RECENT DEVELOPMENTS IN THE DEFINITION OF DESIGN EARTHQUAKE GROUND MOTIONS CALLING FOR A REVISION OF THE CURRENT ETHIOPIAN SEISMIC CODE - EBCS 8: 1995

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ABSTRACT

Recent developments in the definition of design ground motions for seismic analysis of structures are presented. A summary of results of empirical and analytical site-effect studies are provided and recent findings from empirical studies on instrumental records are compared against similar results from earlier studies.

Pertinent changes introduced in recent editions of international codes as a result of these evidences are presented. Comparisons of relevant provisions of EBCS 8: 1995 with those in contemporary American, European and South African codes are made.

The paper presents compelling evidences showing that the amplification potential of site-soils can in general be significantly larger at sites of low-amplitude rock-surface acceleration up to 0.1g than at sites of larger accelerations.

Noting the practical significance of this fact on the seismic design of structures in low to moderate seismic regions, to which many cities and towns of Ethiopia belong, changes to selected provisions of the local code are proposed.

***KEY WORDS**: Earthquake ground motion, return period, response spectra, seismic hazard, site amplification.*

INTRODUCTION

The latest edition of the Ethiopian standard code for building design was issued in 1995 by the Ministry of Works and Urban Development. This document known by the name of the Ethiopian Building Code Standard (EBCS) has a separate volume, EBCS 8, specifically dedicated to the design and construction of buildings in seismic regions [1].

EBCS 8 covers a wide range of issues ranging from basic definitions to detailed requirements. The document stands out as an important reference material having the purpose of ensuring safety to human lives and limiting damages to buildings during earthquakes. It is widely referred to by design engineers not only in Ethiopia, but also in the wider seismic-prone region of East Africa.

Nevertheless, as rightly stated in its Forward, such standards are technical documents which require periodic updating through the incorporation of new knowledge and practice as they emerge. This is especially true in seismic design of structures for the obvious reason that the discipline is still growing and gets refined with further acquisition of data as new earthquakes occur.

EBCS 8 has been in use for the past 16 years without being updated. Meanwhile, a number of devastating earthquakes have rattled many places all over the world. In the past decade alone, several earthquakes of magnitudes up to 7.3 on the Richter scale have surprised Africa – the continent once regarded as an earthquake free zone. Due to increased data base, knowledge on earthquakes and their effects on human life has tremendously improved. As a result, requirements of many design codes have significantly been refined. Some basic provisions in older editions of design codes are discarded. Existing design approaches have been modified and new ones introduced.

This paper attempts to address the basic issue of the definition of design ground motion in EBCS 8 vis-à-vis those in recent editions of selected major international codes. Two major aspects of design ground-motion are dealt with: seismic-hazard definition and consideration of site effect. The documents selected for comparison include the post-1994 editions of the National Earthquake Hazard Reduction Program (NEHRP) of USA [2-6], the 1994 and 2004 editions of the European Norm [7-9] and the 2010 edition of the South African National Standard [10,11].

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The paper starts by briefly reviewing the historical development of empirical studies of ground motion records with emphasis on site-soil effects [12,13]. Obvious differences in results of studies before and after the 1989 Loma-Prieta earthquake are summarized [12-23]. This is supplemented by basic theoretical evidences [16,17,24]. Developments in pertinent provisions of recent seismic codes are summarized. Specifically, new definitions and methods of characterization of site soils are introduced. Significantly improved amplification factors incorporated in contemporary seismic codes are presented [2-11].

Basic design spectra of EBCS 8 for different site-soil conditions are compared with corresponding spectra specified by the selected codes [1-4,7,8,10,25]. It is demonstrated that the EBCS 8 spectra fail to ensure adequate safety for the majority of common buildings ranging from multi-story residential houses through condominiums and school buildings to multi-purpose buildings with fundamental periods up to around 1 second, especially when the structures are founded on softer formations.

Moreover, it is pointed out that the 100-year return period adapted by EBCS 8 to define design ground motions is incompatible with the 475-year return period accepted worldwide and can significantly compromise safety [1,26]. This led to the recommendation that appropriate provisions of EBCS 8 need revision.

BRIEF HISTORICAL DEVELOPMENT OF SITE EFFECT STUDIES

Results of Early Instrumental Studies

Even though the potential of site soils to amplify earthquake ground motions was recognized since around the 1950s, it was only in the early 1970s that notable results of empirical studies on the subject started to emerge. The pioneering works were performed in Japan and the USA.

Hayashi et al (1971), as cited by Seed et al [12], are probably the first to present site-dependent average spectra, which were based on 61 accelerograms from 38 earthquakes in Japan. However, due to the limited size and quality of their data, the authors themselves suggested that their spectral curves be regarded with caution.

A more detailed study was reported by Seed et al at a later time [12]. Based on a total of 104 ground motion records from sites of fairly known geotechnical conditions, the study considered four site groups. Most of the records in the first three site groups were obtained from sites in western United States dominated by the 1971 San Fernando earthquake, whereas many records for the softest site soil group were from the Japanese earthquakes of 1964 Niigata and 1968 Higashi-Matsuyame.

The average spectra for the four site conditions are given in Fig. 1(a) for 5% damping. Significant differences are observed in the spectral shapes of the various site classes. For periods greater than 0.4 to 0.5 s, spectral amplifications are much higher for deep cohesionless soil deposits and soft to medium clay deposits than for stiff site conditions and rock over a wide range of periods. Seed et al [12] pointed out the inadequacy of their records for the fourth soil class and advised against the use of this particular spectral curve until further studies shade better light.

Mohraz [13] concurrently with Seed et al [12] conducted also an independent study on almost the same ground motion data base and came up with similar results.

Based on these results, the Applied Technology Council Project (ATC-3) came up in 1978 with the simplified site-dependent design spectra shown in Fig. 1(b) for three site soil groups: S1 (rock or shallow stiff soils), S2 (deep firm soils) and S3 (7 to 14 m deep soft soils). The spectral curves of S2 and S3 are obtained in such a way that their respective ratios with respect to S1, normally known as ratio of response spectra (RRS), in the velocity-sensitive region are 1.5 and 2.2, respectively. The less reliable fourth soil class was excluded, apparently heeding the advice of the researchers [12].

In general, the ATC-3 spectra are characterized by an ascending straight line for the very short-period range up to around 0.2 s, a constant acceleration for the acceleration-sensitive short-period range and a curve descending for the velocity-sensitive intermediate-period range.

The ATC-3 spectra were integrated in the series of editions of the National Earthquake Hazard Reduction Program (NEHRP) up to 1994 and in the Uniform Building Code (UBC) series up until 1997. In 1988, a fourth soil type, S4, for deep soft clays was included with the aim to address the rather high amplification potential of soft soils as evidenced by the 1985 Mexico City earthquake [5,16,17].

