PROPOSED SEISMIC LOADING PROVISIONS - NATIONAL BUILDING CODE OF CANADA 1985

by

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ABSTRACT

This paper describes the process by which Canadian seismic loading provisions are developed and then details the primary changes being planned for the 1985 edition of the National Building Code of Canada. The most significant change is the inclusion of new seismic zoning maps, based on a new seismic risk methodology, a new probability level, additional seismic zones, and incorporating both horizontal ground velocity and acceleration as zoning parameters. The format of base shear calculation is revised to incorporate these changes, including the specification of a new seismic response parameter. The base shear formula is calibrated to ensure that, on a cumulative basis throughout the country, the level of seismic loading remains unchanged. Additional changes discussed in the paper include the removal of dynamic analysis as a specific option in the dynamic load calculation and some signficant changes in the calculation of torsional effects.

INTRODUCTION

The objective of this paper is to describe and discuss the primary new features of the seismic loading provisions being planned for inclusion in the National Building Code of Canada 1985 (NBCC 1985). The responsibility for recommending changes to the seismic loading provisions of the NBCC rests with the Canadian National Committee on Earthquake Engineering (CANCEE). The recommendations of CANCEE are made to the Standing Committee on Structural Design (SCSD) who in turn are responsible to the Associate Committee on the National Building Code of Canada. Both authors have been members of CANCEE for a number of years and have been intimately involved in the ongoing development of the seismic loading provisions.

The technical background work leading to specific code changes proposals within CANCEE is handled through a number of CANCEE Task Groups which are assigned the ongoing responsibility for reviewing the various aspects of the seismic loading provisions and the associated commentaries (Effects of Earthquakes and Dynamic Analysis for the Seismic Response of Buildings in NBCC 1980). Changes are considered for two primary reasons: a) new information resulting from current research or from experience of actual building behaviour during recent earthquakes, and b) inadequacies in current code provisions. Most of the significant changes being planned for NBCC 1985 have been discussed and considered by CANCEE since 1979. The final version was approved for recommendation to the SCSD at the November 1982 meeting of CANCEE. At the time of writing this paper, the recommendations have been approved by the SCSD, (with minor revisions) and are in the process of being sent out for public comment. The extent of any subsequent revisions will depend on the comments which are received.

This paper includes only the most significant changes being planned for NBCC 1985 and does not include minor adjustments. It also does not include changes in the foundation requirements but is restricted to the structural loading aspects.

SEISMIC ZONING

The current seismic zoning map (contained in NBCC 1980) has been unchanged since it first appeared in NBCC 1970 (1). This was the first strictly probabilistic map and was developed by Milne and Davenport (2) using extreme-value statistics applied to the catalogue of known Canadian earthquakes to compute probability of peak acceleration exceedance at a grid of sites throughout the country. This map displayed contours of peak horizontal acceleration, at a probability of exceedance of 0.01 per annum, that were used as boundaries for the four seismic risk zones.

A recent review of the methods of estimating seismic risk in Canada (3) has shown that the Cornell method (4) is more appropriate for the evaluation of the geographical distribution of seismic risk. The Cornell method enables the incorporation of geological and tectonic information, when available, to assist in defining earthquake source zones, in contrast to the assumption (in the extreme value method) that future large earthquakes will occur in the same locations as the historic events. The Cornell method has been adopted for the new maps; a full description of the methodology and the earthquake source zones used in determining these maps is given by Basham et al (5).

