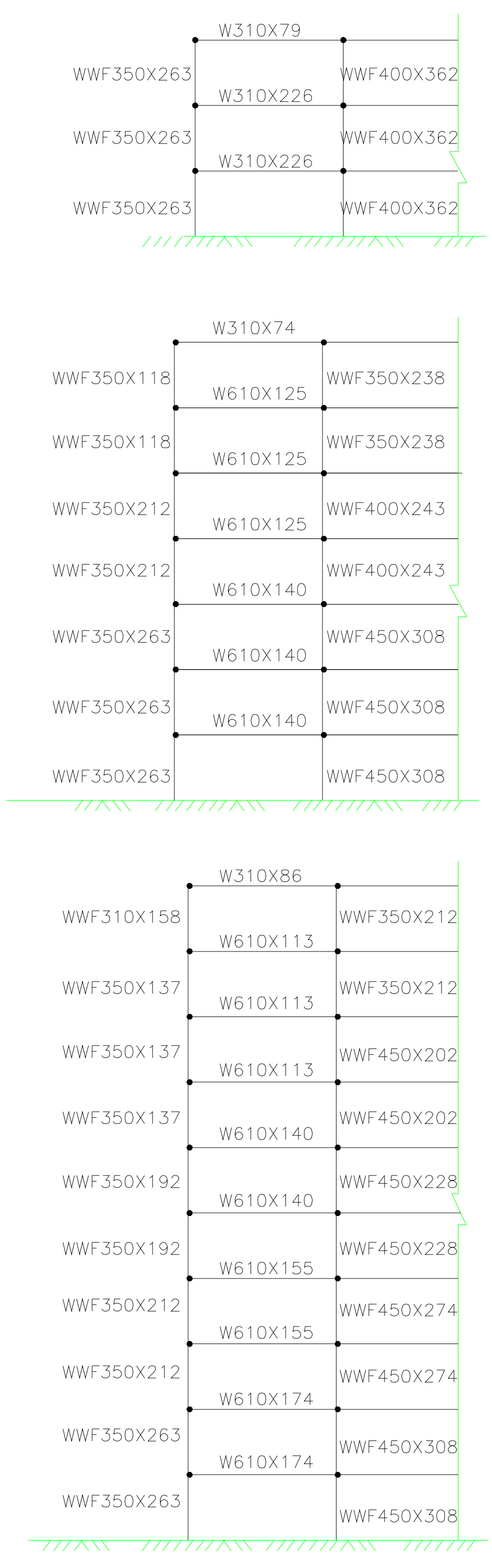


Figure 2. Frames designed in accordance with the 2005-NBCC.

