Determination of seismic design forces by equivalent static load method[1]

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Abstract: In the proposed 2005 edition of the National Building Code of Canada (NBCC), the seismic hazard will be represented by uniform hazard spectra corresponding to a 2% probability of being exceeded in 50 years. The seismic design base shear for use in an equivalent static load method of design will be obtained from the uniform hazard spectrum for the site corresponding to the first mode period of the building. Because this procedure ignores the effect of higher modes, the base shear so derived must be suitably adjusted. A procedure for deriving the base shear adjustment factors for different types of structural systems is described and the adjustment factor values proposed for the 2005 NBCC are presented. The adjusted base shear will be distributed across the height of the building in accordance with the provisions in the current version of the code. Since the code-specified distribution is primarily based on the first mode vibration shape, it leads to an overestimation of the overturning moments, which should therefore be suitably adjusted. Adjustment factors that must be applied to the overturning moments at the base and across the height are derived for different structural shapes, and the empirical values for use in the 2005 NBCC are presented.

Key words: uniform hazard spectrum, seismic design base shear, equivalent static load procedure, higher mode effects, base shear adjustment factors, distribution of base shear, overturning moment adjustment factors.

Résumé : Dans l'édition 2005 proposée du Code National du Bâtiment du Canada (CNBC), le risque sismique sera représenté par un spectre de risque uniforme pour lequel la probabilité d'être dépassé en 50 ans est de 2 %. La valeur du cisaillement de base utilisée par la méthode de construction parasismique à charge statique équivalente sera obtenue à partir du spectre de risque uniforme du site correspondant à la période du premier mode du bâtiment. Puisque cette procédure ignore l'effet des modes supérieurs, le cisaillement de base ainsi dérivé doit être ajusté convenablement. Une procédure permettant de dériver les facteurs d'ajustement du cisaillement de base, pour les différents types de systèmes structuraux, est décrite, et les valeurs du facteur d'ajustement proposées pour le CNBC sont présentées. Le cisaillement de base ajusté sera distribué sur toute la hauteur du bâtiment, en accord avec les clauses de la version actuelle du code. Puisque la distribution spécifiée par le code est basée en premier lieu sur la forme du premier mode de vibration, cela mène à une surestimation des moments de renversement, lesquels devraient donc être ajustés convenablement. Les facteurs d'ajustement qui doivent être appliqués aux moments de renversement localisés à la base du bâtiment et sur toute sa hauteur sont dérivés, ce pour différentes formes structurales, et les valeurs empiriques à utiliser dans le CNBC 2005 sont présentées.

Mots clés : spectre de risque uniforme, cisaillement de base d'une construction parasismique, procédure de charge statique équivalente, effets d'un mode supérieur, facteurs d'ajustement du cisaillement de base, distribution du cisaillement de base, facteurs d'ajustement du moment de renversement.

[Traduit par la Rédaction]

Introduction

The response of a structure to earthquake-induced forces is a dynamic phenomenon. Consequently, a realistic assessment of the design forces can be obtained only through a dynamic analysis of the building models. Although this has long been recognized, dynamic analysis is used only infrequently in routine design, because such an analysis is both complicated and time-consuming. A major complication arises from the fact that most structures are designed with the expectation that they would be strained into the inelastic range when subjected to the design earthquake. Although the ability to carry out a nonlinear analysis has seen significant improvement over recent years, considerable uncertainty persists in modelling the nonlinear behaviour of structural materials and components. In addition, nonlinear response to two different ground motions may differ significantly, even when such ground motions produce similar elastic responses. This means that for the purpose of obtaining design forces, nonlinear analysis must be repeated for several different ground motion records that are representative of the seismicity of the site.

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In view of the difficulties associated with nonlinear analysis, linear dynamic analysis is often carried out to determine the design forces. Computer programs that are capable of carrying out a linear dynamic analysis, either a time-history analysis or a mode-superposition analysis, are widely available, and designers are becoming increasingly comfortable in using them. In view of these facts, the proposed 2005 edition of the National Building Code of Canada (NBCC) will recognize dynamic analysis as the preferred procedure for obtaining the design forces and, in fact, make such analysis mandatory for all irregular structures except those of low height or located in zones of low seismicity. Dynamic analysis will also be mandatory for structures that are regular but tall and are located in zones of high seismicity. Saatcioglu and Humar (2003) discuss the code provisions related to dynamic analysis.

The 2005 NBCC will continue to permit the use of an equivalent static load procedure for the following class of structures: (*i*) structures located in zones of low seismicity where $I_eF_aS_a(0.2)$ is less than 0.35, I_e being the importance factor, F_a the acceleration-related foundation factor, and $S_a(0.2)$ the uniform hazard spectral response acceleration in units of g, the gravity constant, corresponding to a period of 0.2 s; (*ii*) regular structures, located in any seismic zone, that are less than 60 m in height and have a fundamental lateral period less than 2 s; and (*iii*) irregular structures, located in any seismic zone, that are less than 20 m height, have a fundamental lateral period less than 0.5 s, and are not torsionally sensitive.

The equivalent static load procedure proposed for the 2005 NBCC is similar to that in the 1995 NBCC (NBCC 1995). In the 2005 NBCC, however, the seismic hazard at a site will be represented by a uniform hazard spectrum (UHS) and not by an idealized response spectrum as in the 1995 NBCC. The equivalent static load procedure should therefore be appropriately revised so as to be used in association with a UHS. This paper provides details of the revisions proposed to the equivalent static load procedure.

The proposed revisions to the NBCC described in this paper have been developed by the authors for the Canadian National Committee of Earthquake Engineering (CANCEE) and reflect the current views of that committee. Revisions to other aspects of the NBCC and discussion of several issues related to the provisions described in this paper appear in a series of related papers in this issue of the Journal.

Design spectral acceleration curve

The seismic design force for an elastic single-degree-of-freedom (SDOF) system having a specified period and damping can be derived from the spectral acceleration that the system is likely to experience. The design value of the spectral acceleration depends on the seismic hazard at the site. In the 1995 NBCC the seismic hazard was expressed in terms of the peak ground acceleration and peak ground velocity at the site with a 10% probability of being exceeded in 50 years. A design response spectrum was constructed by applying appropriate amplification factors to the ground motion bounds. For use in the equivalent static load method of design, the resulting response spectrum was approximated by a set of empirical expressions.

The seismic hazard calculations for the 1995 NBCC were based on historic data of earthquakes up to the year 1985. Since then, additional data have become available on earthquake events, both Canadian and foreign. Also, new ground motion relations that describe how shaking varies with magnitude and distance have been developed. Taking into account these developments and the innovations in hazard calculation methodology, the Geological Survey of Canada has produced new seismic hazard maps for the country. The hazard is now expressed in terms of site-specific spectral acceleration values at selected values of the period and 5% damping. The curve passing through such values is referred to as a uniform hazard spectrum (UHS). The UHS for a site provides the maximum spectral acceleration that an SDOF system located at the site and having 5% damping is likely to experience during the entire range of earthquakes that it may be exposed to. It is proposed that the probability of exceedance to be used in the derivation of the uniform hazard spectra be 2% in 50 years (Adams and Halchuk 2003).

In the Geological Survey of Canada calculations of uniform hazard spectra, spectral acceleration values have been determined for eight different period values ranging from 0.1 to 2.0 s (Adams and Halchuk 2003). For use with the 2005 NBCC, however, the UHS will be defined through the spectral acceleration values at only four periods, namely 0.2, 0.5, 1.0, and 2.0 s, specified in the table of climatic data for most cities throughout Canada. Adams and Atkinson (2003) provide detailed discussions on hazard calculation, probability of exceedance, and development of UHS.

The Geological Survey of Canada uniform hazard spectra are derived for a specific reference ground condition and have to be modified for cases where the ground condition is different from the reference condition. The provisions related to foundation soil effects, described in another paper in this series (Finn and Wightman 2003), specify two different soil amplification factors, an acceleration-related factor F_a and a velocity-related factor F_v. Using these amplification factors, the design spectral acceleration value $S(T)$ is derived from the uniform hazard spectral response acceleration for reference ground condition, $S_a(T)$, as follows:

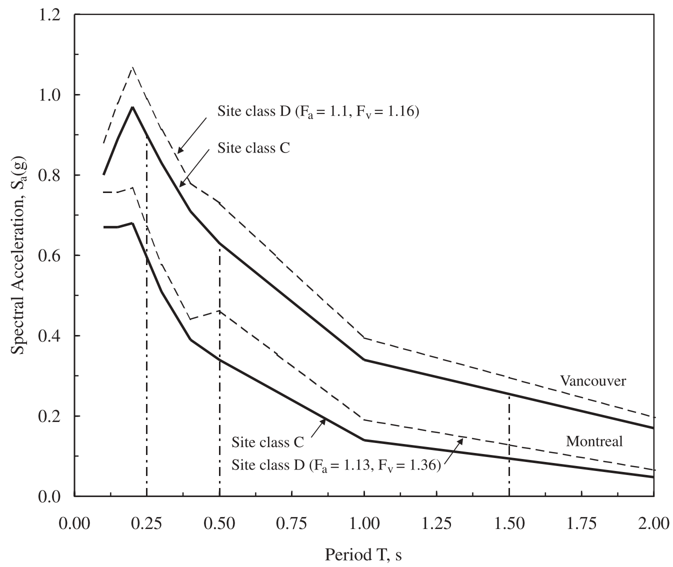
$$
\begin{aligned}
&S(T) = F_a S_a(0.2) && \text{for } T \le 0.2 \text{ s} \\
&S(T) = F_v S_a(0.5) \text{ or } F_a S_a(0.2) \text{, whichever is smaller,} && \text{for } T = 0.5 \text{ s} \\
&S(T) = F_v S_a(2.0) && \text{for } T = 2.0 \text{ s} \\
&S(T) = F_v S_a(2.0)/2 && \text{for } T \ge 4.0 \text{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

where T is the period of the SDOF system. A linear interpolation must be used for intermediate values of T.

It will be noted that for $T \leq 0.2$ s the spectral acceleration curve has a plateau. In reality, for $T < 0.2$ s, the spectral acceleration values generally decrease with the period. It is standard practice, however, to replace the descending branch for $T < 0.2$ s by a plateau, because when a structure having a period $T < 0.2$ s softens during an earthquake it experiences an elongation in its period and may therefore attract increased seismic forces. A plateau is also used for $T > 4.0$ s. This is because of a lack of sufficient seismological data needed to obtain the uniform hazard spectral values for this range. In fact, even the spectral acceleration values for T = 4.0 s have not been obtained by the same rigorous procedure as that used for $T \leq 2.0$ s. The proposed spectral shape for periods between 2.0 and 4.0 s can therefore be considered as only approximate.

For the purpose of comparison, the uniform hazard spectra for Vancouver and Montréal are drawn in Fig. 1 for two different soil types, soil class C, which is the reference soil with $F_a = F_v = 1.0$, and soil class D, which is a softer soil.

Fig. 1. Elastic uniform hazard spectra for Montréal and Vancouver.



Base shear formulation

The elastic base shear V_e produced in an SDOF structure of period T can be obtained from the UHS for the site by using the following equation:

$$[2] \qquad V_e = S(T)W$$

where W is the weight of the structure at the time of the earthquake. In the 2005 NBCC, eq. [2] is modified as follows to obtain the design base shear V in a structure that may have one or more degrees of freedom:

$$[3] \qquad V = \frac{S(T_a)M_v IW}{R_o R_d} \geq \frac{S(2.0)M_v I_e W}{R_o R_d}$$

where T_a is the first mode period of the structure, M_v is the base shear adjustment factor, R_o is the overstrength factor, and R_d is the ductility factor. The proposal also specifies that for structures with $R_d \geq 1.5$, V need not be taken as being greater than two thirds of the value calculated for T_a = 0.2 s. Thus

$$[4] \qquad V \leq \frac{2}{3}\frac{S(0.2)I_e W}{R_d R_o}$$

The rationale for the lower limit specified in eq. [3] is as follows. The Geological Survey of Canada has supplied spectral acceleration values for periods greater than 2.0 s. These values have been derived by an indirect procedure, however, and there is considerable uncertainty associated with them. It is therefore proposed that for the purpose of determining the design base shear the spectral accelerations for periods greater than 2.0 s should be taken as equal to that at T_a = 2.0 s. The rationale for specifying the upper limit in eq. [4] has been described by Heidebrecht (2003).