In addition to adopting a new methodology for the calculation of seismic risk, it was necessary to review the probability level at which the risk calculations are done. First, the expression in terms of probability of exceedance of 0.01 per annum is often converted to a return period of 100 years, which implies a prediction of risk far into the future based on information over an equally long period in the past. For purpose of building design, it is more useful for the probability to be expressed as the probability of exceedance of a strong ground motion in the average lifetime of a building, e.g. 50 years. Making this conversion, a probability of exceedance of 0.01 per annum corresponds to a 40% probability of exceedance in 50 years. However, this relatively high probability of exceedance of seismic ground motion does not imply the same probability of exceedance of seismic design loads, since the design loads are computed by a formula which is only empirically related to the seismic ground motions. The design loads that result from the static provisions of NBCC 1980 were originally set or calibrated empirically based on building practice in California. The probability level that can be associated with these design loads is not known precisely but is much lower than that now used as the basis for seismic zoning maps. To this extent the seismic risk probability level can be considered somewhat arbitrary and required only as a means of assessing relative risk levels across the country. However, experience in recent years has shown that the values of peak ground acceleration provided in the current seismic zoning map are frequently used in non-code applications in the mistaken belief that this will result in levels of protection comparable to those afforded by building structures designed according to the NBCC. The actual levels of protection would be considerably lower and it is therefore desirable to use a probability level for seismic ground motion which is near to that of the design loads. While it is not yet possible to link these in a totally rational process, current experience suggests that a probability of 10% exceedance in 50 years is more nearly appropriate to the effective design levels provided by the current code. This is the level recommended for the new seismic zoning maps, which has the further advantage of corresponding to that adopted by the ATC-3 guidelines in the USA (6), which will facilitate more direct comparison across the Canada-US border.

NBCC 1980 uses PHA (peak horizontal acceleration) to specify the level of strong ground motion that a structure must be designed to withstand without major failure or loss of life. This would be adequate if experience showed that all building damage correlated well with peak acceleration; but this is not the case, especially for modern tall buildings having fundamental periods greater than approximately 0.5 second. Estimates of PHA are most appropriate to periods centred near 0.2 second, while estimates of PHV (peak horizontal velocity) are appropriate to periods centred near 1 second. Thus, using both the parameters PHA and PHV have the potential for significantly improving the seismic provisions contained in NBCC 1980. Additional parameters (e.g. peak displacement, sustained level and duration) are also helpful in fully characterizing the ground motion and estimation damage potential. However, PHA and PHV are considered sufficient for NBCC applications and revised zoning maps have been prepared for both parameters.

NBCC has traditionally used the zonal approach to seismic loading, i.e. uniform seismic loading within each zone bounded by specific upper and lower contours of peak horizontal acceleration. This has the advantage of simplifying the loading calculations and providing relatively consistent seismic loading and design requirements throughout large regions of the country having approximately the same seismic risk. However it does have the disadvantage of rather large step changes of loading across zone boundaries, particularly when the number of zones is relatively small. NBCC 1980 has four zones (0 to 3), with load changes of a factor of 2 across zone boundaries. There is considerable support for a move to a full contour system for calculating loads, but this also has some disadvantages. In certain areas of the country, the gradient of seismic risk is rather steep and there could be significant differences of loading within a single municipality if the contour approach were adopted. Even with the contour approach, there would be a need to have some definite "zone-type" boundaries to allow changes in design requirements to be incorporated.

As a compromise, it was decided to continue to use the zone approach but to increase the number of zones in order to reduce the size of the step changes across the zone boundaries. There are now seven zones (0 through 6) with zone boundaries as shown in Table 1. The units for PHA (g) and PHV (m/s) enable the zonal boundaries for both PHA and PHV to be expressed in the same numbers. This is not only convenient but also has some physical basis. The PHA/PHV ratio of 1, using those units, corresponds to the average of a large number of recorded earthquakes, as can be seen in the shape of the design spectrum included in Commentary K of NBCC 1980 (7). Consequently the units for PHV and PHA can be expressed as non-dimensional ratios to 1 m/s and 1 g respectively, preserving the same PHA/PHV relationship. Table 1 also includes the zonal ratio for each zone, i.e. the single value of PHA or PHV to be used in each zone.

Table 2 gives the values of PHA and PHV and the proposed seismic zones for selected Canadian cities. From the table it can be seen that the PHA/PHV ratio varies considerably (from 0.48 to 2.04 for the selected cities in that table), and that the two parameters need to be zoned separately in order to recognize the differing character of the seismic ground motion at various locations. The ratio is low, i.e. velocity dominates, at sites that are influenced by large earthquakes at a distance (e.g. Prince Rupert). It is high, i.e. acceleration dominates, at sites that are influenced by moderate nearby earthquakes (e.g. Montreal). The corresponding difference in zones can be as large as two zones, e.g. Prince Rupert with Z_a two zones lower than Z_v and Montreal or Ottawa with Z_a two zones higher than Z_v. The application of this new zoning system to seismic loading is discussed in the following section.

