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APPLICATION OF INELASTIC UNIFORM HAZARD SPECTRA IN SEISMIC DESIGN

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ABSTRACT

Elastic spectra for obtaining the design earthquake forces are currently derived by amplifying the peak ground motion bounds for the site under consideration. Recognizing that considerable error may be involved in this process, methods have been developed for deriving the linear elastic spectral values for a given site and for a given hazard level. Such spectra are called Uniform Hazard Spectra (UHS). The UHS directly provide the design elastic base shear. This shear must be adjusted to account for the expected inelastic response and the ductility capacity of the building structure. A method of using the uniform hazard spectral values to obtain design response spectral curves for different values of ductility is presented here. The method uses two spectral values obtained from the hazard maps, the peak spectral acceleration for the site and the spectral acceleration corresponding to a period of 0.5 s. Empirical expressions are developed to represent the design response spectra. It is shown that by using inelastic spectral accelerations rather than the elastic spectral values in association with a reduction factor, the new method provides a more reliable estimate of the design forces.

KEYWORDS

Uniform hazard spectra, Ductility ratio, Inelastic spectra, Seismic design forces, Force modification factor.

INTRODUCTION

The objective of the earthquake-resistant design requirements of National Building Code of Canada (NBCC) (Canadian Commission, 1995) and of the US Uniform Building Code (UBC) (International, 1991) is to ensure that buildings that are designed to satisfy the requirements do not collapse when subjected to the design earthquake. The design earthquake is defined as an event that has 10% probability of exceedance in 50 years. The forces that the building is required to resist without collapse are calculated by an equivalent lateral load analysis procedure. In this procedure, the elastic design base shear is obtained from an idealized response spectrum for a multi-degree-of-freedom system with 5% damping. In both NBCC and UBC this spectrum is derived from two ground motion parameters: the peak ground acceleration, and the peak ground velocity for the seismic zone in which the building is located. The spectral shape is, in effect, derived by applying suitable amplification factors to the ground motion bounds.

The derivation of a spectral shape from peak ground motion bounds is based on studies carried out by Newmark and Hall (1982). In these studies, the spectral shapes were obtained by averaging the spectral curves for ground motion records from a few earthquakes. Most of these earthquakes were in the magnitude range of 6 to 7 and had a distance to the source of about 20 km. It is now recognized that the idealized spectrum derived as above may deviate significantly from the true spectrum for a site, and the error can be as large as 300% (Atkinson, 1991). Spectral shape for a site is governed by the magnitudes and source distances of earthquakes that contribute most significantly to the hazard. If for a given site these parameters are different from those used in deriving the standard spectral shapes, the site specific spectra deviate considerably from the standard spectra.

Since the mid 1970s, methodologies have become available for deriving linear elastic spectra for a given site and for a given hazard level, say 10% probability of exceedance in 50 years. 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. Because UHS provide response parameters that can be used directly in estimating the design earthquake forces, they are preferable to the spectra derived indirectly from peak ground motion bounds.

The Geological Survey of Canada is producing a suite of new seismic hazard maps for Canada, to be released for trial use (Adams et al, 1995). The hazard maps incorporate new geological and tectonic information, as well as earthquake data collected since 1985 when the last such maps were produced. The maps will provide spectral accelerations for a 5% damped elastic SDOF system at several values of the period and for a uniform probability of exceedance of 10% in 50 years. It is expected that the new hazard maps will form the basis of the earthquake design provisions of NBCC 2000. However, before the UHS can be used in design, the site specific response values obtained from such spectra must first be incorporated into a seismic zoning map for the country. The UHS typically provide spectral ordinates for a number of different vibration periods. On the other hand, zoning maps for use in design should be based on a limited number of parameters, perhaps no more than 2.

In United States, the recent version of NEHRP recommended provisions for seismic design (Building Seismic Safety Council, 1994) provides zoning maps of spectral acceleration values at 0.3 and 1.0 s for trial use in design.

An essential step in the use of UHS in design is the development of a methodology to obtain the design elastic base shear from the information contained in the zoning maps, and the adjustment of elastic base shear to account for the expected inelastic response of the building structure. A methodology for achieving these objectives is presented here.