In eq. [3], I_e is a factor that depends on the importance category of the building. For normal buildings and buildings of low human occupancy, I_e is assigned a value of 1.0. For buildings that are likely to be used as post-disaster shelters, for example schools and community centres, and for manufacturing and storage facilities containing toxic, explosive, or other hazardous substances, I_e is taken as 1.3. For post-disaster buildings, such as hospitals, telephone exchanges, power generating stations, water and sewage treatment facilities, and fire and police stations, I_e is assigned a value of 1.5. A value of I_e greater than 1 ensures that the ductility demand imposed on the structure by the design earthquake is lower, reducing the amount of inelastic deformation, yielding, and cracking, so any damage to the building is less likely to impair the functioning of the facility housed in the building or to release any toxic or hazardous substance contained in the building. Factor I_e is similar to I in the 1995 NBCC.

Factor R_d is used to reduce the design forces in recognition of the fact that a ductile structure designed for such lower forces is able to dissipate the energy input by the earthquake through inelastic deformation without collapsing. The value of R_d is dependent on the ductility capacity of the structure. The factor R_o is also used to reduce the design forces but is meant to take into account the dependable overstrength known to exist in structures designed according to the code provisions. Together, factors R_o and R_d replace the factor R/U in the 1995 NBCC. The ductility factor R_d and the overstrength factor R_o and their recommended values are described in detail in another paper in this series (Mitchell et al. 2003). The factor M_v, which does not exist in the 1995 NBCC, is meant to account for higher mode effects present in a multistorey building. It is discussed in detail in the subsequent sections of this paper.

The application of eq. [3] requires that an estimate of the fundamental period T_a be available. The 2005 NBCC provisions for determining the fundamental period are discussed in another paper in this series (Saatcioglu and Humar 2003).

Use of a uniform hazard spectrum in modal analysis

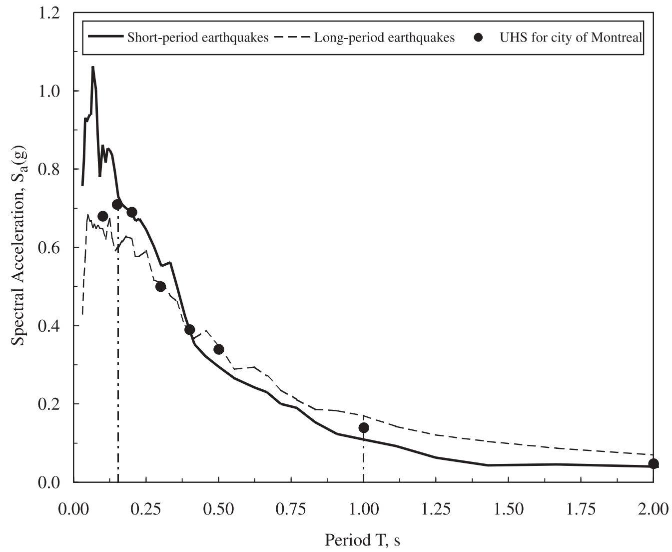
It is important to distinguish between a UHS and a classical response spectrum. A classical response spectrum provides the maximum response of an SDOF elastic oscillator having a range of values of period and specified damping to a single earthquake ground motion. On the other hand, a UHS represents a composite of maximum spectral responses for given damping at different periods. The spectral values at different periods, and even at the same period, may arise from earthquakes having different distances to source and different magnitudes but the same annual probability of exceedance. In general, the short-period UHS values are dominated by earthquakes at close distances, whereas the long-period values are contributed by more distant earthquakes.

It is evident that, as contrasted to a classical response spectrum, a UHS does not correspond to a single earthquake. Thus if a simulated time history is generated to match the UHS, it will correspond to a simultaneous occurrence of a number of potentially damaging events and would therefore be unrealistic. In other words, a number of different time histories must be generated to match the different regions of UHS. Atkinson and Beresnev (1998) have produced physically realistic time histories that not only match the hazard spectrum but also are representative of motions corresponding to the magnitude source distance scenarios for the site. They have also shown that a UHS can be adequately matched with just two types of earthquakes: a lower magnitude, smaller distance earthquake to match the short-period part of the spectrum, and a larger magnitude, greater distance earthquake to match the long-period part of the spectrum.

According to Atkinson and Beresnev (1998), the UHS for 2% probability of exceedance in 50 years can be approximated in the case of eastern Canadian locations by M6.0 events for short-period ranges and M7.0 events for long-period ranges, and in the case of western Canadian locations by M6.5 events for short-period ranges and M7.2 events for long-period ranges. The distances at which these events are placed depend on the seismicity of the site. Figure 2 shows the spectra for the two sets of UHS-compatible records generated by Atkinson and Beresnev for Montréal. One of the two sets is for the short-period range and the other is for the long-period range. In each set there are four different ground motion records. For each set the spectrum plotted in Fig. 2 is an average of the spectra of the four ground motions in the set. For the purpose of comparison, the UHS values are also shown in Fig. 2.

If modal superposition is used to obtain the response of a multistorey building located in Montréal, for example, at least two sets of analyses would be required, one with each of the two simulated spectra shown in Fig. 2. The larger of the response values obtained from the two analyses would provide the design forces. For illustration, consider a shear wall structure located in Montréal. The first two modal periods of the structure are 1.00 and 0.159 s, respectively, as indicated in Fig. 2. Considering only the first two modes, a modal superposition analysis in which the resultant response is obtained by taking the square root of the sum of the squares (SRSS) of modal responses gives a base shear equal to $0.630W$ when the long-period spectrum shown by a broken line is used and $0.730W$ when the short-period spectrum shown by solid line is used. The design shear can thus be taken as $0.730W$. Now if the envelope of the two spectra,

Fig. 2. Short- and long-period acceleration response spectra for Montréal. UHS, uniform hazard spectra.



which is representative of UHS, is used in the mode superposition analysis, the base shear works out to $0.742W$. In the present example, use of UHS in a mode superposition analysis overestimates the base shear by 1.6%.

Because a UHS can be considered as the envelope of maximum spectral acceleration values produced by different earthquakes, it is apparent that the use of UHS in a mode superposition analysis would lead to conservative results. For the spectra shown in Fig. 2 this is so whenever the period is longer than that at the crossover point, that is, longer than about 0.40 s. The degree of conservatism is not large, however. A number of calculations, similar to those illustrated earlier, for different regions of the country have shown that the overestimate is no more than 10% and in most cases is significantly smaller. This degree of conservatism is for most practical purposes quite insignificant. It is therefore acceptable to use a UHS in carrying out a mode superposition analysis, and this assumption has been made in arriving at the results presented in the remaining parts of this paper.

The discussion in this section is limited to the use of UHS for a planar response spectral analysis. The application of such an analysis to asymmetric buildings where modal coupling may occur between torsion and translation is discussed in a companion paper in this issue (Humar et al. 2003). It should also be noted that the 2005 NBCC will not permit the use of the equivalent static load method for determining the design forces in a torsionally sensitive building, in which strong modal coupling may exist (Humar et al. 2003; DeVall 2003)

Effect of higher modes on base shear

In a multistorey building all vibration modes of the building contribute to the base shear. For an elastic structure, the relative contribution of higher modes depends on the spectral shape and on two dynamic characteristics of the system: (*i*) the relative values of the modal periods, and (*ii*) the modal weights for different modes. The two sets of characteristics, in turn, depend on the structural type. For an illustration of these observations consider two widely different

Table 1. Relative modal periods and modal weights for flexural and shear cantilevers.

	Flexural cantilever		Shear cantilever	
Mode	Period	Modal weight	Period	Modal weight
1	1.000	0.616	1.000	0.811
2	0.167	0.188	0.333	0.09
3	0.057	0.065	0.200	0.03
4	0.030	0.032	0.143	0.02
5	0.018	0.020	0.111	0.01

structural types, a shear cantilever representing a shear frame and a flexural cantilever representing a flexural wall. In terms of the dynamic characteristics noted earlier, most structural types will fall between the two extremes. The relative modal periods and modal weights for the first five modes of the two structural types are shown in Table 1. Now consider a building structure with first mode period of 1.50 s, and for simplicity assume that contributions from only the first two modes are significant. The first and second mode periods for structures of the two different types, and the corresponding spectral acceleration values are indicated in Fig. 1. The second mode period of the flexural cantilever is 0.25 s, and that of the shear cantilever is 0.50 s. Using the spectral accelerations obtained from UHS, the design shears for buildings of two different structural types and two different locations, Vancouver and Montréal, have been calculated and are shown in Table 2. For the shear cantilever building in Vancouver, the first mode contributes a major portion of the base shear; the second mode contribution is only 0.28 times the first mode contribution. The base shear estimate obtained by taking SRSS of the modal contributions is smaller than that if the entire response is assumed to be in the first mode, and an adjustment factor, $M_v = 0.84$, must be applied to the latter to obtain a more precise value. For a similar building located in Montréal, the first mode contribution would still be higher than that in the second mode, but compared with the building in Vancouver the second mode makes a more significant contribution, as much as 0.41 times that of the first mode. This difference is related to the spectral shape. Compared with Vancouver, the spectral acceleration for Montréal drops more rapidly with an increase in period, so the spectral acceleration in the second mode is proportionally larger for Montréal. The adjustment factor M_v to be applied to the estimate based on the first mode alone now works out to 0.90.

For a flexural wall building located in Vancouver, the second mode contribution is larger than the first mode contribution. This is because in a wall structure the separation between the modal periods is comparatively large, the second mode period being only 0.167 times the first mode period, compared with 0.333 times the first mode period in a shear frame structure. This, combined with the shape of the design spectrum, whereby the spectral acceleration drops fairly rapidly with an increase in the period, leads to a significantly higher spectral acceleration for the second and higher modes compared with that for the first mode. In addition, when compared with a shear frame, the modal weight for the second mode of a flexural wall constitutes a larger proportion of the total weight. A higher spectral acceleration and a higher modal weight together lead to a larger contribution from the second mode. The adjustment factor M_v to be applied to the response based on first mode alone is still less than 1.

When the flexural wall building is located in Montréal, the second mode contribution is substantially higher than the first mode contribution. This is because of (*i*) the rapidly dropping spectral shape for Montréal, (*ii*) the larger separation between modal periods, and (*iii*) the larger modal weight for the second period. The combined effect of all these factors is such that the adjustment factor M_v works out to 1.35. The foregoing discussion indicates quite clearly that the base shear obtained by using the spectral acceleration value in the UHS corresponding to the first mode period needs to be modified by a factor M_v to get a more precise estimate of the elastic response.

Methodology for estimating the shear adjustment factors

Humar and Rahgozar (2000) have studied the variation of M_v factors for frame and wall models for two locations, Vancouver and Montréal. In their study they used the UHS for these two cities and carried out response spectrum analyses for determining M_v factors. In the present paper this study has been extended in the following manner. First, several different structural configurations, other than a frame and a wall, are considered. These are braced frames, coupled wall systems, and hybrid systems comprising moment frame and a wall. Second, studies have been carried out for 22 different cities, 10 located in the western region of Canada and 12 in the eastern region. A list of the cities included in this study and complete spectral acceleration data for such cities are provided by Adam et al. (1999).