BASE SHEAR CALCULATIONS

With the seismic zoning system established in terms of ground motion parameters for a given probability of exceedence, application to a building code requires a quantitative link between the zoning parameters and the desired response and performance of buildings during earthquakes. It is the purpose of this section to describe the changes to the NBCC 1980 seismic response factor and base shear formula that are required to accommodate the proposed zoning system.

The NBCC 1980 formula for base shear V is

$$(V)_{1980} = A\ S\ K\ I\ F\ W \qquad (1)$$

where A is the acceleration ratio (the 1980 zonal value at a probability of exceedence of 0.01 per annum), S the seismic response factor, K the structural behaviour factor, I the importance factor, F the foundation factor and W the dead load. For buildings of normal importance, and for good quality foundation conditions, both I and F are equal to one. Using these values, since it is beyond the scope of the paper to consider changes in I and F, rearranging equation (1) yields the following normalized base shear coefficient

$$(V/KW)_{1980} = A\ S_{1980} \tag{2}$$

Since it is not intended to consider the effect of varying or modifying K in this paper, it is included in the left hand side of equation (2). This format for the normalized base shear coefficient will be used in the remainder of this paper to discuss the effects of changes in seismic zoning.

The NBCC 1980 seismic response factor is given by

$$S = 0.5\ T^{-1/2} \leq 1.0 \tag{3}$$

where T is the natural period of the building in question. The equality in this expression is applicable to the medium and long period range (velocity amplification), whereas the limiting value is associated with the short period range (acceleration amplification).

It is proposed that the new base shear formula be given in the form

$$V = v\ S_{new}\ K\ I\ F\ W \tag{4}$$

where v is the zonal velocity ratio. A new seismic response factor, S_{new}, is described graphically in Figure 1 in terms of a parameter S_n which is to be determined. The proposed normalized base shear coefficient is therefore given by

$$(V/KW) = v\ S_{new} \tag{5}$$

As can be seen from the foregoing, it is proposed that the seismic forces for long period structures ($T \geq 0.5$ s) be directly proportional to zonal velocities. Forces for short period structures ($T \leq .25$ s) are proportional to zonal accelerations, with the exception that the effective acceleration zone is allowed to deviate by only one zone (up or down) from the velocity zone at any site. The forces in the intermediate period region ($.25$ s $< T < .5$ s) are determined by linear interpolation between the two transition periods (see Figure 1). The advantage of this arrangement is to have a transition region which is in the neighbourhood of the normal response spectrum corner period (approximately 0.4 s) while maintaining the acceleration bound corner period of 0.25 s at the same place as in NBCC 1980. The long period variation of forces with period is the same as in NBCC 1980. This scheme avoids significant shifts in the transitonal period for different Z_a and Z_v combinations, while permitting forces to vary as the Z_a/Z_v ratio varies.

The restriction that the effective acceleration zone can deviate by a maximum of one from the velocity zone in effect at a given site will affect several locations (e.g. Montreal and Ottawa; see Table 2) and requires some explanation. In locations where the actual acceleration/velocity ratio is high, the ground accelerations will often be high frequency and of short duration in character; these accelerations will consequently not produce amplified response to the same extent as velocity. Therefore, it is reasonable to place an upper limit on the "effective" acceleration/velocity ratio. For low actual acceleration/velocity ratios, it is necessary that the structures which would be sensitive to velocity (i.e. $T \geq 0.5$ s) be designed to force levels associated with the velocity in effect at that location. However, it is not deemed appropriate to allow low site accelerations to reduce forces for $T \geq 0.5$ s, which is accomplished by not allowing the "effective" acceleration to be more than one zone lower than the velocity zone. At locations where the velocity zone, Z_v, is zero but the acceleration zone, Z_a, is non-zero, it is considered desirable to require that all structures have a minimum level of seismic resistance. For these cases, the condition should be imposed that $Z_v = 1$.

