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

Derivation of seismic design forces for multistorey buildings from Uniform Hazard Spectral curves

Rahgozar, M.A.[1], Humar, J.L.[2]

ABSTRACT

The use of uniform hazard spectra for obtaining the seismic design forces is being considered for the next version of the National Building Code of Canada. Such spectra provide the spectral accelerations of a single degree-of-freedom system for a range of periods but for a uniform level of hazard. One of the issues that need to be resolved before uniform hazard spectra are used in design is the adjustment required in the base shear to account for the higher mode effects present in a multi-degree-of-freedom system. This issue is examined here through analytical studies of the response of idealised multi-storey building frames to ground motions representative of the seismic hazard in east and west of Canada. Representative values are obtained for the adjustment factors that must be applied to the design base shear and to the base overturning moment.

INTRODUCTION

In the 1995 National Building Code of Canada, the seismic design base shear is obtained from a response spectrum whose shape is related to two ground motion parameters: peak ground acceleration and peak ground velocity for the seismic region in which the building is located. The spectral shape derived from the ground motion parameters may be in considerable error. Since the mid-1970s, methodologies have become available for deriving linear elastic spectra for a given site and a given hazard level directly from the seismological information for the region. Such spectra are called uniform hazard spectra (UHS). They provide spectral accelerations at specified values of the period of an elastic single degree-of-freedom (SDOF) system. It is expected that UHS will form the basis of the earthquake design provisions of National Building Code of Canada (NBCC) 2000.

Before UHS could be used to obtain the seismic forces for which a building must be designed, a number of issues need to be addressed. One of these is the fact that a uniform hazard spectrum provides the spectral acceleration for a SDOF. When a SDOF spectrum is used to obtain the elastic design base shear for a multi-storey building assuming that the entire mass of the building is responding in the first mode, the design base shear may at times be underestimated. In buildings that are expected to undergo inelastic deformations during the design earthquake, the use of a design base shear derived from a SDOF spectrum may lead to excessive ductility demands in some storeys. In recognition of these facts, most seismic codes, including the NBCC, artificially raise the design spectrum in the long period range. A more rational procedure would be to maintain the spectral shape corresponding a SDOF system and to incorporate a specific MDOF modification factor, M_V, in the expression for determining the design base shear. The present paper deals with the derivation of such a factor.

It should be noted that the factor M_V is useful only in providing a correct estimate of the base shear in a multi-storey building. The distribution of this base shear across the height depends on the relative contribution of the different modes of vibration. As a result, the base overturning moment obtained from a design base shear distributed according to just the first mode may be in considerable error, and is usually significantly higher than the true overturning moment. The moment reduction factor, J, in NBCC is meant to suitably adjust the base overturning moment calculated from a shear distribution that is based primarily on first mode response. Values of J are also presented in this paper.

Previously, Nassar and Krawinkler (1991) have studied the variation of M_V factor for moment-resisting frames subjected to selected records from Whittier Narrows Earthquake, October 1, 1987. More extensive studies are carried out in the present work to cover both moment-resisting frames and flexural walls. Also, earthquake records that are representative of the seismicity of Canada are used in the study.

[1] Research Associate

[2] Professor and Chair, Dept. of Civil and Environmental Engineering, Carleton University, Ottawa, Canada

BASE SHEAR FORMULATION

The following expression for the design base shear in a multi-storey building has been proposed (Rahgozar, 1998)

$$V = \frac{S(T)M_V FIW}{RR_d} \tag{1}$$

where the spectral acceleration $S(T)$ is obtained from the UHS for the site. Factor F accounts for the effect of foundation soil, and is different than 1.0, except for firm soils when it is equal to 1.0. Values for this factor will be provided in the code. Factor M_V accounts for higher mode effects in multi-storey buildings. Factor I is an importance factor similar to that in the 1995 version of NBCC, and W is the sum of dead plus applicable part of live load. Factor R is applied when inelastic deformations under the design earthquake are permissible, and is related to the ductility capacity of the structure. For the purpose of code provisions, R may be taken as being independent of the period. Finally, factor R_d accounts for reserve strength in the structure. Rahgozar and Humar (1997, 1998) have addressed the issue of reserve strength (overstrength) in seismic design and provide guidelines for the selection of a value for the reserve strength factor R_d.

