8th Canadian Conference on Earthquake Engineering / 8ième Conférence canadienne sur le génie paraséismique
Vancouver – 1999

Seismic performance of buildings designed to National Building Code of Canada

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ABSTRACT

Buildings designed according to the National Building Code of Canada are analysed to verify whether they achieve the expected levels of performance under earthquake loads. The buildings studied have a lateral load resisting system consisting of moment-resisting frames of concrete. They are assumed to be located in two representative cities of Canada, Victoria in the west, and Montreal in the east. The evaluation of performance is based on the results obtained from push-over and dynamic time history analyses. It is observed that all the buildings studied perform reasonably well. However, buildings in the western region of Canada are more vulnerable to seismic hazard in comparison to those in the eastern region.

INTRODUCTION

Buildings are generally designed to resist the minimum earthquake-induced lateral loads specified in the relevant building codes. However, design to resist the minimum lateral loads alone may not be enough to ensure the desired level of performance of a building structure during an earthquake. Although it is difficult to establish a direct relationship between the method of design and the performance objectives, the necessity of performance based design has been stressed by many researchers and building code agencies (Fajfar et. al. 1996; Heidebrecht 1997). The first step in the development of a performance based design is the definition of seismic hazard for the site of the building. Canadian National Committee on Earthquake Engineering (CANCEE) is currently working on the development of a suitable format for expressing the seismic hazard in Canadian locations in terms of uniform hazard spectra (UHS). The outline of such a format is presented in a preliminary report of the committee (CANCEE, 1996). The second step in performance based design is the definition of an acceptable level of damage in the building when it is exposed to the expected seismic hazard. The acceptable level of damage is related to the performance objectives for the building. Currently, the National Building Code of Canada (NBCC) states the design objectives in terms of the anticipation that the buildings would be able to resist minor levels of earthquake ground motion without damage, resist moderate levels of earthquake ground motion without structural damage, but possibly with some non-structural damage, and resist major levels of earthquake ground motion without collapse, but with some structural as well as non-structural damage. It is important to define the performance objectives in more explicit terms, both qualitative and quantitative. It is also important to ascertain how well the buildings designed according to the provisions of NBCC meet these objectives. The study presented here attempts to do this.

DEFINITIONS AND METHODOLOGY

The seismic hazard is generally expressed in terms of the recurrence interval or a probability of exceedance of the earthquake events occuring at a given location. In a recent report the Vision 2000 Committee of the Structural Engineers Association of California (Vision, 1995) has suggested that the following levels of design earthquakes (expressed in terms of the recurrence intervals) be used in performance based engineering of buildings:

1. Frequent (43 years)
2. Occasional (72 years)
3. Rare (475 years)
4. Very Rare(970 years)
5. Extremely Rare (2500 years)

The Vision 2000 report defines a performance level as the maximum acceptable damage in a building when it is subjected to a specific level of earthquake. The damage suffered by structural and non-structural elements and

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contents as well as the availability of the site utilities necessary to building function are considered in defining the performance levels. The performance levels are defined in terms of the following qualitative measures.

1. Fully operational
2. Operational
3. Life-safe
4. Near collapse
5. Collapse

For engineering applications, the foregoing performance levels need to be expressed in quantitative terms. Vision 2000 report provides some suggestions on the quantitative measures of performance based on drift levels. Other response/damage parameters, such as, the damage index proposed by Park and Ang (1985a, b), can also be used to quantify the performance levels. Table 1 provides some guidelines on quantitative measures of seismic performance of normal building structures. In Table 1 drift indicates interstorey drift and the Park-Ang damage index is the global index.

The evaluation of seismic performance of any structure requires the estimation of its dynamic characteristics and the prediction of its response to the ground motion 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. Static push-over analysis could be used to determine the lateral load resisting capacity of a structure and the maximum level of damage in the structure at the ultimate load. Inelastic time history analyses provide the damage states of the building when it is subjected to various levels of ground motion. Selection of appropriate damage parameters is very important for performance evaluation. Overall lateral deflection and interstorey drift are parameters most commonly used. Overall deflection is not always a good indicator of damage, but interstorey drift is quite useful because it is representative of the damage to the lateral load-resisting system. Maximum values of member or joint rotations, curvature and ductility factors are also good indicators of damage because they can be directly related to the element deformation capacities. Damage index developed by Park and Ang (1985a) is also regarded as a good representation for structural damage, as it accounts for the damage caused by cyclic deformations into the post-yield level.