The value of S_n is determined by calibrating the proposed seismic shear forces to those in effect in NBCC 1980. The calibration is based on the principle that the new seismic forces should be equivalent, in an average way across the country, to those of NBCC 1980. Since the adoption of the new estimates of seismic risk has altered in some detail the geographical distribution of seismic risk within Canada, this equivalence can only be attained in a cumulative sense by summing or integrating these effects across the country.

The approach used here is to calibrate by equating the sum of the weighted base shear coefficients for $T \geq 0.5$ s (1980 and new; i.e. equations (2) and (5)) for the ten Canadian cities in 1980 zones 2 and 3 with populations greater than 100,000 (Chicoutimi, Hamilton, Montreal, Ottawa, Quebec City, St. Catherines, St. John, St. John's, Vancouver and Victoria, according to the 1976 metropolitan census). It is desirable to given more weight to cities in higher seismic zones so the weighting factors were the populations multiplied by the 1980 zonal accelerations. This procedure resulted in $S_n = 0.44$.

Figure 2 shows plots of 1980 and new base shear coefficients for a selected group of Canadian cities which are located in NBCC 1980 zones 2 and 3. The effect of differing Z_a and Z_v combinations can be seen clearly. The comparison of Prince Rupert ($Z_a < Z_v$) and Victoria ($Z_a = Z_v$) shows the effect of different acceleration zones for cities which have the same velocity zone ($Z_v = 5$). A similar comparison can be made for Fredericton ($Z_a > Z_v$) and St. John's ($Z_a = Z_v$).

For the cities included in Figure 2 the largest changes in base shear coefficient from NBCC 1980 occur for Victoria (increase of 65 percent) and St. John's (reduction of 45 percent). The increase for Victoria is due primarily to the inclusion of more zones in the higher risk regions of the country thereby permitting the risk in Victoria to be distinguished from that in Vancouver, whereas both cities are in NBCC 1980 zone 3. The reduction for St. John's arises primarily from a change in seismic risk estimate due to the change in method.

Moderate and long period structures ($T \geq 0.5$ s) in Vancouver, Ottawa and Montreal have very little change in force levels (an increase of about

12%). However, there are increases (55%) for the short period structures in Ottawa and Montreal due to the acceleration zone being higher than the velocity zone. Quebec City has some decrease (18%) for moderate to long period structures and an increase (17%) for low period structures, due to the fact that $Z_a > Z_v$.

It should be noted that both the NBCC 1980 and the above proposed new base shears are unfactored loads; i.e., they need to be multiplied by the load factor of 1.5 to obtain the design base shear. While it is not proposed to include any change to this load factor in NBCC 1985 there are several persuasive arguments for changing it to 1.0 and, in order to retain the same design load, multiplying the base shear formula by 1.5. In this way the specified seismic load would represent the ultimate load, without any additional "factor of safety".

DYNAMIC ANALYSIS

NBCC 1980 (7) makes specific provisions for the use of dynamic analysis for the determination of the base shear, as an alternative to the static formulation given by equation (1). Commentary K gives a detailed description of a dynamic modal analysis procedure which is deemed to be an acceptable way of dynamically computing the base shear. This procedure includes a recommended elastic response spectrum and information on damping ratios and structural ductility factors for various kinds of structures.

However, it has long been recognized that the dynamically determined base shears bear little or no relation to those computed using the static code provisions (9)(10)(11). While there are a variety of specific reasons for these differences, it is clear that the assumptions made in the dynamic calculations have at least as much or more uncertainty than those of the static calculations. The primary advantage of the dynamic calculation is a more accurate representation of the distribution of stress resultants throughout the structure, particularly for structures which are non-uniform in mass and/or stiffness distribution. Consequently, the changes being planned for NBCC 1985 are designed to move in the direction of using dynamic modal analysis for distributional purposes, when needed, and to rely on the static approach completely for the calculation of the value of the base shear. With this in mind, three specific changes are planned for NBCC 1985:

1) Dynamic analysis is to be removed as a specific option for determining the value of the total lateral seismic force V.

2) The distribution of the total lateral seismic force along the height of the building may be determined by dynamic analysis, as an alternate method to the static approach currently specified in Sentence 4.1.9.1 (12) of NBCC 1980 (7).

3) Dynamic analysis is required for the determination of the torsional effects for buildings in which the locus of the mass centers and the locus of the centers of stiffness do not lie approximately on vertical lines.