In the present study the structure is assumed to remain elastic. Based on this assumption, the adjustment factor to be applied to the base shear obtained by assuming that the entire response is in the first mode is given by

$$[5] \qquad M_v = \frac{\sqrt{\sum [S_a(T_i)W_i]^2}}{S_a(T_1)W}$$

where $S_a(T_i)$ is the spectral acceleration corresponding to the ith mode having a period T_i, W_i is the modal weight in the ith mode, and W is the total weight of the structure. In eq. [5] the base shear obtained by taking the square root of the sum of squares of modal base shears is assumed to be a reasonably close estimate of the true value. It is evident from eq. [5] that for a given spectral shape the M_v factor depends on only the modal periods and modal weights.

Description of structural models

A simplified symmetric multistorey building model is selected for the present study. The building floors are assumed to be infinitely rigid in their own planes. The torsional effects are neglected so that the response of the building can be studied by analysing a single planar frame. For the purpose of this study the building model is assumed to consist of two identical parallel single-bay frames. The entire mass of the structure is assumed to be uniformly distributed at the

Table 2. Design shears in a building of two different structural types located in Vancouver and Montréal.

	Period (s)		Modal weight		Spectral acceleration (g)		Base shear				
Structure type	First mode	Second mode	First mode	Second mode	First mode	Second mode	First mode	Second mode	SRSS shear	Base shear assuming entire response in first mode	M_v
Vancouver											
Shear cantilever	1.5	0.5	0.811W	0.090W	0.260	0.660	0.211W	0.059W	0.219W	0.260W	0.842
Flexural cantilever	1.5	0.25	0.616W	0.188W	0.260	0.900	0.160W	0.169W	0.233W	0.260W	0.896
Montréal											
Shear cantilever	1.5	0.5	0.811W	0.090W	0.094	0.340	0.076W	0.031W	0.082W	0.094W	0.873
Flexural cantilever	1.5	0.25	0.616W	0.188W	0.094	0.600	0.058W	0.113W	0.127W	0.094W	1.351

floor levels. The storey height and floor mass are assumed to be uniform across the height of the building.

The lateral force resisting planes in the models being studied are selected to be regular and simple. The resisting planes may comprise columns and beams, bracings, shear walls, or a combination of these elements. The following five different types of lateral force resisting systems are considered: (*i*) moment-resisting frame, (*ii*) concentrically braced frame, (*iii*) flexural wall, (*iv*) coupled flexural wall, and (*v*) hybrid frame–wall system. These systems cover most practical structural systems found in buildings.

It should be noted that in the study of an elastic system such as those considered in this study the important consideration is to produce the relative modal periods and modal weights that are representative of the structural type. The shape of the individual element, the absolute values of the element stiffness, and the number of storeys are irrelevant. The models are designed to have eight or more storeys so that contributions from at least the first eight modes are considered.

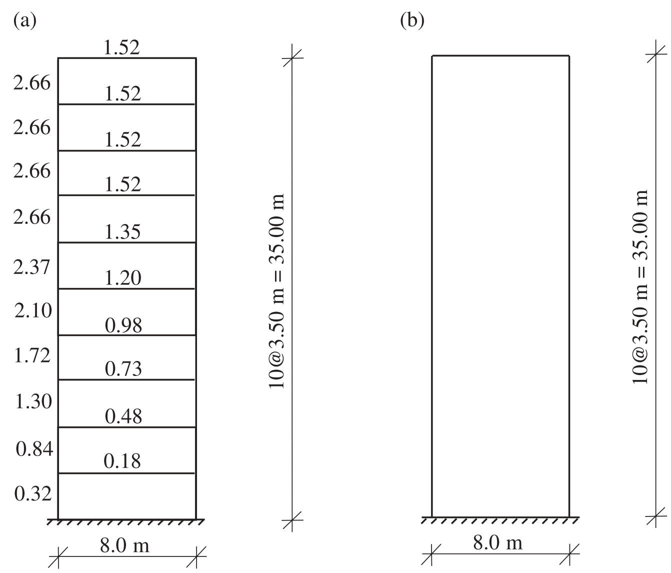
Moment-resisting frame

In this case each of the lateral force resisting planes is a single-bay 10-storey frame comprised of rigidly connected beams and columns. The frame width is taken to be 8.0 m. The ratio of the beam stiffness to the sum of the column stiffnesses in each storey is taken as 1:4. This is representative of a strong column – weak beam system, which is the system preferred by most seismic codes. The relative values of the column and beam stiffnesses across the height of the structure are adjusted such that under a set of storey forces distributed in an inverted triangular shape the interstorey displacements are approximately the same. The displaced shape under the selected forces is thus linear. The fundamental period of the frame is now matched to a specific value by selecting an appropriate value for the modulus of elasticity. In other words, the same frame configuration is used for the entire range of periods studied, but with different values of the modulus of elasticity. The mass tributary to each level of the frame is taken as 55.2 Mg. An elevation of the frame indicating the values of relative element moments of inertia is shown in Fig. 3*a*.

Flexural wall

A 10-storey flexural wall system is studied. The wall has a uniform width of 8.0 m and a uniform thickness of 0.4 m across the height, as shown in Fig. 3*b*. The mass tributary to each storey level is taken as 55.2 Mg. Again, the first mode period of the structure is matched to a specified value by adjusting the modulus of elasticity.

Fig. 3. Elevation of selected structural models: (*a*) moment-resisting frame showing relative values of the moments of inertia; (*b*) flexural wall.



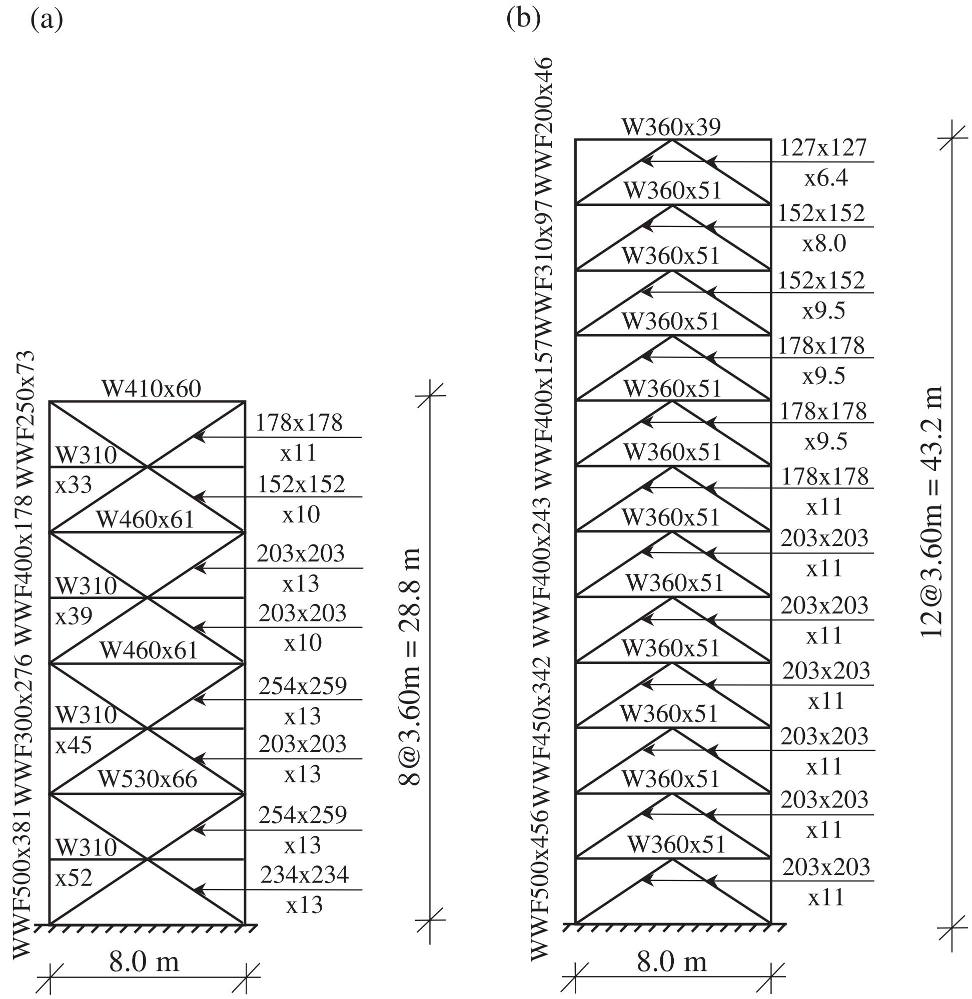
Concentrically braced frame

Two different single-bay braced frame structures are used for the study. Each frame is modelled as a simple vertical truss. The first braced frame, shown in Fig. 4*a*, consists of two-storey cross bracing. It is eight storeys high and has a width of 8.0 m and a storey height of 3.6 m. The section properties of the various elements of the frame are shown in Fig. 4. The mass tributary to each level of the frame is taken as 55.2 Mg. The second braced frame, shown in Fig. 4*b*, employs chevron bracing. It is 12 storeys high and has a bay width of 8.0 m and a storey height of 3.6 m. The cross-section properties of the various members are shown in Fig. 4. The storey tributary mass is taken as 55.2 Mg. For each of the two frames the desired value of the first mode period is achieved by adjusting the value of the modulus of elasticity.

Coupled flexural walls

This structural system consists of two 12-storey flexural walls connected to each other with beams. The connections

Fig. 4. Elevation of selected structural models: (*a*) braced frame with cross bracing; (*b*) braced frame with chevron bracing.



between the beams and walls are considered to be rigid. Each wall is 0.4 m thick and 8.0 m wide. The width of the beams is taken as 0.4 m. Analyses are carried out with six different beam depths: 200, 400, 600, 800, 1000, and 1200 mm. The geometry of the structure and the cross-section properties of individual elements are shown in Fig. 5*a*. The storey tributary mass is taken as 91.5 Mg, and the modulus of elasticity is adjusted to match the specified value of the fundamental period.

Hybrid frame–wall system

This system consists of a 12-storey moment-resisting frame connected by axially rigid links to a flexural wall. The geometry and cross-sectional dimensions are shown in Fig. 5*b*. The wall width is taken as a variable, and analysis is repeated with six different wall widths: 3.0, 4.0, 5.0, 6.0, 7.0, and 8.0 m. The tributary mass of each storey is 91.5 Mg. The modulus of elasticity is adjusted to match the specified value of the fundamental period.

Details of analysis for determining M_v factors

As mentioned in the preceding section, five different types of multistorey building structures were selected for study. The buildings are assumed to be located in 22 cities in Canada, 10 in the western region and 12 in the eastern region. The moment-resisting frames, concentrically braced frames, and flexural wall systems are analysed for seven different values of the fundamental period and for 22 locations cited earlier. The coupled flexural wall and the hybrid systems are analysed for two locations, Montréal in the east and Vancouver in the west, and seven fundamental periods. Six different beam depths are used for the coupled flexural wall model, and six different wall widths are used for the hybrid system. In all cases the seven fundamental periods selected are T_a = 0.5, 0.7, 1.0, 1.5, 2.0, 2.5, and 3.0 s.

