13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 1535

SEISMIC PERFORMANCE OF CONCRETE FRAME/WALL BUILDINGS DESIGNED ACCORDING TO NBCC 2005

Jag Mohan HUMAR[1] and Ashutosh BAGCHI[2]

SUMMARY

The National Building Code of Canada (NBCC) is currently undergoing revision. In the proposed 2005 edition of NBCC, the seismic loading would be based on uniform hazard spectra (UHS) corresponding to 2% probability of exceedance in 50 years. Seismic loading provisions in most existing building codes focus on the minimum lateral seismic forces for which the building must be designed. Specifying the lateral forces alone is not enough to ensure that the desired level of performance will be achieved. In this regard, an assessment of the expected performance of the buildings designed according to the proposed requirements of 2005 NBCC is useful. Two sets of simple twelve storey buildings, one set with concrete moment resisting frames and the other with shear walls are considered here. The buildings are designed for seismic hazard corresponding to Vancouver, representing western Canada, and Montreal representing eastern Canada. Infill panels are included in the moment resisting frame models to simulate the non-structural elements. Spectrum compatible synthetic records are used for evaluating the dynamic response.

INTRODUCTION

Seismic loading provisions in most existing building codes specify the minimum lateral seismic forces for which the building must be designed. The specification of the lateral forces alone is of course not enough to ensure that the desired level of performance will be achieved. It will be some times before fully performance based seismic codes are developed. In the interim it will be useful to carry out an assessment of the performance that can be expected from the buildings designed according to the current codes.

In Canada the seismic design of buildings is performed according to the relevant provisions of the National Building Code of Canada (NBCC). The 1995 edition of NBCC [1] adopts a two parameter seismic zoning approach to quantify the seismic hazard across the country. The level of seismic risk at any site is expressed in terms of both the peak horizontal ground acceleration and the peak horizontal ground velocity, each with a probability of exceedance of 10% in 50 years. In the next version of the code (expected to be published in 2005), the seismic hazard will be represented by site dependent uniform hazard spectra corresponding to a 2% probability of being exceeded in 50 years (a return period of 2500 years). The 2005 NBCC seismic design provisions continue to rely on the specification of minimum lateral seismic forces for which the building must be designed and the acceptable drifts under such forces.

[1] Distinguished Research Professor, Department of Civil and Environmental Engineering, Carleton University, Ottawa, Canada
[2] NSERC Postdoctoral Fellow, ISIS Canada, Department of Civil Engineering, University of Manitoba, Winnipeg, Canada

In view of this an assessment of the performance that could be expected from buildings designed according to the proposed requirements of 2005 NBCC is of interest. This paper focuses on an evaluation of the expected performance of buildings with reinforced concrete moment resisting frames and simple shear walls. Performance is evaluated for buildings situated in two different locations: Vancouver representing a western location, and Montreal representing an eastern location in Canada. The buildings in these two locations have different dynamic characteristics and their performance achievements are found to be significantly different.

SEISMIC DESIGN PROVISIONS OF NBCC 2005

The technical background to the 2005 NBCC is presented in a series of articles in the Canadian Journal of Civil Engineering [2-7]. The equivalent static design provisions of the code are briefly reviewed here. In 2005 NBCC the seismic hazard is expressed in terms of a uniform hazard spectrum (UHS), which provides the maximum expected spectral acceleration S_a of a single-degree-of-freedom (SDOF) system with 5% damping. Site-specific values of the spectral acceleration S_a for the reference ground condition, defined as very firm soil or soft rock, are available from the Geological Survey of Canada [Adams *et al*, 8] and will be specified in the table of climatic data. The spectral values must be modified for the soil at the site. Such modification is carried out by applying the soil factors specified in the code. Two sets of soil factors are specified, factor F_a for the short period range and factor F_v for the long period range. The design spectral acceleration, $S(T)$ is obtained from the following expressions.

$$\begin{aligned} S(T) &= F_a S_a(0.2) \text{ for } T \le 0.2 \text{ s} \\ &= F_v S_a(0.5) \text{ or } F_a S_a(0.2) \text{ whichever is less for } T = 0.5 \text{ s} \\ &= F_v S_a(1.0) \text{ for } T = 1.0 \text{ s} \\ &= F_v S_a(2.0) \text{ for } T = 2.0 \text{ s} \\ &= F_v S_a(2.0)/2 \text{ for } T \ge 4.0 \text{ s} \end{aligned} \tag{1}$$

The elastic base shear, V_e for a single-degree-of-freedom building can be obtained by multiplying the spectral acceleration value $S(T)$ corresponding to the fundamental period of the building T_a with the weight of the building W. Considering the ductility capacity, the over-strength, the higher mode effects, and the importance of the structure, the design base shear is given by

$$V = \frac{S(T_a) M_v I_e W}{R_o R_d} \ge \frac{S(2.0) M_v I_e W}{R_o R_d} \tag{2}$$

where M_v accounts for higher mode effect, I_e is the importance factor, and R_d and R_o account for ductility and overstrength, respectively. The design base shear is distributed along the height of the building according to provisions that are similar to those in 1995 NBCC.

EVALUATION OF THE SEISMIC PERFORMANCE

The evaluation of seismic performance of a structure requires the estimation of its dynamic characteristics and the prediction of its response to the ground motions to which it could be subjected during its service life. The dynamic characteristics, namely the periods and mode shapes are obtained through an eigenvalue analysis. Inelastic time history analyses provide the damage states of the building when it is subjected to various levels of ground motion. Static push-over analysis could be used to determine the lateral load resisting capacity.