When dynamic time-history analyses are used for performance evaluation, selection of appropriate earthquake records is very important. If actual earthquake records are used, only those records may be used whose spectra closely match the uniform hazard spectrum corresponding to the selected hazard level for the site of interest. As an alternative, UHS compatible ground motion time histories, such as those developed by Atkinson et. al. (1998) for Canadian conditions, can be used. The Atkinson records are used in the present study. For any given location these records comprise a set of 4 records corresponding to each level of hazard. The four records in a set are: (1) long duration event, trial 1; (2) long duration event, trial 2; (3) short duration event, trial 1, and (4) short duration event, trial 2.

In the present study, performance evaluation is carried out on a series of building frames designed according to NBCC. The ultimate lateral load carrying capacity of each frame is determined through a static push over analysis using computer program DRAIN-2DX (Prakash et.al. 1993). The seismic performance is then determined through a series of inelastic dynamic analyses using program DRAIN-RC (Alsiwat 1993, Shoostari 1997). Each frame is analysed for three different levels of earthquake hazards. These levels correspond to earthquake return periods of 475 years (UHS-500), 970 years (UHS-1000), and 2500 years (UHS-2500). Each frame is analysed for all four records for a given level of earthquake hazard and the envelopes of damage parameters are used to determine the seismic performance of the frame. The damage parameters used in the evaluation of the performance of a building are: interstorey drift, beam and column ductility demands, and Park-Ang damage index (element level as well as global). The beam and column ductility demand μ is defined as $\mu = \theta_m/\theta_y$, where θ_m is the maximum rotation at the end of a beam-column element, and θ_y is the rotation when yielding starts.

DESCRIPTION OF BUILDING MODELS

Six- and twelve-storey buildings are considered and they are assumed to be located in Victoria in western Canada and Montreal in eastern Canada. A plan view of the buildings considered is shown in Fig. 1a. It has seven 6-meter bays in N-S direction and three bays in E-W direction. The E-W bays consist of two 9-meter office bays and a central 6-meter corridor bay. The storey height is 4.85 m for the first storey and 3.65 m for all other storeys.

The elevations of the frames for six- and twelve-storey buildings are shown in Fig. 1b and c, respectively. The yield stress for reinforcing steel, f_y, is assumed to be 400 Mpa, and the 28-day concrete compressive stress, f'_c, is taken as 30 MPa. The following gravity loads are used in the design. Dead load is assumed to be 3.5 kN/m^2 on the roof and 5.0 kN/m^2 on all other floors. Live load is assumed to be 2.2 kN/m^2 on the roof and 2.4 kN/m^2 on all other floors.

The seismic lateral forces are obtained using the new UHS based methodology proposed by CANCEE (1996) for base shear calculation. The base shear is distributed across the height of the frame, using the procedure suggested by NBCC 1995 to obtain the floor level forces. The western Canadian location, Victoria has a higher level of seismic hazard as compared to the eastern Canadian location, Montreal. Hence for the buildings situated in Victoria all the transverse frames are assumed to be ductile lateral load-resistant, while for Montreal 50% of the transverse frames are assumed to be ductile lateral load-resistant and the rest are designed to take only gravity loads. For Victoria an interior transverse frame is considered for the purpose of evaluation of the seismic performance. For Montreal two transverse frames are considered, one of which is designed as a lateral load-resistant frame, and the other is designed as a lean-to frame (designed for gravity loads alone). These two frames (lateral load-resisting and gravity frames) can be considered together in the analyses, by connecting them through rigid links.

