Evaluation of earthquake design requirements on the basis of a site specific investigation

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ABSTRACT: This paper describes a methodology for determining seismic design requirements for a specific geographical location. This methodology involves the selection of an ensemble of strong ground motions which have similar seismo-tectonic characteristics (e.g. magnitude and epicentral distance) to those which dominate the seismic risk at the particular site. This ensemble of records is used to make statistical evaluations of the design requirements, i.e. response spectra and base shear of typical multi-degree of freedom structures. The paper illustrates this methodology by application to the city of Ottawa. Results are compared with the provisions of the 1985 edition of the National Building Code of Canada. It is concluded that a site specific evaluation of seismic design requirements provides important information which cannot be determined from the normal building code design provisions.

1 INTRODUCTION

Seismic design requirements are typically stated in two parts:

a) specification of one or more parameters to define the seismic risk at a specific site or location, and

b) specification of a process to use the seismic risk parameters (and other information concerning the type of structure, foundation, etc.) to determine the actual forces (e.g. base shear) for which the structure is to be designed.

In Canada, the application of these requirements for the design of building structures is specified in the National Building Code of Canada (NBCC 1985) (Associate Committee on the National Building Code 1980). Part a) is satisfied by the specification of zonal velocity and acceleration ratios, "a" and "v", which correspond to the horizontal ground accelerations and velocities respectively. These zonal ratios are based on probabilistic seismic ground motion maps of Canada (Heidebrecht et al. 1983) which define these parameters, at a probability of exceedance of 10% in 50 years, in terms of velocity-related and acceleration-related seismic zones, Z_v and Z_a.

$Z_a = Z_v$ when a, expressed as a decimal percentage of g, is numerically equal to v, expressed as a ratio to 1 m/s. When $a > v$, then $Z_a > Z_v$; similarly $Z_a < Z_v$ corresponds to $a < v$.

Part b) is specified in NBCC 1985 by a simple formula for the base shear, V, which includes the zonal velocity ratio "v" and a seismic response factor S. This latter factor represents the amplification of the seismic ground motion and is a function of the fundamental natural period, T, of the structure. It is independent of the actual value of the ground motion parameters, although it is defined in terms of the relative values of the seismic zones Z_a and Z_v. The Supplement to NBCC 1985 also defines a response spectrum which can be used for dynamic analysis, e.g. to determine the vertical distribution of lateral seismic forces in non-uniform structures, or to determine the torsional effects when the mass-stiffness eccentricity is not uniform throughout the height of the building. This design spectrum has a constant shape and is simply scaled according to the value of "v".

The building code approach, as described above, treats parts a) and b) of the design requirements as being essentially independent. While NBCC 1985 includes a limited dependence of the seismic response factor S on the relative values of zonal velocity and acceleration, most seismic codes provide a single function, equivalent to S, which is independent of the actual value of the seismic ground motion. In reality, the characteristics of earthquakes which influence structural response depend very much on both the level of seismic motion and the seismo-tectonic conditions, e.g. magnitude of earthquake and epicentral distance to the site in question.

The assumption of the independence of the seismic risk and the structural response process may be satisfactory for normal building structures, but is not likely to be adequate for the design of more critical structures. It is the primary objective of this paper to describe a methodology to determine site specific seismic design requirements and to illustrate the application of these requirements for one Canadian location, i.e. the city of Ottawa. A subsidiary objective is to use this methodology to evaluate the suitability of