DERIVATION OF INELASTIC DESIGN FORCES FROM UHS

Methodology

In deriving M_V for a building that is expected to become inelastic during an earthquake, it is useful to compare the strength requirements for the multi-storey building with that for an associated SDOF system. The associated SDOF system is defined as one having in its linear range, the same period and damping as the multi-storey building. The weight of the associated SDOF system is equal to the total weight of the multi-storey building. The strength demand for a SDOF system is calculated from Eq. 2 with the spectral acceleration S_a obtained from an inelastic spectrum for the design earthquake corresponding to the target ductility μ_t.

$$V_s = S_a(T, \mu_t)W \tag{2}$$

As stated earlier, to keep the maximum ductility demand in a multi-storey building within the target ductility μ_t, the design base shear for the associated SDOF system should be adjusted by applying a multiplier M_V, usually greater than 1, to obtain the design base shear for the multi-storey building.

The methodology for deriving M_V involves inelastic response analyses of simplified and idealised multi-storey building frames subjected to a series of earthquake ground motions. For each ground motion record, the design base shear is obtained from an inelastic response spectrum for the associated SDOF system and for target ductility μ_t. This base shear is distributed across the height of the frame according to the provisions of NBCC. An elastic analysis for these distributed forces provides the required member strengths as well as the yield level interstorey drifts to be used in the subsequent calculations of storey ductilities. A nonlinear dynamic analysis of the multi-storey frame is now carried out to obtain its response to the selected earthquake record. This provides the maximum ductility demand in any of its storeys. If this demand is different from the target ductility, the design base shear is suitably adjusted. Corresponding to the revised base shear, a new set of design forces, design strengths and yield level interstorey displacements are found. Another inelastic response analysis is then carried out to obtain the revised storey ductility demands. The process is repeated until the maximum storey ductility is equal to or slightly less than the target ductility. The ratio of the dynamic base shear, V_m, obtained at this stage to the original base shear in the associated SDOF system provides the value of M_V corresponding to the selected earthquake. The process is repeated with other earthquake records, and a mean value obtained for M_V. Details of the idealised frames studied, and the results obtained from the dynamic analyses are presented in the following sections.

The ground motions used in the present study are the UHS compatible simulated ground motions developed by Atkinson and Beresnev (1998). Two sets of records, one for Vancouver and Victoria in the West and another for Montreal, Ottawa, and Quebec City in the East are considered. In each set, there are four records, two governing the response in the long period range, and two governing the response in the short period range of the UHS.

Multi – storey models

Simplified multi-storey moment-resisting frame and wall models are used here in order to gain an insight into basic inelastic dynamic behaviour patterns. The models represent regular 2-dimensional single-bay bents. Each model has a bay size of 8.0 m and a uniform storey height of 3.5 m. The mass at each floor is 27.59 tonne. Axial deformations in the beams and columns in the moment-resisting frames, and in flexural walls are neglected. Inelastic deformations in members of the frame are assumed to be concentrated in plastic hinges at their ends. These plastic hinges have an elasto-plastic moment-rotation relationship with zero strain hardening. The effect of gravity loads is not considered, and the P-Delta effect is neglected. For each structural model, four structural heights, namely, 5, 10, 20, and 30 storeys, are considered.

Simplifications made in the modelling are justified on several accounts. First, the contributions of some of the factors, such as for example consideration of gravity load in the design of members, strain hardening, and P-Delta effect are normally accounted for in deriving a reserve strength factor to be applied to the base shear (Rahgozar and Humar, 1997, 1998), and the focus here is on an assessment of the effects of the higher structural modes on the ductility and strength demands in MDOF systems of different structural types. Second, in view of the many uncertainties involved in the determination of the earthquake hazard and the complexity of the structural response, a simplified method of design that is suitable for the design codes is best derived from idealised models that still capture the essential dynamic characteristics.