It is well known that when only the bare concrete frames are assumed to contribute to the lateral stiffness, the calculated building period is significantly smaller than the period determined from the empirical formulas given in the NBCC. It is sometimes reasoned that the presence of nonstructural elements makes the building considerably stiffer and makes the building period closer to that determined from NBCC. In the present study the effect of such nonstructural elements is also taken into account. Infill panels of masonry are included in some frame models to simulate the effect of non-structural elements. The modulus of elasticity of masonry used in the infills is assumed to be $750f_m$ for concrete blocks and $500f_m$ for clay masonry, where f_m is the compressive strength of masonry. Clay masonry with $f_m = 8.6$ MPa and a thickness of 100 mm is used in all the cases considered here. The lateral load-resisting frames are designed as fully ductile with a force modification factor, R equal to 4, as provided in NBCC-95. The frames are designed according to the capacity design philosophy, so that the total flexural capacity of a column exceeds the sum of the flexural capacities of the beams meeting the column at a joint. Two ductile lateral load-resisting frames, six and twelve storeys in height, respectively, are designed for a location in Victoria. For Montreal two ductile lateral load-resisting frames and two gravity frames are designed. The capacity design philosophy is used in the design of ductile frames, and the provisons of CSA Standard A23.3-94 (CSA 94) are followed in the design of concrete members.

PERFORMANCE OF BUILDINGS IN WESTERN CANADA

A modal analysis of the frame being studied is carried out first to determine the number of infill panels that would be required to bring the period of the frame closer to the value recommended by NBCC. Three infill panels in the middle bay are required for the six storey frame, while six infill panels in the middle bay are necessary for the twelve-storey frame. Two levels of strength for the structural materials (factored and nominal) are considered. The following four models are studied: (1) bare frame with factored material strength, (2) bare frame with nominal material strength, (3) infilled frame with factored material strength, and (4) infilled frame with nominal material strength

Push-over analyses, in which the base shear is distributed along the height according to the provisions of NBCC are carried out to determine the ultimate capacity. Push-over curves, or capacity curves, representing the variation of base shear with the roof displacement (i.e., lateral drift) in an internal lateral load-resisting frame are shown in Fig. 2a. Curves for both bare and infilled frames are shown. In fact, there are two curves for each type of frame, one in which the factored value of material strength is used as implied in the design, and the other in which nominal value of material strength is used. It is observed that the inclusion of infill panels improves the capacity of the frame drastically. The effect of infill panels is generally not considered in the design. In reality they contribute a great deal of strength to the overall capacity of a frame (Drysdale et. al. 1994) provided that they are positively connected to the frame. The push-over curves for the interior transverse frame of a twelve-storey building are shown in Fig. 2b. In this case also the infilled frame has significantly higher strength than the bare frame.

The bare and the infilled frame models are analyzed for all three levels of earthquake hazards. Seismic performance of six storey building located in Victoria is summarized in Table 2. It is noticed that the infill panels consistently improve the performance of the building. Seismic performance of twelve-storey building located in Victoria under various levels of earthquake hazard is summarized in Table 3.

The analytical results show that under UHS-500 events (the one used for the design of buildings studied) the buildings remain fully operational or operational, depending on whether the effect of infill panels is considered are not. At higher levels of seismic hazard, they suffer significant damage. Under UHS-1000 the buildings can be considered as being life safe, while under UHS-2500 they are either near collapse or have collapsed.

It is observed that the infill panels generally help a frame in achieving a better performance, but in some cases they may prove to have a detrimental effect. The detrimental effect of infill panels is observed when the 12 storey frame models are subjected to UHS-1000 events. These events cause moderate damage in the bare frame models, but they cause heavy damage in the infill frame models. A unique result is obtained in this case because of the shape of UHS-1000, that shows a reduced response corresponding to the period of a twelve-storey bare frame.

PERFORMANCE OF BUILDINGS IN EASTERN CANADA

As stated earlier, for buildings located in Montreal half of the frames are designed to resist the lateral loads due to earthquake along with the gravity loads tributary to them. The other frames are designed to resist only gravity loads. A number of different frame models are analysed, consisting either of a lateral load-resisting frame alone, or a combination of one lateral load-resisting frame and one gravity frame. In the latter case, the frames are connected by a rigid axial link at each storey level. In some models infill panels are introduced in the lateral load-resisting frame. Modal analyses are carried out for models consisting of the bare lateral load-resisting frame alone, the bare lateral load-resisting frame together with a gravity frame, and the infilled frame together with a gravity frame. Such analyses allow the determination of the number of infill panels required to bring the periods of the frames closer to the values determined on the basis of empirical equations in NBCC. In the six-storey frame four infill panels in the middle bay are required. In the twelve-storey frame nine panels in the middle bay and three panels in each of the exterior bays are necessary. Two material strength levels are considered, namely factored strengths and nominal strengths. Following six cases are considered for each building: (1) bare ductile lateral load-resisting frame with factored material strength, (2) bare ductile lateral load-resisting frame with nominal material strength, (3) bare ductile lateral load-resisting frame together with a gravity frame and with factored material strength, (4) bare ductile lateral load-resisting frame together with a gravity frame and with nominal material strength, (5) infilled frame together with a gravity frame and with factored material strength, and (6) infilled frame together with the gravity frame and with nominal material strength.

