Ground motion amplification factors for the proposed 2005 edition of the National Building Code of Canada[1]

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Abstract: Foundation factors are used in seismic codes to capture the amplification effects of local soil conditions on ground motions and, hence, on seismic design forces. Recent developments in categorizing site conditions for seismic codes and assigning intensity- and frequency-dependent amplification factors to the various site classes are presented to provide a basis for understanding the new foundation factors proposed for the 2005 edition of the National Building Code of Canada.

Key words: design spectra, site characterization, amplification factors.

Résumé : Des facteurs de fondation sont utilisés dans les codes sismiques afin de capturer les effets d'amplification que les conditions de sol locales ont sur le mouvement du sol, et par conséquent, sur les forces utilisées en conception sismique. De récents développements en matière de classement des conditions de site pour les codes sismiques et d'attribution aux différentes classes de sites de facteurs d'amplification dépendant de l'intensité et de la fréquence sont présentées. Ils procurent une base afin de comprendre les nouveaux facteurs de fondation proposés pour la prochaine édition du Code national du bâtiment du Canada en 2005.

Mots clés : spectres de dimensionnement, caractérisation du site, facteurs d'amplification.

[Traduit par la Rédaction]

Introduction

Site conditions play a major role in establishing the damage potential of incoming seismic waves from major earthquakes. Damage patterns in Mexico City after the 1985 Michoacan earthquake demonstrated conclusively the significant effects of local site conditions on seismic response of the ground. Peak accelerations of incoming motions in rock were generally less than 0.04*g* and had predominant periods of around 2 s. Many clay sites in the dried lakebed on which the original city was founded had site periods also around 2 s and were excited into resonant response by the incoming motions. As a result, the bedrock outcrop motions were amplified about five times. The amplified motions had devastating effects on structures with periods close to site periods. In the 1989 Loma Prieta earthquake, major damage occurred on soft soil sites in the San Francisco – Oakland region where the spectral accelerations were amplified two to four times over adjacent rock sites (Housner 1989) and caused severe damage. Clearly, seismic design should incorporate the amplification effects of local soil conditions. The crucial question is how can this be done effectively without unduly complicating the structural design process or increasing the cost of engineering services significantly.

Theoretical basis of site amplification

The effects of site conditions on seismic ground motions are usually interpreted to mean how the waves from the underlying rock are affected by the geometrical and geological structures of the softer surface deposits during wave transmission to the surface. The basic mechanism of amplification is best illustrated by examining the effect of an undamped elastic surface layer on incoming bedrock motions. Consider the elastic layer shown in Fig. 1 characterized by a thickness, H, a shear wave velocity, V_{ss}, and a density, ρ_s. Let the shear wave velocity and density in the bedrock be denoted by V_{sr} and ρ_r, respectively.

Okamoto (1973) has shown that if the bedrock motion is a harmonic wave with a period equal to the fundamental period of the elastic surface layer ($T = 4H/V_{ss}$), then the amplification factor $A = a_t/a_r$ for the motion at the surface of the soil layer is

$$A = 2/\kappa \quad [1]$$

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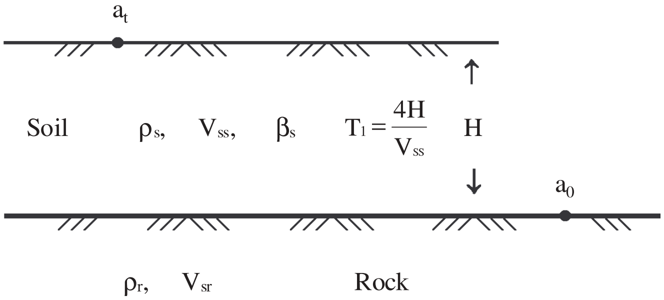
Fig. 1. Elastic layer on elastic half-space.



where κ, the impedance ratio, is $\rho_s V_{ss}/\rho_r V_{sr}$; a_t is the surface acceleration; and a_r is the incoming bedrock acceleration to the upper layer. The factor 2 in eq. [1] results from wave reflection at the surface of the soil layer. This equation represents the combined effects of the impedance ratio, input motion – soil layer resonance, and the effects of the free surface. Most strong-motion instruments are located on rock or stiff soil sites and provide the database for predicting ground motions on such sites. Therefore ground motions for seismic design on softer sites are determined by first estimating what the motions would be at the site on a rock or stiff soil outcrop and then estimating how much these motions would be amplified on passing through the soft overlying soils. Therefore, the crucial question is what is the amplification ratio A between the surface acceleration a_t and outcrop acceleration a_o shown in Fig. 1? If the soil layer has a critical damping ratio β_s, the amplification factor A of a_t with respect to the outcrop motion a_0 is