This last change will be discussed in more detail in the following section.

TORSIONAL EFFECTS

The provisions to account for torsional effects in NBCC 1980 incude a formula for the computation of structural eccentricity, which is given below:

$$e = \sum_{i=x}^{N} F_i e_{ix} / [\sum_{i=x}^{N} F_i] \qquad (6)$$

where F_i = lateral force applied at floor i
e_{ix} = distance between the centre of mass at floor i and the centre of rigidity at floor x, and
N = no. of floors

The design eccentricity is then computed by whichever of the following expressions provides the greater stress

$$e_x = 1.5e + 0.05D \quad \text{or} \qquad (7a)$$

$$e_x = 0.5e - 0.05D \qquad (7b)$$

where D = the plan dimension of the building parallel to the applied forces.

The torsional moment distribution which is to be applied simultaneously with the lateral forces is given by

$$M_{tx} = (V - \sum_{i=1}^{x-1} F_i)\, e_x \qquad (8)$$

= interstorey shear at floor x multiplied by the design eccentricity at floor x.

If the design eccentricity exceeds 0.25D, the adverse effects of torsion, as computed above, are to be doubled, or alternatively, torsional effects are to be determined by a dynamic analysis.

A critical evaluation of the NBCC 1980 torsional provisions was given by Tso and Meng (12). It was shown that for regular, asymmetrical buildings, i.e., buildings having centres of mass and centres of rigidity of the floors lying along two vertical lines, the NBCC 1980 torsional provisions are generally applicable. Inaccuracies of the code provisions arise in two circumstances. The code is nonconservative when the structural eccentricity is small and when the torsional and lateral frequencies are close to one another (i.e., modal coupling occurs). The code is overly conservative when the structural eccentricity is large because NBCC 1980 requires the doubling of the torsional effect for design.

Due to the difficulty in defining the centers of rigidity for irregular buildings, there is no generally accepted procedure to estimate the quantities e_{ix} in equation (6). Using the traditional procedure of calculating the center of rigidity, based on the floor plans of each floor, equation (6) was applied to buildings with eccentric setbacks. It was found that the torsional moments in the setback portions were grossly underestimated (12).

As a result of this study, it was agreed that there should be significant changes to the torsional provisions. The planned changes are summarized below and are discussed in detail by Tso (13).

First, it is planned to eliminate the formula given by equation (6). The definition of e as the "distance between the location of the resultant of all forces at and above the level being considered and the centre of rigidity at the level being considered" is deemed to be sufficient for buildings which have relatively constant eccentricity over the height of the building.

Second, the accidental eccentricity in equations (7a) and (7b) is increased from 0.05D to 0.10D. This will improve the situation for small eccentricity. Third, the provision for doubling torsional effects in case of high eccentricity is to be removed. Figure 3 shows the current and proposed provisions in graphical form. This figure also includes the results of an analytical study by Dempsey and Tso (14) for structures having different aspect ratios. While there are still situations in which the proposed eccentricity is lower than that which is determined by dynamic analysis, it is clearly an improvement over the current provisions. At the same time, the provisions retain their basic format and simplicity. It should be noted that the adoption of these provisions will result in a design eccentricity which is similar to that used in the Mexican Code (15).

As noted in the section on Dynamic Analysis it will be required that dynamic analysis be used when the locus of the mass centers and the locus of the centers of stiffness of the different floors do not lie approximately on vertical lines. The reason for this is that the studies already mentioned (12)(13)(14) have shown that the static provisions, with the proposed adjustments, are generally satisfactory when the eccentricity is reasonably constant over the height of the building. When this is not the case, or the eccentricity at each floor is ill-defined, there is no simple static formula for torsional effects which can ensure that these effects are adequately included in the analysis process. Consequently, it is necessary to require that a dynamic analysis be done to determine the torsional effects.

SUMMARY

The authors have attempted to describe the primary changes planned to be introduced in the seismic provisions of NBCC 1985. These include improvements in the seismic risk distribution across the country, improvements in seismic load calculation by incorporating both ground acceleration and velocity considerations, clarification of the role of dynamic analysis and improvements in the calculation of torsional effects.