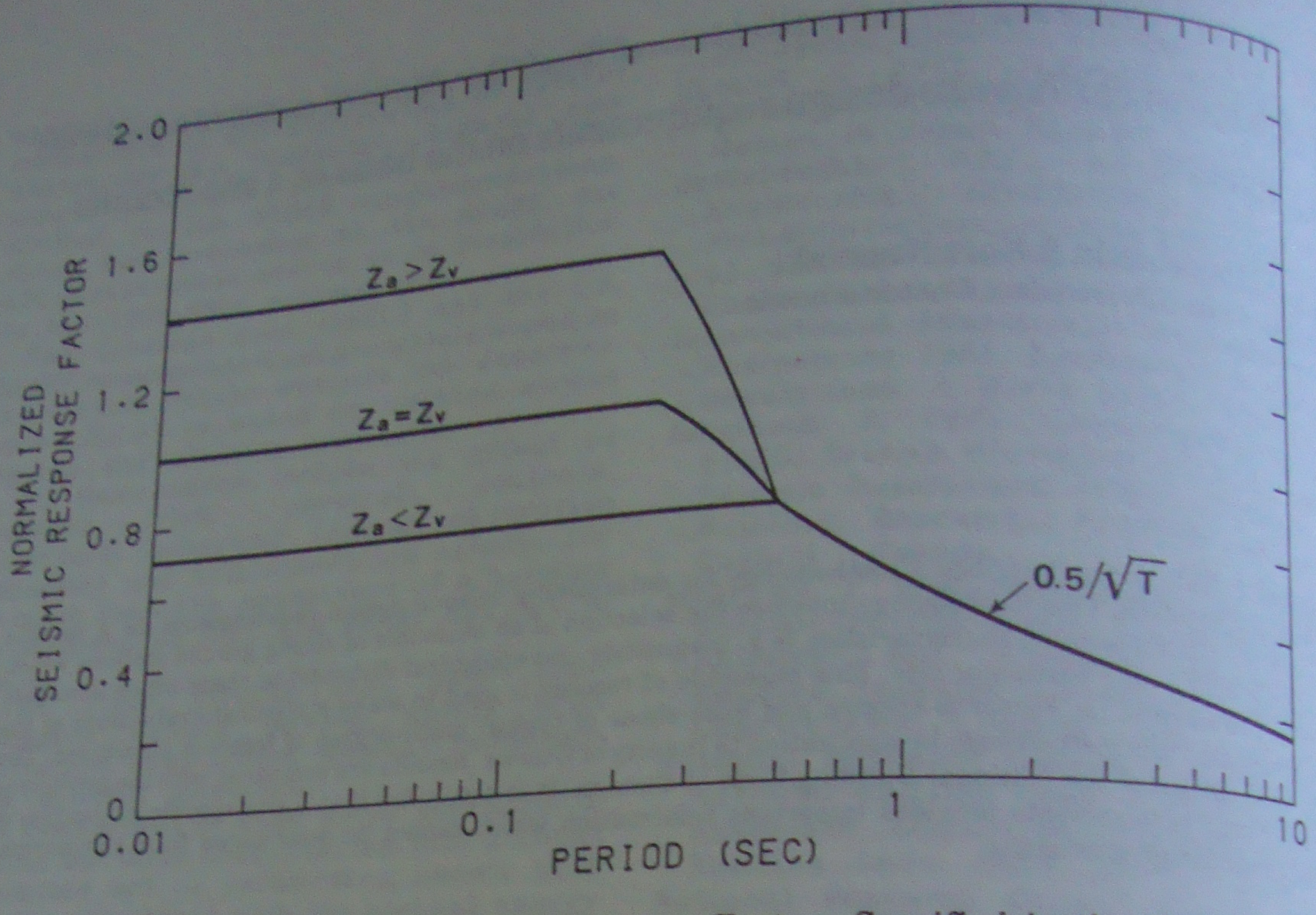

Fig. 1 Normalized Seismic Response Factors Specified in the National Building Code of Canada, 1985

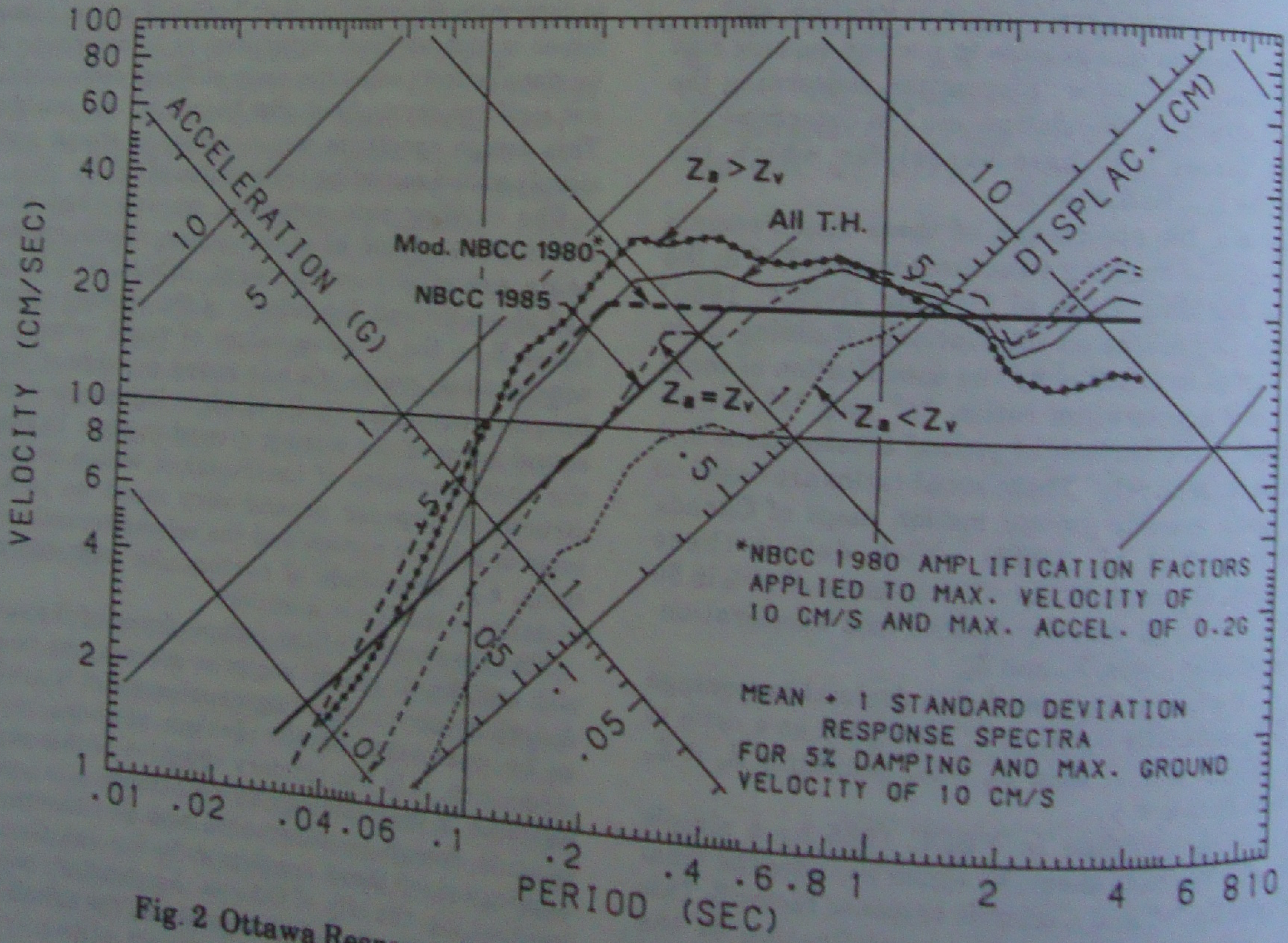

Fig. 2 Ottawa Response Spectra for Different Zonal Combinations

the process used in the building code approach, with particular reference to the S factor and the response spectrum specified in NBCC 1985.