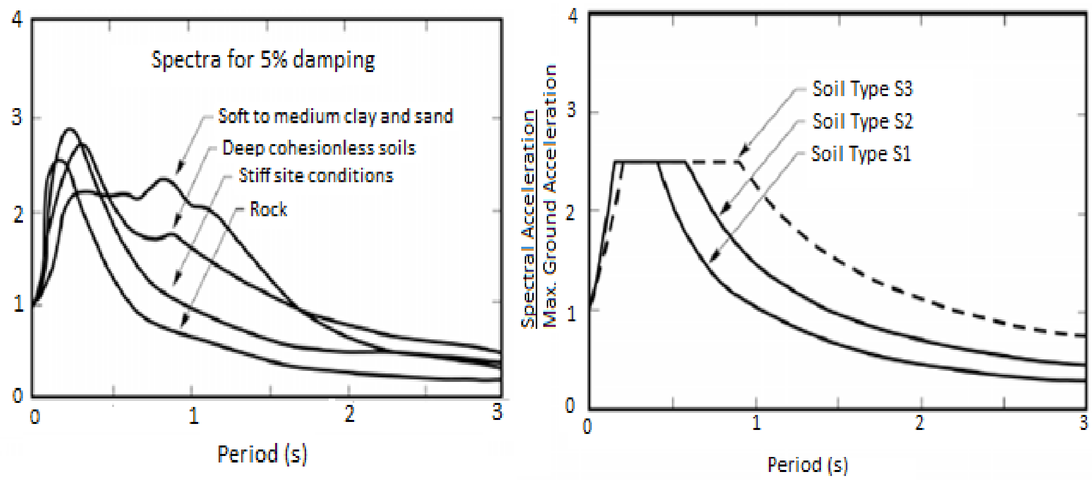


Figure1. (a) response spectra for different site conditions after Seed et al [12]; (b) The design spectra proposed by ATC-3:1978 Project [16,17]

It is important to note that the site-dependent spectral values in Fig. 1 are normalized with respect to the peak-ground acceleration, and thus the spectral curves are all anchored to unity at T=0. This has the effect of concealing inherent amplifications in the short-period range so that only amplifications in the intermediate velocity-sensitive range are observed. This will be clearer in the subsequent sections.

Results of Recent Empirical Studies

During the 7.1-magnitude Loma-Prieta earthquake of 1989, most damages linked to site-soil amplification and liquefaction took place in the bay area of San Francisco and Oakland located about 100 km NW of the epicenter. Much of the recorded evidence was also obtained from this area [14-18]. This, together with evidences from laboratory and analytical studies, encouraged a critical review of the single-factor amplification concept that endured up until that time and described above. A number of studies conducted on the enlarged data base shaded more light on site effects than ever before.

Idriss [14,15] studied the amplification of rock-surface accelerations using records from this and the 1985 Mexico earthquake both of which are associated with small rock-level accelerations. His main findings are that soil sites have the ability to amplify rock-surface accelerations of up to around 0.4g.

A more important outcome of post-Loma-Prieta studies, especially for engineers, is the rather high amplification of response spectra by soft soil sites. Average spectral accelerations of ground motion records due to the Loma-Prieta earthquake from thick soil sites near the San Francisco bay area and Oakland are provided in Fig. 2 for a damping of 5% in comparison with the corresponding average spectra of adjoining rock sites.

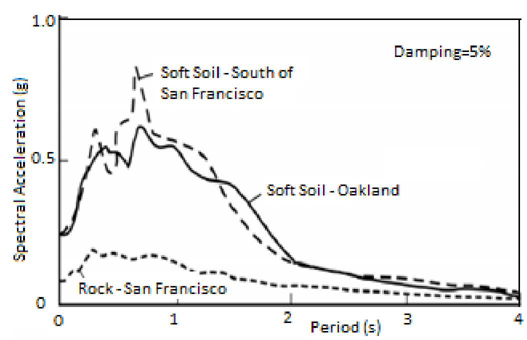


Figure 2 Comparison of average soil-site spectra in Oakland and San Francisco areas with average rock-site spectra in the region during 1989 Loma-Prieta earthquake [16,17]

The figure shows that the rock-surface acceleration (as $T \rightarrow 0$) is 0.08 to 0.1 g and amplified two to threefold by the soil. A similar degree of amplification is seen for periods up to about 0.2 s. The response spectra in the period range of 0.2 to 1.5 s are amplified to a much larger degree. Similar trends, but with lesser degree of amplification, were observed for stiff soil sites, though not presented here [16,17].

Comparison of the spectral curves in Fig. 2 with those in Fig. 1 shows that the short-period amplifications were not revealed in the early studies. Also, the amplifications in the velocity-sensitive region were underestimated. For this reason, the single-factor approach is no more found adequate to account for site-soil effects and has long been abandoned. This fact led to the introduction of new site-dependent design spectra in US seismic codes since 1994 and in other national and regional design codes.

BASIC THEORETICAL EVIDENCE IN DYNAMIC SITE RESPONSE

A simple one-dimensional model of a homogenous soil layer overlying a rock formation subjected to a vertically propagating sinusoidal shear wave can be used to provide a basic understanding of the amplification potential of soft-soil sites [16,17,24]. Roesset [24] showed that the ratio of the amplitudes of the sinusoidal accelerogram at the soil surface, a_A, to that at the rock, a_B, is a function of the soil shear-wave velocity, v_s, and the soil material damping ratio, β_s. Plots of this maximum amplification ratio as a function of v_s for selected values of β_s are presented in Fig. 3.

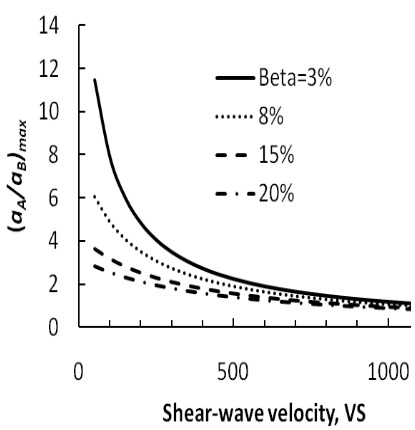


Figure 3 Ground amplification ratio at resonance versus shear-wave velocity for different damping ratios

Approximating RRS_{max} by this amplitude ratio – an assumption that has been found to be fairly reasonable for a preliminary estimate - this rudimentary model fairly accurately predicts RRS_{max} = 9 for the Mexico City soft soil site, for which a shear-wave velocity of 80 m/s and β_s = 3% are employed. This compares well with RRS_{max} of 8 to 20 actually recorded at the soil sites in Mexico City during the 1985 earthquake. Similarly, the model predicts RRS_{max}=4 for San Francisco Bay area corresponding to representative values of v_s =150 m/s and β_s = 8% for the area. The model once again predicted well RRS_{max} observed at this site during the Loma-Prieta earthquake that ranges from 3 to 6 [16,17].

These results indicate that, for places where previous seismic records are not available, prior knowledge of the representative shear-wave velocity of the site and its damping behavior can provide a good idea of its amplification potential. Such an exercise is particularly useful for Ethiopia, where none to few recorded strong ground motion records are available to conduct statistical studies.

According to laboratory evidences, the material damping ratio, β_s, is a nonlinear function of the plasticity and the strain level of the soil. Highly plastic soils (PI > 50%) exhibit small damping and behave nearly linearly over a wide range of strains. For highly plastic clays, β_s can be less than 3% for strains up to 0.1%.

Soils of high PI are not uncommon in urbanized seismic regions of Ethiopia, a typical example being the dark and light grey expansive soils covering a big part of Addis Ababa and its environs. At some locations, this formation can be several tens of meters thick and in a rather soft state over a significant depth. The damping potential of such soils can be quite low and their amplification potential very high.