Characteristics of moment – resisting frames

The elastic member stiffnesses in each storey of the frames studied are selected so that, under the 1995 NBCC equivalent static load pattern, the interstorey drift in every storey is identical. The ratio of the beam stiffness to the sum of the column stiffnesses in each storey is set to be 1/8. This implies that the columns are stiffer than the beams, typical of earthquake-resistant construction. The condition of identical storey drift and the selected value of the beam to column stiffness ratio allow the determination of the relative stiffnesses of the elements. The absolute values of the stiffnesses are now adjusted so that the first mode period of the structure is equal to that given by the NBCC recommended expression $T = 0.1N$ s, where N is the number of storeys. Because of the way in which the stiffnesses are selected, the first mode shape is close to a straight line.

The models are designed so that plastic hinges develop simultaneously at all beam ends and at the first storey column bases under NBCC lateral load pattern. The moment capacities at plastic hinges are thus selected to be equal to the design moments at the corresponding member ends. The concept of strong-column-weak-beam is followed and the column strengths at all locations, other than at the base of the columns, are arbitrarily increased so that plastic hinges will form in beams only (as well as at base supports). A bilinear moment-rotation hysteresis model with zero strain-hardening ratio is assumed for each plastic hinge. At lower values of the base shear, the relationships between the applied shear and storey displacements are linear. As the base shear is increased, the storey displacements approach yield level, and at a certain value of the base shear all storeys yield simultaneously. The interstorey displacements at this stage are considered as being the yield displacements.

Characteristics of flexural walls

The flexural walls used here have a uniform stiffness along the height. Once the mass at each floor is known, the stiffness of the wall can be determined by setting the first mode period to $T = 0.1N$ s.

The member strengths are selected in a manner similar to that used for the moment-resisting frames. The models are designed so that plastic hinges develop simultaneously in each storey at the top and bottom of the wall under the 1995 NBCC equivalent static lateral load pattern. The moment capacities at plastic hinges are thus determined as being equal to the design moments at the corresponding member ends. For each storey, the yield displacement is the interstorey displacement when simultaneous yielding takes place in all the storeys. A bilinear moment-rotation hysteresis model with zero strain-hardening ratio is assumed at each plastic hinge location.

Shear adjustment factor

The variation of M_V with period and ductility for moment-resisting frames is shown in Figs. 1a and b, respectively for the west and the east of Canada. Factor M_V is seen to increase monotonically with both the period and the ductility. The M_V values are the lowest for the elastic case, less than 1.0 for the entire period range both in the West and the East. For higher ductilities M_V is generally higher in the East than in the West. The highest value of M_V is 1.8 for the West and 2.0 for the East.

The variation of M_V with period and ductility for flexural walls is shown in Figs. 2a and b, respectively for the West and the East. Again the factor M_V increases with both the period and the ductility. The values of M_V are significantly higher for flexural walls than for moment-resisting frames. The highest value for the West is about 2.7, while for the East it is 5.8. Also, the effect of an increase in target ductility on M_V is more substantial for flexural walls than for frame type structures.

The following observations can be made in respect of the upper bound values of M_V for different structural types. These values are based on a target ductility of 4, and are therefore conservative when the target ductility is lower. For moment-resisting frames, M_V can be taken as 1.0 for periods up to 1.0 s; then increasing linearly to 1.75 for a period of 3.0 s. Identical values are proposed for the West and the East.

For flexural walls in the West, M_V may be taken as increasing from 1.0 at $T = 0.2$ s to 1.2 at $T = 0.5$ s and then to 2.75 at $T = 3.0$ s. For walls in the East, M_V may be taken as increasing from 1.0 at T = 0.2 s to 1.7 at T = 0.5 s and then to 5.8 at $T = 3.0$ s.