Damage parameters

Selection of appropriate damage parameters is very important for performance evaluation. Overall lateral deflection, ductility demand, and inter-storey drift are commonly used. Damage index developed by Park and Ang [9] is regarded as a good representation for structural damage in a concrete element, since it accounts for the damage caused by cyclic deformations into the post-yield level. Table 1 shows some guidelines relating the quantitative damage parameters with the qualitative performance levels as suggested by Vision 2000 Committee [10].

UHS compatible ground motion time histories

Inelastic time history analysis requires that one or more appropriate earthquake records be selected. Atkinson *et al* [11] have produced physically realistic time histories, which not only match the hazard spectrum, but also are representative of motions for specified magnitude distance scenarios in the regions of interest. In the absence of a database of actual ground motion records that are compatible to the location-based seismic hazard, Atkinson's records are used for evaluating the dynamic response.

Table 1: Performance levels and permissible damage

Damage parameter	Performance level				
	Fully operational	Operational	Life safe	Near collapse	Collapse
Drift					
(a) Transient	< 0.2%	< 0.5%	< 1.5%	< 2.5%	> 2.5%
(b) Permanent	Negligible	Negligible	< 0.5%	< 2.5%	> 2.5%

Computer modelling

Modelling a structure and representing it in a computer program for suitable analysis are important steps in performance evaluation. In the present study, DRAIN-2DX [12] is used for modal and push-over analyses, and DRAIN-RC [13] is used for inelastic dynamic analysis of frames. The moment-curvature relationship and moment-axial force interaction curves for reinforced concrete beam-column elements are obtained by a sectional analysis program developed by the first author.

BUILDINGS USED IN THE PRESENT STUDY

Description

The present study focuses on performance evaluation of twelve storey buildings with concrete moment resisting frames (CMRF), and shear wall frames designed according to the provisions of 2005 NBCC. The buildings are assumed to be situated in Vancouver in the west of Canada, and Montreal in the east of Canada. The geometric details of the buildings are shown in Figs. 1 and 2. The buildings have several six-meter bays in the N-S direction and 3 bays in E-W direction. The E-W bays consist of two nine-meter office bays and a central six-meter corridor bay. The storey height is 4.85 m for the first storey and 3.65 m for all other storeys.

The yield stress, f_y for reinforcing steel, and the 28-day concrete compressive stress, f_c' are assumed to be 400 MPa and 30 MPa, respectively. Live load on the roof is assumed to be 2.2 kN/m^2; on other floors it is 4.8 kN.m^2 on the corridor bay and 2.4 kN/m^2 on the other bays.

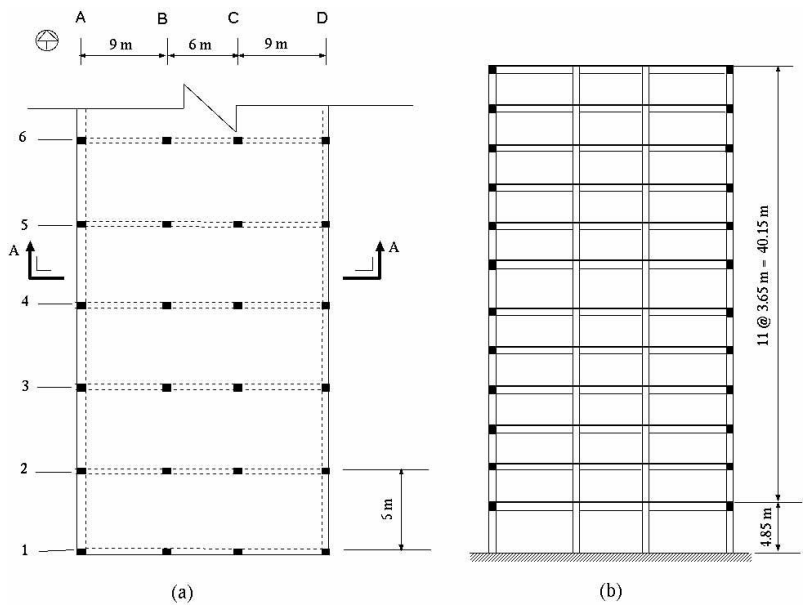

Figure 1: Layout of the building with moment resisting frames, (a) plan, and (b) elevation

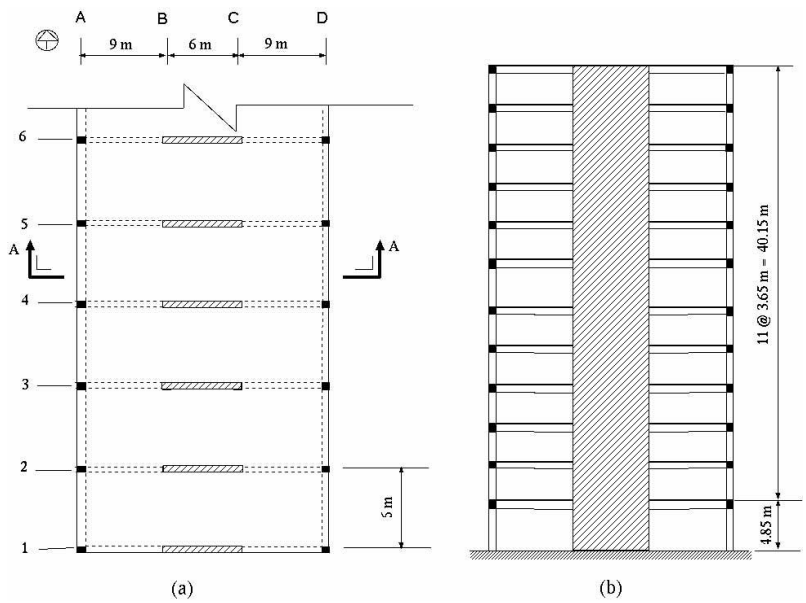

Figure 2: Layout of the building with shear wall (Vancouver), (a) plan, and (b) elevation

Modelling

For simplicity the exterior and interior ductile frames are kept similar. Thus a single frame along with the floor mass tributary to it can be used in the analysis, and the analysis procedure becomes two-dimensional. For consistency with this procedure, accidental torsion is not considered in obtaining the design forces.