THE NEW APPROACH TO ACCOUNT FOR SITE EFFECT

Evaluation of Improved Site Coefficients

A more practical approach for the evaluation of site amplification factors in regions, where sufficient earthquake records and geotechnical data are available, is the calculation of statistical averages of RRS for site soils grouped according to their dynamic behavior. A number of empirical studies conducted after the Loma-Prieta earthquake suggested that average amplification factors of soil sites are proportional to the mean shear-wave velocity, v_S, of the upper 30 m thickness raised to a certain negative exponent, which is dependent on the period band and the intensity of the rock acceleration [16-23]. It was thus found important that site soils are classified on the basis of this important parameter.

The empirical study of Borcherdt [18] in particular suggested the following generic best-fit relations for the two amplification factors, denoted by F_a and F_v, as a function of v_s and the rock-surface shaking intensity:

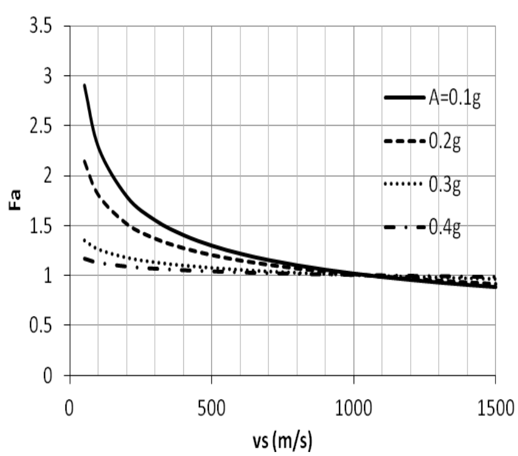
$$F_a = \left(\frac{1050}{v_S}\right)^{m_a} \quad ; \quad F_v = \left(\frac{1050}{v_S}\right)^{m_v} \tag{1}$$

The factor F_a is applicable for the acceleration-sensitive short-period region (about 0.1 to 0.5 s) and F_v for the velocity-sensitive intermediate-period region (about 0.4 to 2 s). The values of the exponents, m_a and m_v, are provided in Table 1.

Table 1: Values of the exponents in Borcherdt's regression relations of Equation (2) [18]

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

The plots of Eq. 1 are given in Fig. 4 which shows that F_v is consistently larger than F_a for v_s up to around 1000 m/s - v_S of the reference rock site. Both factors tend to unity with v_S approaching 1000 m/s and decrease with increasing intensity of rock shaking. Please note the similarity of these curves to the theoretical curves of Fig. 3 demonstrating that increasing rock-shaking intensity is associated with increased damping.



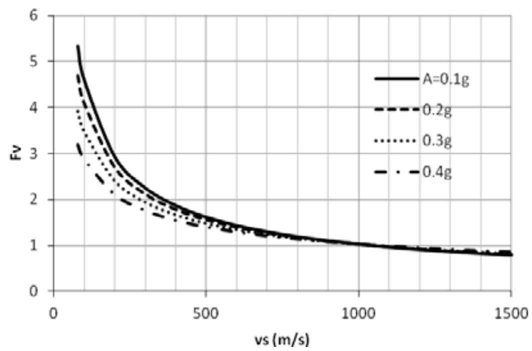


Figure 4 Variation of spectral amplification factors versus v_S for short and long period ranges and for a range of intensity of rock shaking (Re-plotted after Borcherdt [18]

The New System of Soil Classification

For a generally stratified formation of *n* layers each having a thickness of h_i and a shear-wave velocity of v_{Si} within the upper 30 m thickness, v_S can be established using the following relationship [2-4, 8, 10, 16, and 17]:

$$v_S = 30/t_{30} = 30 \Big/ \sum_{i=1}^{n} \left(h_i / v_{Si}\right) \tag{2}$$

The terms in the summation represent the time taken for the shear wave to travel through each individual layer. The shear-wave velocity computed in this manner is based on the time, t_{30}, taken by the shear wave to travel from a depth of 30 m to the ground surface, and is thus not computed as the mere arithmetic average.

This approach also allows for the use of more readily measurable quantities such as the standard penetration test blow count, *N*, for granular deposits or undrained shear strength, S_u, for saturated cohesive soils though they are less reliable due to the inherent double correlations. The representative values are determined in a manner similar to Eq. 2

Based on a landmark consensus reached by geotechnical engineers and earth scientists in the USA in the early 1990s, five distinct soil and rock classes, A to E, are identified in accordance with this approach and provided in Table 2. Corresponding approximate soil classes as per older methods are also provided in the first column for comparison purposes. A sixth much softer site class, F, is also defined that requires site-specific studies. It is described in detail in NEHRP documents [2, 3, and 4].

Table 2: Site soil classes as per the recent NEHRP editions [2, 3, and 4]

Pre-1994 Site Class (approximate)	New NEHRP Site Class	Description	v_S (m/s)	SPT blow count, *N*	S_u (kPa)
S1	A	Hard Rock	>1500	-	-
	B	Rock	760 – 1500	-	-
S1 and S2	C	Soft rock/very dense soil	360 – 760	>50	>100
	D	Stiff soil	180 – 360	15 – 50	50 – 100
S3 and S4	E	Soft soil	<180	<15	<50
	F	Soils requiring site-specific study			

While this method of site classification is not entirely correct from a theoretical perspective, the general consensus is that the stiffness of the shallow soil as measured by vs is the most reliable single site parameter to best characterize site amplification potential [16 - 20]. In addition, vs is readily measured in the field.

The New Site Amplification Factors

Using the average v_s of each soil class given in Table 2, the site amplification factors can now be established by reading from Fig. 4 for the representative value of rock-motion intensity considered. The discrete values so obtained according to Borcherdit [18] and adopted by NEHRP [2, 3, 4] are given in Table 3. The effective peak acceleration, $A_{n,}$ and the effective velocity related acceleration, A_v, are rock-level seismic hazard parameters employed to characterize site seismicity of US for 90% probability of not being exceeded in 50 years (475 years return period) [2].

The new amplification factors exhibit the following main features [5, 6, and 25]:

1. The three (later on four) site categories in earlier codes are replaced by six new categories A to F. Soil classes C to E amplify the rock motion significantly, especially when the rock shaking intensity is small.

2. Two seismicity dependent site coefficients, F_a and F_v, replace the single site coefficient, S, in older codes. F_a is for the acceleration-sensitive region and F_v is for the velocity-sensitive region. Both factors decrease with increasing seismicity due to increased damping, and F_v is almost always larger than F_a for all sites

3. While the old factor, S, assumed values up to 1.5 (or 2.2), the new factors, F_a and F_v, take values of up to 2.5 and 3.5 for short-period and intermediate-period bands, respectively. This results in much larger seismic design forces for many classes of structures on soft formations especially in

Table 3: Values of the site coefficient F_a and F_v according to NEHRP 1994 [2]

Soil Profile Type	F_a for Shaking Intensity, A_a					F_v for Shaking Intensity, A_v				
	≤ 0.1	0.2	0.3	0.4	≥0.5	≤ 0.1	0.2	0.3	0.4	≥0.5
A	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0	1.7	1.6	1.5	1.4	1.3
D	1.6	1.4	1.2	1.1	1.0	2.4	2.0	1.8	1.6	1.5
E	2.5	1.7	1.2	0.9	[b]	3.5	3.2	2.8	2.4	SR
F	SR= Site-specific geotechnical studies and dynamic site-response analysis required									

less seismic regions. For this reason, the seismic design of structures in less seismic regions has become much more stringent than ever before.