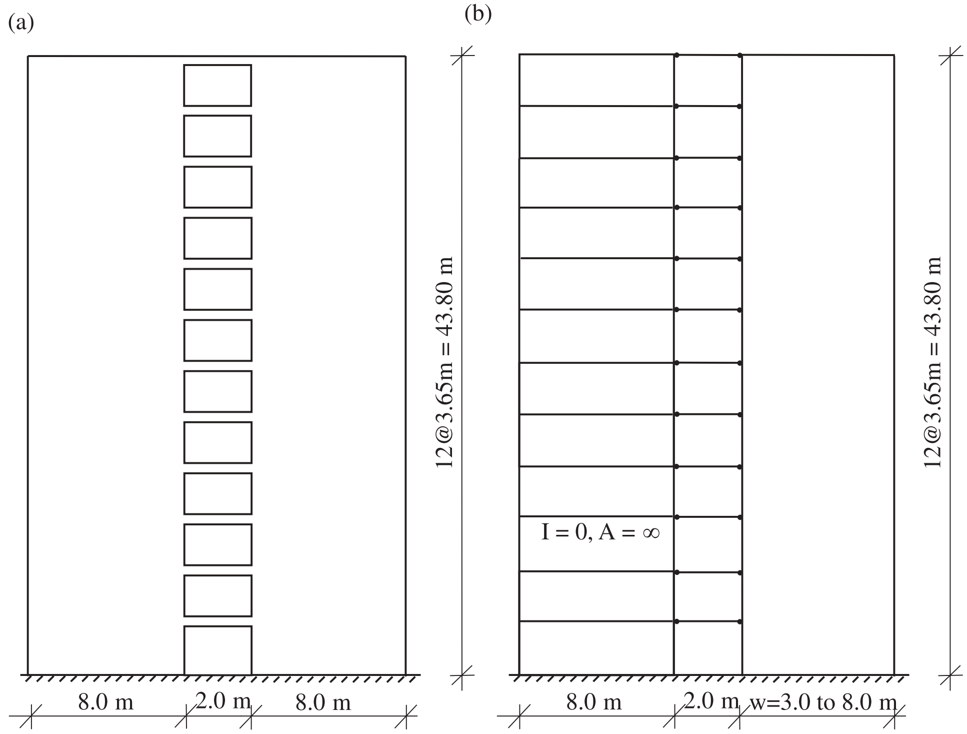
For any given structural system, selected location, and fundamental period, the elastic code base shear V_{bc} is obtained by using eq. [2], so

$$[6] \qquad V_{bc} = S_a(T_a)W$$

A response spectrum analysis is carried out using the selected UHS and contribution from all of the modes to obtain a more precise estimate of the base shear, V_{be}. The base shear adjustment factor is calculated from

$$[7] \qquad M_v = \frac{V_{be}}{V_{bc}}$$

Fig. 5. Elevation of selected structural models: (*a*) coupled flexural walls; (*b*) hybrid frame–wall system.



Modal periods and modal weights

Free vibration analyses of the structural models yield the modal periods and the modal weights required in the determination of M_v factors. The relative modal periods are presented in Table 3 for the five selected systems and the first 10 modes. The modal weights are shown in Table 4. Several observations can be made from the data presented. Compared with the moment-resisting frame, the flexural wall exhibits a larger spread between the first mode period and the higher mode periods. Also, the higher mode weights are relatively large in flexural walls compared with those in moment-resisting frames. Thus, in a flexural wall higher modes account for 35.50% of the total weight, whereas in a moment-resisting frame they account for only 25.90% of the total weight. A braced system lies between a moment-resisting frame and a flexural wall system. The coupled wall system shown in Tables 3 and 4 behaves like a moment-resisting frame. The hybrid system shown is similar in its behaviour to that of a flexural wall system.

Table 3. Modal periods for the five selected structural systems (T_1 = 0.5 s).

	Modal periods (s)				
Mode	MRF	Braced frame 2	Wall	Coupled walls (b = 0.6 m)	Hybrid system (w = 8.0 m)
1	0.500	0.500	0.500	0.500	0.5
2	0.194	0.161	0.079	0.146	0.087
3	0.109	0.086	0.028	0.071	0.031
4	0.070	0.059	0.014	0.041	0.016
5	0.048	0.046	0.009	0.026	0.01
6	0.035	0.038	0.006	0.018	0.006
7	0.026	0.033	0.004	0.013	0.005
8	0.020	0.029	0.003	0.010	0.004
9	0.016	0.026	0.003	0.008	0.003
10	0.013	0.024	0.002	0.006	0.002

Analytical results for M_v factors

Analytical results for base shear adjustment factors for a moment-resisting frame, a concentrically braced frame with cross braces, and a flexural wall corresponding to fundamental periods T_a = 0.5, 0.7, 1.0, 1.5, 2.0, 2.5, and 3.0 s are presented in the form of graphs in Figs. 6–8 for the 22 cities. The curves related to Montréal and Vancouver are shown in bold lines. The following observations are made on the basis of results presented.

The base shear adjustment factor M_v is period dependent and increases with an increase in period. The rate of increase of M_v is higher for the eastern region than for the western region. In the west, for example, the M_v factor varies between 0.687 and 1.197, whereas in the east it ranges from 0.748 to 3.701. The factor M_v is also strongly dependent on the type of ground motion. For the same fundamental period, structures subjected to records in the east usually have a larger M_v factor than their counterparts in the west. This can be attributed to the difference in the spectral shapes for the two regions. Thus, in the east the spectrum drops more rapidly with an increase in period compared with that in the west. A consequence of this is that the higher mode contribution is more predominant in the east than in the west.

Proposed empirical expressions for shear adjustment factor

Seismic codes that permit the use of an equivalent static load method of design, in which the spectral acceleration

Table 4. Modal weights in percent of total weight for the five selected structural systems.

Mode	Modal weight (as percent of total weight) MRF	Braced frame 2	Wall	Coupled walls (b = 0.6 m)	Hybrid system (w = 8.0 m)
1	74.10	67.70	64.50	75.30	64.6
2	11.80	21.20	19.80	11.30	19
3	5.10	5.60	6.80	5.10	6.7
4	3.00	2.30	3.50	2.90	3.4
5	2.00	1.20	2.10	1.80	2.1
6	1.40	0.80	1.40	1.20	1.4
7	1.00	0.40	0.90	0.90	1
8	0.80	0.40	0.60	0.60	0.7
9	0.60	0.20	0.40	0.40	0.5
10	0.20	0.10	0	0.3	0.4
Total	100	99.90	100	99.80	99.8

values are determined from a design spectrum representing the response of an SDOF system, usually include a procedure to account for the effect of higher modes. In the 1995 NBCC this is accomplished by somewhat arbitrarily raising the spectrum in the long-period range, where the higher mode effects may lead to an increase in the design base shear. The National Earthquake Hazard Reduction Program guidelines (NEHRP 1997), on which the U.S. codes are based, use a similar indirect procedure. As stated earlier, the higher mode effects depend on a number of factors, including the fundamental period, the type of structural system, and the shape of the response spectrum. Since a simple adjustment in the shape of the design spectrum cannot account for all of the factors, the use of design forces obtained from the adjusted spectrum will not ensure a uniform level of protection. The Canadian National Committee on Earthquake Engineering (CANCEE) decided to move toward added rationality in accounting for higher mode effects, without introducing undue complexity in the design process. The committee took the view that the UHS should be specified in its original form as obtained from hazard analysis. In addition to maintaining transparency in the design process, this would permit the use of UHS in a response spectrum analysis and in producing spectrum-compatible ground motions for a time history analysis. It follows that specific provisions must be developed to account for the effect of higher modes. At the same time, an opportunity exists to relate the higher mode effects to the fundamental period, structural type, and spectral shape.

As stated earlier, the shape of UHS is different for each site. Consequently, the higher mode effects will also be different for each site. Clearly, it would be impractical to take this into account in a simplified method of design. Fortunately, in Canada the spectral shapes can be grouped into just two distinct categories, one for the eastern region and one for the western region. The characteristics of the UHS for sites in one region are more or less similar. Higher mode effect factors have therefore been developed for the two regions and for the structural types identified in earlier sections.

Based on the analytical results obtained for the 22 cities, empirical expressions are derived for M_v for use in design. The empirical values are shown in Table 5 and in Figs. 6–8 for three structural types: moment-resisting frame, braced frame, and wall. The values vary with period and differ for the western and eastern regions. In Table 5, the two regions are identified by the spectral ratio $S_a(0.2)/S_a(2.0)$. This ratio is less than 8.0 for the western region and greater than 8.0 for the eastern region where the spectrum drops more rapidly with an increase in the period. As a conservative estimate M_v is selected to be no less than 1. In other words, a multi-degrees-of-freedom (MDOF) system is designed for at least the same shear as a SDOF system having a period equal to the fundamental period of the former. The factor M_v is assumed to be constant for $T_a \geq 2.0$ s. As stated earlier, considerable uncertainty is associated with the spectral acceleration values for periods greater than 2.0 s. It has therefore been proposed that for the purpose of design the spectral accelerations for periods greater than 2.0 s should be taken as equal to that at $T_a = 2.0$ s. With this assumption, M_v can be taken as being constant for $T_a > 2.0$ s. The analytical values of M_v for $T_a > 2.0$ s do not influence the design expressions. They have been included here for the sake of comparison with the design values. It may be noted that even when a dynamic analysis is used in design, the 2005 NBCC requires that the dynamic shears, moments, and forces be tied back to the base shear obtained from the equivalent static load method of analysis (Saatcioglu and Humar 2003).

In some cases the calculated value of M_v is less than 1 for the entire period range. In such cases M_v is taken as 1 for all periods. When M_v increases with an increase in period to a value greater than 1 at 2 s, a value of $M_v = 1$ applies in the short-period range. In most cases M_v is less than 1 for periods up to 1 s, which is therefore taken as the boundary of the short-period range. For periods between 1 and 2 s a straight-line interpolation is used. In general, the design value corresponding to a period of 2 s is close to the mean for that period and the straight line joining the design values at 1 and 2 s is representative of the data. A single straight line, rather than a series of straight lines joining the mean values at selected periods, is used to maintain simplicity. In a few cases where the calculated values for a large urban centre, such as Montréal, lie significantly above the mean, the straight line representing M_v is selected to be close to the values for the urban centre under reference. The foregoing discussions and Table 5 form the basis for the 2005 NBCC provisions related to base shear adjustment factor.

Distribution of shear

According to the 1995 NBCC, the base shear is distributed across the height of the structure considering a vibration shape that is representative of the first mode of the structure. The force at floor level i is given by

$$F_i = \frac{w_i h_i}{\sum w_i h_i} V \qquad [8]$$

where w_i is the weight assigned to the ith storey, and h_i is the height of the ith storey above the base.

For uniform floor masses and uniform storey heights, the distribution shape given by eq. [8] is an inverted triangle. This linear shape provides a reasonably good approximation

Fig. 6. Variation of M_v factor with period for moment-resisting frame structures: (*a*) cities in the west of Canada; (*b*) cities in the east of Canada.