DESIGN METHODOLOGY

Inelastic Response Spectra

In the interest of economy, most buildings are designed to have a strength that is a fraction of the strength required to resist the forces derived from an elastic response spectrum. This implies that a building structure is expected to undergo inelastic deformation during the design earthquake. For a given structure and earthquake motion, the amount of inelastic deformation depends on the ratio of the strength of the structure to the strength required to keep its response within elastic limit. In general, the smaller is this ratio, the greater is the inelastic deformation. Figure 1 shows the relationship between the displacement of a SDOF structure and the lateral force acting on it during an earthquake. If the structure is elastic, the maximum value of the lateral force or base shear is V_e and the corresponding displacement is Δ_e. The minimum strength required to keep the structure elastic is thus V_e. Assume that the structural strength provided is V_y, with $V_y \leq V_e$. Then if the lateral force-displacement relationship for the structure is elasto-plastic in nature, the structure will yield at a base shear of V_y and will undergo further displacement at a constant value of shear equal to V_y. Let the maximum displacement be denoted by Δ_i. This displacement is comprised of two parts, yield displacement Δ_y, and plastic or inelastic displacement Δ_p. As stated earlier, the smaller is the ratio V_y/V_e, the greater is the inelastic deformation, Δ_p.

Inelastic deformation causes damage to the structure. The level of damage should be kept within the capacity of the structure to sustain such damage without collapse. Various measures have been adopted to assess the damage level and the damage capacity. The simplest of these measures is the so called ductility, defined as the ratio of the total deformation to the yield deformation and denoted by μ. Thus

$$\mu = \frac{\Delta_i}{\Delta_y} \tag{1}$$

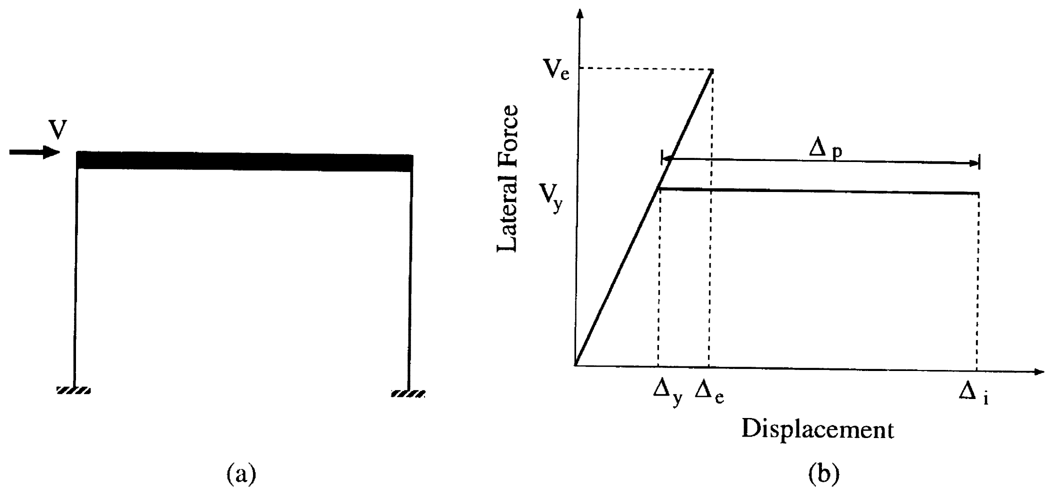

Figure 1: Lateral force-displacement relationship for a single story frame

The objective of the earthquake design is to determine the value of V_y that will limit the ductility demand to within the ductility capacity. In the 1995 NBCC, strength V_y is determined from V_e, the elastic base shear, by applying a reduction factor R to the latter, so that

$$V_y = \frac{V_e}{R} \tag{2}$$

Equation 2 permits the use of elastic response spectra for determining the design strength of an inelastic structure. In the current NBCC, the R values are somehow related to ductility capacity, although engineering judgment has perhaps played a key role in the selection of the specified values. The recommended values of R are independent of the period of the structure. Many researchers (Nassar and Krawinkler, 1991) have, however, shown that R is strongly dependent on the period, and for a given μ, it generally decreases with the period. Factor R is also affected by the nature of earthquake motion, or specifically, the a/v ratio, where a is the peak ground acceleration and v the peak ground velocity. Evidently, there is considerable uncertainty associated with the selection of R.