4. The older qualitative site classification method is replaced by a new unambiguous and more rational classification method using representative shear-wave velocities of the upper 30 m geological formation. Alternatively, though less preferable, average SPT blow counts and/or undrained shear strength can be used to classify sites (See Table 2).

It is important to note that results of later studies on an enlarged data base including records from more recent earthquakes like Northridge 1994 have not suggested significant changes to the values of the above site amplification factors [4, 19, and 20].

DESIGN SPECTRA IN SEISMIC CODES

The Design Spectra of NEHRP

As noted earlier, the ATC-3: 1978 Spectra were for the first time replaced by new design spectra in a 1994 document issued through a long-term federal project of the US Government known by the name of the National Earthquake Hazard Reduction Program (NEHRP), which was initiated in 1985 to replace the mission of ATC. NEHRP incorporated the new results based on the 1989 Loma Prieta earthquake. As presented above, the new results clearly demonstrated that the site-dependent design spectra that were in use up to that time were inadequate [5,6,16,17]. Furthermore, NEHRP has since its inception consistently employed a 475-year return period in defining the design ground motion [2-4].

The basic elastic design spectrum of NEHRP 1994 that for the first time made use of the above values of amplification factors is given by the following relationship [2]:

$$C_{se} = \frac{1.2c_v}{T^{2/3}} \leq 2.5C_a$$

$$C_v = F_v A_v;\; C_a = F_a A_a \qquad (3)$$

Note that F_a is applied on the constant part of the spectrum, whereas F_v is applied on the descending segment. A plot of Eq. 3 normalized with respect to C_a against period is given in Fig. 5(a) for C_v/C_a=1. This plot shows the shape of the basic elastic design spectral curve.

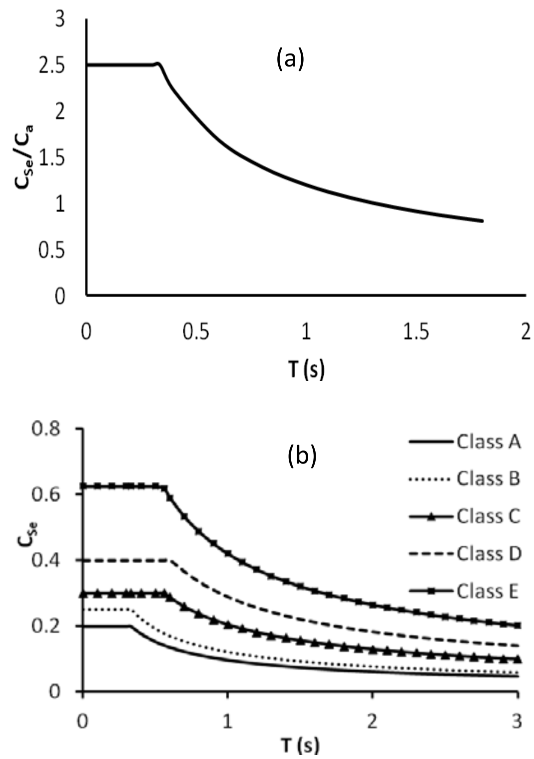


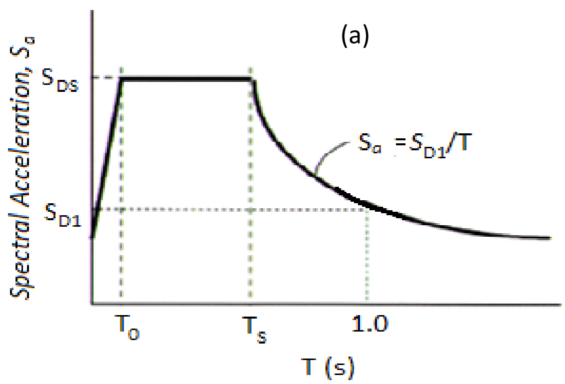
Figure 5 Elastic design spectra according to NEHRP 1994 (a) Basic [2]; (b) For A_a=A_v=0.1

Spectral curves corresponding to the five possible soil classes A to E can be plotted from Eq. 3 for a given earthquake shaking intensity. Such design spectra for a seismic region characterized by A_a=A_v=0.1 are given in Fig. 5(b). Similar curves can be prepared for other seismic regions. This is to be compared to the three spectral curves of ATC-3 given in Fig. 1, where amplification occurs in the declining section only.

The basic design spectrum in NEHRP 1997 [3] has shown substantial changes as shown in Fig. 6(a), in which two key spectral ordinates in the figure, S_{DS} and S_{D1}, are introduced as given by

$$S_{DS} = \frac{2}{3}S_{MS} = \frac{2}{3}(F_d S_s)$$

$$S_{S1} = \frac{2}{3}S_{M1} = \frac{2}{3}(F_v S_1) \quad (4)$$



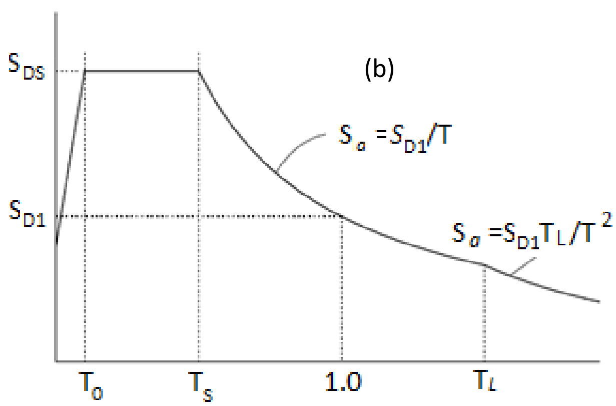


Figure 6 The basic design spectral curves according to (a) NEHRP 1997 [3]; (b) NEHRP 2003[4]

The transition periods in the figure are obtained from

$$T_0 = 0.2S_{D1}/S_{DS}; \qquad T_S = S_{D1}/S_{DS} \quad (5)$$

S_S and S_1 are mapped spectral accelerations in terms of fractions of g for the short and intermediate-period regions represented by 0.2 s and 1 s, respectively. These spectra correspond to the Maximum Considered Earthquake (MCE) and replace the effective accelerations, A_a and A_v, of the 1994 version to characterize the seismic hazard. The MCE corresponds to a 2% probability of being exceeded in 50 years (or 2500 years return period) to be adjusted later to 475-years return period by multiplying by 2/3. S_{MS} and S_{1S} are the corresponding spectra that account for site-soil effect [3]. The coefficients F_a and F_v are the same site soil amplification factors of NEHRP 1994 given in Table 3. Note also that the descending right part is varying according to T^{-1} and no more according to $T^{-2/3}$. Similar to Fig. 5(b), a set of five curves can be plotted from Eqs. 4 and 5 for the five different soil groups in a given seismic region. The basic design spectral curve in Fig. 6(a) has remained the same in the subsequent editions of NEHRP since 2000, except for the introduction of a flatter curve varying according to T^{-2} for the displacement-sensitive long-period period region beyond T_L as shown in Fig. 6(b) [4].