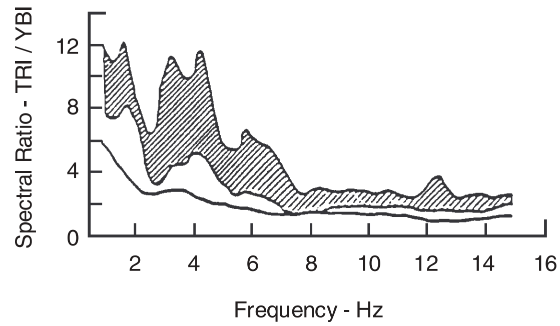
$$[2] \qquad A = \frac{1}{\kappa + \beta_s \pi/2}$$

These theoretical results show that the important parameters controlling ground motion amplification in elastic surface soil layers are (*i*) the relationship between the predominant period of the outcrop motions and the fundamental period of the surface layer ($T = 4H/V_{ss}$), (*ii*) the impedance between the surface layer and the base material, and (*iii*) the damping in the surface layer. Therefore, the key site parameters controlling the amplification of the outcrop motions are H, V_{ss}, κ, and β_s.

Under strong shaking, the response of the soil will be nonlinear. The shear modulus and damping are strain dependent, and therefore the larger strains, associated with strong shaking, reduce the effective shear moduli and increase the damping. The shear strength of the soil also puts a limitation on the magnitude of the surface acceleration because the seismic waves cannot generate shear stresses greater than the mobilized shearing resistance of the soil. Field evidence to show the effect of soil nonlinearity on ground motion amplification factors will now be reviewed.

The nonlinear behaviour of soils causes the amplification factors to be dependent on the intensity of shaking. This was demonstrated very clearly by Jarpe et al. (1989) by comparing the amplification factors for a site on Treasure Island in San Francisco Bay relative to the rock motions at adjacent Yerba Buena Island, using data from the main shock of the 1989 Loma Prieta earthquake and seven subsequent aftershocks. The amplification factors for surface motions re-

Fig. 2. Amplification of strong and weak motions at the Treasure Island site (Jarpe et al. 1989). TRI, Treasure Island surface motions; YBI, Yerba Buena Island rock motions.



corded at the Treasure Island site during the 1989 Loma Prieta earthquake are shown in Fig. 2. The solid line shows the variation in the north–south spectral ratio for the first 5 s of the shear wave in the main shock before any liquefaction took place at the site.

The shaded area in Fig. 2 shows the 95% confidence region for the spectral ratios of seven aftershocks. The amplification factors are drastically reduced in the strong-motion phase, although still 2 or greater over a wide frequency band of engineering interest. The reduction in amplification with increased intensity of shaking is due to the nonlinear stress–strain response of the soil, resulting from reduced effective shear moduli and increased damping. The peak acceleration at the surface is only 0.16*g*, so the amplification factors are associated with fairly low levels of earthquake shaking.

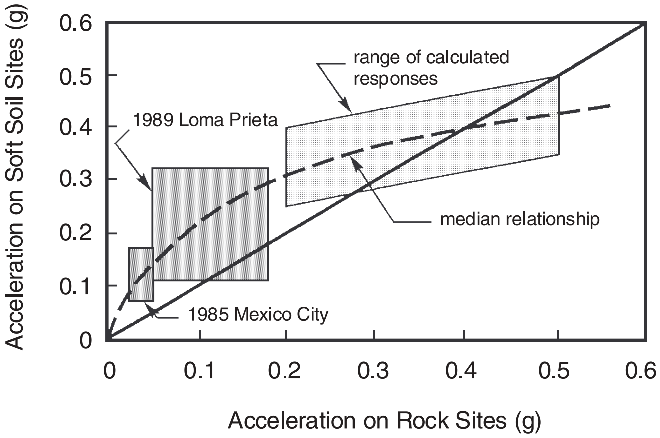
Idriss (1990) has summarized conveniently the relationship between peak accelerations on soft soil sites and those on associated bedrock sites in Fig. 3. The median curve is based on data recorded in Mexico City during the 1985 Michoacan earthquake and strong-motion data from the 1989 Loma Prieta earthquake. The part of the median curve for peak rock accelerations greater than 0.2*g* is based on one-dimensional site response analyses using the SHAKE computer program (Schnabel et al. 1972). The curve suggests that, on the average, the bedrock accelerations are amplified in soft soils until the peak rock accelerations reach about 0.4*g*. The higher amplification ratios between rock and soil sites, in the range of 1.5–4.0, are associated with levels of rock acceleration less than 0.10*g*, when the response is more nearly elastic. The increased nonlinearity of soft soil response at the higher accelerations reduces the amplification ratios because of the increase in hysteretic damping and the reduction in effective shear moduli.