It is possible through inelastic analysis to directly determine the maximum required strength for a given structure so that the ductility demand on it during a given earthquake is equal to a target ductility. For a given period and damping ratio and a given earthquake motion, such inelastic analysis provides V_y corresponding to a ductility μ. By repeating such calculations for different periods and different ductility ratios, a series of constant ductility inelastic response spectra can be obtained.

Because the use of elastic spectra with reduction factor R to obtain the design strength for an inelastic structure involves considerable uncertainty, it is preferable to obtain the design strength directly from inelastic response spectra produced for a range of values of the ductility μ. It is expected that, in the future, ground motion relations will become available for inelastic spectral values. However, inelastic response spectra can also be derived from the corresponding elastic spectra. Factor R may be considered as the ratio of elastic spectral acceleration for a given period to the corresponding inelastic spectral acceleration associated with a specified inelastic damage level. In the present study ductility μ has been used as the measure of such damage, but other measures such as hysteritic energy absorption, accumulated ductility level etc. can equally well be used. Recent studies (Bazzurro and Cornell, 1994) have shown that the mean value of R, although dependent on the period, is insensitive to the magnitude of earthquake event and distance to the source. This implies that if an ensemble of earthquake records provide an elastic spectrum that matches the UHS for a particular site, the same ensemble can be used to derive the inelastic response spectra for that site.

It should be noted that a UHS is the envelope of responses produced by several different earthquakes with varying magnitude and source distance. Thus the maximum short period spectral values may result from short distance earthquakes, while the long period values may be contributed by more distant earthquakes. In the present study, a suite of 15 high a/v records from McMaster University (Naumoski *et al*, 1988) and another suite of 15 intermediate a/v records from the same source are used to derive two mean elastic response spectra. The high a/v ratio spectral curve is scaled so that the maximum spectral value is equal to the maximum hazard spectral acceleration, S_m, for the given site. The intermediate a/v ratio spectral curve is scaled to have a 0.5 s spectral acceleration value equal to the 0.5 s uniform hazard spectral value $S_a(0.5)$ for the site. The envelope of the two curves provides the elastic response spectrum for the site under consideration. This envelope response spectrum is found to match reasonably with the UHS developed by Canadian Geological Survey for a number of cities spread throughout the country. As an example, the elastic spectrum envelops derived as above are compared with UHS curves for Vancouver and Montreal respectively in Figs. 2a and b. In each case, the match is quite good.