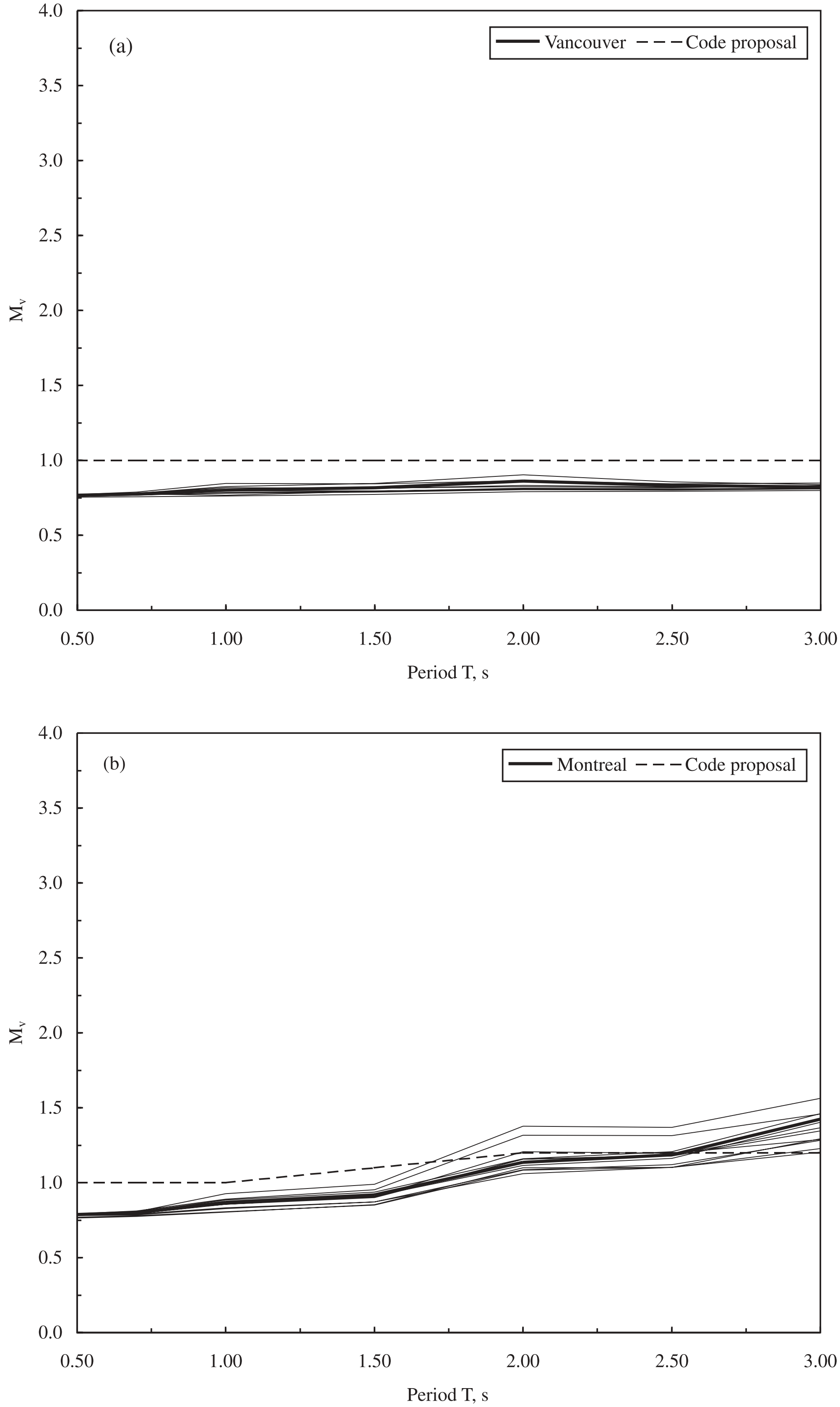


Fig. 7. Variation of M_v factor with period for braced frame structures: (*a*) cities in the west of Canada; (*b*) cities in the east of Canada.

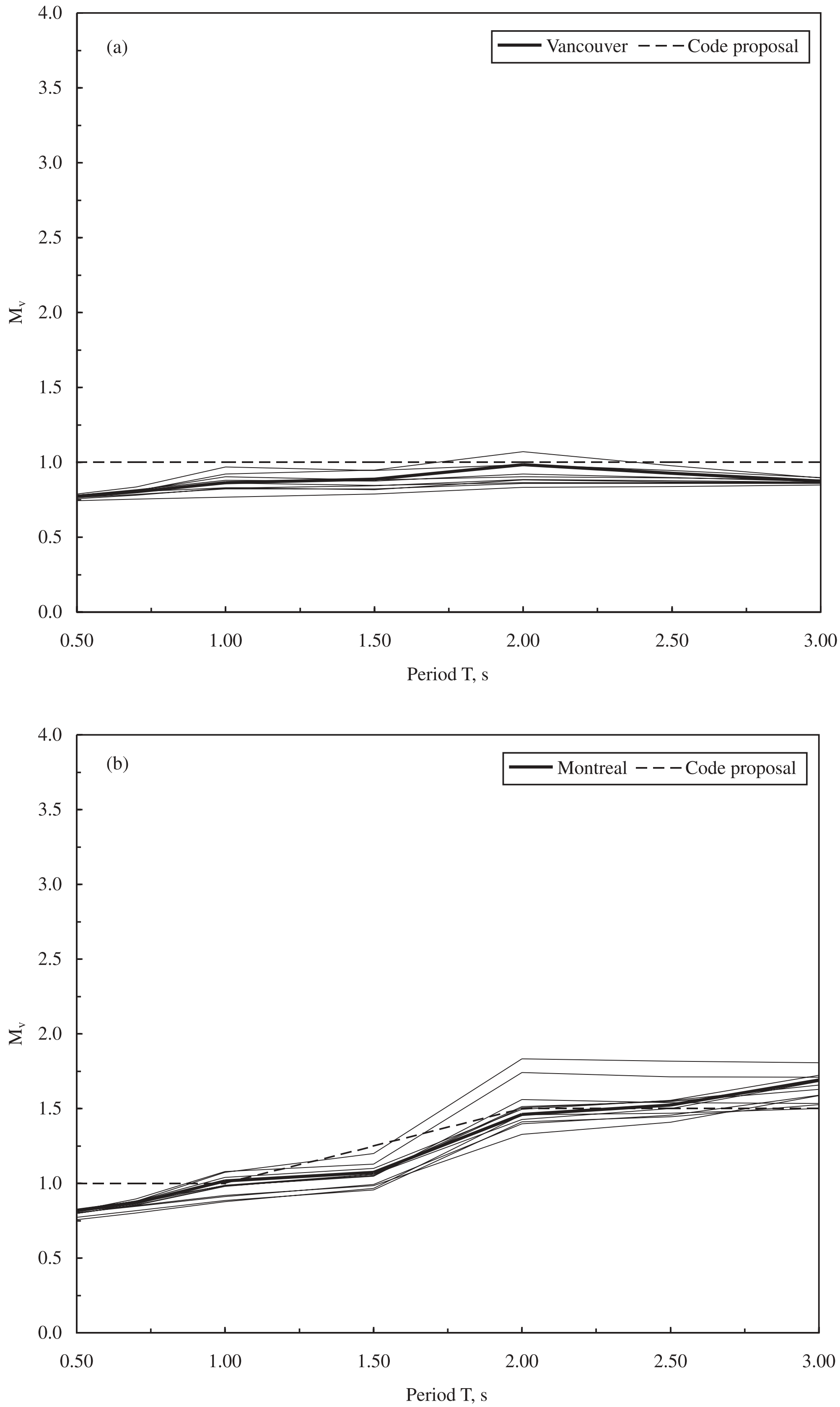


Fig. 8. Variation of M_v factor with period for wall structures: (*a*) cities in the west of Canada; (*b*) cities in the east of Canada.

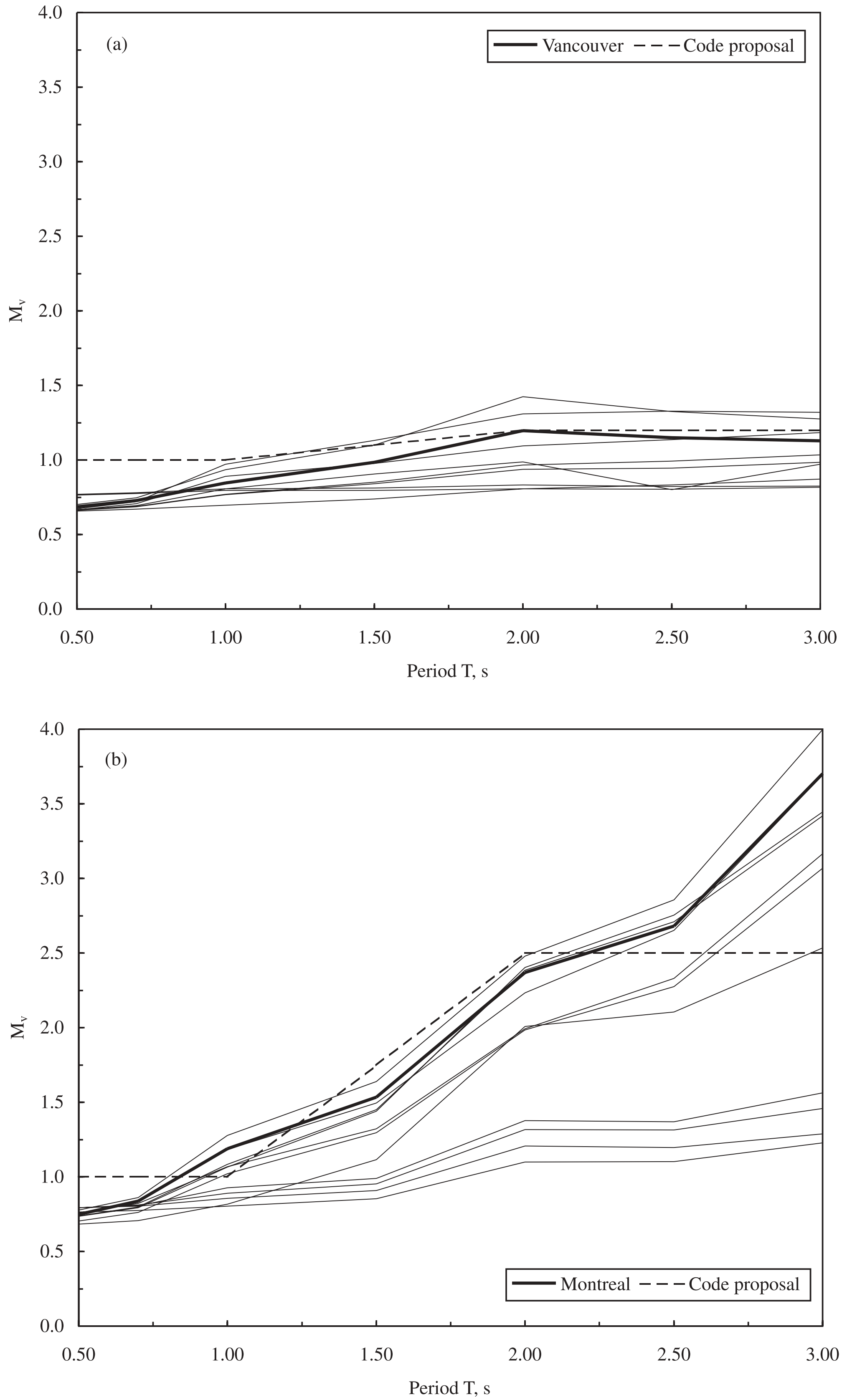


Table 5. Proposed base shear and overturning moment adjustment factors M_v and J for different structural systems.

		M_v		J	
$S_a(0.2)/S_a(2.0)$	Type of lateral force resisting system	$T \leq 1.0$	$T \leq 2.0$	$T \leq 0.5$	$T \leq 2.0$
< 8.0	Moment-resisting frames or "coupled walls"[a]	1	1	1	1.0
	Braced frames	1	1	1.0	0.8
	Walls, wall–frame systems, other systems[b]	1	1.2	1	0.7
> 8.0	Moment-resisting frames or "coupled walls"[a]	1	1.2	1	0.7
	Braced frames	1.0	1.5	1.0	0.5
	Walls, wall–frame systems, other systems[b]	1.0	2.5	1.0	0.4

Note: Values of M_vS between periods of 1.0 and 2.0 s and values of J between periods of 0.5 and 2.0 s are to be obtained by linear interpolation.

[a]Coupled wall is a wall system with coupling beams where at least 66% of the base overturning moment resisted by the wall system is carried by axial tension and compression forces resulting from shear in the coupling beams.