Building structures with infill panels in the moment resisting frames are also studied. Each ductile moment resisting frame in Vancouver is assumed to contain infill panels of 100 mm thickness placed in the middle bay for the full height of the building. In a frame located in Montreal infill panels of 200 mm

thickness are used, again in the middle bay and for the full height. Since the number of ductile lateral load resisting frames in the CMRF building in Montreal is half of that for the building in Vancouver, the amount of infill panels expressed in terms of the wall cross section to floor area ratio (WFAR) works out to be same in both buildings, and is 0.41%. Infills are assumed to be built from clay masonry. The compressive strength, f_m of clay masonry is assumed to be 8.6 MPa and the modulus of elasticity to be 500 f_m [Shoostari, 13]. The infill panels are modelled using equivalent struts described by Stafford-Smith [14].

In general, the period of a bare frame in its fundamental mode of vibration is higher than the value obtained using the expression recommended by NBCC 2005. Non-structural elements in a frame play a major role in stiffening the structure, significantly reducing its fundamental period of vibration. It is presumed that the code expressions for calculating the fundamental period take into account the stiffening effect of non-structural elements. In the present study, infill panels are included in a frame to simulate the effect of non-structural elements

In a real building many different non-structural elements including infills contribute to the stiffness. Since, in the present study the infill panels are used to account for the effects of all non-structural elements and to bring down the period closer to the empirical NBCC value, the quantity and placement of the infill panels may be different from that actually present in the corresponding building, and may seem unrealistic in some cases.

For the shear wall buildings, two different models are considered, one in which the shear wall alone resists the lateral loads, and the other in which the shear wall as well as the beams and columns in the same frame resist the lateral loads.

Design of lateral load-resisting frames

The building frames are designed such that the lateral load case (that is, $D+0.5L+E$, where D, L and E are dead, live and seismic lateral loads respectively), not the gravity load case ($1.25D+1.5L$) governs the design of the ductile lateral load resisting frames. The western Canadian location, Vancouver has a higher level of seismic hazard as compared to the eastern Canadian location, Montreal. Hence for the buildings situated in Vancouver all transverse frames are assumed to be ductile lateral load resisting. For Montreal 50% of the transverse frames are assumed to be ductile lateral load resisting and the remaining are designed to take only gravity loads so that the design of ductile frames is governed by the lateral load case. An interior transverse frame is considered for the purpose of evaluation of the seismic performance. Gravity frames (for Montreal) are not considered to be part of the lateral load resisting system, and they are not included in the analysis. Wind load is not considered in the design, since the objective of this study is to evaluate the minimum level of seismic protection available to a building. In a case where wind load governs the design, the structure is expected to have a higher level of seismic protection.

The empirical expressions of NBCC 2005 for calculating the fundamental periods of concrete moment resisting frames and shear wall frame buildings are given in Equations 3(a) and 3(b), respectively, where, T_a is the fundamental period, and h_n is the height of the building above its base in meters.

$$T_a = 0.075(h_n)^{3/4} \tag{3a}$$

$$T_a = 0.050(h_n)^{3/4} \tag{3b}$$

It may be noted that the period given by Eqs. 3a and 3b depends on the height of the building alone, not on any other geometric or strength parameters. Consequently, the design forces for buildings of same height, will be calculated on the basis of identical periods, even when the buildings are located in different seismic zones.

In the calculation of the design base shears for the buildings used in this study, firm (reference) ground condition is assumed, and the following values are assigned to the different parameters: I_e = 1.0, R_o = 1.7, R_d = 4.0 for a moment resisting frame and 3.5 for a shear wall frame. The floor level lateral forces are obtained by distributing the design base shear across the height according to a procedure similar to the one suggested in NBCC 1995. Site-specific spectral accelerations used for Vancouver and Montreal are given in Table 2.

Table 2: Design Spectra for Vancouver and Montreal (Adams *et al* [2])

	$S_a(0.2)$	$S_a(0.5)$	$S_a(1.0)$	$S_a(2.0)$	$S_a(\geq 4.0)$
Vancouver	0.96	0.66	0.34	0.180	0.090
Montreal	0.69	0.34	0.14	0.048	0.024

Table 3: Beam and column details – moment resisting frames

Location	Member description	(Width, mm) x (Depth, mm)	Reinforcement details*
Vancouver	Beam (Storey 1-6)	400x600	8 #20 bars at top, 4 #20 bars at bottom
	Beam (Storey 7-11)	400x580	6 #20 bars at top, 3 #20 bars at bottom
	Beam (Roof)	400x550	5 #20 bars at top, 3 #20 bars at bottom
	External column	540x540	4 #25 and 8 #20 bars
	Internal column	600x600	12 #25 bars
Montreal	Beam (Storey 1-6)	400x560	7 #20 bars at top, 4 #20 bars at bottom
	Beam (Storey 7-11)	400x550	6 #20 bars at top, 3 #20 bars at bottom
	Beam (Roof)	400x600	5 #20 bars at top, 3 #20 bars at bottom
	External column	550x550	12 #20 bars
	Internal column	600x600	12 #20 bars

* 4L #10 stirrups @ 100 for beams and 4L #10 ties @100 for columns are used as lateral reinforcements

For the purpose of design, the member forces are determined using a linear elastic analysis, where the effect of cracking in concrete in a moment resisting frame is accounted for by assuming that the effective moments of inertia, I for the beams is $0.35I_g$ and that for the columns and walls is 0.70 I_g, where I_g is the gross value of the moment of inertia. $P-\Delta$ effect is taken into account in calculating the member forces. The assumed values of I for beams and columns, as mentioned above, are used for the initial design only. More accurate values of I, and other parameters that determine the moment curvature relationship for each element are calculated by sectional analysis. The frames are designed according to the capacity design philosophy specified in CSA Standard A23.3-94 [15], so that the total flexural capacity of the columns meeting at a joint exceeds the sum of the flexural capacities of the beams meeting at the same joint. Details of reinforcement in the ductile moment resisting frames for buildings located in Vancouver and Montreal are given in Table 3. The reinforcement details for the shear wall buildings are shown in Fig. 3.