Base overturning moment adjsutment factor

The dynamic analysis carried out for the purpose of determining the base shear in a structure also provides the overturning moments M_{bm} at the base. The application of forces obtained by distributing V_m according to NBCC gives a base moment, M_{bs}, which, in general, is quite different from M_{bm}. A factor $J = M_{bm}/M_{bs}$ must be applied to the base moment determined by equivalent lateral load method in order to determine the true base moment. Figures 3 and 4 present the J factor for the frame and wall structures respectively for different values of the target ductility ratio. Parts (a) of these figures present the results for the west of Canada, while parts (b) show the results for the east of Canada. For comparison the J factor specified by NBCC 95 is also shown in these figures.

The following observations can be made on the basis of the results presented here. For moment-resisting frames, J varies between 0.85 to 1.0, except in the East where it is as low as 0.7 for the elastic case and a period of 3.0 s.

For flexural walls, J is both period and ductility dependent. It decreases monotonically with increase in either the period or the ductility ratio. Factor J is in all cases lower for East than for West. For a period of 3.0 s and a ductility of 4, it is 0.18 in the East and 0.37 in the West. In general, it is observed that the higher M_V is, the smaller is J. The J factor is also less than the code prescribed value. As noted earlier, the use of a larger J does not necessarily lead to a safer design, because it may increase the tendency for a brittle shear failure.

CONCLUSIONS

The results presented here show that the design base shear obtained from a UHS based on the response of a SDOF system, must be suitably adjusted to account for the higher mode effects in a multi-storey building. This is particularly true for flexural wall structures located in east of Canada, both because the higher modes make a relatively larger contribution in such structures, and the UHS for eastern locations fall quite rapidly with period. Along with the adjustment of the base shear, it is also necessary to appropriately scale the overturning moments obtained when the adjusted base shear is distributed according to the NBCC. The results presented, while being indicative of the trends, need to be corroborated by more extensive studies.

REFERENCES

Atkinson, G.M. and Beresnev, I.A. 1998. Compatible ground-motion time histories for new national seismic hazard maps. Canadian Journal of Civil Engineering, 25, 305-318.

Nassar, A.A. and Krawinkler, H. 1991. Seismic demand for SDOF and MDOF systems. John A. Blume Earthquake Engineering Centre, Stanford University, Calif., Report No. CA94305-4020.

Rahgozar, M.A. 1998. Ductility and strength demands for building structures subjected to earthquake forces. Ph.D. thesis submitted to Carleton University, Ottawa, Canada

Rahgozar, M.A. and Humar, J.L. 1997. Accounting for overstrength in seismic design of concrete building structures. Al-Azhar Engineering Fifth International Conference, December 1997, Cairo, Egypt.

Rahgozar, M.A. and Humar, J.L. 1998. Accounting for overstrength in seismic design of steel structures. Canadian Journal of Civil Engineering, **25**, 1-15.