The main effects on the overall seismic loading are due to signficant changes in the geographical distribution of seismic risk throughout the country, as seen in Figure 2. Otherwise, designers will see very little in the way of change in the level of seismic loading. The code retains its basic simplicity for regular structures but will require additional calculations for irregular structures, as seen in the description of the torsional provisions given above.

It is clear that this is not the "final word" in seismic code provisions. Some further directions which are seen as being desirable are being explored by CANCEE and are described in a subsequent paper (16).

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Table 1

Definition of Seismic Zones

Seismic Zone Z_a, Z_v	Range of Peak Acceleration and Velocity in g and m/s, respectively	Zonal Ratio
0	<.04	0
1	.04 to <.08	0.05
2	.08 to <.11	0.10
3	.11 to <.16	0.15
4	.16 to <.23	0.20
5	.23 to <.32	0.30
6	≥.32	0.40

Table 2

Ground Motion Parameters and Seismic Zones

for Selected Canadian Cities

City	PHA* g	PHV* m/s	PHA/PHV	Proposed Zones Z_a	Z_v	1980 NBCC Zone
Inuvik	.060	.083	0.72	1	2	3
Prince Rupert	.13	.27	0.48	3	5	3
Victoria	.28	.26	1.08	5	5	3
Vancouver	.21	.21	1.00	4	4	3
Calgary	.019	.040	0.48	0	1	0
Toronto	.056	.038	1.47	1	0	1
Ottawa	.20	.098	2.04	4	2	2
Montreal	.18	.097	1.86	4	2	2
Quebec City	.19	.14	1.36	4	3	3
Fredericton	.096	.066	1.45	2	1	2
Halifax	.056	.056	1.00	1	1	1
St. John's	.054	.052	1.04	1	1	2

*PHA (peak horizontal acceleration) and PHV (peak horizontal velocity) computed at a 10% probability of exceedance in 50 years.