The Eurocode Design Spectra

The 1994 edition of the European seismic code (EC 8) employed three site classes, A, B and C, similar to those in ATC-3, 1978 [7]. However, while the ATC-3 spectra shown in Fig. 1 have a common plateau to all site classes, EC 8: 1994 paradoxically specifies a smaller maximum value and a smaller amplification factor over the entire period range for the softest site class C as shown in Fig. 7(a), in which the spectra are normalized with respect to the design ground acceleration. In light of the background material given above, such a representation of the dynamic behavior of soft formations is obviously faulty. Similar views have recently been expressed by Rey et al [9], who attribute this pitfall to lack of sufficient *ad hoc* studies prior to the publication. These spectra are no more in use in Europe.

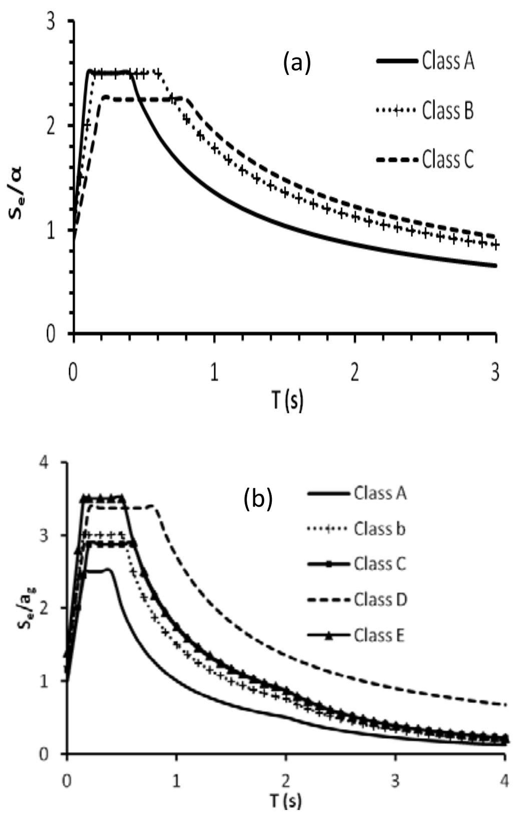


Figure 7 Normalized elastic response spectra: (a) EC 8: 1994 [7]; (b) EC 8: 2004 – Type 1 (for $M_S \geq 5.5$; re-plotted after [8])

The more recent edition of EC8 issued in 2004 has not only rectified this problem but also introduced the new soil classes of NEHRP with some modifications [2,3,4,8] (see Fig. 7(b)). According to the new EC 8, all rock and rock-like geological formations with $v_s > 800$ m/s are categorized under Ground Type A. This is unlike the provision for two distinct rock site classes of A and B in the recent NEHRP editions. Each soil class in EC 8, 2004 is assigned a constant amplification factor for the entire period range. In general, these factors are lower than the corresponding NEHRP factors.

Two types of spectra, Type 1 and Type 2, are proposed by EC 8: 2004 for regions with predominant earthquakes of surface-wave magnitudes larger than 5.5 and less than 5.5, respectively. Fig. 7(b) presents Type 1 spectra for the five soil classes. A segment descending according to T^{-2} is included for the periods longer than 2 s. The Type 2 spectra proposed for less seismic regions are similar in shape to the Type 1 spectra but with larger amplification factors and reduced control periods.

The Design Spectra of SANS 10160-4: 2010

This standard for seismic actions was published very recently (June 2010) and makes up one of the eight parts of the South African National Standard SANS 10160 series - *Basis of structural design and actions for buildings and industrial structures* [10]. It supersedes the older version SABS 0160:1989 [10,11].

The code adapted Type 1 basic spectrum of EC 8: 2004 with a slight modification of the left linear part. It has also directly adopted Ground Types A to D of EC 8, 2004 and the corresponding amplification factors and control periods omitting Ground Types E and F. Since the plots of the response spectra are similar to those in Fig. 7(b), they are not presented here.

The seismic hazard is represented in terms of reference peak ground acceleration, a_g, for Ground Type 1 (rock site) and given in form of a seismic hazard map based on a 475-year return period. Noteworthy is that this return period was also used in the superseded 1989 edition [11]. Two major zones are distinguishable: Zone I of natural seismic activities and Zone II of mining-induced and natural seismic activities. The majority of Zone I is assigned a_g=0.1g with sites of a_g values less than 0.05g being rare.

Given the relatively stable seismic nature of South Africa, the attention given to seismic design in the country is quite instructive to the more seismic nations in East Africa. This provides an additional perspective to critically evaluate the rather liberal seismic hazard definition of EBCS 8 and its provisions for site effects.

The Design Spectra of EBCS 8, 1995

The normalized elastic design spectra, S_d, of the Ethiopian Building Code Standard, EBCS 8 (1995), proposed for dynamic analysis are given in Fig. 8(a).

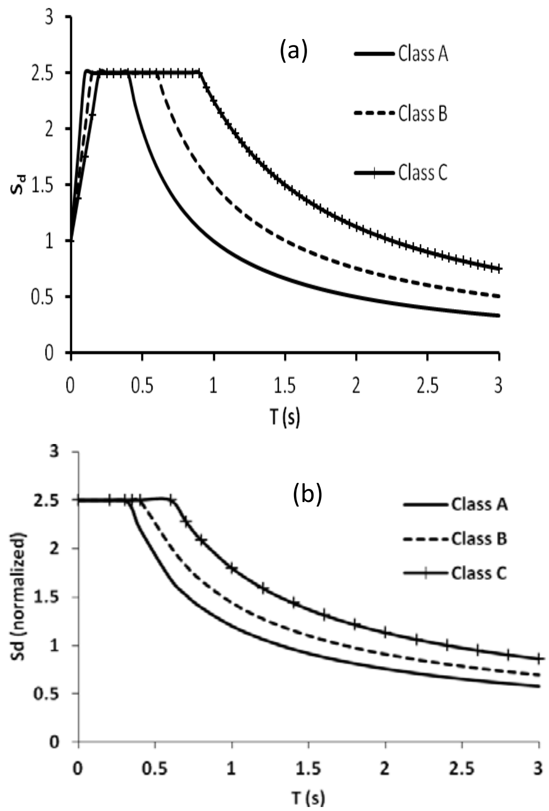


Figure 8 The design spectra of EBCS 8 (1995) (a) for dynamic analysis; (b) for static analysis [1,25]

Excepting for some minor differences, the EBCS 8 spectra are practically identical to the already obsolete ATC-3 (1978) spectra given in Fig. 1. The design spectra proposed for pseudo-static analysis are also given in Fig. 8(b) for comparison purposes. The left linear part is omitted in this case, the right side descends according to $T^{-2/3}$ instead of T^{-1} and the amplification factors reduced.

COMPARISON OF EBCS 8 DESIGN SPECTRA WITH THE REST

In this section, a comparative study of EBCS 8 spectra against those specified by NEHRP 2003, EC8: 2004 and SANS 2010 is presented.