The soils in the database used by Idriss (1990) vary significantly in properties, and the sites vary in geological structure. Thus, although the curve in Fig. 3 is useful in preliminary site evaluation, it is too general for estimating amplification factors for the different classes of soft soil sites encountered in practice. In many building codes, including the present edition of the National Building Code of Canada (NBCC 1995), the amplification effects of local soil conditions are represented by foundation factors. The variety of soil conditions are compressed into four distinct site categories and an amplification factor for long-period motions, termed a foundation or site factor, is associated with each

Table 1. Foundation factors, *F* (NBCC 1995).

Category	Type and depth of soil measured from the foundation or pile cap level	*F*
1	Rock, dense and very dense coarse-grained soils, very stiff and hard fine-grained soils; compact coarse-grained soils and firm and stiff fine-grained soils from 0 to 15 m deep	1.0
2	Compact coarse-grained soils, firm and stiff fine-grained soils with a depth greater than 15 m; very loose and loose coarse-grained soils and very soft and soft fine-grained soils from 0 to 15 m deep	1.3
3	Very loose and loose coarse-grained soils with depth greater than 15 m	1.5
4	Very soft and soft fine-grained soils with depth greater than 15 m	2.0

Fig. 3. Accelerations on soft soil and associated rock sites (Idriss 1990).



site category. How the system works will be illustrated using the relevant provisions of the NBCC.

Foundation factors in the 1995 NBCC

The seismic provisions of the 1995 NBCC incorporate the effects of local soil condition on design ground motions by classifying the wide variety of possible soil conditions into four categories and assigning a foundation factor, *F*, to each category. The foundation factors vary from 1.0 to 2.0 as shown in Table 1. Sites underlain by deposits of very soft to soft, fine-grained soils with depths greater than 15 m are assigned a foundation factor $F = 2.0$. The first three foundation factors are based primarily on research on site effects reported by Seed et al. (1976) and Mohraz (1976). The factor $F = 2.0$ was added as a result of the observation of large amplifications of incoming earthquake motions in the clay deposits of Mexico City during the 19 September 1985 earthquake in Mexico (Building Seismic Safety Council (BSSC) 1998). Justification of the value of $F = 2.0$ for peak rock accelerations of about 0.20*g* or less can be seen in the charts by both Jarpe et al. (1989) and Idriss (1990) presented earlier. It should be noted that the foundation factors are applied only to the longer period motions because of the cap on short-period motions in the 1995 NBCC.

The use of broad and distinctly different soil categories has the advantage that rather distinct patterns of ground response are associated with each type. A disadvantage is that it is sometimes difficult to decide into what category complex site conditions should be assigned.

In the 1995 NBCC, the equivalent lateral seismic force representing elastic response, V_e, is given by

$$V_e = vSIFW \tag{3}$$

where *v* is the zonal velocity ratio, *S* is the seismic response factor, *I* is the importance factor, *F* is the foundation factor, and *W* is the weight of the building. The factor *F*, representing site conditions, appears directly in the equation for calculating the design base shear and therefore has a direct impact on the seismic design loads. The code foundation factor, *F*, has a maximum value of 2 and decreases to 1 below periods of 1 s.