Push-over curves, representing the variation of the base shear with the roof displacement are shown in Fig. 3a. Three pairs of curves are shown in this figure; the first pair is for the bare ductile lateral load-resisting frame alone (designated as single frame), the second pair is for the bare frame ductile lateral load-resisting frame together with a gravity frame, and the third pair is for the infilled frame together with a gravity frame. The push-over curves for twelve-storey frames are shown in Fig. 3b.

Six models, as described earlier, are analysed for the evaluation of seismic performance of the six-storey building located in Montreal. Analyses are carried out for 3 levels of earthquake hazards; UHS-500, UHS-1000 and UHS-2500. A summary of the performance levels achieved by the different models of six-storey building is given in Table 4, while a summary of the performance levels achieved by the twelve-storey building is given in Table 5.

The analytical results show that the buildings do not suffer significant damage under the design level earthquake or under the higher earthquakes. In most cases the buildings are seen to be fully operational under UHS-500 and UHS-1000 events, and remain operational under UHS-2500 events. This indicates that the buildings in eastern Canada are not vulnerable to the seismic hazard in that region.

DISCUSSION AND CONCLUSIONS

Performance evaluation of buildings designed according to the seismic provisions of NBCC is carried out through a series of analytical studies that include push-over analysis, modal analysis and inelastic dynamic analysis. It is observed that the buildings in western Canada are more vulnerable to seismic hazard as compared to those in eastern Canada. Non-structural elements have 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. It is observed that the lateral load capacity of a frame increases dramatically when infill panels are included. However, there may be occasional cases, when infill panels prove to be detrimental to the performance of a building.

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Table 1: Performance Levels and Permissible Structural Damage

System Description	Performance Level				
	Fully operational	Operational	Life safe	Near Collapse	Collapse
Drift a) Transient b) Permanent	 $< 0.2\%$ Negligible	 $< 0.5\%$ Negligible	 $< 1.5\%$ $< 0.5\%$	 $< 2.5\%$ $< 2.5\%$	 $> 2.5\%$ $> 2.5\%$
Park and Ang Damage index	< 0.20	< 0.40	< 0.70	< 1.0	> 1.0

Table 2: Performance of the six storey building located in Victoria

Frame type	Performance level		
	UHS-500	UHS-1000	UHS-2500
Bare frame	Operational	Life safe	Near collapse
Infilled frame	Fully operational	Operational	Life safe

Table 3: Performance of the twelve storey building located in Victoria

Frame type	Performance level		
	UHS-500	UHS-1000	UHS-2500
Bare frame	Operational	Fully operational	Collapse
Infilled frame	Fully operational	Life safe	Near collapse

Table 4: Performance of the sic storey building located in Montreal

Frame type	Performance level		
	UHS-500	UHS-1000	UHS-2500
Bare frame (single)	Fully operational	Fully operational	Operational
Bare and grav frame	Fully operational	Fully operational	Operational
Infilled and grav frame	Fully operational	Fully operational	Operational

Table 5: Performance of the twelve storey building located in Montreal

Frame type	Performance level		
	UHS-500	UHS-1000	UHS-2500
Bare frame (single)	Fully operational	Fully operational	Fully operational
Bare and grav frames	Fully operational	Fully operational	Operational
Infilled and grav frames	Fully operational	Fully operational	Fully operational