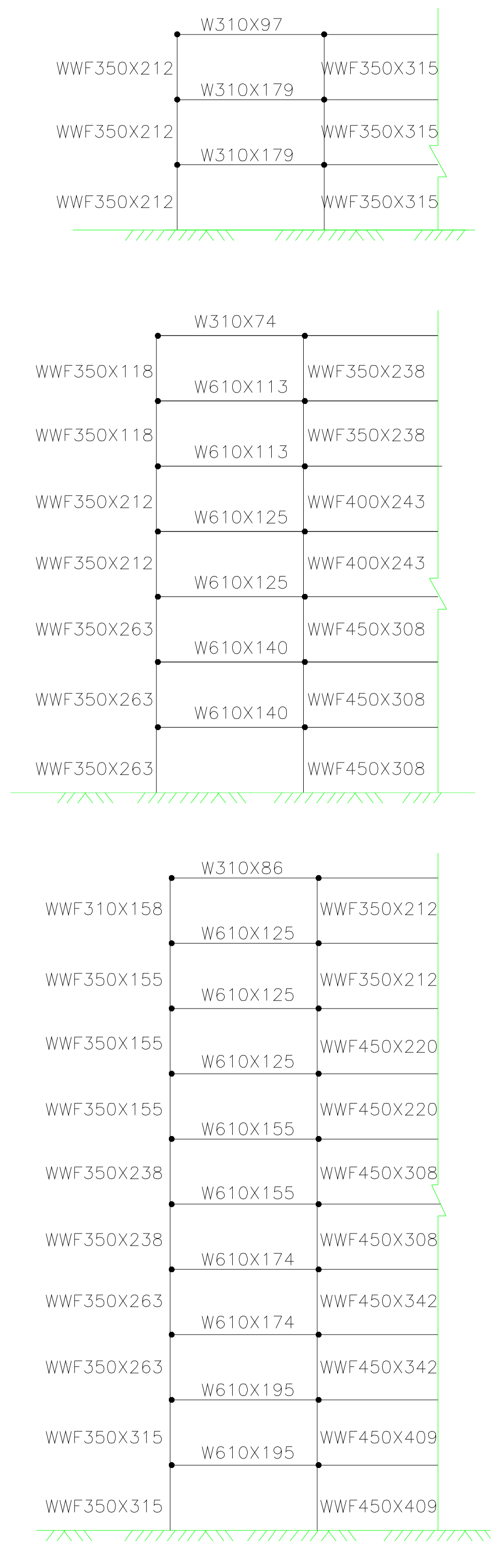


Figure 3. Frames designed in accordance with the 1995-NBCC.