EBCS 8 Versus NEHRP 2003

The basic design spectrum of NEHRP in all its editions since 2003 remained almost unchanged. This spectrum as it appears in NEHRP 2003 [4], is given in Fig. 6(b) and can be expressed as

$$S_a = \begin{cases} \left(0.6\left(T/T_0\right)+0.4\right)S_{DS}; & 0 \le T \le T_0 \\ S_{DS}; & T_0 < T \le T_S \\ S_{D1}/T; & T_S < T \le T_L \\ S_{D1}\,T_L/T^2; & T > T_L \end{cases} \qquad (6)$$

For T = 0, Eq. (6) yields the design spectral ordinate for an ideally rigid structure undergoing the same motion as its foundation which we can denote by S_{a0}. With this and the introduction of Eq. 4 in Eq. 6, we obtain:

$$S_{a0} = 0.2\dot{6}F_a S_S \qquad (7)$$

For a rigid structure on the reference ground type, Class B, F_a takes the value of unity (See Table 3), and the design spectral ordinate S_{a0} should be equal or the same as the peak ground acceleration (PGA) of the site for the design earthquake. This enables us to estimate the value of S_S from Eq. 7 for a known PGA of a site.

As per the existing seismic hazard map of Ethiopia which is based on a return period of 100 years, the capital, Addis Ababa, located in Seismic Zone 2, is assigned a PGA of 0.05g. With this value inserted in Eq. 7 for S_{a0}, the corresponding maximum value of spectral acceleration for short period according to NEHRP 2003 would be obtained as S_S=0.188*g*. The corresponding one-second spectral acceleration, S_1, can be extrapolated from Table 3 as 0.072g. With these inserted in Eq. 4, the design spectral values S_{DS} and S_{D1} for Zone 2 are obtained as

$$S_{DS} = \frac{2}{3}\left(F_s X\, 0.188\right) = 0.125F_a$$
$$S_{D1} = \frac{2}{3}\left(F_v X\, 0.072\right) = 0.048F_v \qquad (8)$$

Similarly, the transition periods can be computed by substituting Eq. 8 back into Eq. 5. The values of S_{DS}, S_{D1}, T_0 and T_S computed in this manner are substituted in Eq. 6 and the resulting expressions plotted for the different site soils. These are given in Fig. 9 together with the EBCS 8 spectra for a PGA of 0.05g specified for Zone 2. Comparisons for other seismic zones can be made in a similar way.

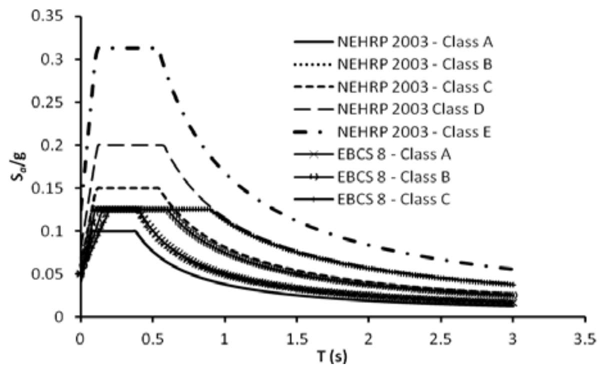


Figure 9 Comparison of EBCS 8 spectra and NEHRP spectra adapted to a return period of 100 years for Zone 2 (including Addis Ababa)

The plots show that the introduction of the NEHRP 2003 site factors demands design forces up to 150 % in excess of what is currently required by EBCS8. The largest spectral discrepancies occur in a very important period range encompassing buildings of small to moderate height of up to around 12 stories with a fundamental period of up to around 1 s built on NEHRP Site Classes D and E. Evidently, such buildings are the most frequently built structures including residential houses, condominiums, apartments, office flats, public offices, hotels, hospitals and many others. Thus, the implications of the above results are not difficult to figure out.

EBCS Versus EC 8 and SANS 2010

Comparison of the EBCS spectra with the European and South African spectra is more direct forward, as all of these documents use rock-level PGA to characterize seismicity.

Type I spectra of EC 8: 2004 are compared in Fig. 10 with EBCS spectra, which show that buildings in the short-period region designed in accordance with EBCS 8 could be underdesigned by up to 40 %. A similar comparison with Type II spectra of EC 8, 2004 indicates larger differences of up to 80 %. These discrepancies are comparatively smaller than the discrepancies observed with NEHRP spectra, because the NEHRP site amplification factors are consistently larger than the EC 8 amplification factors. Comparison of the EBCS 8 spectra with the SANS spectra gives identical results as in Fig. 10 with Site Class E omitted.

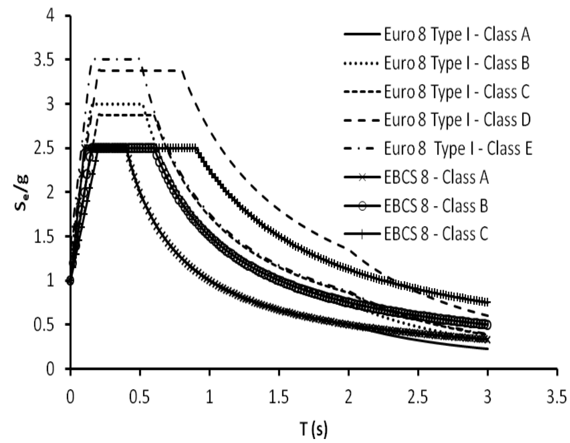


Figure 10 Comparison of EBCS 8 spectra with Type I EC 8: 2004 spectra adapted to 100-year return period

Note that the comparisons in Fig. (9) and (10) are conducted without considering the difference in the definition of the return period. This issue is treated in the next section.

Influence of Seismic Hazard definition

Seismic Hazard Maps of Ethiopia

The seismic hazard map of Ethiopia as provided in EBCS 8, 1995 is presented in Fig. 11 [1]. This map is based on a 100-year return period or approximately 50 % of being exceeded in 50 years. According to this map, each seismic zone of 1 to 4 is assigned a constant bedrock acceleration ratio, α_0, of 0.03, 0.05, 0.07 or 0.1, whereas Zone 0 is considered seismic free. Addis Ababa belongs to Zone 2 with $\alpha_0 = 0.05$. Many cities and big towns like Mekele, Dese, Semera, Adama, Awasa and Arba Minch, of which some are capitals of federal states, all belong to Zone 4 with α_0=0.1.