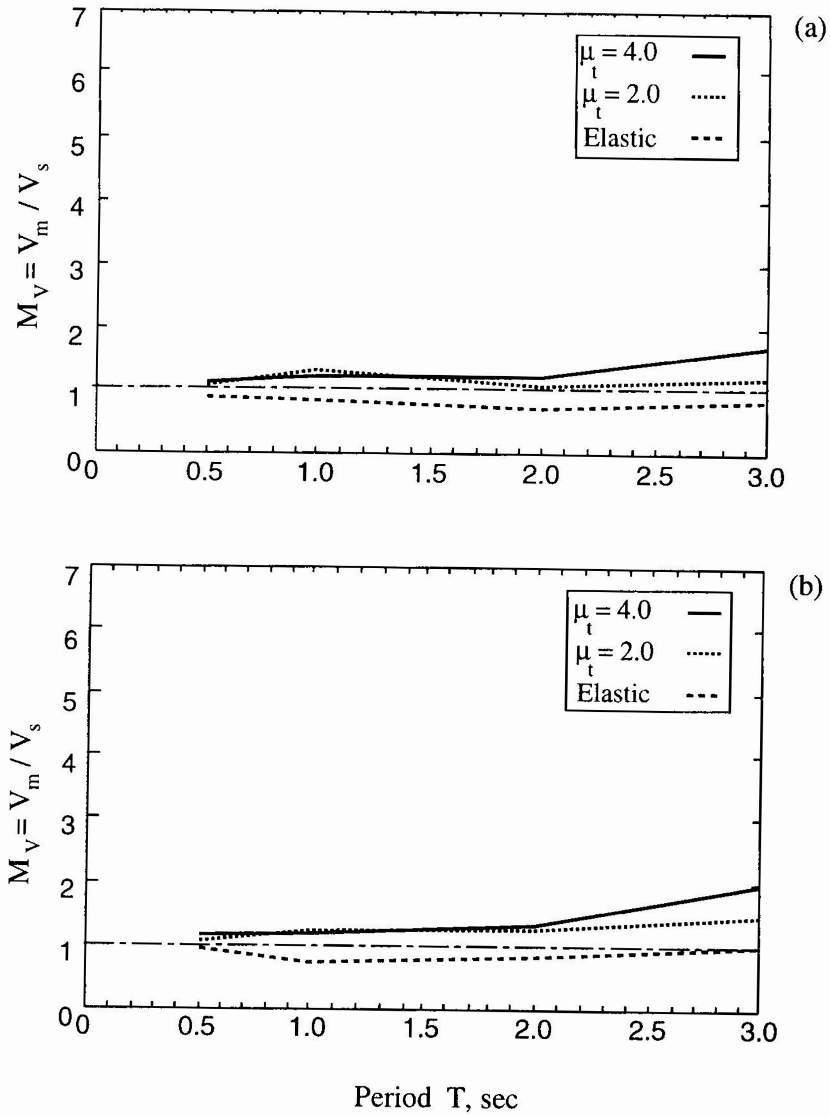

Figure 1 : Variation of M_V factor with period for different target ductility ratios for moment-resisting frames (a) west of Canada and (b) east of Canada.