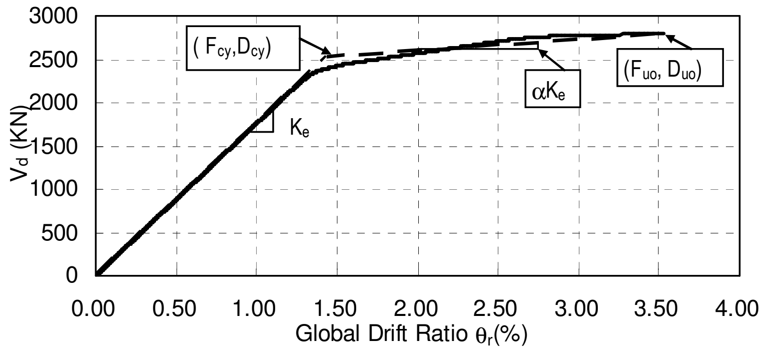


Figure 4. Illustration of capacity curve obtained from the NSPA

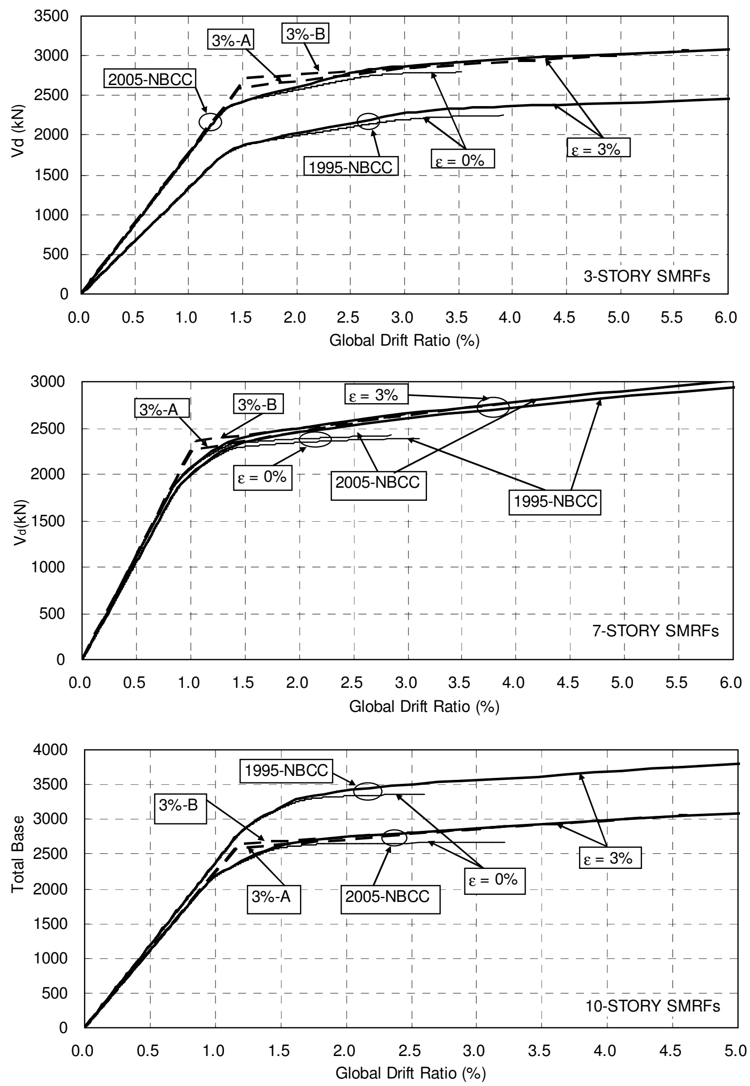


Figure 5. Capacity curve in terms of the global drift ratio (%).

By ignoring the uncertainty in material properties and geometrical variables, and considering that material resistance and section geometrical variables equal to their factored design values, and nominal values, respectively, the NSPA is carried out. The obtained capacity curve, which is referred to as the factored (design) capacity curve, is shown in Fig. 5 for each designed frame with ε = 0% and ε = 3%. The figure shows that the capacities at yield for 3SF-05 and 7SF-05 are higher than those of the 3SF-95 and 7SF-95. However the capacity at yield for 10SF-05 is lower than that of the 10SF-95. To distinguish the characteristics of this factored capacity curve from other possible capacity curves, an additional subscript f is to be used with symbols F_{cy}, D_{cy}, K_e, α, D_{uo}, and F_{uo} for its characterization.

Note that the results shown in the figure suggest that the displacement ductility capacity of the structure designed according to the 2005-NBCC is, in general, lower than or close to the one designed based on the 1995-NBCC.

NSPA Capacity Curve by Considering Uncertain Structural Properties

The material properties and the structural geometry of a built structure are not exactly equal to those designed. The material properties such as the yield strength F_y and modulus of elasticity E, and the geometry properties such as the sectional area, A, moment of inertia, I_x, and plastic section modulus, Z_x, are uncertain and, are likely to affect the structural responses. It is of interest to investigate the impact of these uncertain variables or degrees of uncertainties in these variables on the capacity curves. A literature review (Galambos and Ravindra 1978, Ellingwood et al. 1980, Kennedy and Aly 1980, Song and Ellingwood 1999, Ellingwood 2001) has lead to the typical statistics and probabilistic model for the mentioned random variables shown in Table 2, which is adopted in the present study.

Table 2. Adopted statistics and probabilistic models for the structural steel properties.

Variables	W-shape structural steel		WWF-shape structural steel		Distribution Type
	Mean/nominal	cov	Mean/nominal	Cov	
Yield strength, F_y	1.11	0.06	1.11	0.06	Lognormal
Modulus of elasticity E	1.04	0.05	1.04	0.05	Uniform
Section area, A	1.01	0.03	1.01	0.03	Lognormal
Moment of inertia, I_x	1.03	0.04	1.03	0.04	Lognormal
Modulus of plasticity, Z_x	1.03	0.04	1.03	0.04	Lognormal

It is noted that the mean capacity of the structure could be approximately evaluated by applying the NSPA using the mean values of the random variables, which can be calculated using the nominal values and the adopted the mean to nominal ratios shown in Table 2. In such a case, the parameters characterizing the obtained capacity curves will be represented by symbols F_{cy}, D_{cy}, K_e, α, D_{uo}, and F_{uo} with an addition subscript m. The obtained values F_{cym}, D_{cym}, K_{em}, α_m, D_{uom}, and F_{uom} for the designed frames are shown in Table 3 for ε=0% and compared with those of F_{cyf}, D_{cyf}, K_{ef}, α_f, D_{uof}, and F_{uof}.