2 METHODOLOGY FOR DEVELOPMENT OF SITE SPECIFIC DESIGN REQUIREMENTS

As indicated above, the seismo-tectonic environment influences both the seismic risk and the characteristics which affect the structural response to earthquakes which are felt at a particular site. The seismic risk calculations yield the values of the peak horizontal ground accelerations and velocities, designated here as PHA and PHV respectively, for a specific site at a specified probability of exceedance (Basham et al. 1985). Because these calculations are based on the assessment of magnitude-recurrence relationships for particular seismo-tectonic regions and on appropriate attenuation laws, it is possible to determine the magnitudes and epicentral distances for the seismo-tectonic regions which make dominant contributions to the ground motion at the specified probability of exceedance. This information has been provided for a number of Canadian cities by Basham (1983).

A data base of strong seismic ground motions (Elop and Heidebrecht 1986) is used to select an ensemble of actual strong motion records which were obtained at combinations of epicentral distance (R) and magnitude (M) which are similar to those which make a dominant contribution to the ground motion in the seismic risk investigation, as outlined above. A range of both M and R in the neighbourhood of the combination(s) of M and R which make up the dominant contribution are used to search the data base. It is important that these records be on "rock" in order to eliminate the effects of local site amplification on the characteristics of the strong motion records. It is also important that the records have a minimum intensity to qualify as "strong" seismic ground motions; a minimum peak acceleration of 0.025 g is used in this study. An ensemble of at least 10 time histories (TH) is desirable in order to do a statistical analysis of the results; however larger ensembles are preferred if the data is available.

The ensemble of TH which simulate the seismo-tectonic conditions at a particular site (referred to here as the Site Ensemble Time Histories, SETH) are then used as the excitations for a subsequent investigation of the design requirements for that site. Regardless of the actual intensity of the recorded TH's, each TH is scaled to the same peak velocity "v". In this study, the scaled "v" is that value which corresponds to the zonal velocity ratio specified in NBCC 1985. This allows for easier comparision with the design requirements specified in NBCC 1985. Velocity scaling is used, in preference to acceleration scaling, because most engineered structures have fundamental periods which are in the velocity-amplification region of the spectrum, i.e. $T > 0.3$ to 0.4 s.

The investigation of design requirements includes two distinct components: a) the determination of response spectra, and b) the determination of the response of typical elastic multi-degree of freedom structures. In each case, a level of damping is chosen which is representative of that which would be expected during the response to moderate or strong seismic ground motion. In this study, damping at 5% of critical is used throughout. This is also the level of damping which is used in the response spectrum specified in NBCC 1985.

For both components, the mean plus one standard deviation level (M+SD) response parameters for the SETH are used to represent the "design level" response. Design spectra are commonly specified at this level (Rosenblueth 1980) in order to ensure that there is a relatively small probability that the response will be above the specified design level. For a normally distributed set of results, the M+SD level corresponds to 84.1% of all responses being below the specified level.

Uniform wall and frame structures are used to represent typical elastic multi-degree of freedom structures. In this context, wall structures are flexural cantilevers and frame structures are shear cantilevers. These two types of structural systems represent the two extremes of dynamic characteristics of structural systems. Uniform flexural cantilevers have the lowest ratios of higher to fundamental periods whereas uniform shear cantilevers have the highest ratios. Similarly, uniform flexural cantilvers have the lowest fundamental mode participation factor whereas uniform shear cantilevers have the highest. Five modes are used to determine the dynamic response of these systems. Modal responses are computed by a numerical integration (linear acceleration method) time-history analysis and are then combined to determine the total response time-history. Base shear is used as the structural response parameter in this paper.

The base shears of elastic multi-degree of freedom structural systems having a range of fundamental natural periods are expressed in terms of seismic response factor (S) spectra as defined in NBCC 1985. This paper uses a normalized version of the S factor specified in NBCC 1985, as outlined below. The base shear V in NBCC 1985 is specified as follows:

$$V = v\,S\,K\,I\,F\,W \qquad [1]$$

in which

v = zonal velocity ratio
S = seismic response factor
K = structural system coefficient
I = importance factor
F = foundation factor, and
W = dead load.

For convenience, the above equation is modified in this study as follows:

$$V = 0.44\,v\,S^{*}\,K\,I\,F\,W \qquad [2]$$

in which the normalized seismic response factor S^{*} is defined as

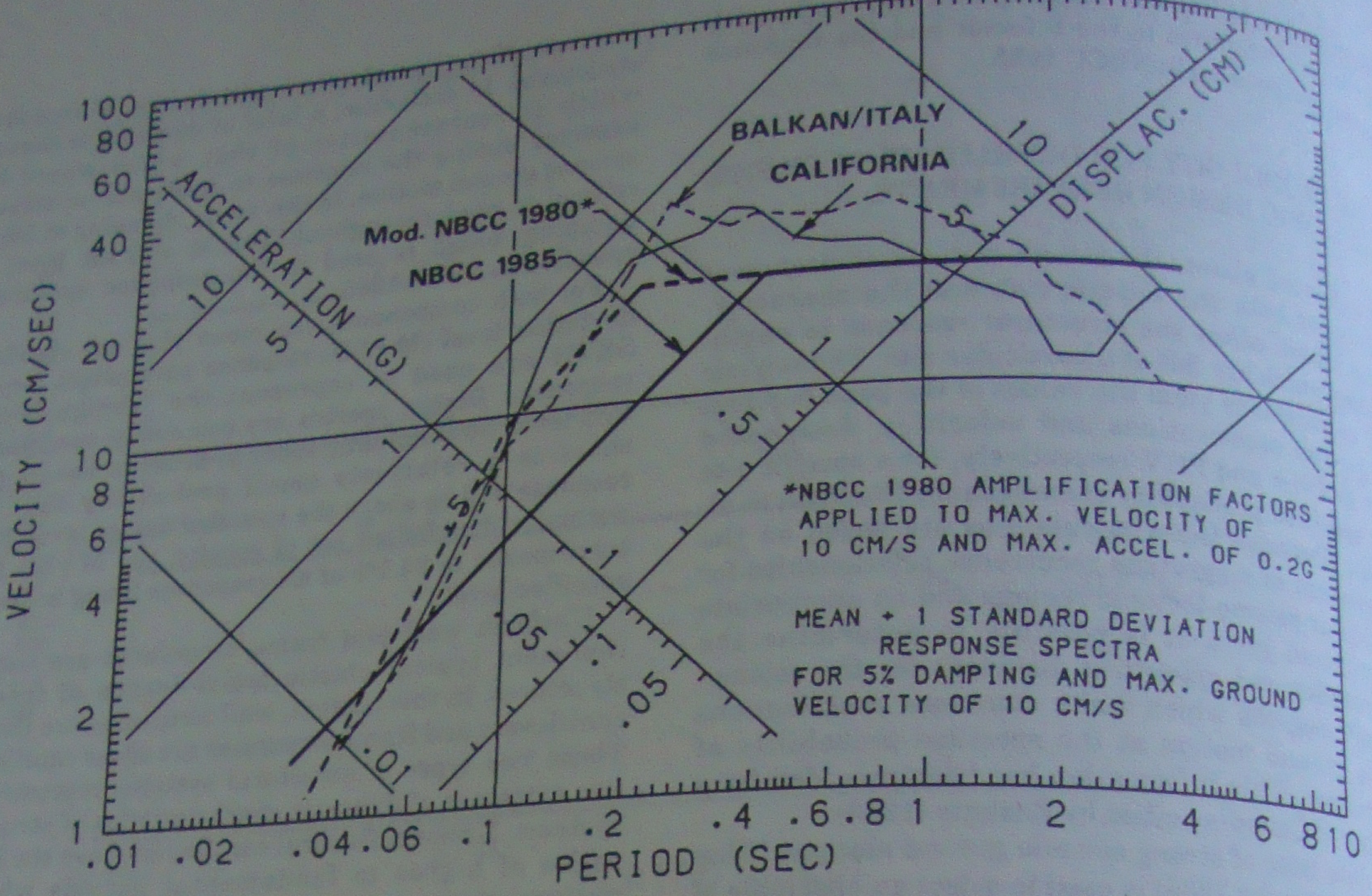

Fig. 3 Ottawa Response Spectra Based on Ensembles from Different Geographical Sources Regions