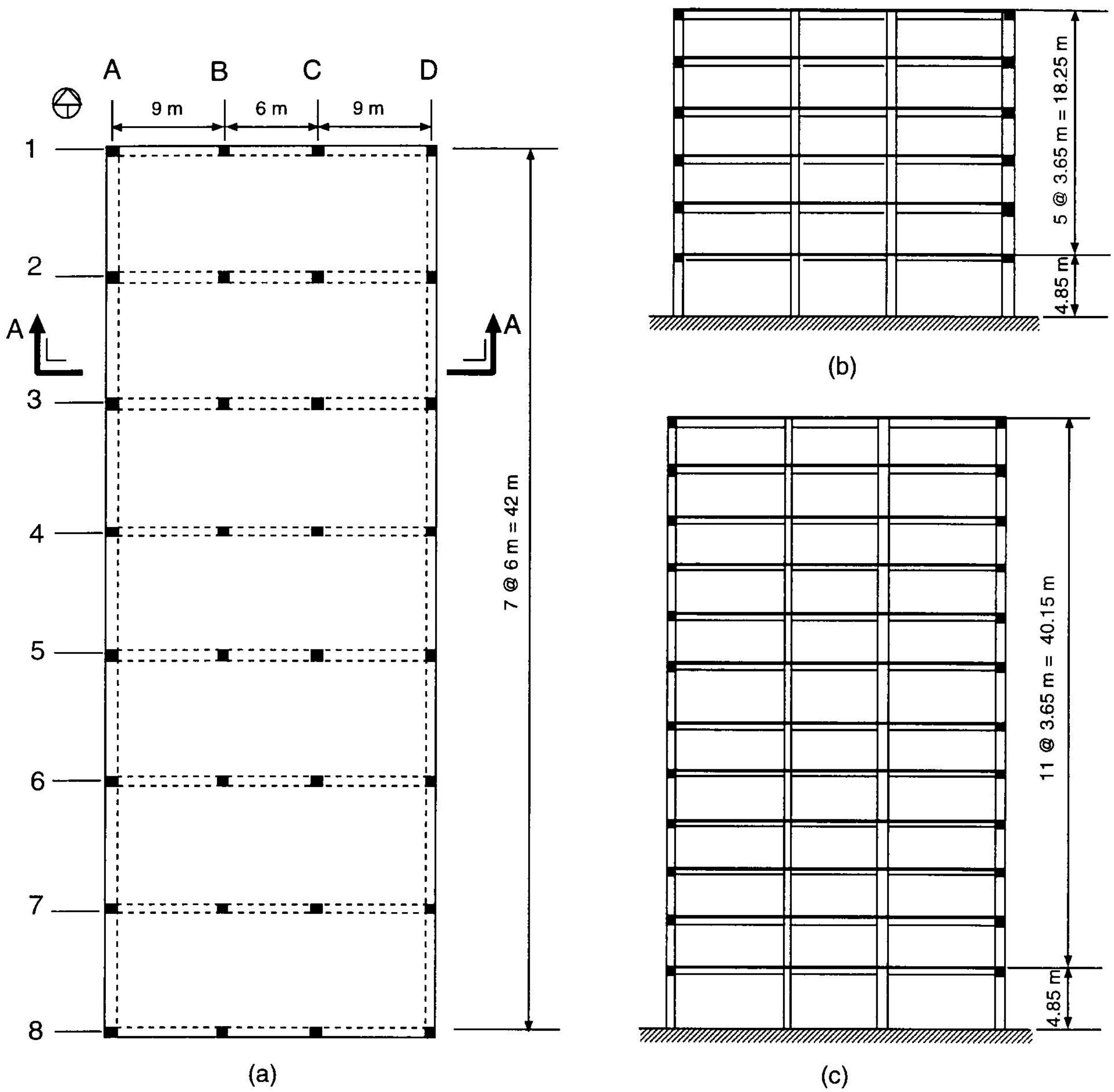

Fig. 1 Plan and elevations of the buildings: (a) plan, (b) elevation of six storey building, (c) elevation of twelve storey building