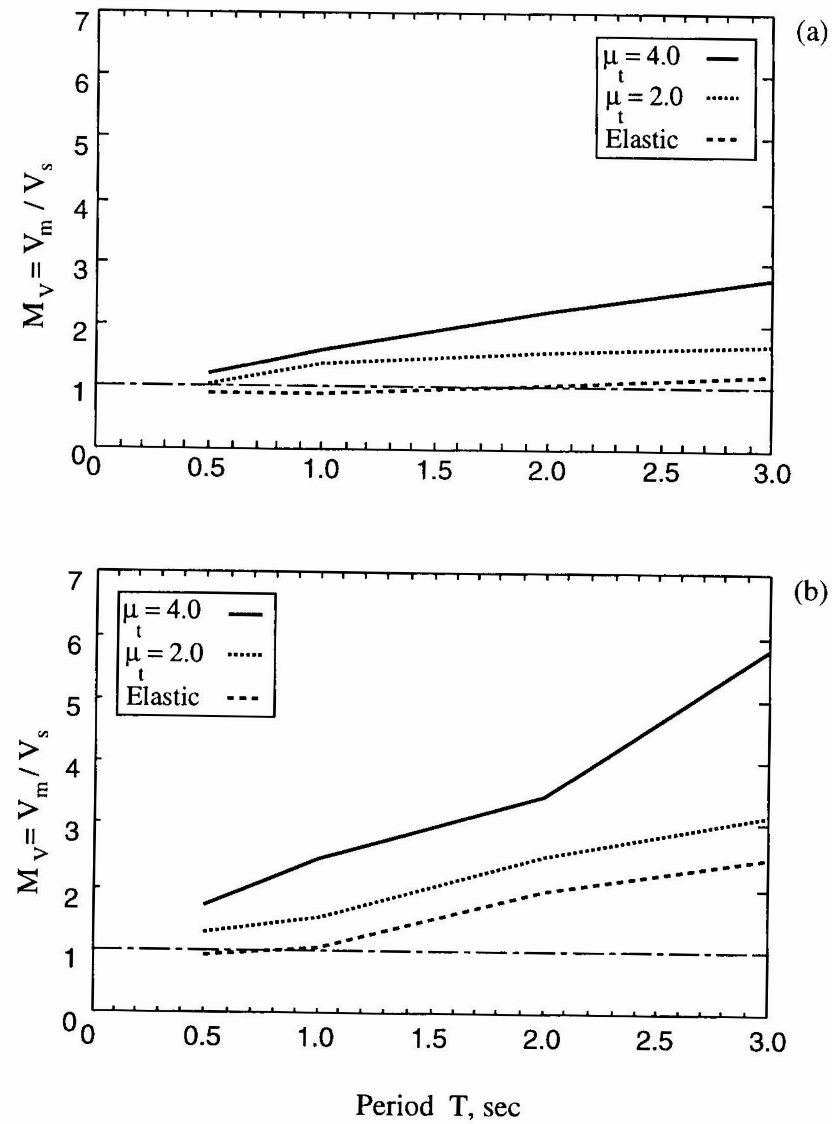

Figure 2 : Variation of M_V factor with period for different target ductility ratios for flexural walls (a) west of Canada and (b) east of Canada