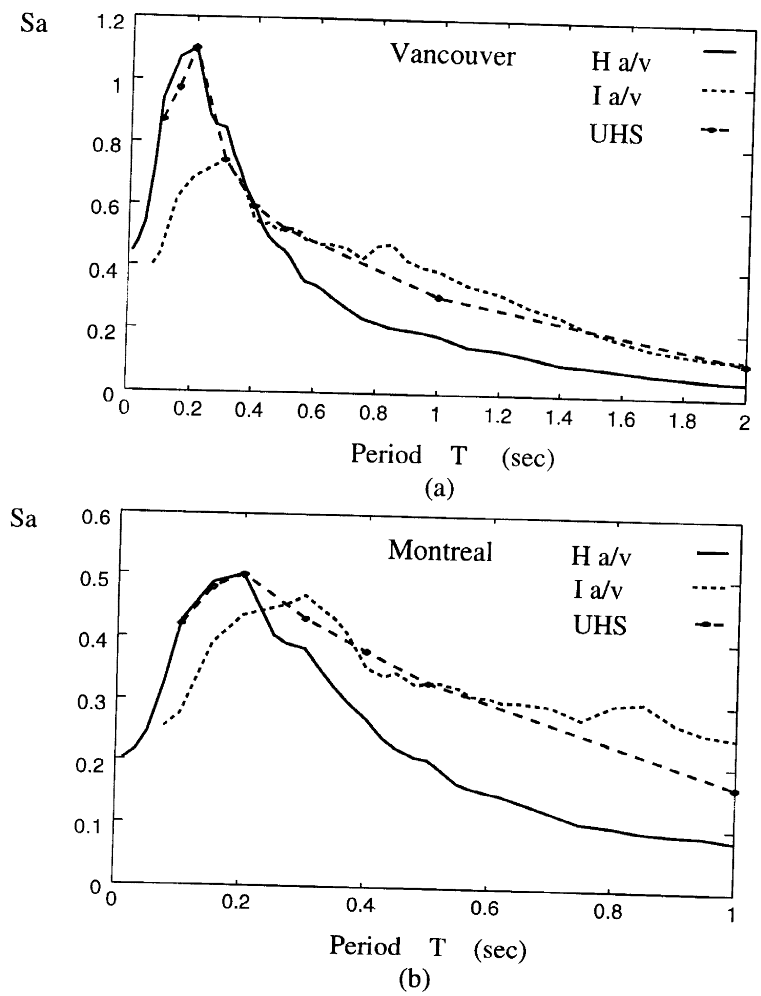

Figure 2: Comparison of scaled mean elastic spectrum curves of high and intermediate a/v ratio records with Uniform Hazard spectra for (a) Vancouver and (b) Montreal

Considering that the suites of records used in the study provide a good representation of the UHS, the same suites are used to derive the inelastic response spectra for six different values of the ductility capacity μ, namely 1, 2, 3, 4, 5 and 6. For use in the design code, curves are fitted to the inelastic spectra derived as above. Using empirical equations for these curves, the inelastic design base shear coefficient can be represented as

$$V = CW \tag{3}$$

$$C = \mathrm{Max}\{C_H S_m, C_L S_a(0.5)\} \leq \gamma S_m \quad \text{for} \quad T < 0.5\mathrm{s} \tag{4a}$$

$$C = C_L S_a(0.5) \quad \text{for} \quad T \geq 0.5\mathrm{s} \tag{4b}$$

where

$$C_H = A - BT \tag{5a}$$

$$C_L = \frac{\alpha}{T^{2/3}} \tag{5b}$$

and parameters γ, A, B and α have the following values

μ	γ	A	B	α
1	1.000	1.460	2.280	0.630
2	0.500	0.716	1.155	0.308
3	0.366	0.457	0.731	0.219
4	0.321	0.369	0.620	0.165
5	0.295	0.339	0.647	0.143
6	0.281	0.322	0.651	0.127

The spectral curves obtained from Eq. 4 are compared in Fig. 3 with the response spectrum envelopes for Vancouver for ductility values, $\mu = 1$ and $\mu = 4$. The match is quite good in each case. It will be observed that the empirical expressions provide spectral values that are conservative in the long period range. As will be discussed later, this characteristics is useful in the design of muti-degree-of-freedom (MDOF) systems.

Figures 4 and 5 show the elastic and inelastic design spectra obtained from Eq. 4 for two locations, Vancouver and Montreal. In parts (a) of the figures, elastic spectra for $\mu = 1$ are compared with the corresponding UHS as well as the provisions of NBCC. Parts (b) and (c) of the figures show the inelastic spectra for ductility values of 1, 2, 3, 4, 5 and 6.

Comparison with Inelastic Spectra Derived by using R

In the current NBCC, the elastic seismic forces that have been reduced by R to account for inelasticity are further reduced by U, often referred to as the overstrength factor (Nassar and Krawinkler, 1991; Tso and Naumoski, 1993). It is implied in this two stage reduction that R provides a measure of the ductility capacity. Thus μ in the new provision can be taken as R of the current NBCC.

It would be of interest to compare the inelastic spectra obtained by the methodology presented here and those obtained by applying the factor R to an elastic spectrum. This has been done for Vancouver and Montreal and the results are shown in Figs. 4 and 5. For a low ductility value of 2, the two set of spectra are almost identical. For higher ductilities and short periods the inelastic spectra obtained by the method presented here are significantly higher than those derived indirectly by using the modification factor R.