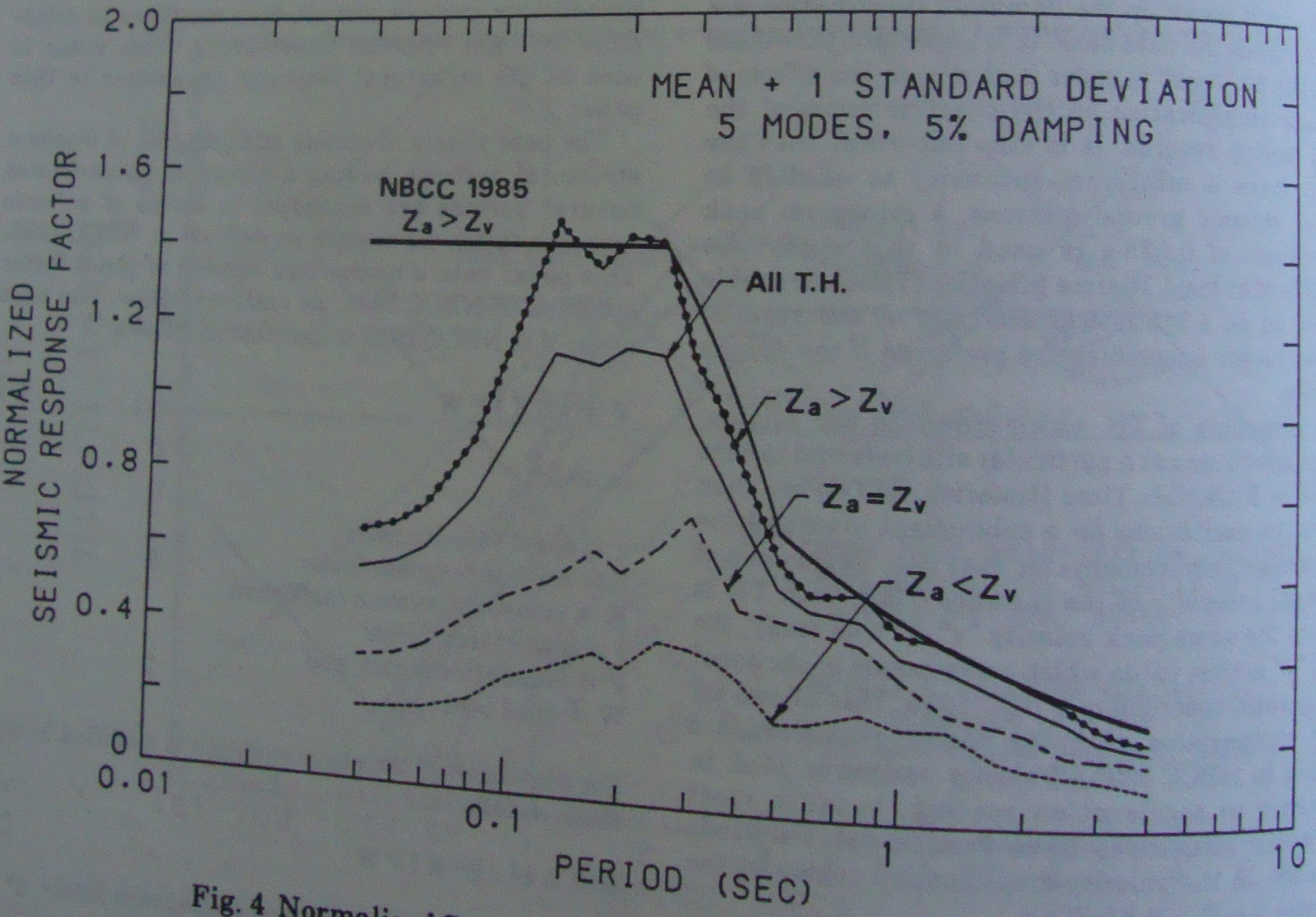

Fig. 4 Normalized Seismic Response Factors for Wall Structures

$$S^* = S/0.44 \tag{3}$$

The reason for doing so is that S^* has a maximum value of 1.00 for $Z_a = Z_v$. This allows for easier comparisons of S^* (as in NBCC 1985) and SRF (as determined from the dynamic analysis of frame and wall systems). Fig. 1 illustrates the S^* spectra for the three combinations: $Z_a < Z_v$, $Z_a = Z_v$, and $Z_a > Z_v$.

In order to calculate the dynamic SRF, it is first necessary to determine the equivalent elastic base shear condition in the NBCC 1985 base shear. From other studies (Rainer 1986), it has been determined that the factored base shear (using a load factor of 1.5 applied as a multiplied to V) should be elastic when the factor K is approximately equal to 5. Using unit values for I and F, which represent the case of structures of normal importance on rock or stiff soil foundations, this results in the elastic base shear V_e as follows:

$$V_e = 3.3\, v\, S^*\, W \tag{4}$$

which can be rewritten in the form

$$S^* = V_e/3.3\, v\, W \tag{5}$$

If the dynamic base shear V_d is substituted into the above, then the dynamically determined SRF can be obtained as follows

$$SRF = V_d/3.3\, v\, W \tag{6}$$

The SRF is independent of the actual value of the peak ground velocity "v", provided that the dynamic response V_d is computed for the same value of "v" as is used in the denominator of eq. [6].

In this study, the foregoing methodology is illustrated by application to the city of Ottawa. The response spectrum obtained on the basis of the site specific investigation is compared with that specified in NBCC 1985. Comparison is also made with a design spectrum created using the velocity and acceleration amplification factors specified in NBCC 1980 (Associate Committee on the National Building Code 1980). The SRF spectrum is also compared with that specified in NBCC 1985, i.e for $Z_a > Z_v$.

3 SELECTION OF SITE ENSEMBLES OF TIME HISTORIES

Table 1 summarizes the seismic risk parameters and the resulting Site Ensembles of Time Histories (SETH) for Ottawa. As can be seen from this table, the dominant contributions to the ground motions are slightly different for PHA and PHV. Also, in the case of velocity,there are two combinations of M and R which make up the dominant contribution. This is because the seismicity is due to several independent earthquake source zones. The Ottawa seismicity is such that 75% of the dominant contribution is due to an earthquake of M = 6 at an R of 70 km; this arises from the Western Quebec (WQU) seismic source zone. Approximately 20% of the dominant contribution is due to an M 6.9 earthquake at an R of 140 km; this arises from the Charlevoix (CHV) seismic source zone. Detailed descriptions of the seismic source zones used in calculating the seismic risk in Canada are given by Basham et al. (1982).

Table 1. Summary of seismic risk parameters and site ensemble selection for Ottawa.

Description		Item	Value
10% in 50 yr. motions		PHV (m/s)	0.10
		PHA (g)	0.20
NBCC Seismic Zoning:		Z_v	2
		v	0.10
		Z_a	4
		a	0.20
Dominant Contributions to Ground Motions at Probability of Exceedance of 10% in 50 yrs. (Basham 1983)	to PHV	%	75
		M	6.6
		R(km)	70
		%	20
		M	6.9
		R(km)	140
	to PHA	%	95
		M	6.6
		R(km)	60
Ranges Used to Select Site Ensembles of Time Histories (SETH)		M	6.0–7.0
		R(km)	40–80
Total No. of T.H.			56
Mean Values		M	6.4
		R(km)	54.9
Distribution of T.H. by Geographical Source Region		California	38
		Balkan/Italy	18
Distribution of T.H by Seismic Zonal Combination*		$Z_a < Z_v$	11
		$Z_a = Z_v$	18
		$Z_a > Z_v$	27