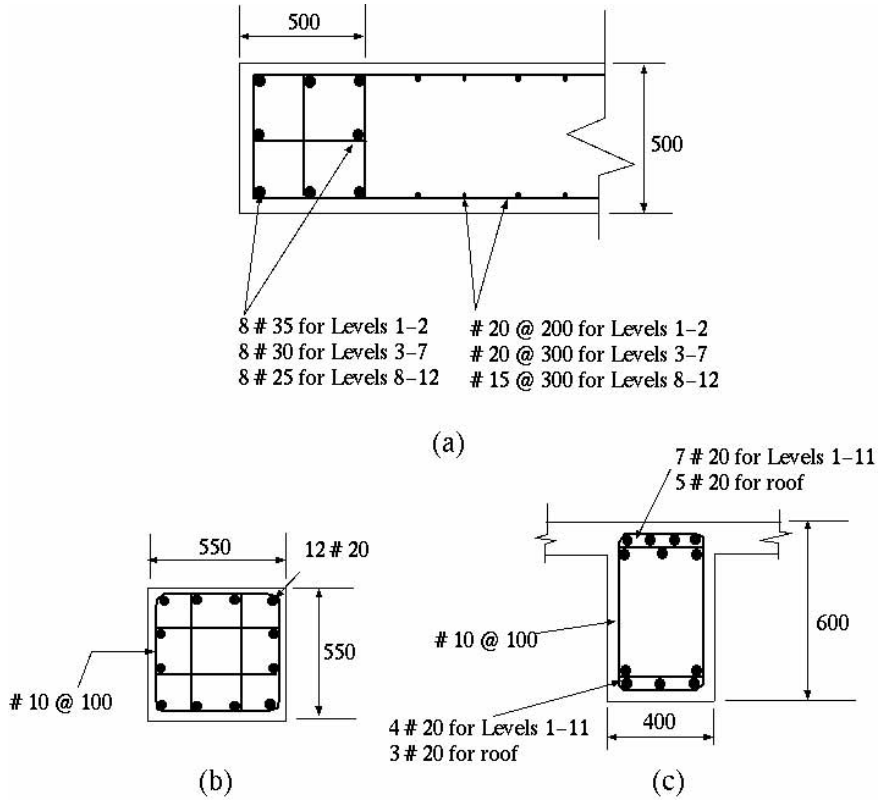

Figure 3: Details of reinforcement in a shear wall frame

ANALYSIS OF THE BUILDING FRAMES

Modal analysis is carried out at first to find the fundamental period of each building model. This is followed by a non-linear push-over analysis and non-linear dynamic analyses for the selected ground motion histories. For both push-over and dynamic analyses, gravity loads corresponding to the lateral load case (*i.e.*, D+0.5L) are used and the $P-\Delta$ effect is considered. The envelopes of damage parameters obtained from the dynamic analyses are evaluated to determine the qualitative performance of the buildings. The results are presented in the following paragraphs.

PERFORMANCE OF THE CMRF BUILDING IN WESTERN CANADA

Modal Analysis

Modal analyses of the bare and infilled frame models of the building with moment resisting frames in Vancouver are carried out to determine the period of the building. The periods of the bare frame and infilled frame are 2.782 s and 1.470 s, respectively. The corresponding empirical NBCC value is 1.30 s.

Lateral Load-Resisting Capacity

Push-over curves, representing the variation of base shear with the roof displacement in an internal lateral load-resisting frame are shown in Fig. 4(a) for both the bare frame and infilled frame. It is observed that the inclusion of infill panels drastically improves the capacity of the frame. Although not considered in the design infill panels contribute a great deal of strength to the overall capacity of a frame [Fajfar *et al,* 16], provided they are restrained against out of plane failure.

The design base shear, V_d is 511.2 kN for the frame being considered. A maximum interstorey drift of 2% occurs at a roof displacement close to 1% of the building height for the bare frame and 0.9% for the infilled frame. The base shear at this level of interstorey drift (*i.e.*, 2% of storey height) is $1.5V_d$ for the bare frame, and 4.8 V_d for the infilled frame. It will be observed that beyond an interstorey drift of 2% $P-\Delta$ effects cause the stiffness of the frame to become negative, signifying that the frame may be entering a region of possible instability. The infilled frame has significantly higher strength as compared to the bare frame.

Dynamic Response and Seismic Performance

Dynamic response analysis is carried out for a 2500-year earthquake ground motion. The maximum values of the inter-storey drift are shown in Fig. 4(b). Envelope values of other damage parameters are available but are not shown here for want of space. Both bare and infilled frame models suffer considerable damage under the 2500-year earthquake. In the case of bare frame, the maximum value of interstorey drift approaches 2.0% of storey height. Ductility demand in some of the beams exceeds 7.0, and maximum ductility demand at the base of the first storey columns is around 4.0. The global damage index is close to 0.6. The values of maximum ductility demand and damage index in beams and columns suggest that the building still has some lateral strength. The level of performance achieved by the structure under UHS-2500 events can be categorized as *life safe*. However, it is unsafe for post-earthquake occupancy.