Multi-Degree-Of-Freedom Systems

The spectral acceleration curves presented in the previous section are applicable to a SDOF system. The required strength for such a system having a given period and ductility capacity can be obtained by entering the spectral curve for that ductility. In fact, the spectral curve provides the maximum acceleration, S_a, in units of g, which can be interpreted as the strength required per unit weight. The SDOF strength spectra need to be modified before they can be applied to a multistorey building. For this purpose it is useful to compare the strength requirement for a multistorey building with that

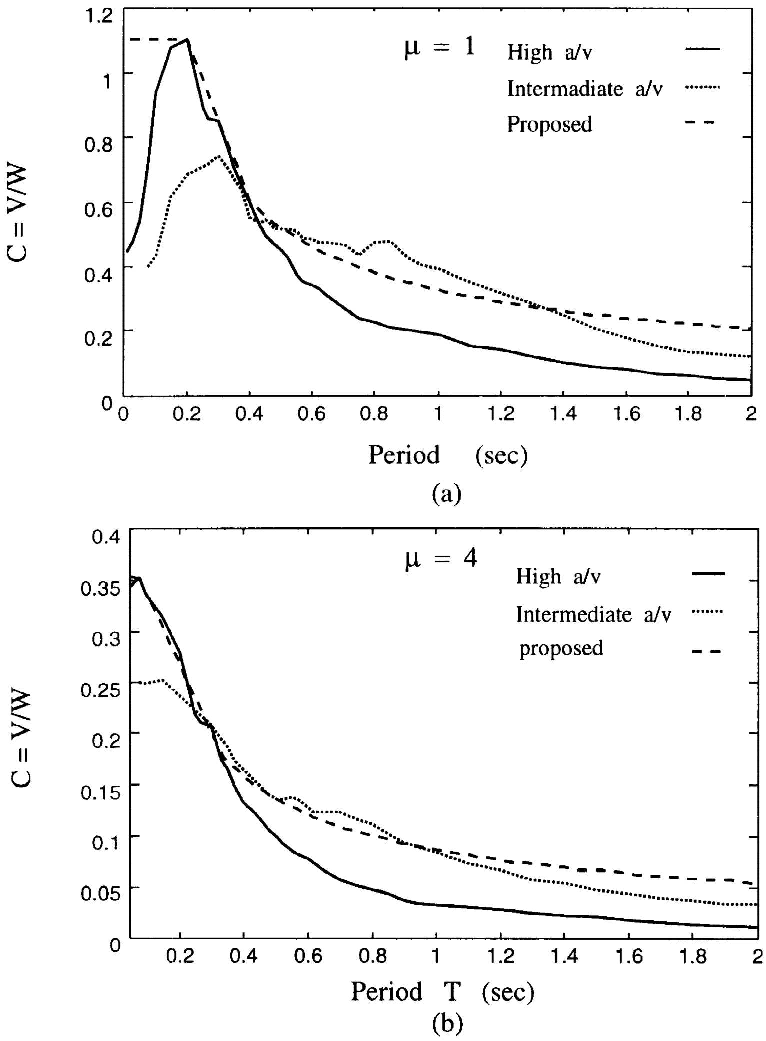

Figure 3: Comparison of the proposed spectral curves with the response spectrum envelopes for Vancouver for two ductilities, (a) $\mu = 1$ and (b) $\mu = 4$.

for an associate SDOF system. The associated SDOF system is defined as one having, in its linear range, the same period and damping as the multistorey building. The weight of the associated SDOF system is equal to the total weight of the multistorey building. The ductility demand in a multistorey building having the same strength (base shear capacity) as the associated SDOF system can be quite different from that in the SDOF system. In fact, ductility requirement varies across the height of the multistorey building and may be smaller or greater than the ductility requirement for the associated SDOF system. The demand is, in general, higher in the lowest and uppermost stories, the first storey usually being the most critical.

It is apparent that if the ductility demand in a multistorey building is to be limited to that in the associated SDOF system, the design strengths of multistorey frames have to be suitably adjusted. It may be possible to achieve the target ductilities by redistribution of the strengths across the height. Such a procedure will, however, be quite complex given that the ductility demands are influenced by many different parameters. The alternative is to increase the design base shear by an appropriate value, keeping the distribution of design strength unchanged.