[b]For hybrid systems, use values corresponding to walls or carry out a dynamic analysis.

of the first mode. The code recognizes that the first mode distribution fails to account for the effect of higher modes, which tend to increase the shear in the upper storeys. Also, the higher mode effect becomes more significant as the fundamental period increases. These factors are taken into account in the code by specifying that a portion of the base shear, F_t, be assigned to the top floor level and the remaining shear distributed according to eq. [8], with V replaced by $V - F_t$. The top force F_t is given by

$$[9]\quad \begin{aligned} F_t &= 0 && T_1 \leq 0.7 \\ F_t &= 0.07T_1V && 0.7 < T_1 < 3.6 \\ F_t &= 0.25V && T_1 \geq 3.6 \end{aligned}$$

As pointed out earlier, the contribution of higher modes depends on both the characteristics of the structure and the shape of the response spectrum. Higher mode effects are more predominant in a flexural wall structure than in a shear frame structure. Similarly, the response of a structure located in the eastern region of Canada is affected more by contributions from higher modes when compared to a structure located in the western region. This is because of the different shapes of the spectra in the two regions. Obviously a single formula for distribution of base shear across the height of a structure cannot capture the variation caused by different structural characteristics and different structural shapes. In general, for a given base shear, the NBCC distribution assigns smaller shears to the upper stories and larger shears to the lower stories than would be provided by a modal analysis. These differences are larger for long periods, eastern regions of Canada, and shear wall type structures. Despite these discrepancies, the 1995 NBCC distribution is judged to be adequate for design purposes (Humar and Rahgozar 2000) and remains unchanged in the 2005 edition.

The manner in which the base shear is distributed across the height of a structure affects the estimated overturning moment produced at various storey levels. For a given base shear the largest overturning moments are produced when the shear is distributed according to the first mode; the moments become proportionately smaller when higher mode contributions are accounted for in the distribution of base shear. Since the NBCC distribution of shear is based predominately on the first mode, the overturning moments calculated from the resulting storey-level forces overestimate the true moments. The 1995 NBCC specifies correction factors to be applied to the calculated moments so as to obtain more realistic estimates. A factor J is applied to the base overturning moment and a factor J_x is applied to the overturning moment at level x. The 1995 NBCC overturning moment reduction factors need to be revised in order for them to be used in association with seismic base shear calculated from a UHS.

Methodology for estimating *J* factors

The methodology used for calculating the J factors is similar to that used for calculating the M_v factors. For a given building model, the base shear obtained by modal analysis, that is $M_vV_{bc} = V_{be} = \sqrt{\Sigma[S_a(T_i)W_i]^2}$, is distributed according to eqs. [8] and [9] to yield storey-level forces. These forces are used to calculate the storey-level overturning moments M_{xc} and base overturning moment M_{bc}. More precise estimates of these parameters obtained from response spectrum analysis are designated as M_{xe} and M_{be}, respectively. The storey-level overturning moment adjustment factor J_x and the base overturning moment adjustment factor J are now given by

$$[10a]\quad J_x = \frac{M_{xe}}{M_{xc}}$$

$$[10b]\quad J = \frac{M_{be}}{M_{bc}}$$

Analytical results for *J* factors

Analytical results for base overturning moment reduction factors for different structural models are presented in the form of graphs in Figs. 9–11 for 22 locations. The following observations are made. The adjustment factor J decreases with an increase in period. The rate of decrease is higher for the eastern region than for the western region. Thus, factor J varies from 1.030 to 0.601 in the west and from 0.950 to 0.304 in the east. Factor J is also strongly dependent on the type of ground motion. For the same value of T_a, structures in the east usually have a lower J factor than their counter-

Fig. 9. Variation of *J* factor with period for moment-resisting frame structures: (*a*) cities in the west of Canada; (*b*) cities in the east of Canada.

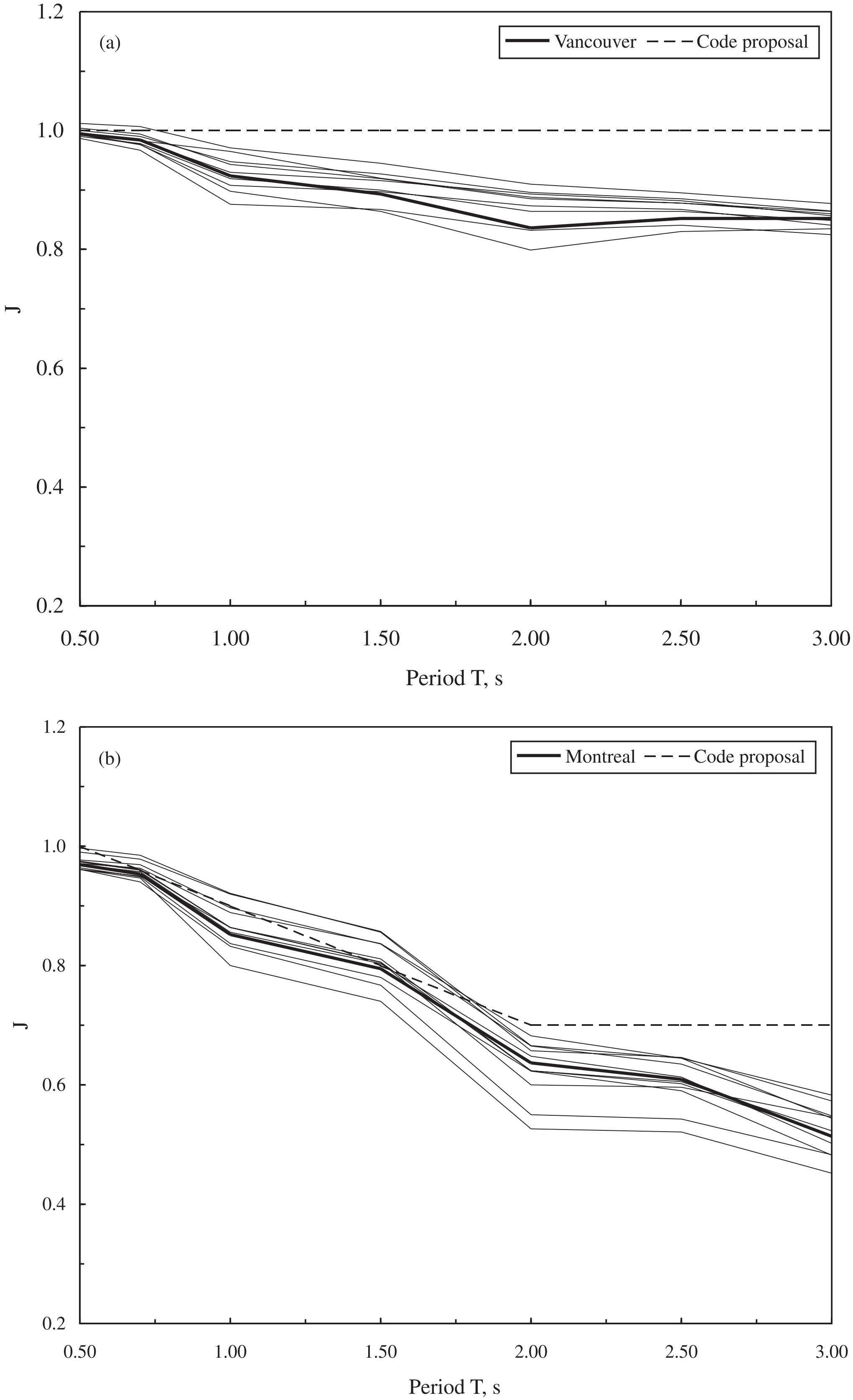


Fig. 10. Variation of J factor with period for braced frame structures: (*a*) cities in the west of Canada; (*b*) cities in the east of Canada.

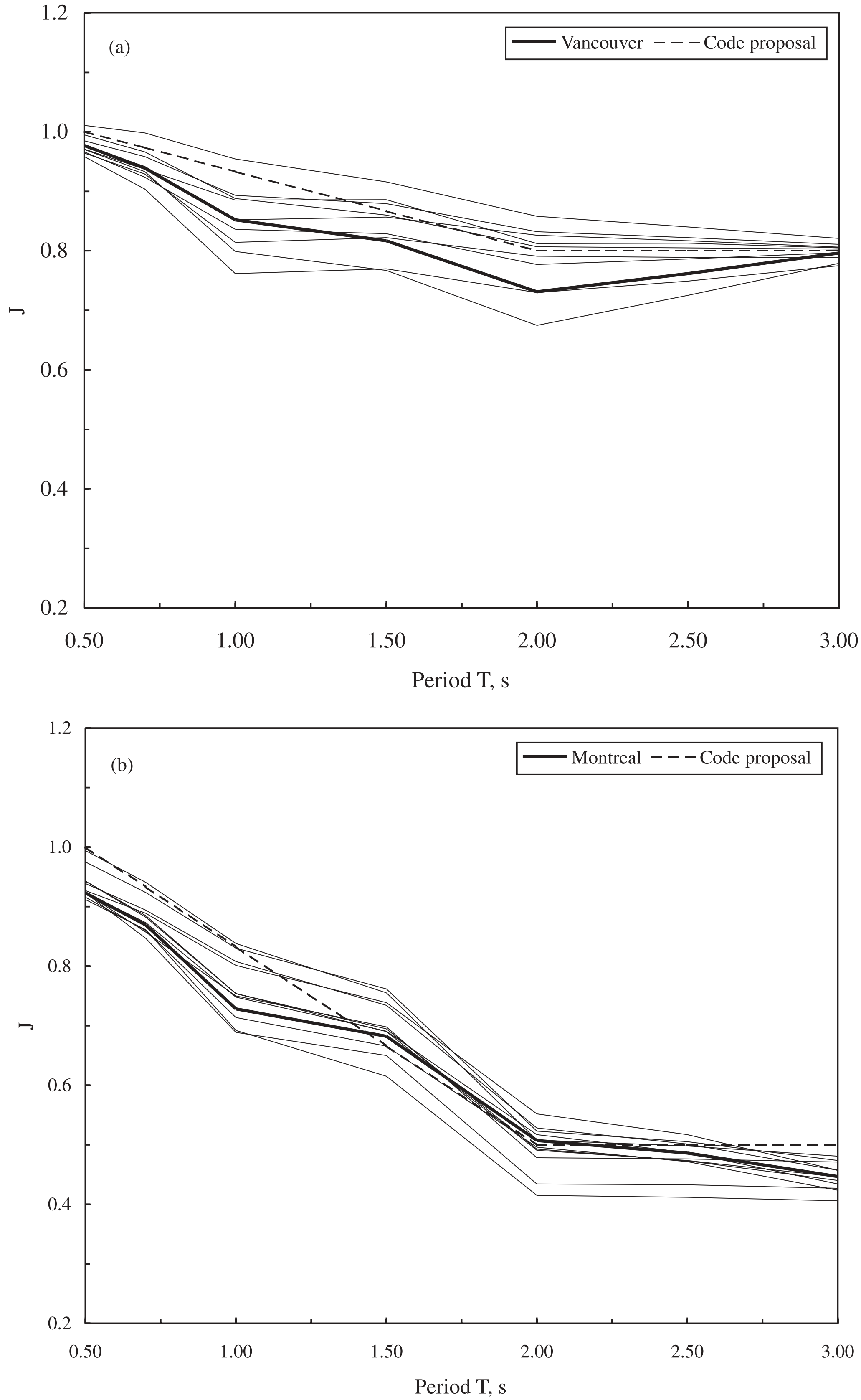
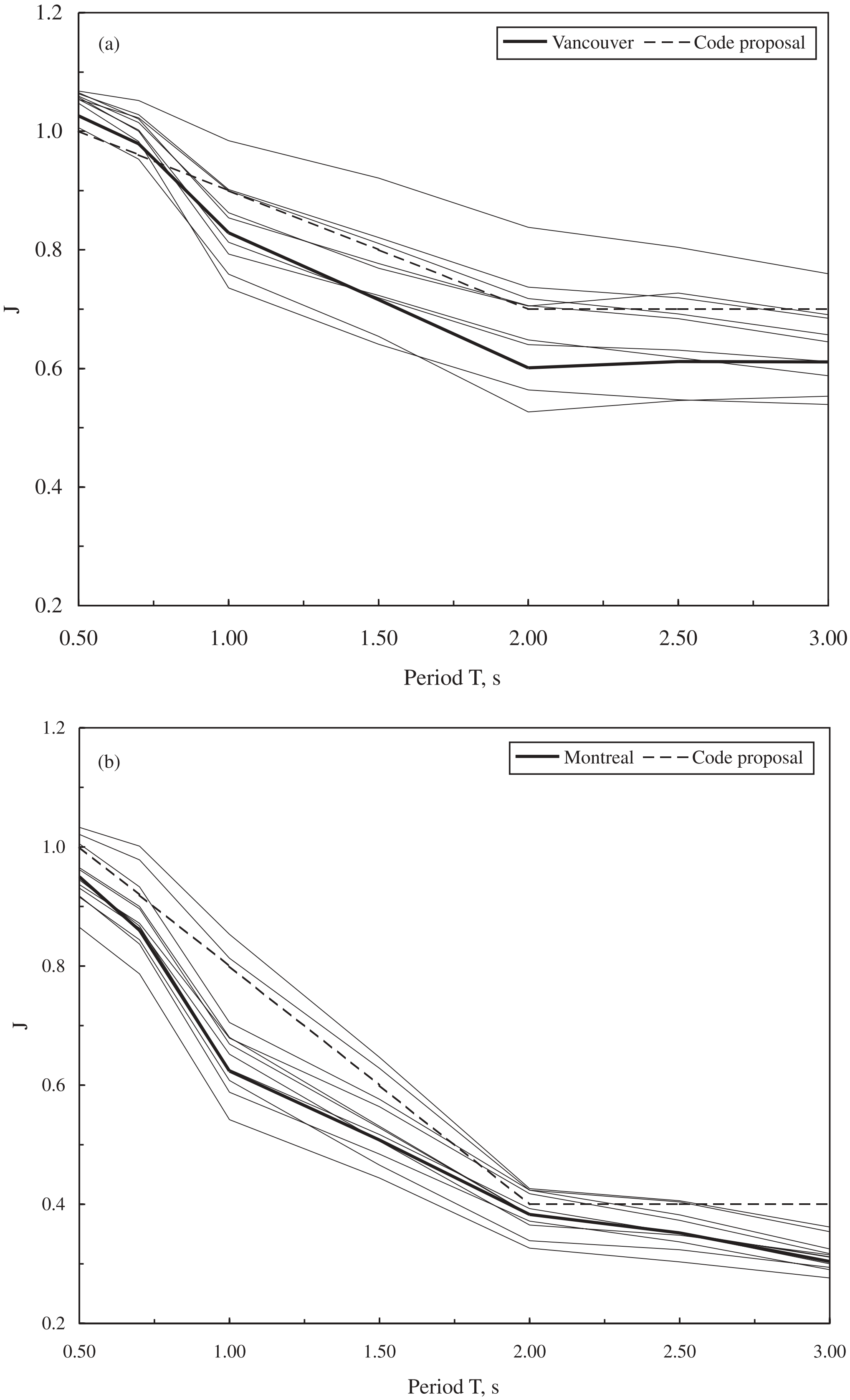