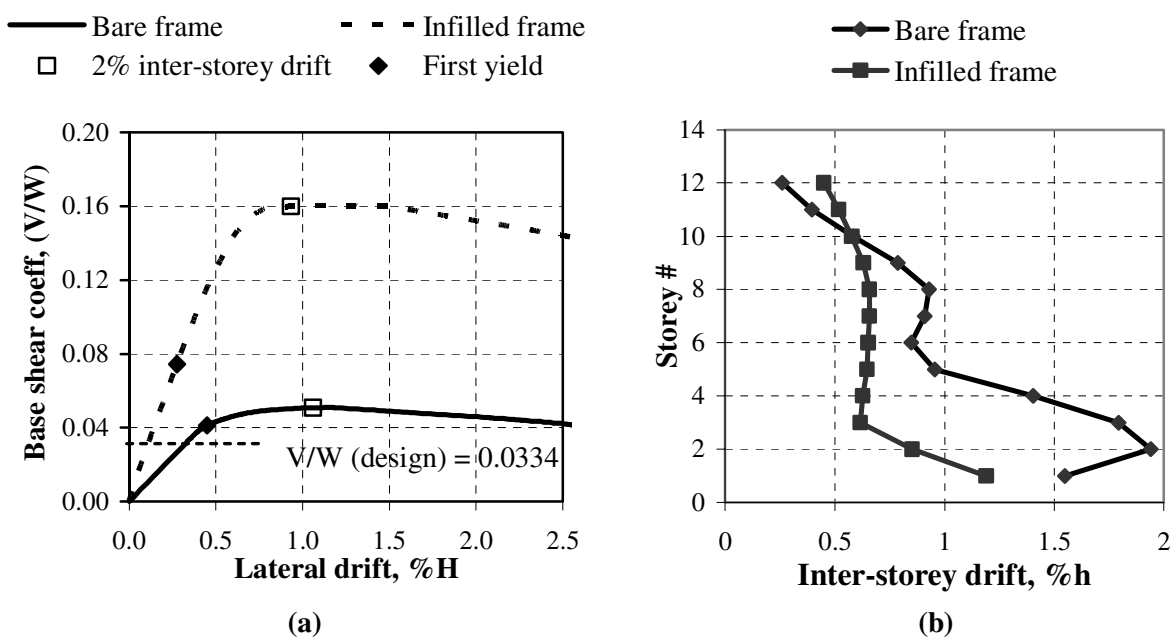

Figure 4: Response of the CMRF building in Vancouver, (a) static pushover curves, and (b) inter-storey drift envelope obtained from a dynamic analysis

The introduction of infill panels has a positive effect on the performance of the building. In this case, the maximum interstorey drift is close to 1.2%. The ductility demand in some of the beams is close to 2.0, and the ductility demand in columns is negligible. The damage index is close to 0.5. Considering the response of the infilled frame model, the performance of the building under UHS-2500 can be said to be *life safe*.

PERFORMANCE OF THE CMRF BUILDING IN EASTERN CANADA

Modal Analysis

Modal analyses of the bare and infilled frame models of the building with moment resisting frames in located in Montreal are conducted to determine the periods. The periods of the bare and infilled frame models are 4.412 s and 1.985 s, respectively, while the empirical NBCC value is 1.3 s.

Lateral Load-Resisting Capacity

Push-over curves, representing the variation of the base shear with the roof displacement are shown in Fig. 5(a). The maximum interstorey drift of 2% occurs at a roof displacement close to 1.0% of the building height for both bare and infilled frame models. The design base shear, V_d is 404.2 kN for the ductile frame. The base shear corresponding to 2% interstorey drift is $1.6V_d$ for the bare frame model and $8.1V_d$ for the infilled frame model. Beyond an interstorey drift of 2% $P-\Delta$ effect causes the lateral stiffness of the frame to become negative.

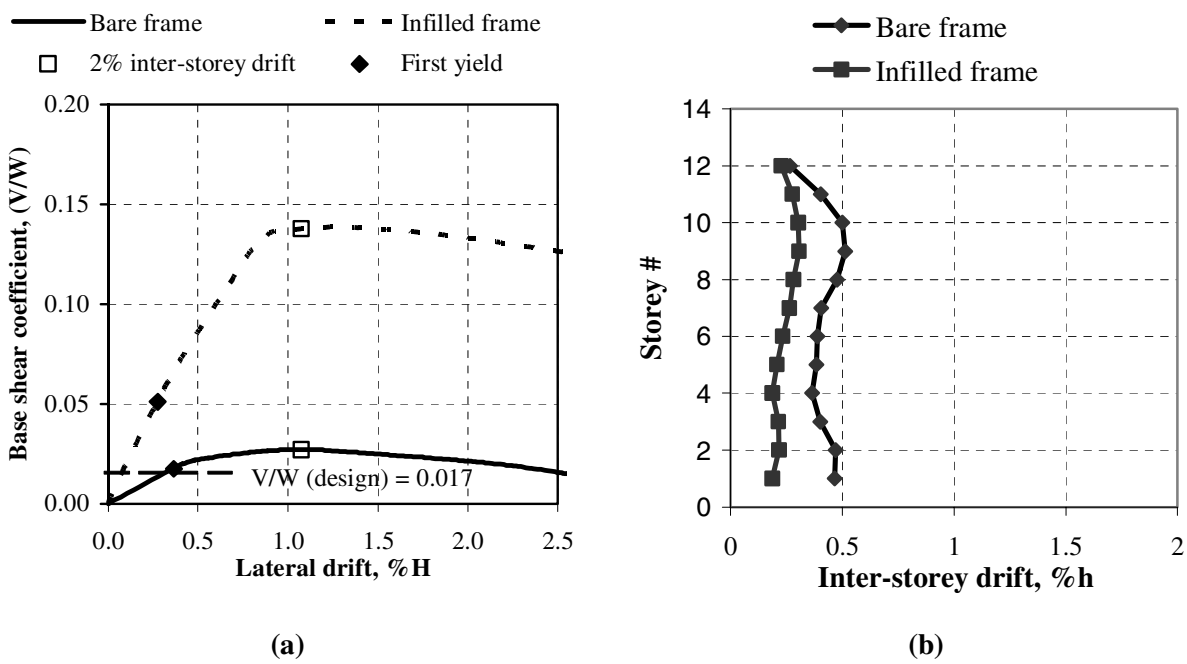

Figure 5: Response of the CMRF building in Montreal, (a) static pushover curves, and (b) inter-storey drift envelope obtained from a dynamic analysis