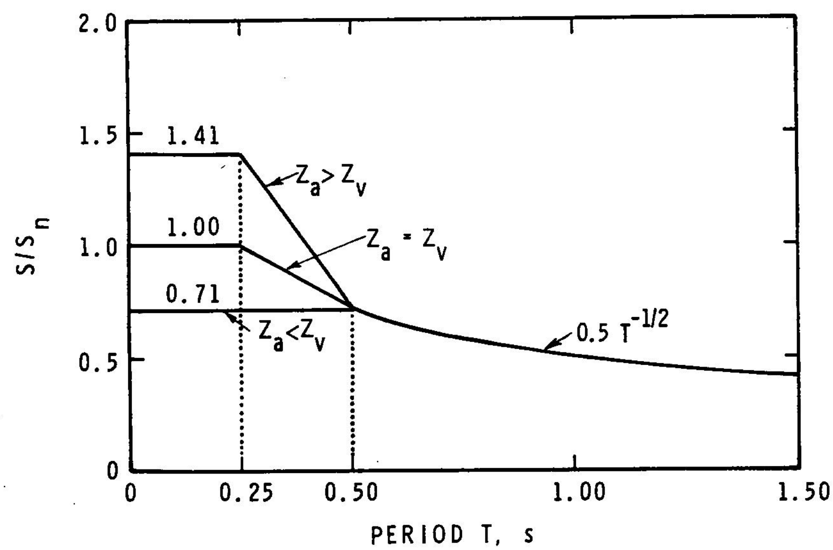

FIGURE 1 Proposed Seismic Response Factor

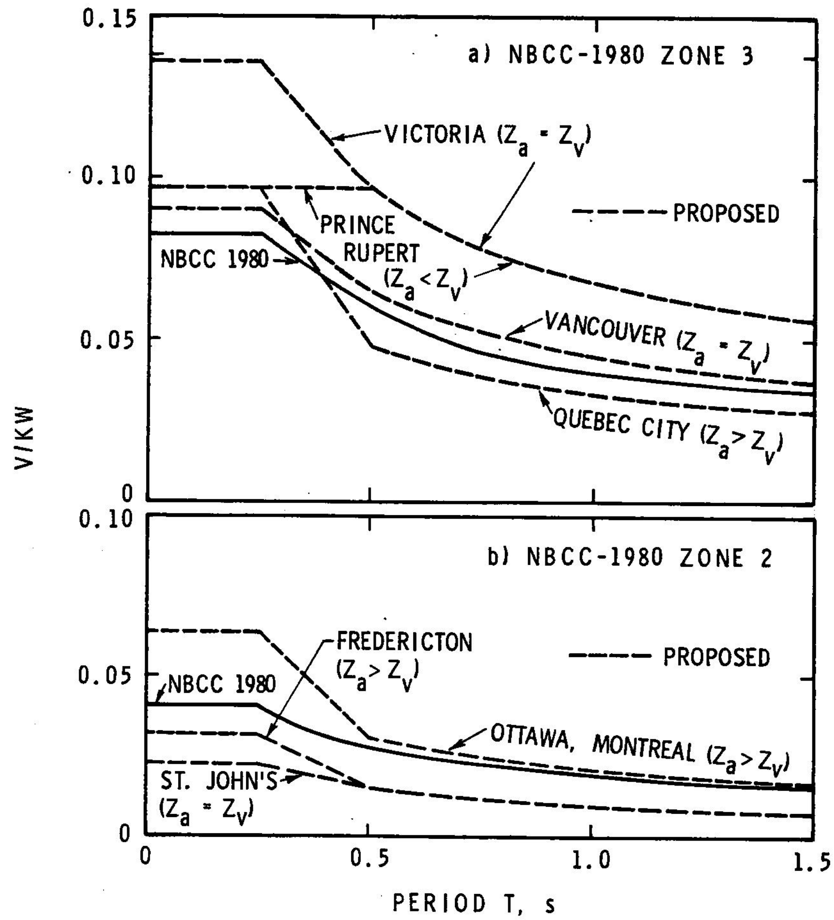

FIGURE 2 Comparison of Proposed Normalized Base Shear for Selected Cities in Zones 2 and 3 of NBCC 1980

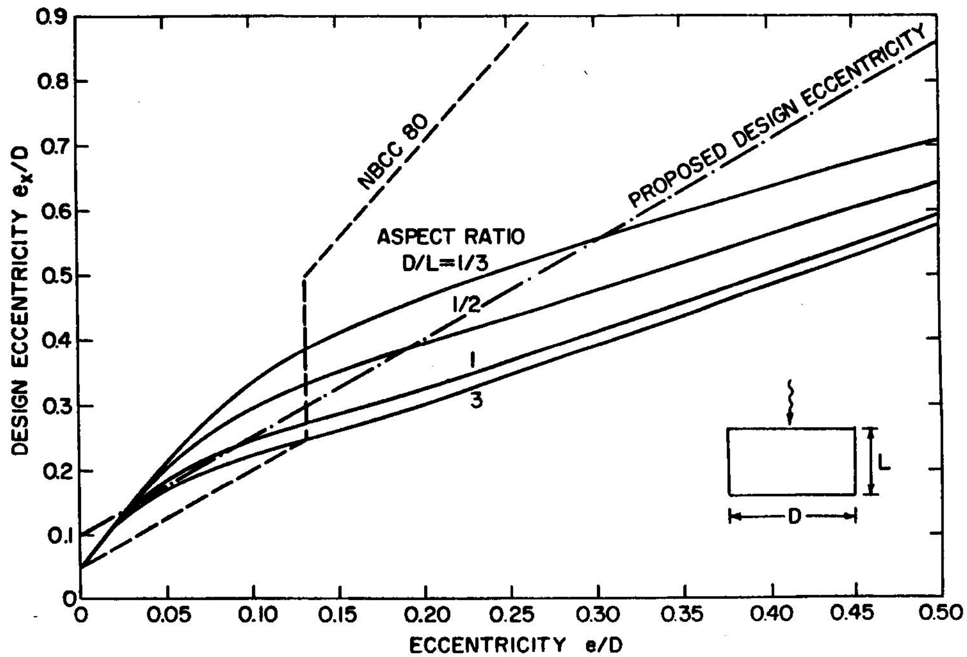

FIGURE 3 Proposed and NBCC 1980 Design Eccentricities Compared With Those Determined by Dynamic Analysis