Further, if an accurate estimate of the mean characteristics of F_{cy}, D_{cy}, K_e, α, D_{uo}, and F_{uo} is desirable, one could use the simulation technique to assess the expected capacity curves. This is done by sampling the values of the random variables shown in Table 2 and then performs the NSPA with the sampled values. By sample enough capacity curves, one could calculated expected values of F_{cy}, D_{cy}, K_e, α, D_{uo}, and F_{uo}, denoted by $E(F_{cy})$, $E(D_{cy})$, $E(K_e)$, $E(\alpha)$, $E(F_{uo})$, and $E(D_{uo})$, where $E(\ )$ represents the expectation. The characteristics of the capacity curve obtained in this manner with a simulation cycle of 50 is also shown in Table 3 for ε=0%. Comparison of the results shown in Table 3 suggests that the characteristics of the mean capacity curve are very close to those of the capacity curve estimated using the means of the random variables. This implies that the latter provide a good approximation to the former.

Table 3. Characteristics of the capacity curves for ε=0%.

Storey		1995-NBCC			2005-NBCC		
		f	*M*	E( )[a]	*f*	*m*	E( )
3	F_{cy} (kN)	1989	2506	2487, 0.02	2540	3225, 0.02	3239
	D_{cy} (%)	1.49	1.80	1.79, 0.02	1.44	1.71, 0.02	1.73
	K_e (GN/m)	1.11	1.16	1.16, 0.01	1.47	1.56, 0.02	1.56
	α(%)	8.00	6.75	6.92, 0.17	6.92	6.52, 0.16	6.77
	F_{uo}(kN)	2248	2755	2752, 0.02	2794	3494, 0.02	3496
	D_{uo} (%)	3.91	4.44	4.59, 0.06	3.53	3.92, 0.05	4.14
7	F_{cy} (kN)	2262	2857	2846, 0.02	2285	2899, 0.02	2915
	D_{cy} (%)	1.07	1.26	1.27, 0.02	1.02	1.23, 0.02	1.24
	K_e (GN/m)	0.76	0.81	0.80, 0.01	0.80	0.85, 0.01	0.85
	α(%)	2.96	2.98	2.88, 0.26	3.33	3.21, 0.28	3.18
	F_{uo}(kN)	2390	2991	2978, 0.02	2421	3030, 0.02	3043
	D_{uo} (%)	3.11	3.26	3.44, 0.18	2.84	2.92, 0.23	2.95
10	F_{cy} (kN)	2864	3687	3595, 0.02	2587	3241, 0.03	3189
	D_{cy} (%)	1.08	1.30	1.27, 0.02	1.16	1.37, 0.03	1.36
	K_e (GN/m)	0.67	0.71	0.71, 0.01	0.56	0.59, 0.01	0.59
	α(%)	9.21	13.1	13.0, 0.36	1.72	4.11, 0.26	4.24
	F_{uo}(kN)	3085	3889	3851, 0.02	2665	3362, 0.02	3325
	D_{uo} (%)	1.98	2.08	2.01, 0.11	3.21	2.60, 0.23	2.64

Note: *f* =calculated using factored capacity curve; *m*=calculated using the capacity curve evaluated at the mean values of the random variables; E( )=calculated using sampled capacity curves. a) the second value represents the coefficient of variation.

Table 4. Characteristics of factored (design) capacity curve and mean capacity curves.