Dynamic Response and Seismic Performance

The maximum values of the inter-storey drift produced by a 2500-year earthquake are shown in Fig. 5(b). Envelope values of other damage parameters are available but are not shown here for want of space. Under the 2500-year earthquake the frame models suffer some minor damage. There may be cracking in various members, but none of the members yield. The inter-storey drift ratio in the bare frame model barely exceeds 0.5% of storey height. The structure remains mostly elastic. The performance level in this case can be said to be *operational*, and some minor repair work may be necessary for the normal functioning of the building. The response of the infilled frame model is even lower and its performance can be categorized as *fully operational*.

PERFORMANCE OF THE SHEAR WALL BUILDING IN WESTERN CANADA

Modal Analysis

Modal analysis of the shear wall and wall-frame models of the shear wall building in Vancouver is conducted to determine the periods. The fundamental periods of the wall and wall-frame models are 2.275 s and 1.823 s, respectively, while the empirical NBCC value is 0.864 s.

Lateral Load-Resisting Capacity

Push-over curves of the wall and wall-frame models are shown in Fig. 6(a). There are two curves in the plot, one corresponding to the wall model and the other corresponding to the wall-frame model. The design base shear, V_d is 947.1 kN. The roof displacement of the wall model corresponding to a maximum inter-storey drift value of 2% of building height is close to 1.6% of storey height, and the base-shear at that drift level is $1.43V_d$. The wall-frame model shows significant increase in the lateral load carrying capacity. The roof displacement corresponding to 2% inter-storey drift is close to 1.7% of the building height, and the base-shear at that drift level is 2.33 V_d.

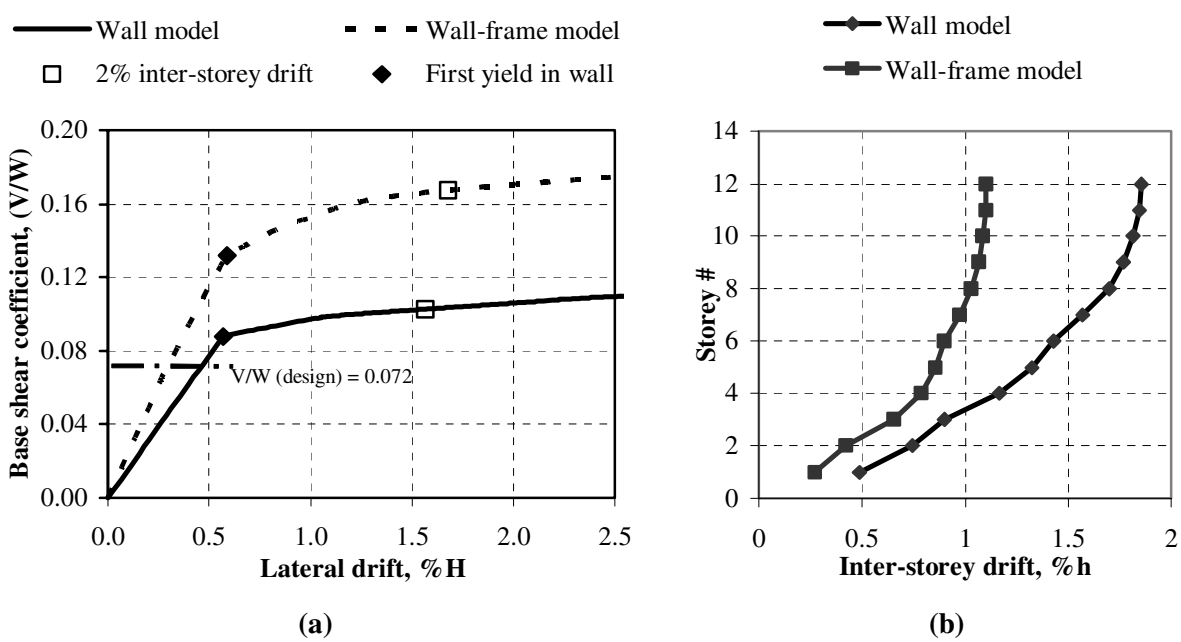

Figure 6: Response of the shear wall building in Vancouver, (a) static pushover curves, and (b) inter-storey drift envelope obtained from a dynamic analysis

Dynamic Response and Seismic Performance

The maximum values of the inter-storey drift produced by a 2500-year earthquake are shown in Fig. 6(b). Envelope values of other damage parameters are available but are not shown here for want of space.

The maximum value of inter-storey drift in the wall model is close to 2 %, and the maximum value of shear wall ductility is close to 4.5. The performance level achieved by the building can be categorized as *Life safe*, and the building will not be available for post-earthquake occupancy. The wall-frame model records a relatively lower level of damage. The maximum inter-storey drift marginally exceeds 1.0%. The maximum shear wall ductility is 2.5. The maximum value of beam ductility is somewhat higher than 2.0,

but the columns remain elastic. The performance level of the building under UHS-2500 events can be said to be *life-safe* in this case too.