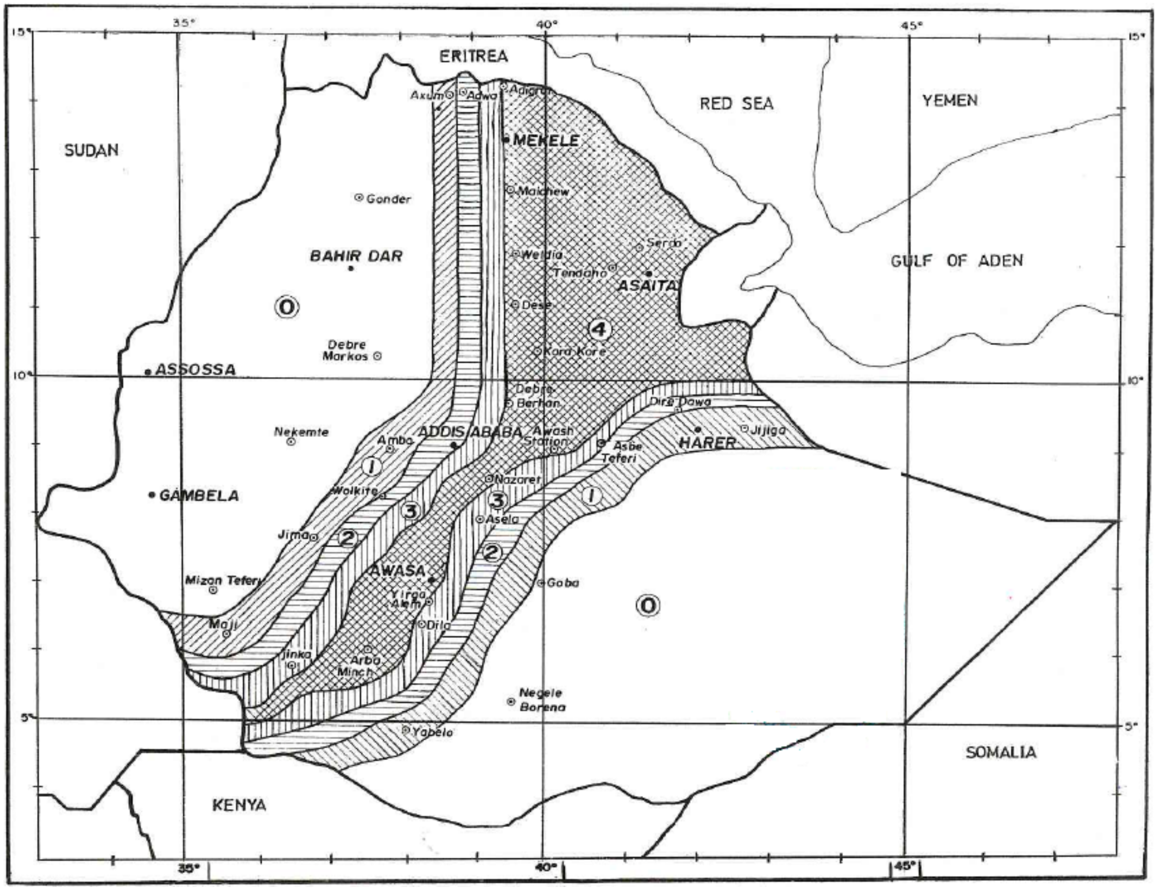


Figure 11 Seismic hazard map of Ethiopia for 100-year return period as per EBCS 8: 1995 [23]

A recent helpful compilation of worldwide seismicity is provided by the Global Seismic Hazard Assessment Program (GSHAP), which was launched by the International Lithosphere Program (ILP) with the support of the International Council of Scientific Unions (ICSU), and endorsed as a demonstration program in the framework of the United Nations International Decade for Natural Disaster Reduction (UN/IDNDR). It had the objective of mitigating the risk associated with the recurrence of earthquakes by promoting a regionally coordinated, homogeneous approach to seismic hazard evaluation. The project was operational from 1992 to 1999 [26].

The major output of the GSHAP is the global seismic hazard map for a 475-year return period. As noted above, this level of hazard has been widely accepted all over the world as a design-level earthquake and incorporated in US codes for more than three decades now. In contrast, the EBCS 8, 1995 employs a return period of just 100 years. Reference documents could not be found providing a rational explanation for taking such a bold decision involving risks on the safety of life and property.

The data base of GSHAP is accessible to users [26]. A seismic hazard map for Ethiopia prepared by the author using the appropriate data is given in Fig. 12, in which five distinct seismic regions are identified with different ranges of PGA values as shown in the legend. Note that the ratio of the PGA to the gravitational acceleration, g, corresponds to α_0 - the bedrock acceleration ratio in EBCS 8.

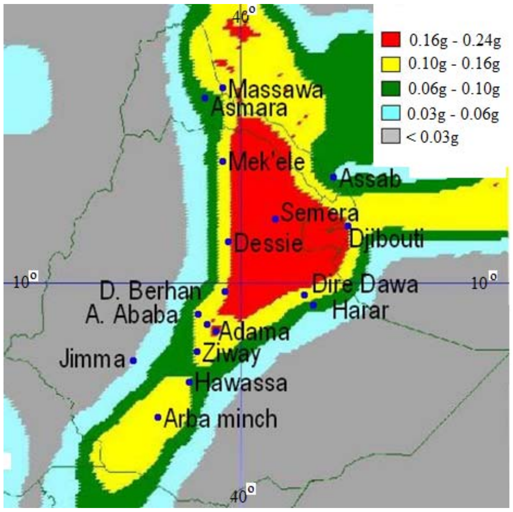


Figure 12 The seismic hazard map of Ethiopia based on the GSHAP data for a return period of 475 years

Comparison of Fig. 11 with Fig. 12 shows that not only corresponding seismic regions are assigned much higher values of PGA in the GSHAP map, but also are the entire size and extent of the individual seismic zones changed. According to the new map, the most seismic area of the country is concentrated near and around the Afar region characterized by a PGA of 0.16g to 0.24g. This alone entails an increase in seismic force demand of 60 to 140 % in this region without including site effect. The capital, Addis Ababa, belongs to the second most seismic zone with PGA in the range of 0.1g to 0.16g. This again implies an increase of 100 to 220 % in seismic hazard level with an average increase of 160 %. Several rapidly growing towns including, Mekele, Dese, Debre Berhan, Ziway, Hawasa, Arbaminch and Dire Dawa belong to this seismic zone, while Semera, the current capital of the Afar Region, is in the heart of the most seismic zone.

Returning to the spectral comparison, the combined influence of the new site classification system and the new seismic hazard definition is studied next. Considering 0.1g as the lowest-estimate PGA of the region, to which Addis Ababa belongs, Eq. 7 yields a corresponding short-period spectral acceleration, S_S, of 0.45g.The one-second spectral acceleration, S_1, can be interpolated as 0.18g.

With these values inserted in Eq. 4, the design spectral values S_{DS} and S_{D1} are obtained and the transition periods easily computed as before. These quantities are substituted in Eq. (6) and the resulting expressions plotted for the different site soils. These are presented in Fig. 13 together with the EBCS 8 spectra for a PGA of 0.05g specified for Zone 2 including Addis Ababa.

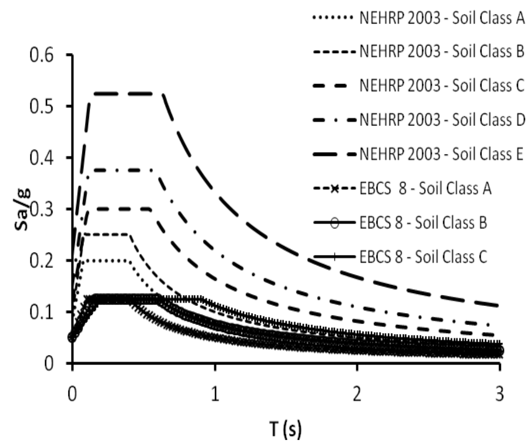


Figure 13 Comparison of EBCS 8 spectra with NEHRP spectra adapted to GSHAP zoning Of Ethiopia: Spectra for the second most seismic region (including Addis Ababa)

The plots show a very significant difference between the two sets of design spectra. Design base shear computed in accordance with EBCS 8 spectra fulfills only a fraction of the base shear demanded by NEHRP requirements, in some cases being as low as 24 %. All ranges of buildings on any soil formation are affected by the inadequate provisions of EBCS 8. Similar comparisons made with the European and South African spectra confirm these discrepancies.