Fig. 11. Variation of *J* factor with period for wall structures: (*a*) cities in the west of Canada; (*b*) cities in the east of Canada.



parts in the west. This is because the spectrum drops more rapidly with an increase in period in the east than in the west, so the higher mode contribution is more predominant in the east. Based on the analytical results obtained for the 22 cities, empirical expressions are derived for the J factor to be used in design. These values are listed in Table 5 and shown as broken lines in Figs. 9–11.

The variation of J_x obtained from eq. [10*a*] across the height of a structure is shown in Fig. 12 for a flexural wall having a fundamental period of 2.0 s, for both the western and eastern regions. The factor J_x is significantly greater than 1 in the upper storeys. Evidently, this is because the NBCC distribution underestimates the shear in the upper storeys. However, the absolute values of overturning moments are not large in upper storeys, hence from the point of view of design it is sufficient to take $J_x = 1$ in these storeys. Taking this into account and based on the shape of the variation of J_x, the following simple expressions are proposed for J_x:

$$[11a] \quad J_x = 1.0 \qquad \text{for } h_x \geq 0.6h_n$$

$$[11b] \quad J_x = J + (1 - J)(h_x / h_n) \qquad \text{for } h_x < 0.6h_n$$

where h_n is the total height of the structure.

Coupled flexural walls

Coupled flexural wall structures are often used in buildings to resist the lateral loads. The model considered here is the 12-storey coupled walls system shown in Fig. 5*a*. Response spectrum analyses are carried out for different combinations of connecting beam depths (d = 0.2, 0.4, 0.6, 0.8, 1.0, and 1.2 m) and different fundamental time periods (T_a = 0.5, 0.7, 1.0, 1.5, 2.0, 2.5, and 3.0 s) for two Canadian cities, Vancouver in the west and Montréal in the east.

Figure 13 shows the variation of the factors M_v and J with period for six different coupled flexural walls systems located in Montréal. For the sake of comparison, Fig. 13 also shows the values of M_v and J for a moment-resisting frame and an isolated flexural wall. Figure 13 shows that, as expected, the response of the coupled wall system lies between that of a moment-resisting frame and that of an isolated flexural wall. When the beam is very flexible (d = 0.2 m), the system behaves like an isolated wall; when the beam is stiff (d = 1.2 m), the system behaves like a moment-resisting frame. In most practical structures the beams in a truly coupled system would be stiff enough that the coupled wall system can be assumed to behave like a moment-resisting frame.

For the 2005 NBCC it is proposed that a coupled wall system be treated as a moment-resisting frame for deriving the values of M_v and J. The criterion for classifying a system as a coupled wall system is similar to that specified in the Canadian Standards Association concrete design code CSA A23.3, namely that the coupling beams should be stiff enough so that at least 66% of the base overturning moment is carried by axial tension and compression forces resulting from shear in the coupling beams.

Hybrid systems

A hybrid system is defined here as one in which the lateral forces are shared by moment-resisting frames and flexural walls. An idealized model of a hybrid system is shown in Fig. 5*b*. The selected model is a 12-storey structure with a storey height of 3.65 m. Response spectrum analyses are carried out for different combinations of wall widths (w = 3.0, 4.0, 5.0, 6.0, 7.0, and 8.0 m) and different fundamental time periods (T_1 = 0.5, 0.7, 1.0, 1.5, 2.0, 2.5, and 3.0 s) for two Canadian cities, Vancouver in the west and Montréal in the east.

Figure 14 shows the variation of M_v and J with the fundamental period for different hybrid systems located in Montréal. For the purpose of comparison, Fig. 14 also shows the values of M_v and J for moment-resisting frame and flexural wall systems. As would be expected, the response of a hybrid system lies between that for a flexural wall and that for a moment-resisting frame. When the wall width is very small (2.0 m), the system behaves essentially like a moment-resisting frame. When the wall width is larger, 7.0 or 8.0 m, the system behaves essentially like a wall. In most practical structures the flexural wall will be considerably stiffer than the frame, so the hybrid system will behave more like a wall. In the 2005 NBCC a hybrid system will be treated as a wall system for determining the values of M_v and J.

Summary and conclusions

The equivalent static load procedures specified in many seismic codes including the 1995 NBCC require that the elastic base shear be obtained from a design response spectrum using the first mode period of the structure under consideration. The elastic base shear is then reduced by a factor that reflects the capacity of the structure to undergo inelastic deformation without collapse. In the 1995 NBCC the elastic response spectrum is obtained by applying appropriate amplification factors to the peak ground motion bounds. In recent years, methodologies have been developed that allow the direct determination of maximum spectral accelerations for specified values of the period and damping and for a given probability of exceedance. A plot of such spectral accelerations against the period is referred to as a UHS. The earthquake design provisions of the 2005 NBCC will be based on the use of UHS to define the seismic hazard and to obtain the design forces.

Since the spectral acceleration values for different periods may be contributed by different earthquakes, a UHS is different from the response spectrum of a single earthquake. Consequently, when used in a modal analysis a UHS provides somewhat conservative values for the response of an MDOF system. Such conservatism, however, is not excessive, and the results of a modal analysis based on a UHS may be considered quite appropriate for use in design.

In an equivalent static procedure of design based on UHS the elastic base shear will be determined by using the spectral acceleration corresponding to the first mode period of the structure. This process ignores the effect of higher modes on response; consequently, the base shear derived from the first mode period must be suitably adjusted. Adjustment factors for the base shear are derived in this paper

Fig. 12. Height-wise distribution of storey level overturning moment modification factor J_x: (*a*) cities in the west of Canada; (*b*) cities in the east of Canada (flexural wall, T_1 = 2.0 s).

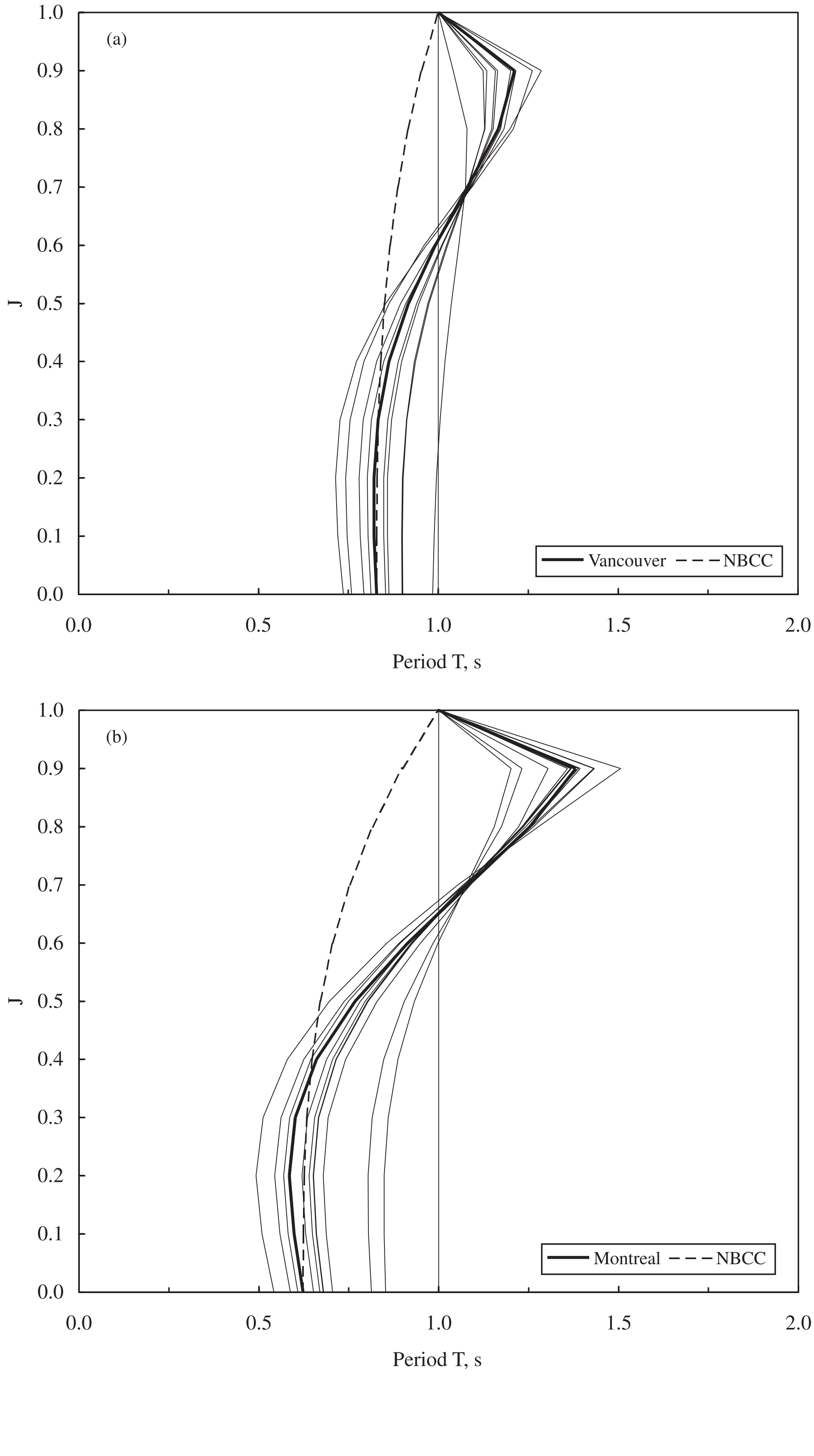


Fig. 13. Variation of adjustment factors with period for coupled wall structures for Montréal: (*a*) M_v factor; (*b*) *J* factor.