More comprehensive approaches to foundation factors

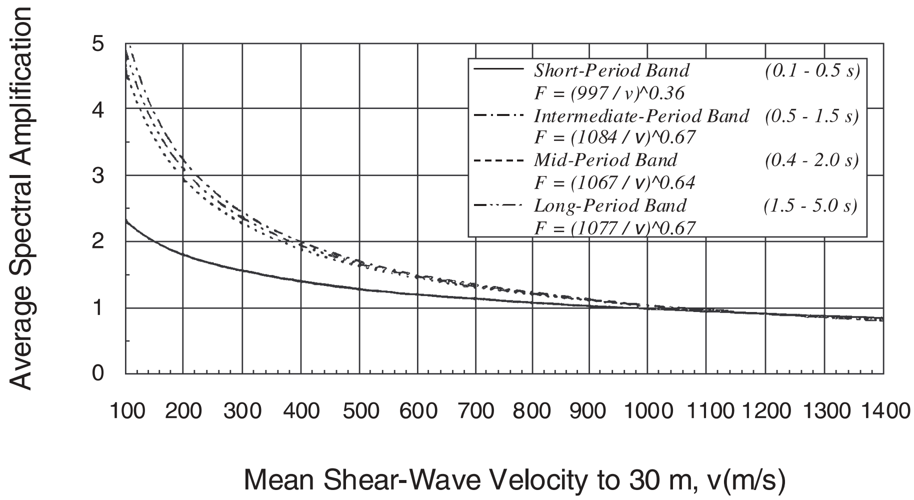
There are two key elements in establishing a reliable foundation factor: characterize the site condition quantitatively, and then assign a numerical value to the foundation factor for that soil category that is dependent on frequency and intensity of shaking. Two new procedures have been proposed for developing foundation factors in this way. One, by Borcherdt (1992, 1994), leads to a continuous distribution of foundation factors; the other, proposed by the National Earthquake Hazard Reduction Program (NEHRP 1994), is still based on a limited number of site categories, but the categories are defined quantitatively and therefore the ambiguities of the categories in the 1995 NBCC are avoided. These two approaches will now be reviewed. They are the basis for the new foundation factors recommended for the 2005 NBCC.

Continuous foundation factors

Borcherdt (1992, 1994) offered an alternative approach to site characterization and the specification of the amplification factors. He used the time-averaged shear wave velocity in the top 30 m of a site, $\overline{V}_{30}$, as a continuous measure of site conditions and developed frequency-dependent amplification factors that are continuous functions of $\overline{V}_{30}$. Amplification factors were determined for 35 instrumented sites using records obtained during the 1989 Loma Prieta earthquake. The amplification is determined with respect to the Franciscan rock formation in California. The shear wave velocity in the Franciscan rock is less than 1100 m/s. Amplification factors were determined by averaging the amplification of Fourier spectra over the different period ranges shown in Fig. 4.

The short-period amplification factor, designated F_a, corresponds to the average Fourier spectral ratios for recorded motions over the period range 0.1–0.5 s. The mid-period factor, designated F_v, is similarly defined for the period range 0.4–2.0 s. The variation of these factors and the corresponding factors for intermediate- and long-period ranges is

Fig. 4. Average horizontal amplification factors, *F*, with respect to firm to hard rock sites as a function of average shear wave velocity in the top 30 m for different period ranges (from Borcherdt (1994), used by permission of Earthquake Engineering Research Institute).



shown in Fig. 4. There is very little difference between the amplification curves for the longer periods.

Figure 4 illustrates that for the softer soils there is clearly a major difference between the short-period and long-period amplification factors. Clearly there is a need to have different amplification factors for the short-period range corresponding to the approximately constant spectral acceleration segment of the response spectrum, and for the longer periods corresponding to the more or less constant velocity range. This essentially follows the approach advocated by Newmark and Hall (1982).

Therefore, it was considered sufficient to use two amplification factors, F_a and F_v, to describe the amplification of outcrop motions in the short- and long-period ranges, respectively.

The amplification curves in Fig. 4 are valid up to peak bedrock accelerations of 0.1*g*. The functional forms of these curves are

[4] $$F_a = (1050/\overline{V}_{30})^{m_a}$$

[5] $$F_v = (1050/\overline{V}_{30})^{m_v}$$

where $m_a = 0.35$ and $m_v = 0.65$ are the exponents determined by Borcherdt to give the best fit to the field data for input ground motions on rock $A_a = A_v = 0.10g$.

Fitting the curve to the data was a two-stage process. First, the shape of the curve was selected a priori as $(V_{ref}/\overline{V}_{30})^m$, where V_{ref} is the mean shear wave velocity for the selected reference ground condition and m is either m_a or m_v as defined above, and then values of the parameter m were selected to give the best fit to the data. There is considerable scatter, however, in the data about the empirical curves in Fig. 4 (Borcherdt 1992).