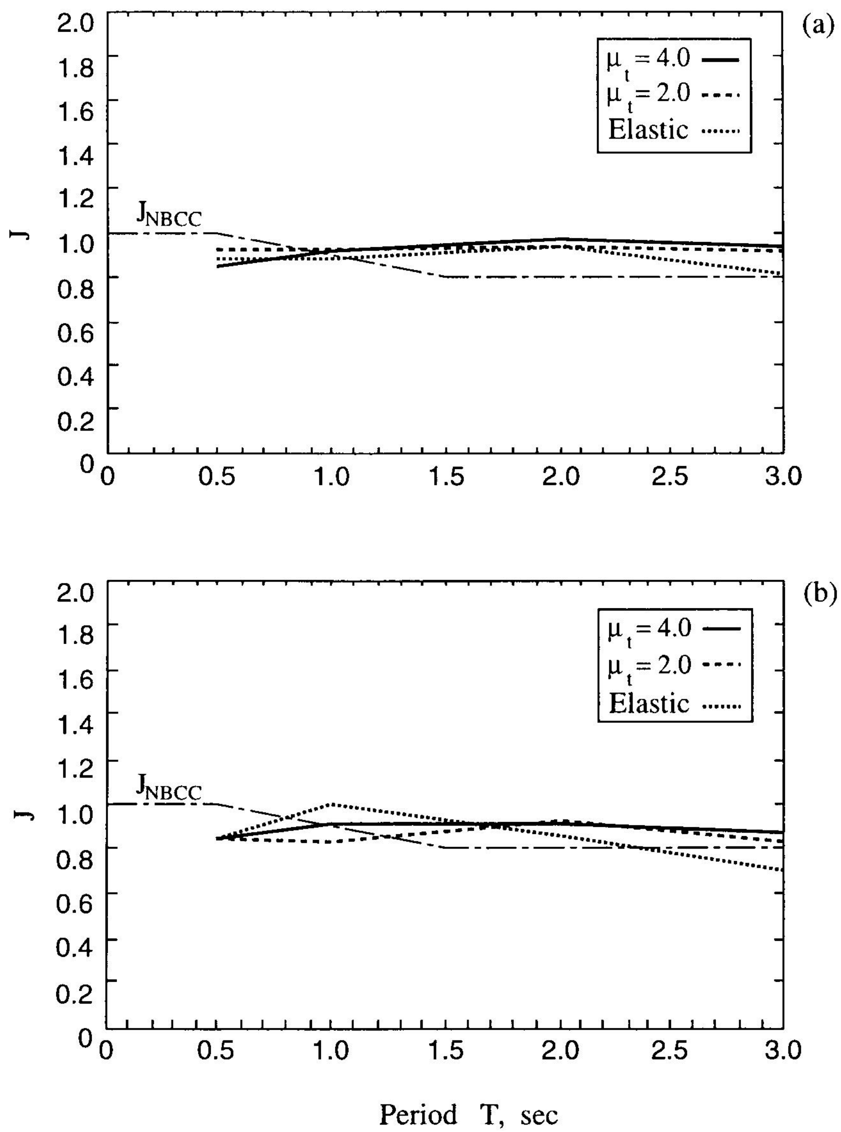

Figure 3: Variation of base overturning moment reduction factor J with period for different target ductility ratios for moment-resisting frames in (a) the west of Canada and (b) the east of Canada

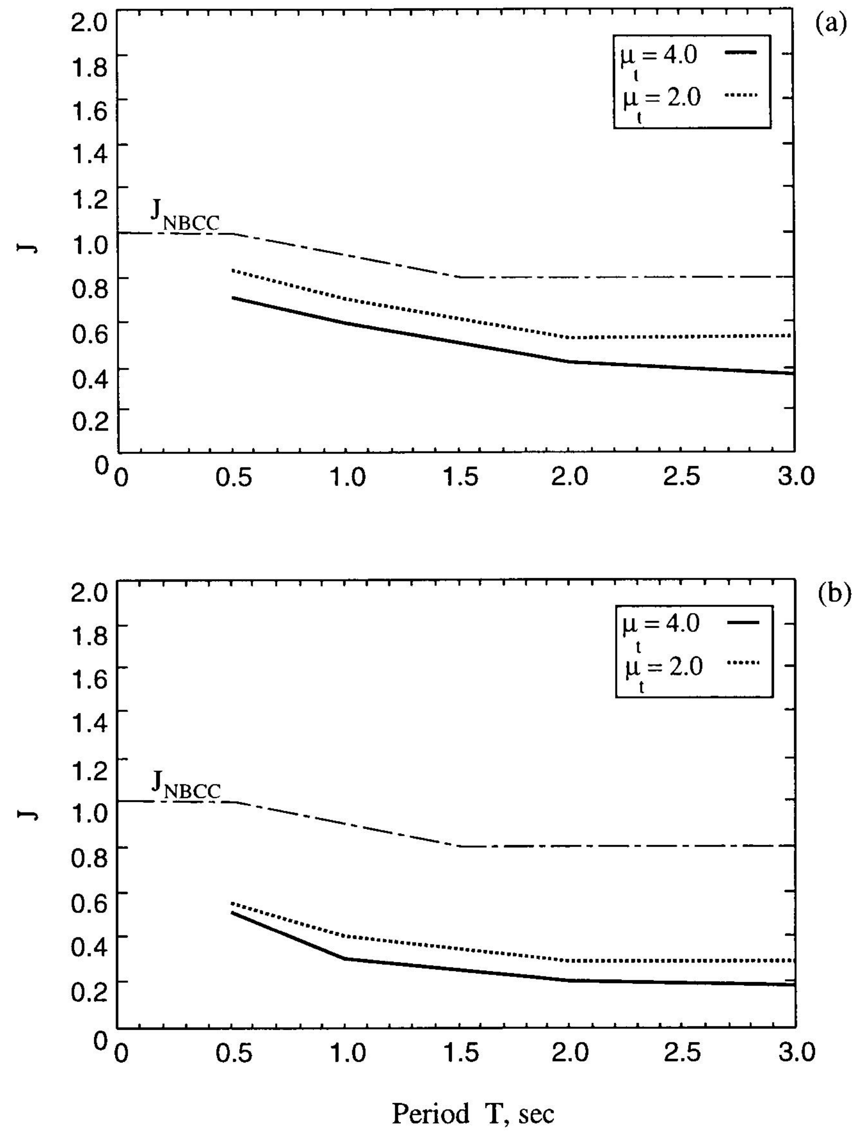

Figure 4: Variation of base overturning moment reduction factor J with period for different target ductility ratios for flexural walls in (a) the west of Canada and (b) the east of Canada