As stated earlier, the ductility demands in multistorey buildings are influenced by many different parameters in a complicated manner. These parameters include the characteristics of the earthquake motion, the distribution of strength and stiffness of the building structure, and the relative magnitude of gravity loads. However, studies on simplified models of building frames are useful in establishing the general trends and in assessing the strength demands for inelastic multistorey buildings in comparison to those in the associated SDOF systems. Such studies have recently been carried out by Nassar and Krawinkler (1991). They performed inelastic analyses of simplified and idealized multistorey frames

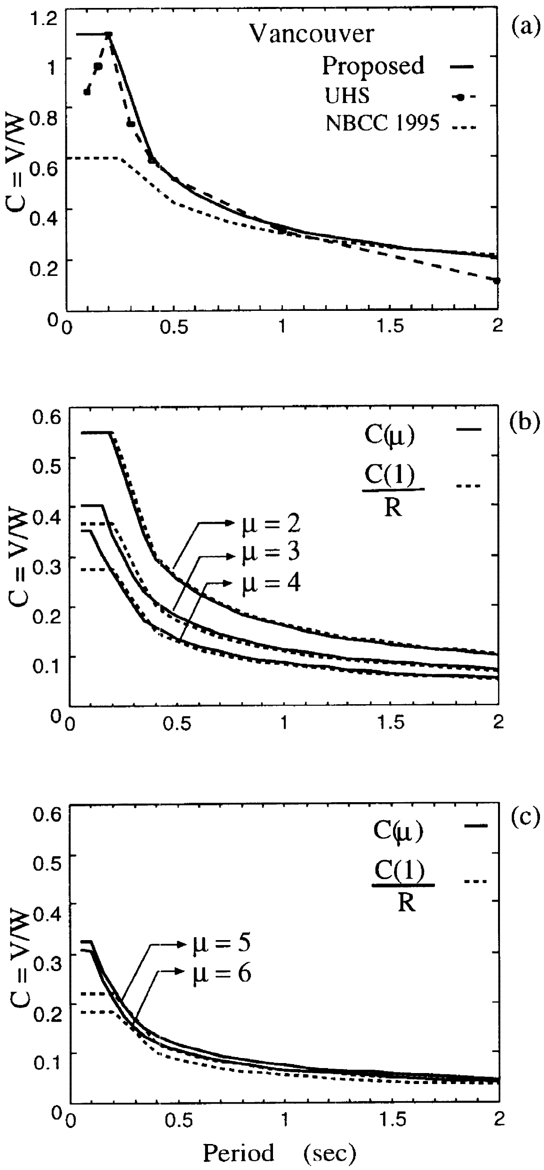

Figure 4: Elastic and inelastic spectra for Vancouver

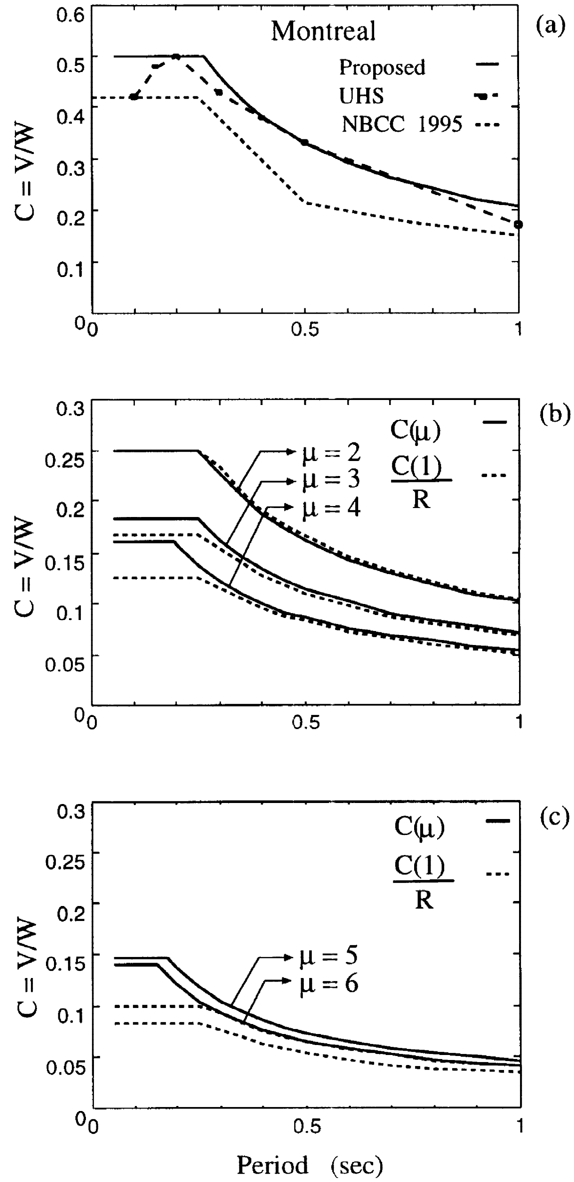

Figure 5: Elastic and inelastic spectra for Montreal

subjected to a series of earthquake ground motions. The results obtained by the two researchers show that the design base shear in the MDOF system needs to be modified in order to limit the storey ductility demands at the base to the prescribed target ductility ratio, which is also the ductility ratio in the associated SDOF system. The ratio of the base shear of MDOF system to that of the SDOF system for identical ductility demands is referred to as the strength ratio. The strength ratio increases with period as well as with ductility. In general, the strength demand is higher for a weak column model than for a weak beam model. Also, strain hardening has a beneficial effect, reducing the amount of extra strength required.

It is apparent that the response spectra for SDOF systems need to be scaled upward before they can be used for a MDOF system. The amount of such scaling should depend on both the ductility and the period. In the interest of maintaining the simplicity of design provisions, it is useful to assess the magnitude of scaling factor for a ductility which can be considered a reasonable upper bound. For this purpose, $\mu = 4$ can be considered as the target ductility, and the strain hardening ratio can be taken to lie between 0 to 10%. For these values, the scale factors for weak column model were found by Nassar and Krawinkler (1991) to be approximately 1.2 for a period of 1.0 s, and 1.5 for a period of 2.0 s. The scale factors for weak beam model were lower.

A comparison of the proposed elastic spectrum curves and the UHS shown in Figs. 4 and 5 shows that the proposed curves are above the UHS in the long period range, and the ratios of the proposed spectral values to the UHS values at periods of 1.0 and 2.0 s are close to 1.2 and 1.5. The proposed spectra can therefore be considered as being appropriate for multistorey buildings.