*$Z_a < Z_v$ corresponds to $A/V < 0.8$
$Z_a = Z_v$ corresponds to $0.8 =< A/V <= 1.25$
$Za > Zv$ corresponds to $A/V > 1.25$

Because there are several combinations of M and R making up the dominant contributions, it was necessary (for the purpose of selecting the SETH from the data base) to choose ranges of M and R which covered the most significant of the dominant contributions. In the Ottawa case, the 20% contribution due to M of 6.9 at an R of 140 km was neglected in order to have a

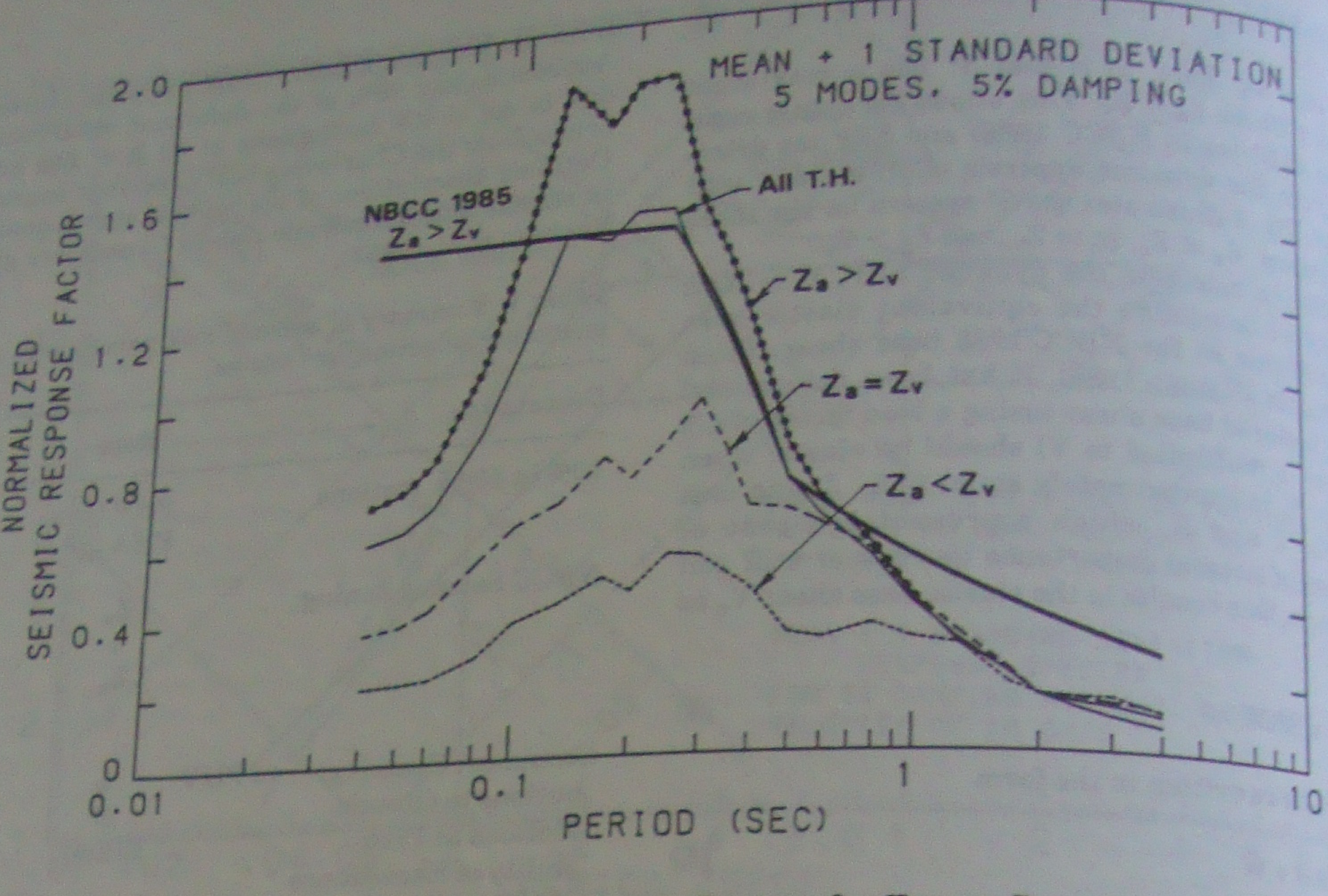

Fig. 5 Normalized Seismic Respnse Factors for Frame Structures

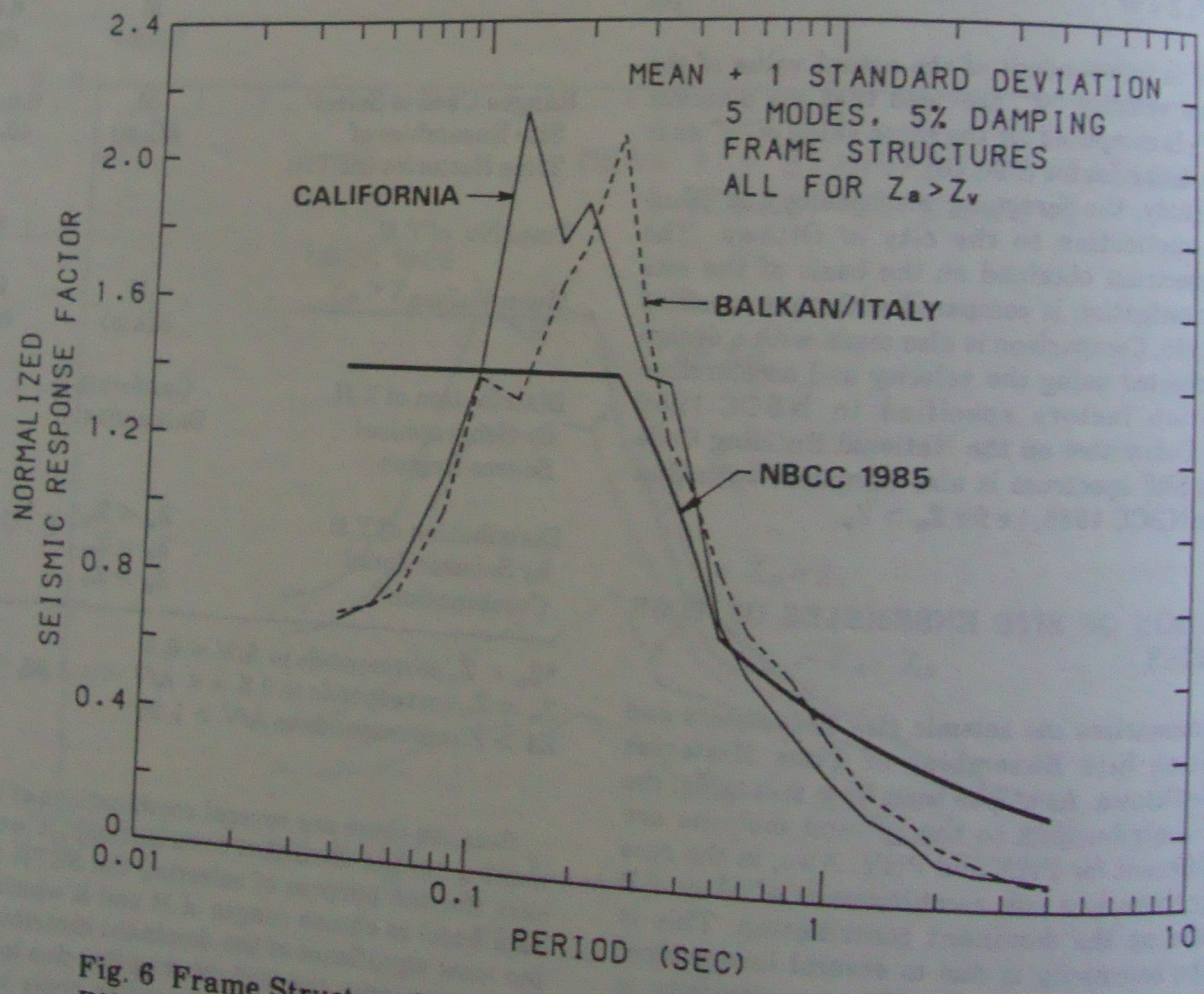

Fig. 6 Frame Structure Seismic Response Factors Based on Ensembles from Different Geographical Regions

more concentrated range of those two parameters, i.e. M from 6.0 to 7.0 and R from 40 to 80 (inclusive in both instances).

Table 1 also summarizes the characteristics of the SETH selected to simulate the Ottawa seismo-tectonic conditions. The mean values of M and R give some indication of how closely the SETH match the combinations of M and R which make a dominant contribution to the ground motion. Since the SETH for Ottawa are dominated by records from California (in particular those obtained during the M 6.4 San Fernando 1971 event), it is not surprising that the mean value of M is 6.4. While the California records dominate, it should be noted that the SETH has significant number of TH from other geographical regions; these are from the Balkan and northern Italy. The Balkan and northern Italian records were recorded in Yugoslavia from earthquakes occurring in Greece, Italy and Yugoslavia (Petrovski, Naumoski and Stamatovska 1984).