PERFORMANCE OF THE SHEAR WALL BUILDING IN EASTERN CANADA

Modal Analysis

Modal analyses of the shear wall and wall-frame models of the shear wall building in Vancouver are conducted to determine the periods. The periods of the wall and wall-frame models are 3.217 s and 2.578 s, respectively, while the empirical NBCC value is 0.864 s.

Lateral Load-Resisting Capacity

The design base shear, V_d for a shear wall frame of the building is 868.1 kN. The maximum value of roof-displacement in the wall model corresponding to an inter-storey drift of 2% is 1.6%; the corresponding value for the wall-frame model is 1.7%. The base shear coefficient at this drift level for is $1.56V_d$ for the wall model, and $2.54V_d$ for the wall-frame model.

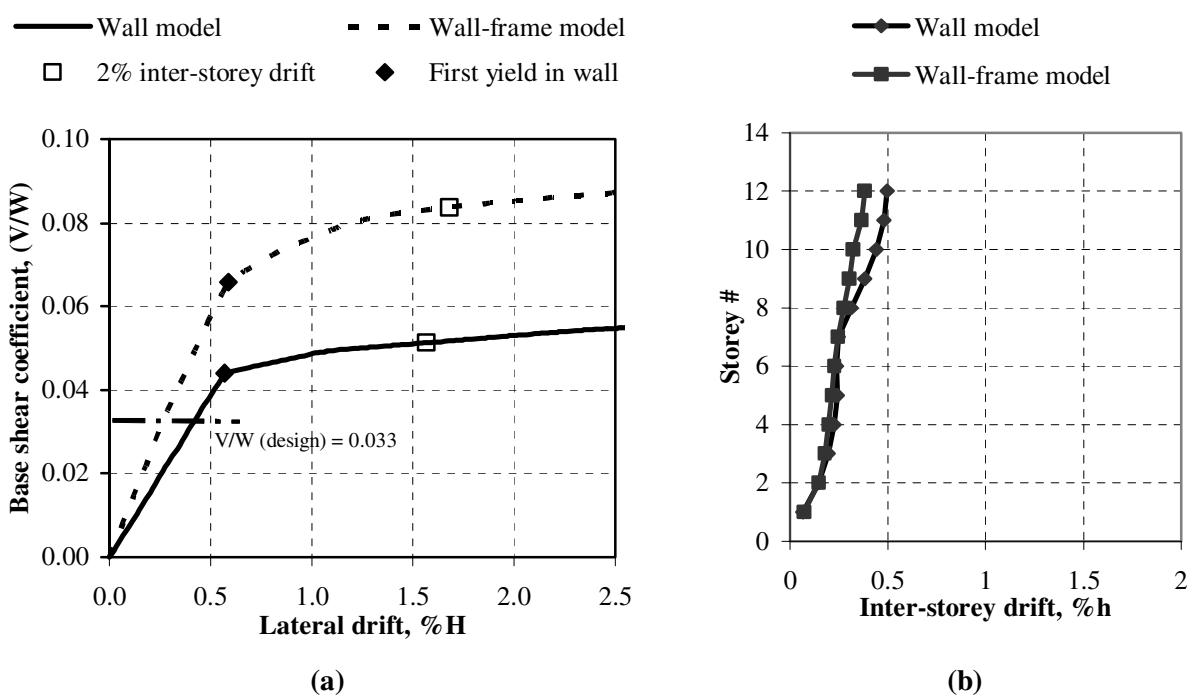

Figure 7: Response of the shear wall building in Montreal, (a) static pushover curves, and (b) inter-storey drift envelope obtained from a dynamic analysis

Dynamic Response and Seismic Performance

The damage suffered by the building under the 2500-year earthquake is moderate. The maximum value of inter-storey drift is close to 0.5% in the wall model but below 0.5% in the wall-frame model. The wall suffers extensive cracking, but remains elastic in all the cases. Some of the beams in the wall-frame model just yield but the columns remain elastic. The performance level achieved by the building can be said to be *operational* for both the wall and wall-frame models.

DISCUSSION AND CONCLUSIONS

Seismic performance of buildings with concrete moment resisting frames and shear walls are presented in this paper. Vancouver and Montreal are chosen as the representative locations in terms of seismic hazard levels for western and eastern Canada, respectively. In the case of a building with moment resting frames located in Vancouver, all the transverse frames are assumed to be ductile lateral load resisting. For the building in Montreal, only half of the transverse frames are considered to be ductile lateral load-resisting, while the remaining frames are assumed to be capable of resisting only gravity loads. In the case of a building with shear walls located in Vancouver, all the transverse frames are assumed to contain a shear wall. For the building in Montreal, only half of the transverse frames are assumed to contain a shear wall, while the remaining frames are assumed to be capable of resisting only gravity loads. The lateral load-resisting systems are designed according to the seismic provisions of the upcoming version of NBCC (to be published in 2005) based on the seismic hazard level corresponding to 2% probability of exceedance in 50 years (2500 return period).

Static push-over analyses are performed to determine the capacity of each frame. A series of nonlinear dynamic analyses are carried out for various models of a building in order to determine the extent of damage induced by the given level of seismic hazard.

Non-structural elements have a significant influence on the capacity and performance of a frame. They also influence the period of a structure. Infill panel elements are used in this study to simulate the effect of non-structural elements in moment resisting frames. The number of infill panel is chosen such that the period of the structure is close to the NBCC 1995 values. From the push-over curves, it is clear that the inclusion of infill panels in the building models increases the strength and stiffness of the models, but may slightly reduce the ductility capacity of the overall system. Hinge patterns, not shown here, show that while the pattern of yielding in the bare frame exhibits the effect of capacity design, the yielding pattern in infilled frames is not as predictable. A similar observation has been made by Fajfar *et al* [16].