1994 NEHRP foundation factors

In 1992, a 3 day meeting was held at the University of California entitled *Workshop on site response during earthquakes and seismic code provisions*. The workshop was attended by code committee members of the major U.S. groups involved in the development of building codes, geotechnical engineers and seismologists engaged in research on site effects, and representative users of code guidelines from major consulting firms. Draft proposals for new definitions of site categories and values of the associated foundation factors were presented for discussion by Borcherdt (1992), Dobry et al. (1992), and Seed (1992). These proposals had much in common, and a consensus was reached on what modifications should be made to the existing seismic provisions for site effects in the United States (Martin and Dobry 1994). The recommendations outlined in Martin and Dobry (1994) with some minor modifications including expanded definitions of site categories were adopted in the 1994 NEHRP guidelines (NEHRP 1994).

The new site categories adopted by NEHRP (1994) are shown in Table 2. Site categories were specified primarily in terms of the average shear wave velocity, $\overline{V}_{30}$, which is calculated from the time taken for the shear wave to travel from a depth of 30 m up to the ground surface. Thus $\overline{V}_{30} = 30/[\Sigma(h/V_s)]$, where h and V_s represent the thickness and shear wave velocities, respectively, of the individual layers between the ground surface and 30 m depth. To facilitate the use of the site categories in practice, complementary descriptions were developed for the site categories in terms of standard penetration resistance and undrained shear strength. The appropriate ranges in these parameters for the different site categories are also given in Table 2. If shear wave velocity data are unavailable, then appropriate values can be estimated for each layer, using the average standard penetration resistance $\overline{N}$ or the average undrained shear strength $\overline{S}_u$, and then a $\overline{V}_{30}$ value calculated as noted earlier. Site-specific geotechnical investigations and dynamic site response analyses are recommended for soils falling into category F.

Derivation of amplification factors F_a and F_v

A two-factor approach was adopted for constructing the free-field acceleration response spectrum as shown in Fig. 5. The factor F_a is used for the short-period part of the response spectrum, and the factor F_v for the long-period part of the response spectrum. Values of F_a and F_v are given in Tables 3 and 4, respectively, for different levels of peak ground accelerations. The values for F_a are mean values.

Table 2. Site classification for seismic site response (NEHRP 1994).

Site class	Site class name and generic description	Site class definition
A	Hard rock	$\overline{V}_{30} > 1500$ m/s
B	Rock	$760 < \overline{V}_{30} \le 1500$ m/s
C	Very dense soil and soft rock	$360 < \overline{V}_{30} \le 760$ m/s, $\overline{N} > 50$, or $\overline{S}_u > 100$ kPa
D	Stiff soil	$180 < \overline{V}_{30} \le 360$ m/s, $15 \le \overline{N} \le 50$, or $50 \le \overline{S}_u \le 100$ kPa
E	Soil profile with soft clay	$\overline{V}_{30} < 180$ m/s; plasticity index PI > 20, water content w > 40%, and $\overline{S}_u < 25$ kPa
F	Site-specific geotechnical investigations and dynamic site response analyses: (*i*) soils vulnerable to potential failure or collapse under seismic loading (liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils, etc.); (*ii*) peats and (or) highly organic clays (H > 3 m of peat and (or) highly organic clay, where H is thickness of soil); (*iii*) very high plasticity clays (H > 8 m with PI > 75); (*iv*) very thick "soft – medium-stiff clays" (H > 36 m)	

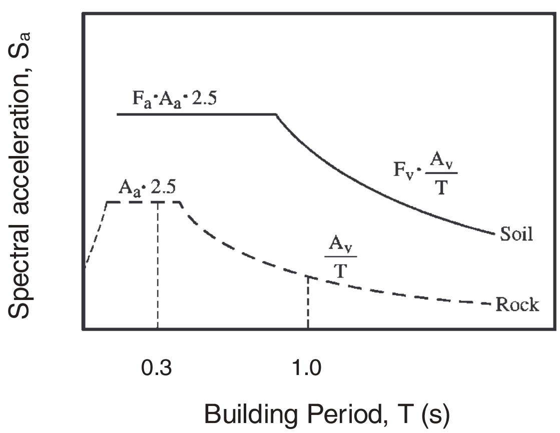


Fig. 5. Design spectra based on period-dependent site amplification factors (NEHRP 1994).