Table 1 also describes the distribution of the A/V ratios of the SETH in terms of the equivalent combinations of Za and Zv. A is the peak acceleration of the record, in g, and V is the peak velocity in m/s. The breakdown of the SETH into those subsets is important because it allows the evaluation of the effect of the A/V ratio on the seismic design requirements at a particular site.

Table 2 gives a more detailed breakdown of the geographical sources and the A/V ratios of these records. A detailed listing of all of the earthquake records used in this study can be obtained from the authors.

Table 2. Detailed descriptions of site ensembles of time histories for Ottawa.

Earthquake	M	No. TH	Zonal Comb.* A	B	C
California Records:					
San Fernando, 1971	6.4	34	9	11	14
Long Beach, 1933	6.3	2	2		
Central Cal., 1962	6.0	2		1	1
California Total		38	11	12	15
Balkan/Italy Records:					
Montenegro, Apr. 1979†	7.0	6		3	3
Montenegro, May 1979†	6.4	2		1	1
Kopaonik, May 1980†	6.1	2		1	1
Friuli, May 1976+	6.2	4		1	3
Friuli, Sept. 1976+	6.1	2			2
Volvi, June 1978#	6.0	2			2
Balkan/Italy Total		18	0	6	12
Total		56	11	18	27

*Zonal Combinations: A: $Z_a < Z_v$
B: $Z_a = Z_v$
C: $Z_a > Z_v$

†Yugoslavia
+Italy
#Greece

4 RESPONSE SPECTRA

The response spectra obtained by statistically analysing the 5% damped spectra for the SETH are shown in Fig. 2; the spectra are scaled to a maximum ground velocity of 0.1m/s corresponding to the zonal velocity ratio "v" = 0.1, as given in Table 1. As indicated previously, these are shown at the M+SD level for the full SETH and also for several subsets. The subsets include the three zonal combination cases (i.e. $Z_a > Z_v$, $Z_a = Z_v$, and $Z_a < Z_v$). Also, Fig. 3 shows a comparison between California and Balkan/Italian records within the $Z_a > Z_v$ subset.