SUMMARY AND CONCLUSION

The current Canadian and US practice in seismic design is to obtain the earthquake base shear from an elastic spectrum whose shape is related to the peak ground motion bounds for the site under consideration. Realizing that the determination of spectral shape by amplifying the peak ground motion bounds is subject to considerable error, new methodologies have been developed that allow direct determination of the linear elastic spectra for a given site and a given probability of exceedance. Using the new methodology and additional geological evidence and records of ground motion collected since 1985, when the last seismic zoning maps of Canada were produced, the Geological Survey of Canada is developing new seismic hazard maps for Canada. These maps will provide spectral accelerations for a 5% damped elastic SDOF systems at several values of the period. This will allow the construction of elastic acceleration spectra commonly referred to as the Uniform Hazard Spectra.

A design methodology that allows the determination of design base shear from uniform hazard spectral values is presented in this paper. The first step in the development of the proposed methodology is the determination of elastic spectral curves for two sets of selected earthquake records having different a/v ratios, high and intermediate. The records with high a/v ratios are scaled to the maximum spectral acceleration at the site, while the intermediate a/v ratio records are scaled to the site spectral acceleration at a period of 0.5 s. The envelope of the two spectra is shown to closely match the UHS. The same suites of earthquakes are then used to produce the inelastic response spectra for different ductilities. Empirical expressions are developed to represent both the elastic and the inelastic spectra. The proposed expressions are related to just two ground motion parameters, the maximum spectral acceleration and the spectral acceleration at a period of 0.5 s. The empirically obtained elastic spectrum curves are compared with the UHS as well as the current NBCC seismic coefficient curves. A comparison is also made between the inelastic spectra derived by the methodology presented here and those derived by using the reduction factor. As would be expected, in the short period range, the new methodology gives significantly higher design forces. The results presented here show that the use of inelastic spectral curves in place of the elastic curves along with a modification factor provides a more rational method of obtaining the design forces.

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