The values of F_v derived in the research studies were highly variable depending on site conditions and input motions. Therefore, F_v values are given at the mean plus one standard deviation level.

According to Joyner and Boore (2000), the amplification factors F_a and F_v were derived as follows. For A_a and A_v = 0.10g, F_a and F_v were based on Loma Prieta field data. For site class E and mapped ground accelerations greater than 0.10g, F_a and F_v were based on equivalent linear and nonlinear analyses conducted by Dobry et al. (1992) and Seed (1992). F_a and F_v for site classes C and D were derived using the general form of eqs. [4] and [5] shown in eqs. [6] and [7]:

[6] $$\log F_a = m_a(\log V_{ref} - \log \overline{V}_{30})$$

[7] $$\log F_v = m_v(\log V_{ref} - \log \overline{V}_{30})$$

The values of F_a and F_v for site class E were used in eqs. [6] and [7] to determine values of m_a and m_v for mapped accelerations A_a and A_v greater than 0.10g. These values were then inserted in eqs. [6] and [7] to obtain F_a and F_v for site classes C and D for the corresponding accelerations. The m_a and m_v values for the acceleration categories shown in Tables 3 and 4 are given in Table 5.

Table 3. Values of F_a as a function of site conditions and shaking intensity A_a (NEHRP 1994).

	A_a				
Site class	0.1g	0.2g	0.3g	0.4g	0.5g
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	—[a]
F	—[a]	—[a]	—[a]	—[a]	—[a]

[a]Site-specific geotechnical investigations and dynamic site response analyses should be performed.

Table 4. Values of F_v as a function of site conditions and shaking intensity A_v (NEHRP 1994).

	A_v				
Site class	0.1g	0.2g	0.3g	0.4g	0.5g
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	—[a]
F	—[a]	—[a]	—[a]	—[a]	—[a]

[a]Site-specific geotechnical investigations and dynamic site response analyses should be performed.

Amplification factors recommended for the 2005 NBCC

The Canadian Committee on Earthquake Engineering (CANCEE) advises on the seismic provisions of the National Building Code of Canada. CANCEE essentially adopted the NEHRP (1994) provisions for establishing the free-field acceleration response spectrum and the NEHRP site classes for the 2005 NBCC. The short- and long-period amplification factors F_a and F_v, respectively, were adopted also with some minor modifications. The 2005 NBCC factors are in Tables 6 and 7. The intensity of shaking in these

Table 5. Values of exponents m_a and m_v at different shaking intensities.

Rock acceleration (g)	m_a	m_v
0.1	0.35	0.65
0.2	0.25	0.60
0.3	0.10	0.53
0.4	–0.05	0.45

Table 6. Values of F_a as a function of site class and spectral acceleration at T = 0.2 s ($S_{0.2}$).

	$S_{0.2}$				
Site class	≤0.25	0.50	0.75	1.00	≥1.25
A	0.7	0.7	0.8	0.8	0.8
B	0.8	0.8	0.9	1.0	1.0
C	1.0	1.0	1.0	1.0	1.0
D	1.3	1.2	1.1	1.1	1.0
E	2.1	1.4	1.1	0.9	0.9
F	—[a]	—[a]	—[a]	—[a]	—[a]

Note: Use straight-line interpolation for intermediate values of $S_{0.2}$.

[a]Site-specific geotechnical investigation and dynamic site response analyses should be performed.

Table 7. Values of F_v as a function of site class and spectral acceleration at T = 1.0 s ($S_{1.0}$).

	$S_{1.0}$				
Site class	≤0.25	0.50	0.75	1.00	≥1.25
A	0.5	0.5	0.5	0.6	0.6
B	0.6	0.7	0.7	0.8	0.8
C	1.0	1.0	1.0	1.0	1.0
D	1.4	1.3	1.2	1.1	1.1
E	2.1	2.0	1.9	1.7	1.7
F	—[a]	—[a]	—[a]	—[a]	—[a]

Note: Use straight-line interpolation for intermediate values of $S_{1.0}$.