CONCLUSIONS AND RECOMMENDATIONS

Recent changes in the definition of design ground motions have been presented. Results of empirical site-effect studies together with basic analytical evidences on site response are provided. Differences in results of empirical studies on recent instrumental records against results from earlier studies are highlighted. Changes introduced in recent editions of international codes as a result of such evidences are presented.

Comparisons of relevant provisions of EBCS 8, 1995 with those in contemporary American, European and South African codes demonstrate that seismic loads of most buildings designed in accordance with EBCS 8 are significantly underestimated. This is especially the case when the site soil overlying the bedrock is medium stiff to soft and is relatively thick. Most vulnerable buildings are those with fundamental periods up to around 1 second that encompass most commonly constructed buildings.

The two main culprits in EBCS 8 for these pitfalls are the rather old and inadequate provisions for site amplification effects and the 100-year return period of the design-level earthquake.

The outcomes of the study strongly suggest that there is an urgent need to revise EBCS 8, 1995 with the objective to account for the above two major issues among others. The post-Loma-Prieta studies on site effects provided sufficient evidence suggesting the use of higher site amplification factors in all period ranges. This has already been addressed in contemporary major seismic codes worldwide including in Africa. EBCS 8 should follow suite, especially with the current construction boom and the relaxed quality control in sight.

Furthermore, it is proposed that a new nation-wide seismic-hazard study is conducted based on an updated catalogue, employing state-of-the-art hazard analysis methods and using appropriate attenuation rules. The GSHAP study results can be used as a good benchmark for this purpose.

It is also strongly recommended that the rather risky 100-year return period, which is currently in use, is critically revisited in consultation with policy makers, property owners, financiers, insurers and other stakeholders.

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REFERENCES

[1] Ministry of Works and Urban Development, *"Design of Structures for Earthquake Resistance"*, Ethiopian Building Code Standard (EBCS 8), Addis Ababa, 1995.

[2] Building Seismic Safety Council (BSSC), *"1994 Edition NEHRP Recommended Provisions for Seismic Regulations for New Buildings"*, FEMA 222A and 223A (Provisions and Commentary), Washington DC, 1995.

[3] Building Seismic Safety Council (BSSC), *"1997 Edition NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures"*, FEMA 302 and 303 (Provisions and Commentary), Washington DC, 1998.

[4] Building Seismic Safety Council (BSSC), *"2003 Edition NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures"*, FEMA 450 (Provisions and Commentary), Washington DC, 2004.

[5] Ghosh, S., "*Trends in the Seismic Design Provisions of US Building Codes*," PCI Journal, Sept.-Oct. 2001, pp. 98-102.

[6] Ghosh, S., "*Update on the NEHRP Provisions: The Resource Document for Seismic Design*," PCI Journal, May-June, 2004, pp. 96-102.

[7] European Committee for Standardization, Eurocode 8, "*Design Provisions for Earthquake Resistance for Structures (ENV 1998)*", Brussels, May 1994.

[8] European Committee for Standardization, Eurocode 8, "*Design of Structures for Earthquake Resistance (EN 1998-1: 2004)*", Brussels, 2004.

[9] Rey, J., Faccioli, E. and Bommer, J., "*Derivation of Design Soil Coefficients (S) and Response Spectral Shapes for Eurocode 8 Using the European Strong-Motion Database*", Journal of Seismology, Vol. 6, 2002, pp. 547-555.

[10] South African National Standard, SANS 10160-1, *"Basis of Structural Design and actions for Buildings and Industrial structures"*, SABS Standards Division, Pretoria, 2010.

[11] Wium, J., "*Background to SANS 10160 (2009): Part 4 Seismic Loading*", Journal of the South African Institution of Civil Engineering, Vol. 52, No. 1, 2010, pp. 20-27.

[12] Seed, H., Ugas, C. and Lysmer, J., "*Site Dependent Spectra for Earthquake-Resistant Design*", Bulletin Of Seismological Society of America, Vol. 66, No. 1, 1976, pp. 221-244.

[13] Mohraz, B., "*A study of Earthquake Response Spectra for Different Geological Conditions*", Bulletin of Seismological Society of America, Vol. 66, No. 3, 1976, pp. 915-935.

[14] Idriss, I.M., "*Response of soft soil sites during earthquakes*", Proceedings of the Symposium to Honor Professor H. B. Seed, Berkeley, May, 1990, pp. 273-289.

[15] Idriss, I.M., "*Earthquake ground motions at soft soil sites*", Proceedings of the Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Missouri, Vol. III, 1991. pp. 2265-2273.

[16] Dobry, R., Borcherdt, R., Crouse, B., Idriss, I., Joyner, W., Martin G., Power, M., Rinne, E. and Seed, R., "*New Site Coefficients and Site Classification System Used in Recent Building Seismic Code Provisions*", Earthquake Spectra, Vol. 16, No. 1, February 2000, pp. 41-67,

[17] Dobry, R. and Susumu, I., "*Recent Development in The Understanding of Earthquake Site Response and Associated Seismic Code Implementation*", In Proc GeoEng2000, An International Conference on Geotechnical & Geological Engineering: Melbourne, Australia, 2000, pp. 186-129.

[18] Borcherdt, R., "*Estimates of Site-Dependent Spectra for Design (Methodology and Justification)*", Earthquake Spectra, Vol. 10, No. 4, 1994, pp. 617-653.

[19] Borcherdt, R. and Fumal, T., "*Empirical Evidence from the Northridge Earthquake for Site-Specific Amplification Factors Used in US Building Codes*" *in* Proc. 12 World Conference on Earthquake Engineering, Auckland, NZ, 2000, pp. 1-6.

[20] Borcherdt, R., "*Empirical Evidence for site Coefficients in Building Code Provisions*", Earthquake Spectra, Vol. 18, No. 2, 2002, pp. 189-217.

[21] Crouse, C. and McGuire, J., "*Site Response Studies for the Purpose of Revising NEHRP Seismic Provisions*", Earthquake Spectra, Vol. 12, No. 2, 2002, pp. 407-439.

[22] Rodriguez-Marek, A., Bray, J. and Abrahamson, N., "*Characterization of Site Response General Categories*", PEER Report 1999/03, Pacific Earthquake Engineering Research Center, Berkeley, California, 1999.

[23] Stewart, J., Liu, A. and Choi, Y., "*Amplification factors for spectral acceleration in tectonically active regions*" Bull. Seism. Soc. Am., 93 (1), 2003, pp. 332-352.

[24] Roesset, J., "*Soil Amplification in Earthquakes", in* Numerical Methods in Geotechnical Engineering, ed. pp. 639-682, McGraw Hill, New-York, 1977.

[25] Worku, A., "*Comparison of Seismic Provisions of EBCS 8 and Current Major Building Codes Pertinent to the Equivalent Static Force analysis*", Zede, Journal of the Ethiopian Engineers and Architects, Vol. 18, 2001, pp. 11-25.

[26] http://www.seismo.ethz.ch/static/gshap/Gshap98-stc.html