		1995-NBCC			2005-NBCC		
	Parameter	ε = 0%	ε= 3% Procedure A	ε = 3% Procedure B	ε = 0%	ε = 3% Procedure A	ε = 3% Procedure B
3-storey	F_{cyf}/F_d	3.75	3.75	4.04	4.79	4.89	5.13
	F_{cym}/F_d	4.73	4.78	5.05	6.08	6.25	6.52
	$E(F_{cy})/F_d$	4.69	4.77	5.00	6.11	6.12	6.42
	F_{cym}/F_{cyf}	1.26	1.27	1.25	1.27	1.28	1.27
	$E(F_{cy})/F_{cyf}$	1.25	1.27	1.24	1.28	1.25	1.25
	D_{cym}/D_{cyf}	1.21	1.22	1.19	1.19	1.20	1.19
	$E(D_{cy})/D_{cyf}$	1.20	1.22	1.19	1.20	1.18	1.18
7-storey	F_{cyf}/F_d	2.38	2.64	2.73	3.67	3.70	3.77
	F_{cym}/F_d	3.01	3.02	3.09	4.65	4.23	4.35
	$E(F_{cy})/F_d$	3.00	3.00	3.07	4.71	4.67	4.78
	F_{cym}/F_{cy}	1.26	1.27	1.28	1.27	1.28	1.27
	$E(F_{cy})/F_{cy}$	1.26	1.26	1.27	1.26	1.28	1.26
	D_{cym}/D_{cyf}	1.17	1.08	1.07	1.19	1.20	1.19
	$E(D_{cy})/D_{cyf}$	1.19	1.08	1.07	1.19	1.20	1.19
10-storey	F_{cyf}/F_d	2.28	2.30	2.41	3.84	3.34	3.41
	F_{cym}/F_d	2.93	2.91	3.10	4.74	4.82	5.01
	$E(F_{cy})/F_d$	2.86	2.9	3.07	4.58	4.79	4.91
	F_{cym}/F_{cy}	1.29	1.27	1.29	1.23	1.44	1.47
	$E(F_{cy})/F_{cy}$	1.26	1.26	1.28	1.19	1.43	1.44
	D_{cym}/D_{cyf}	1.20	1.18	1.20	1.19	1.10	1.22
	$E(D_{cy})/D_{cyf}$	1.18	1.18	1.20	1.19	1.10	1.22

During the analysis, it was also observed that the coefficient of variation of the above mentioned parameters characterizing the capacity curve is relatively small (see Table 3).

To better appreciate the differences between the factored (design) capacity curve to the mean capacity curve, the values of F_{cym}/F_d and F_{cym}/F_{cyf} based on the results shown in Tables 2 and 3 are calculated and shown in Table 4. Also, the above analyses are repeated by considering ε=3%, and the obtained results are also included in Table 4. It can be observed from the table that on average the strength of the designed structure is about 6 or 4 times of F_d (the minimum design base shear force) for the 3-, 7- and 10-storey SMRFs designed according to the 2005-NBCC. This indicates that the reliability of the structures by considering the strength requirement alone could be relatively higher. In relative terms, these frames are more conservative than the SMRFs designed according to the 1995-NBCC, since the latter is associated with the ratio of the strength of the designed structure to F_d that is about 4.5 and 3.0 for the 3- and 7-storey SMRFs. Also, the table suggests that the capacity at yield of the designed SMRFs defined by F_{cym} is about 1.26 times of F_{cyf}. The F_{cym}/F_{cyf} is close to the ratio between the mean to the factored design yield strength of material 1.23. Therefore, it would be expected that if a design is governed by the strength requirement specified in the design code, the mean to design strength of the structure is about 1.23.

Conclusions

Six ductile steel moment resisting frames (3-storey, 7-storey, and 10-storey) located in Victoria, B.C. are designed according to the 1995 edition and 2005 edition of the National Building Code. Assessment of the capacity curves of the frames by considering and ignoring the structural material properties and geometrical variable is carried out using the nonlinear static pushover analysis. The results suggest that:

a) The capacities of SMRFs designed according to the 2005-NBCC are highly depend on the first mode vibration period of the structure. For ductile SMRFs with short period (e.g., the 3-storey frame), the strength of the system is higher than that designed according to the 1995-NBCC. However, it is not the case for ductile SMRFs with longer period (e.g., 10-storey frame). This implies that the design requirement in the 2005-NBCC is not necessarily more stringent than that in the 1995-NBCC;

b) For all the designed frames, the lateral load capacity of the designed frames is much higher than the minimum required design base shear force. This is because the designs are governed by drift requirements. For 3-storey, 7-storey, and 10-story frame designed according to the 2005-NBCC, the ratio of the expected capacity at yield to the minimum required design base shear force is about 6.0, 4.5 and 4.5. These ratios are about 4.5, 3.0, and 3.0 if the 1995-NBCC is employed. These values are much greater than code suggested values of R_o and $1/U$, and are likely to impact the probability of incipient collapse. However, if a design is governed by the strength requirement specified in the design code, the ratio of the expected capacity at yield to design strength of the structure (which equal to the minimum required design base shear force) is about 1.23. In such a case, the use of $1/U$ and R_o to represent the overstrengthening factor in both codes may not be conservative; and

c) The magnitude of the uncertainty in the parameters characterizing the capacity curve caused by the uncertainty in structural properties and geometric variables is not very significant as compared to that of seismic excitations.