[a]Site-specific geotechnical investigation and dynamic site response analyses should be performed.

tables is defined by the short-period (T = 0.2 s) and long-period (T = 1.0 s) spectral accelerations $S_{0.2}$ and $S_{1.0}$, respectively. The probabilistic spectral accelerations at these two periods define the uniform hazard spectrum adopted in the 2005 NBCC. The seismic hazard mapping by the Geological Survey of Canada to determine these values is described by Adams and Halchuk (2003). Numerical values for major population centres across Canada will be provided in the 2005 NBCC. CANCEE adopted site class C in NEHRP (1994) as the reference site for amplification for the 2005 NBCC, instead of site class B as used in the NEHRP (1994) provisions. Site class C is very similar to the reference site in the current code. Therefore, for all intensities of earthquake shaking, the site factor for site C is 1.0. This neglects the deamplifications for this site class under strong shaking given in the NEHRP (Tables 3 and 4). The CANCEE amplification factors shown in Tables 6 and 7 for site classes D and E were determined by maintaining the relative amplifications between these classes and site class C as found in the NEHRP Tables 3 and 4.

Concluding remarks

The effect of local soil conditions on ground motions may be incorporated into building codes using site amplification factors. The factors are associated with a limited number of site classes that are intended to encompass the wide variety of site conditions encountered in practice. These factors measure the amplification of design motion parameters for a given site class with respect to the mapped code design values for the site class selected as the reference or standard site. The standard site in the 2005 NBCC is the equivalent of soft rock or stiff soil. The site categories in the 2005 NBCC are defined primarily by ranges in the average shear wave velocity, $\overline{V}_{30}$, in the top 30 m at the site. Since $\overline{V}_{30}$ may often not be available, the site categories are also defined using the average standard penetration resistance, $\overline{N}_{60}$, for cohesionless soils and the average undrained shear strength, $\overline{S}_u$, for cohesive soils. The two latter parameters are often available from routine site investigations.

The site classes adopted in the 2005 NBCC are those recommended in NEHRP (1994). The foundation factors in the 2005 NBCC were obtained by adjusting the factors recommended by NEHRP (1994) and adopted in BSSC (1998) and the 1997 Uniform Building Code (ICBO 1997) to the Canadian reference site. In addition, other site classes are identified for which site-specific dynamic response studies are recommended for determining the amplification factors.

The amplification factors recommended by NEHRP (Tables 3 and 4) have some limitations. Only the factors for mapped ground motions less than 0.10g are based on field data. They rely exclusively on data from just one case, the Loma Prieta earthquake. Factors for the softest site class, class E, were determined by hundreds of dynamic response analyses for mapped ground motions ranging from 0.10g to 0.5g. Factors for site classes C and D for mapped ground motions ranging from 0.10g to 0.5g were obtained by interpolation. Therefore, there is considerable uncertainty associated with the foundation factors.

Despite the limitations cited earlier, the provisions for determining amplification factors recommended in the 2005 NBCC are an improvement over the provisions in the current code. The major benefit is the way in which the new site classes are defined. By using values of soil parameters such as shear wave velocity, penetration resistance, and shear strength to define site classes, many of the troublesome ambiguities associated with the descriptive definitions in the current code have been removed. The new foundation factors are intensity dependent and reflect the effects of nonlinearity during strong shaking by a reduction in amplification. Lastly, by selecting site class C as the reference site for the 2005 NBCC, the maximum values of the new amplification factors are not significantly different from the values in the present code. Therefore, there is no significant penalty attached to the use of the new factors.

Addendum

A crucial report by Borcherdt (2002) became available as this paper was being processed for publication. His conclusions are so critical for assessment of the amplification factors recommended for the 2005 NBCC that the publisher agreed to add the following comments.

Since the form of the site classes and the specifications for foundation factors were set in 1992, much new field data have become available that are still being processed. The Northridge earthquake in 1994 has provided much data on the seismic response of the stiffer site classes and at very high levels of shaking. The Kobe earthquake of 1995 has provided similar data for the stiffer site classes but more importantly for the soft class E sites. The data from these earthquakes are being studied to verify the validity of the foundation factors proposed by NEHRP and, therefore, by implication the factors proposed in the 2005 NBCC. Borcherdt (2002), who was one of the leading figures in the development of the original NEHRP factors, reports that data from the 17 January 1994 Northridge earthquake support the NEHRP amplification factors adopted in U.S. codes for site classes C and D for base accelerations up to 0.5*g* and that no changes in the present code provisions are justified.

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