The building with moment resisting frames located in Vancouver suffers significant damage under 2500-year earthquake and the performance level achieved by the building is *life safe*, for both bare and infilled frame models. The corresponding building in Montreal does not suffer significant damage and remains *operational*.

The shear wall building in Vancouver suffers significant damage under the 2500-year earthquake and the performance level achieved by the building is *life safe.* The corresponding building in Montreal attains the *operational* level of seismic performance.

The buildings in eastern Canada do not suffer significant amount of damage under the 2500-year earthquake hazard. This indicates that the buildings in eastern Canada, when designed according to NBCC, are not vulnerable to seismic hazard in that region. This can be attributed to the difference between the design period calculated from the empirical expressions given in the building code and the analytical period of a building, and the spectral shapes in eastern and western Canada.

Because of the lower level of seismic hazard in Montreal, only the alternate frames are designed to resist the lateral loads. Consequently the mass tributary to a lateral load-resisting frame of a building in Montreal is twice as much as that in Vancouver. The analytical period of the Montreal building therefore becomes comparatively longer. On the other hand, the periods calculated by the NBCC expressions are identical for the corresponding buildings in Montreal and Vancouver, being dependent only on the height. Thus, for the building in Montreal the difference between the analytical and NBCC periods is considerably larger than that in Vancouver. The difference between the NBCC period, which determines

the design strength, and the analytical period which determines the intensity of seismic input in a dynamic analysis, implies that a Montreal building has a larger overstrength compared to the corresponding Vancouver building. This difference between the design strength and seismic demand for Montreal is further amplified because of the spectral shapes in the Eastern region of Canada, where the spectrum descends rapidly with increasing period.

Most seismic codes including NBCC employ a single empirical expression for determining the fundamental period of a building irrespective of the nature of seismic hazard, or the shape of the hazard spectrum. Thus, buildings of equal heights are assigned the same period even when located in zones of different seismicity. On the other hand a building located in a zone of low seismicity will be designed for lower earthquake forces and is therefore likely to be less stiff than a similar building located in a zone of higher seismicity. A modal analysis will thus provide a larger value for the fundamental period of the building located in the zone of low seismicity, and hence a larger difference between the periods determined from the code formula and the modal analysis. Because of this increased difference between the two periods, buildings in zones of low seismicity and designed for a base shear based on the empirical expression of a seismic code for determining the fundamental period will often have considerable overstrength and will be able to resist an earthquake of intensity significantly higher than the design earthquake.

Although the trend in seismic behaviour of the buildings in western and eastern Canadian locations is expected to be similar to the cases presented here, it may not be possible to reach a general conclusion based on the limited number of cases studied here. A number of buildings with different height and configuration need to be studied to assess the performance characteristics of the buildings designed according to the new seismic guidelines of NBCC 2005.

REFERENCES

1. Canadian Commission on Building and Fire Code. “National Building Code of Canada.” National Research Council of Canada, Ottawa, Ontario, 1995.

2. Adams J, Atkinson G. “Development of seismic hazard maps for the proposed 2005 edition of the National Building Code of Canada.” Canadian Journal of Civil Engineering, 2003; 30: 255-271.

3. Heidebrecht AC. “Overview of the seismic provisions of the proposed 2005 edition of the National Building Code of Canada.” Canadian Journal of Civil Engineering, 2003; 30: 241-254.

4. Humar J, Mahgoub MA. “Determination of seismic design forces by equivalent static load method.” Canadian Journal of Civil Engineering, 2003; 30: 287-307.

5. Saatcioglu M, Humar J. “Dynamic analysis of buildings for earthquake resistant design.” Canadian Journal of Civil Engineering, 2003; 30: 338-359

6. Finn WDL, Wightman A. “Ground motion amplification factors for the proposed 2005 edition of National Building Code of Canada.” Canadian Journal of Civil Engineering, 2003; 30: 272-278.

7. De Vall R. “Background information for some of the proposed earthquake design provisions for the 2005 edition of the National Building Code of Canada.” Canadian Journal of Civil Engineering, 2003; 30, 279-286.

8. Adams J., Halchuck S. “Fourth generation of the seismic hazard maps of Canada: Values for over 650 localities intended for the 200f National Building Code of Canada.” Geological Survey of Canada Open File 4459, Natural Resources Canada, Ottawa, 2003.

9. Park YJ, Ang AHS. “Seismic damage model for reinforced concrete.” ASCE Journal of Structural Engineering 1985; 111: 722-739.

10. Vision 2000 Committee. “Performance based seismic engineering of buildings.” Structural Engineers Association of California (SEAOC), Sacramento, California, 1995.

11. Atkinson GM, Beresnev I. “Compatible ground motion time-histories for new National Hazard Maps.” Canadian Journal of Civil Engineering, 1998; 25: 305-318.

12. Prakash V, Powell GH, Campbell S. “DRAIN-2DX Base Program Description and User Guide: Version 1.10.” Report No. UCB/SEMM-93/17, Department of Civil Engineering, University of California, Berkeley, 1993.

13. Shoostari A. “Seismic drift demand of reinforced concrete buildings.” Ph.D. Thesis, Department of Civil Engineering, University of Ottawa, Canada, 1998.

14. Stafford-Smith B. “Behaviour of square infilled frames.” ASCE Journal of Structural Engineering, 1996. 91, 381-403.

15. Canadian Standard Association (CSA). “Design of Concrete Structures, CSA Standard A23.3-94.” Canadian Standards Association, Rexdale, Ontario, 1994.

16. Fajfar P, Gaspersic P, Drobnic D. “A simplified nonlinear method for seismic damage analysis of structures.” Fajfar P, Krawinkler H, Editors. Seismic design methodologies for the next generation of codes. Rotterdam: AA Balkema, 1997: 183-193.