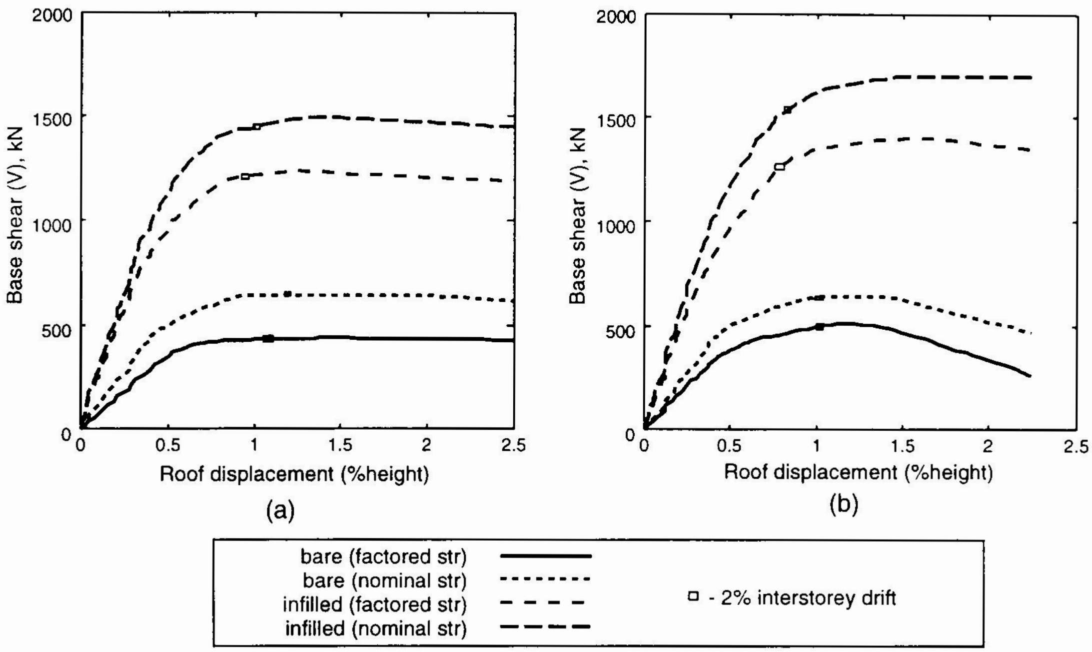

Fig. 2 Push over curves for the building frames in Victoria: (a) six storey, (b) twelve storey

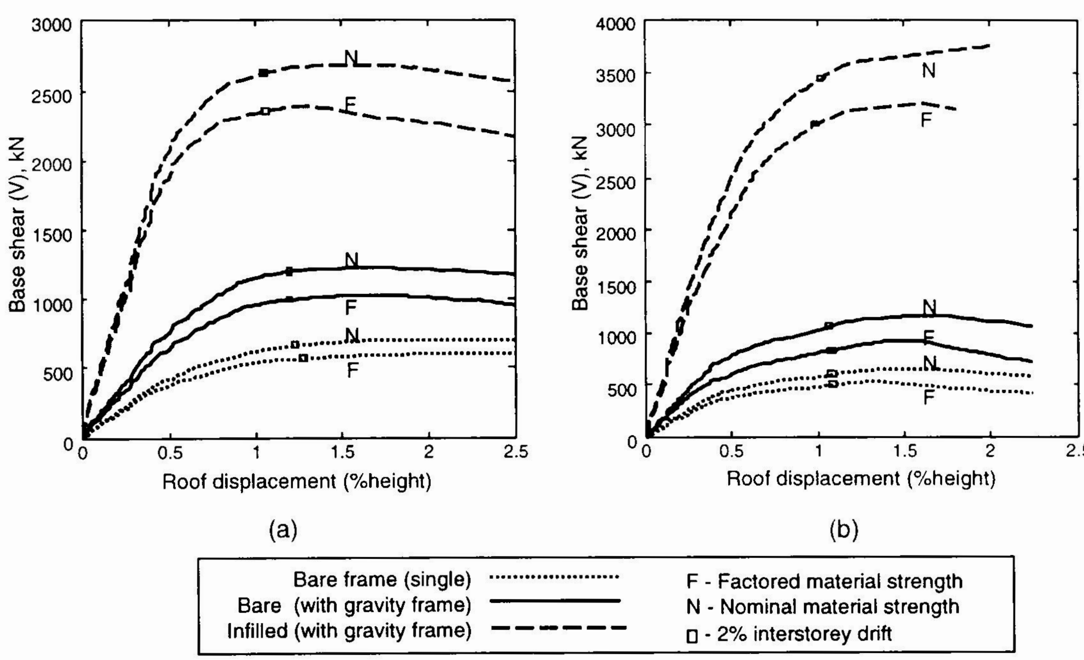

Fig. 3 Push over curves for the building frames in Montreal: (a) six storey, (b) twelve storey