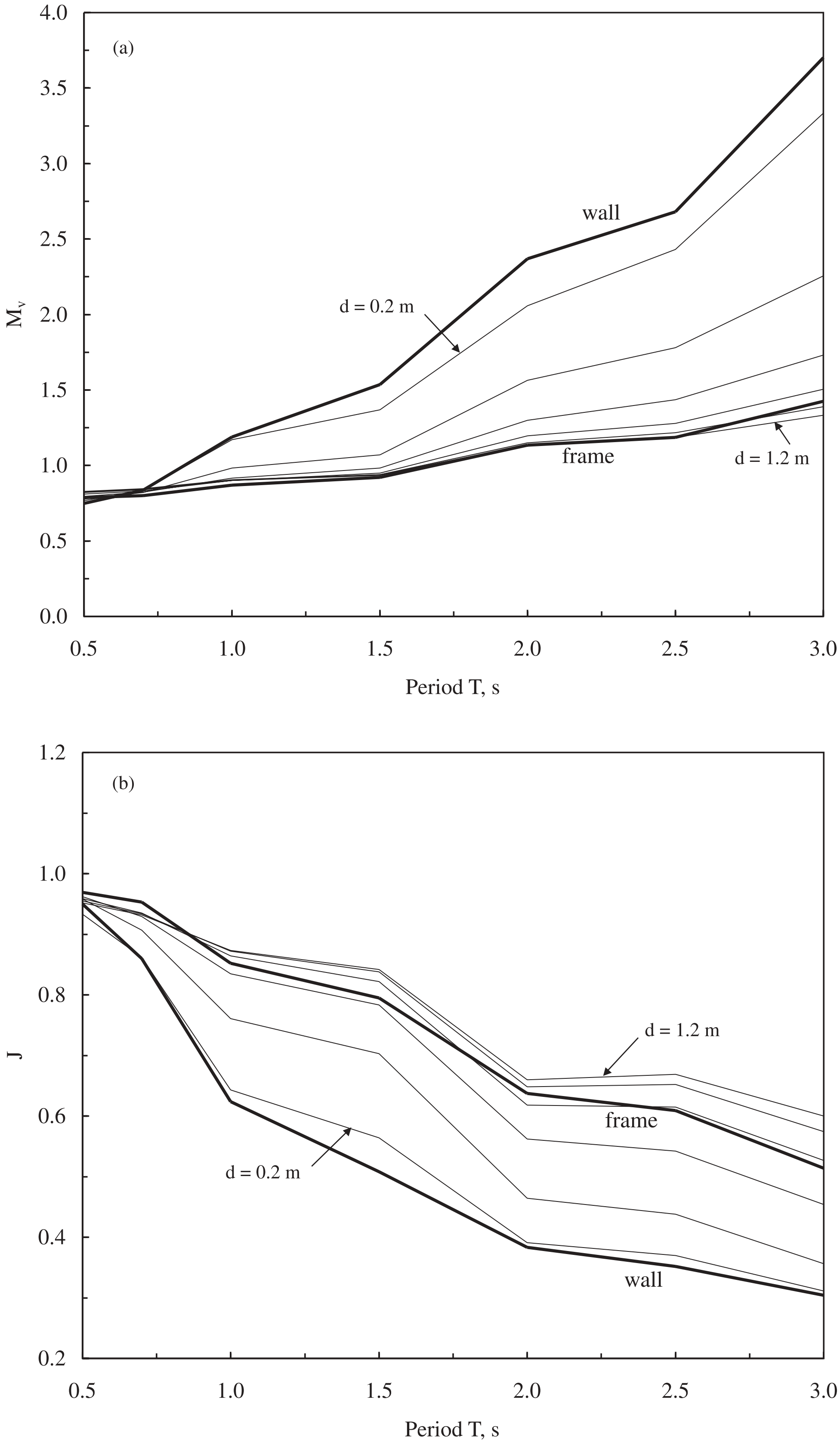
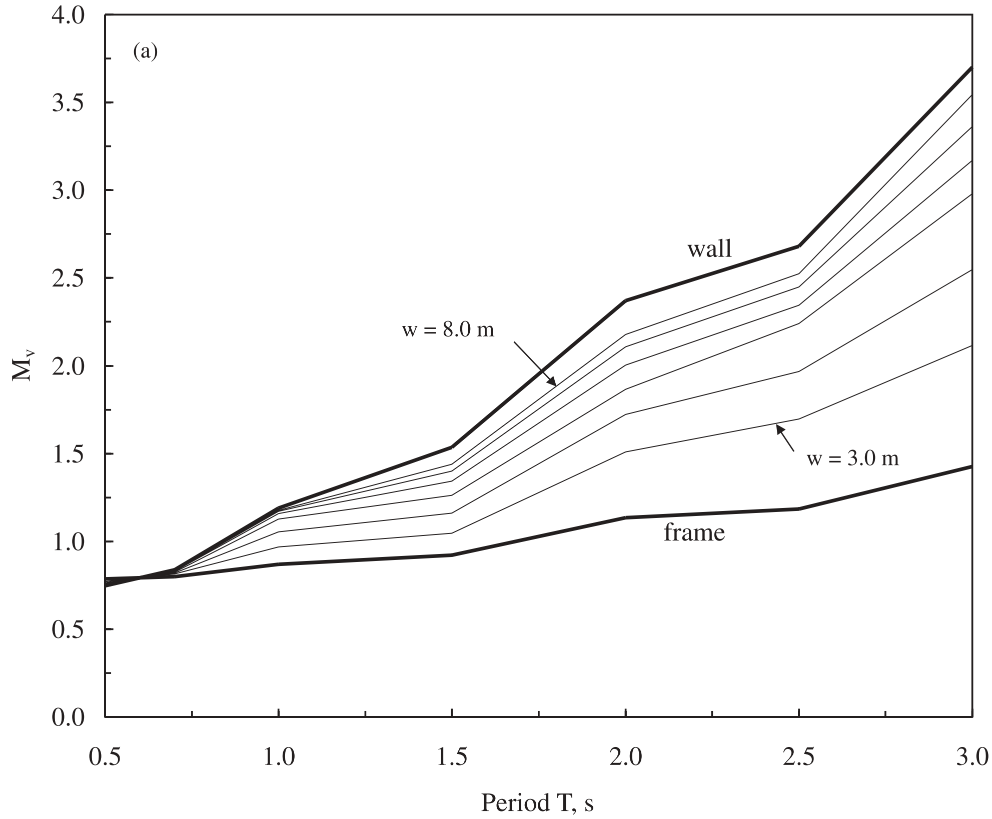
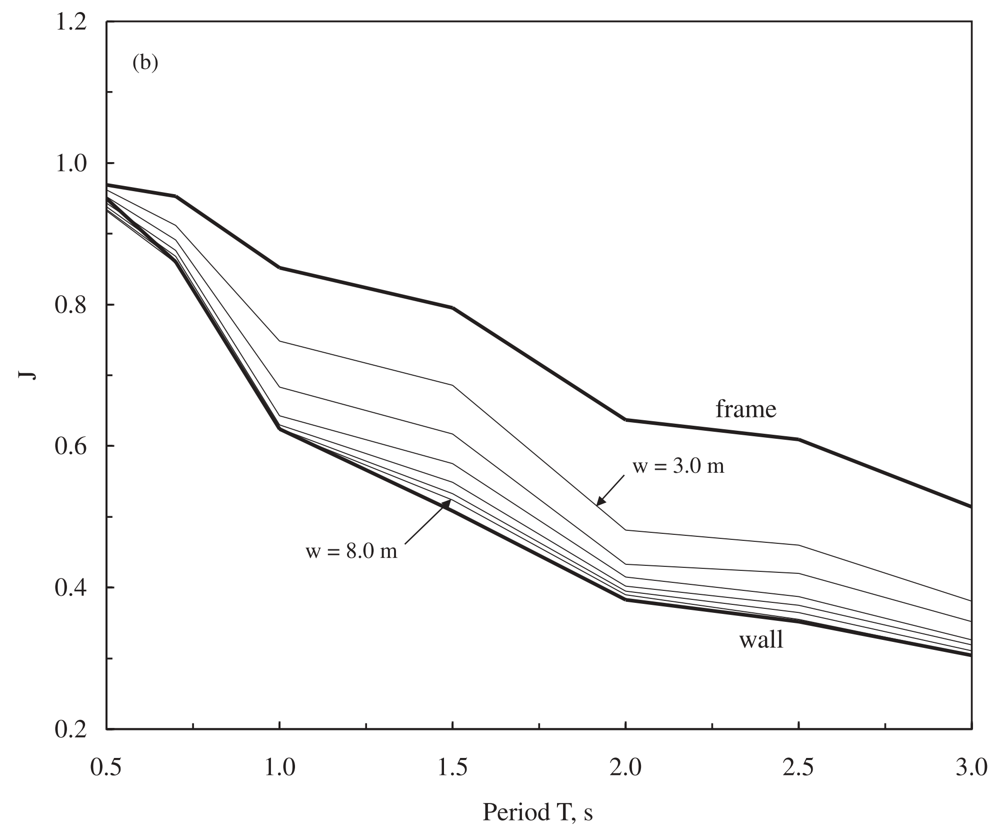


Fig. 14. Variation of adjustment factors with period for hybrid structures for Montréal: (*a*) M_v factor; (*b*) *J* factor.





for several different structural types including moment-resisting frames, braced frames, flexural walls, coupled flexural walls, and hybrid systems.

The corrected base shear can be distributed across the height of a structure according to the NBCC procedures. Because the NBCC distribution is primarily in the form of the first mode, however, the resulting overturning moments generally overestimate the true moments, which arise from a combination of various modes. Adjustment factors to be applied to the overturning moments that have been determined from the NBCC distribution are also derived in this paper for the various structural types.

The base shear and overturning moment adjustments presented in this paper form the basis for the corresponding provisions in the 2005 NBCC. The following conclusions are drawn from the results presented in this paper:

(1) The base shear adjustment factor M_v and the overturning moment reduction factor J are both dependent on the characteristics of the lateral force resisting system. The factor M_v is largest for a flexural wall system and smallest for a moment-resisting frame. On the other hand, J is smallest for a flexural wall and largest for a moment-resisting frame.

(2) The factors M_v and J also depend on the first mode period T_a. Thus M_v increases with an increase in T_a, whereas J decreases with an increase in T_a.

(3) The factors M_v and J strongly depend on the shape of the response spectrum. Compared with the western regions of Canada, the UHS for the eastern regions drops more rapidly with an increase in period. Thus the higher mode contribution is more predominant in the east; as a consequence, M_v values are larger and J values smaller for the eastern region.

(4) The distribution of shear across the height of a structure as specified in the current provision of the 1995 NBCC is reasonable but underestimates the shear in the upper storeys while overestimating it in the lower storeys.

(5) The underestimation of shear in the upper storeys also leads to underestimation of the overturning moments. It is therefore proposed that the expression for J_x be revised. A new formula for J_x that is simpler than the current version is proposed.

(6) The dynamic behaviour of a practical coupled flexural wall system is expected to be similar to that of a frame, so M_v and J for a moment-resisting frame can be applied to a coupled wall structure as well.

(7) The dynamic behaviour of a practical hybrid system comprising a flexural wall and a moment-resisting frame is expected to be closer to that of a flexural wall, so the factors M_v and J for walls can also be applied to a hybrid system.

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