These figures also include the spectrum recommended in NBCC 1985 and a "design spectrum" (DS) which has been obtained by taking the velocity and acceleration amplification factors (AF) given for 5% damping in the NBCC 1980 recommended spectrum (Associate Committee on the Building Code 1980) and applying these to the zonal "v" and "a" values given in Table 1. This DS has been included to enable the results of this site specific investigation to be compared with a spectrum whose shape is dependent only upon the two peak ground motion parameters "v" and "a". It should be noted that the NBCC 1985 spectrum has a spectral shape which is independent of the A/V ratio, i.e. it is scaled only by the zonal "v".

Fig. 2 shows clearly that there is a very real distinction between spectra for the different zonal combinations. The spectrum for the full Ottawa SETH falls between that for $Z_a = Z_v$ and $Z_a > Z_v$. Given that the seismic risk evaluation for Ottawa results in Z_a being two units higher than Z_v, it seems clear that the SETH subset for $Z_a > Z_v$ should be used as the basis for design in Ottawa. Comparing the DS with that spectral subset, it appears that the DS is quite satisfactory in the acceleration amplification region but that it provides inadequate amplification in the velocity region. Increasing the velocity AF from 2 to 3 would provide a satisfactory DS to simulate the conditions in Ottawa. The NBCC 1985 spectrum does not recognize the high A/V ratio in Ottawa and therefore provides inadequate amplification in both the velocity and acceleration region.

Comparing the further SETH subsets for California and Balkan/Italian records (for Za > Zv) shown in Fig. 3 indicates that, while the shapes of the two subset spectra are slightly different, there is no significant difference in the maximum AF between the two cases. It is postulated here that the seismicity in Ottawa is likely to be slightly more like that in the Balkan/Italy region than that in California. Consequently, a DS in which both the velocity and acceleration AFs are equal to 3 would be appropriate.

5 SEISMIC RESPONSE FACTORS

Figs. 4 and 5 show the SRF resulting from the Ottawa SETH for uniform wall and frame structures respectively. Also included on these figures is the normalized seismic response factor S* specified in

NBCC 1985 for $Z_a > Z_v$. These figures show that the SRF for frame structures are larger than those for wall structures for periods of 1 s or less, with the opposite being true for larger periods. For T > 1 s, the wall structure SRF, both for the full SETH and for the various subsets, are enveloped by the S* specified in NBCC 1985. Because of this, and because the largest values of SRF occur at periods below 1 s, subsequent discussion of SRF will be based entirely the results for frame structures.

Fig. 5 indicates that the frame structure SRF for the $Z_a > Z_v$ subset of the SETH reach peaks of approximately 1.85 in the period range $0.1\ s < T < 0.25\ s$; this exceeds the specified S* of 1.41 by about 35%. This indicates that the low period S* will somewhat underestimate the response of frame structures. It should be noted that the zonal A/V for Ottawa (a/v, from Table 1) is 2.0, whereas the S* for $Z_a > Z_v$ implies an A/V of approximately 1.4. Consequently, it is not surprising that structural response in the acceleration amplification region is inadequately predicted by the specified S*. This is due to the fact that Z_a in Ottawa is 2 zones higher than Z_v, whereas the S* for $Z_a > Z_v$ is intended to cover the normal case of Z_a being 1 unit higher than Z_v.

Fig. 6 shows the frame structure SRF for the California and Balkan/Italy subsets (for $Z_a > Z_v$) of the Ottawa SETH. The individual subsets have SRF peaks exceeding 2; however, these peaks occur at different periods. The peak of the Balkan/Italy SRF is at a higher period ($T = \sim 0.25$ s) than that for the California records, which is consistent with the shape of the spectra shown in Fig. 3. The composite effect, as shown in Fig. 5, is to produce an overall $Z_a > Z_v$ SRF which is essentially flat in the period range $0.1\ s < T < 0.25\ s$.

On the basis of the results presented in this section, it is expected that the elastic base shears for frame structures situated in locations with Z_a two zones higher than Z_v will exceed those predicted using S* as specified in NBCC 1985. It is therefore recommended that S* be revised for this zonal combination. A peak value of 2.0 (rather than 1.41) for $T < 0.25$ s, with a linear transition from that period down to the current value of S* at T = 0.5, is recommended as being suitable for frame structures. This peak value is equal to the equivalent A/V ratio for this situation.

6 CONCLUSIONS

On the basis of the investigation detailed in this paper, it is concluded that a site specific investigation of seismic design requirements is important in establishing seismic design requirements which recognize particular seismo-tectonic environments. The methodology proposed in this paper enables the seismo-tectonic environment to be simulated by a judicious use of existing strong motion records.

The illustrated application of this methodology to the city of Ottawa indicates that such an investigation is particularly important when the A/V ratios are expected to be significantly higher than 1, i.e. corresponding to $Z_a > Z_v$ in the seismic zoning system used in NBCC 1985. The results indicate that the seismic response factor specified in NBCC 1985, for the case of Z_a two zones larger than Z_v, are too low for structures having fundamental periods less than 0.25 s.

The results also show that the design spectrum needs to incorporate ground acceleration and velocity as independent parameters with appropriate amplification factors in each case. In establishing a design spectrum for Ottawa, the velocity amplification factors (for 5% damping) commonly used in constructing design spectra are found to be too low; increases from 2 to 3 would be appropriate.

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