Acknowledgments

The financial supports of the Natural Science and Engineering Research Council of Canada and the University of Western Ontario are gratefully acknowledged.

References

Ellingwood, B. 2001. Earthquake risk assessment of building structures. *Reliability Engrg. And System Safety* 74(3), 251-262.

Ellingwood, B., Galambos, T.V., MacGregor, J.G., and Cornell, C.A. 1980. Development of a Probability Based Load Criterion for American National Standard A58. Spec. Pub. No. 577. *National Bureau of Standards*, Washington, D.C.

FEMA-356 2000. *Pre-standard and Commentary for the Seismic Rehabilitation of Buildings, FEMA 356-357.* American Society of Civil Engineers. Reston, VA.

FEMA-450 2004. *NEHR recommended provisions for seismic regulations for new buildings and other structures, FEMA 450*. Building Seismic Safety Council, National Institute of Building Sciences. Washington, D.C.

Galambos, T.V. and Ravindra, M.K. 1978. Properties of steel for use in LRFD. *Journal of the Structural Division*, **104**(9): 178–189.

Heidebrecht, A. 2003. Overview of seismic provisions of the proposed 2005 edition of the National Building Code of Canada. *Canadian Journal of Civil Engineering*, 30, 241–254.

Hong H.P., and Jiang J. 2004. Ratio between inelastic and elastic responses with uncertain structural properties. *Canadian Journal of Civil Engineering*, **31**(4): 703-711.

Humar, J.L. and Mahgoub, M.A. 2003. Determination of seismic design forces by equivalent static load method. *Canadian Journal of Civil Engineering*, 30, 287–307

Kennedy, D.J.L. and Gad Aly, M. 1980. Limit states design of steel structures –performance factors. *Canadian Journal of Civil Engineering*, 7, 45–77.

Kim, J. and Choi, H. 2005. Response modification factor of chevron-braced frames. *Engineering Structures*. 27, 285-300.

Krawinkler, H. 1996. Pushover Analysis: Why, How and When Not to Use it. *Proceedings of the 65th Annual Convention of the Structural Engineers Association of California*, Hawaii, 17-36.

Krawinkler, H. and Seneviratna, G.D.P.K. 1998. Pros and cons of a pushover analysis of seismic performance evaluation. Engineering Structures, Vol. 20, pp. 452-464.

Mitchell, D., et. al. 2003. Seismic force modification factors for the proposed 2005 edition of the National Building Code of Canada. *Canadian Journal of Civil Engineering,* 30, 308-327.

NBCC 1995. *National building code of Canada 1995.* Canadian Commission on Building and Fire Codes, National Research Council of Canada. Ottawa.

NBCC 2005. *National building code of Canada 2005*. Canadian Commission on Building and Fire Codes, National Research Council of Canada. Ottawa.

Prakash, V., Powell, G.H., and Campbell, S. 1993. DRAIN-2DX Base Program Description and User Guide, Version 1.10, *Structural Engineering Mechanics and Materials*, Report No. UCB/SEMM-93/17, University of California at Berkeley, CA.

Song, J. and Ellingwood, B. 1999. Seismic reliability of special moment steel frames with welded connections I. *J. Struct. Engrg.* ASCE, 125(4), 57-371.

Sues, R.H., Wen, Y.K. and Ang, H-S. 1985. Stochastic evaluation of seismic structural performance, *J. Struct. Eng.*, ASCE 6:1204-1218

Wang, W. 2006. Probabilistic assessment of steel frames designed according to NBCC seismic provisions, *Ph.D. thesis*, University of Western Ontario, London, Ontario.

Wen Y.K. 1993. Reliability-based design under multiple loads, *Structural Safety*, **13**:3-19.