J Seismol (2006) 10:335–351
DOI 10.1007/s10950-006-9020-4

ORIGINAL ARTICLE

A comparative study of the design spectra defined by Eurocode 8, UBC, IBC and Turkish Earthquake Code on R/C sample buildings

Adem Doğangün · Ramazan Livaoğlu

Received: 8 November 2004 / Accepted: 20 April 2006

Abstract Earthquake codes have been revised and updated depending on the improvements in the representation of ground motions, soils and structures. These revisions have been more frequently seen in recent years. One of the key changes in earthquake codes has been performed on the design spectra. In this paper, the design spectra recommended by Turkish Earthquake Code and three other well known codes (Uniform Building Code, Eurocode 8, and International Building Code) are considered for comparison. The main purpose of this study is to investigate the differences caused by the use of different codes in the dynamic analysis and seismic verification of given types of buildings located at code defined different sites. The differences in expressions and some important points for elastic and inelastic spectra defined by the codes are briefly illustrated in tables and figures. Periods, base shears, lateral displacements and interstory drifts for the analyzed buildings located at code defined ground type are comparatively presented.

Keywords Design spectra · Eurocode 8 · Ground types · IBC · Seismic codes · Seismic response of building · Turkish earthquake code · UBC

A. Doğangün (✉) · R. Livaoğlu
Karadeniz Technical University, Department of Civil Engineering, 61080 Trabzon, Turkey

Introduction

Earthquake codes are periodically revised and updated depending on the improvements in the representation of ground motions, soils and structures. Moreover, these revisions have been made more frequently in recent years. The Turkish Earthquake Code (TEC, 1998) was also revised in 1997 and has been in effect since 1998. Unfortunately, two destructive earthquakes [Kocaeli and Düzce] occurred in Turkey in 1999 one year after the enforcement of TEC. These earthquakes resulted in more than 18,000 recorded deaths and 50,000 serious injuries. More than 51,000 buildings were either heavily damaged or totally collapsed. Some other 110,000 buildings moderately or lightly suffered damage. The damage inflicted by the Kocaeli earthquake on structures in general, and on modern RC buildings in particular, is perhaps the worst seen in many years, and certainly the worst in Europe in recent history. Average total loss may be in the range of 16 Billion USD, about 7% of the Nation's GDP (Erdik, 2004). A number of separate teams from different disciplines conducted damage surveys and reported some conclusions following the two major earthquakes as briefly summarized below.

Early reports stressed that quality of construction material were poor and that there were many structural mistakes and deficiencies due to the non-compliance with the earthquake code. It was concluded that the nature of the strong-motion was also a major contributing factor to the level of damage. Some of the teams indicated that soil effects played an important role on damage with different forms such as liquefaction, bearing capacity loss, subsidence and lateral spread. In some studies, response spectra computed from recorded accelerograms were compared with design spectra derived from TEC. It was concluded that in a period range of about 1.0–1.5 sec, which corresponds to the pulse periods of the Bolu and Düzce records the capacity demand is considerably in excess of the design spectrum curve (Akkar and Gülkan, 2002). It was also concluded that the acceleration response spectra for the Düzce motion exceeded the UBC (1997) design spectrum at periods less than 0.5 s and greater than 3.0 s (Erdik, 2004). Another spectra-related study in which the observed and simulated displacement spectra were compared with the Eurocode 8 design displacement spectrum for the Bolu expressway viaduct was carried out by Faccioli et al. (2002). One another conclusion was that peak acceleration for most near-source records were not so high as expected and seemed to become saturated for increasing magnitudes. However, the peak velocities and corresponding drift demands were usually considerable, and this was confirmed by structural damage (Akkar and Gülkan, 2002).

Under the light of the observations and lessons from the 1999 earthquakes, many studies concerning TEC have been carried out up to now and a number of improvements in TEC might be brought up to the agenda in the near future. As known, one of the key changes in the earthquake codes has been generally performed on the design spectra. The requirements recommended by EC8 are also very important for the Turkish community; so it will be helpful to establish a comparison between the design spectra recommended by the EC8 (2004) and TEC. The influence of local ground conditions on the seismic action is generally considered depending on the ground types described by the stratigraphic profiles and parameters. That is why the emphasis has been placed on the differences caused by the use of spectra given in the TEC and other well known codes such as UBC, EC8 and IBC(2003) in the seismic analysis of sample buildings.

The effects of ground types defined in the codes on the seismic response of buildings were also investigated for both different building configurations and different supporting soils. Base shears and interstory drifts for considered ground types and buildings are compared to reveal the differences. As design requirements given in the earthquake codes are significantly different, the seismic design of buildings is out of this paper scope. However, authors tried to consider basic design requirements recommended in the codes for selection of the building systems.

Seismic hazard is expressed in TEC and EC8 by a single parameter, namely the reference peak ground acceleration a_{gR} at the surface on rock for a reference mean return period. The reference return period recommended for the non-collapse performance level is the 475 year, corresponding to 10% probability of exceedance in 50 years in a Poisson occurrence process both TEC and EC8. A comparative study of the seismic hazard assessments in European National Seismic Codes including TEC was carried out by Garcia-Mayordomo et al. (2004). In EC8 the design ground acceleration (a_g) is equal to a_{gR} times the importance factor γ_I. A single parameter, however, is insufficient to characterize the ground motion hazard, because the frequency content as well as the amplitude of the ground motion affect structural response. The standard method as in TEC and in EC8 is to describe the frequency content by a response spectrum. Two influences on frequency content are recognized by TEC and EC8, namely the type of ground present at the site under consideration and the magnitude of earthquake. The former is accounted for by describing various ground types as mentioned below:

Code defined ground types

The site conditions have been classified into different categories in earthquake codes. These categories are named ground types, soil profile types, local site classes or subsoil classes. In this paper, the term of ground types is selected in accordance with EC8. Table 1 presents ground types and shear wave velocities given in the codes including FEMA 368 (2001). As seen from this table, TEC gives more information about ground types depending on the topmost layer thickness of soil (h_1). Four and six ground types are defined in TEC and US codes, respectively. It should be noted that in the 1998 edition of EC8 only three types of A, B and C were defined. However, five main ground types as to be A, B, C, D, E and two special ground types S_1 and S_2 have been described in the final version of EC8.

The site classification system is based on definitions of site classes in terms of a representative average shear wave velocity, Standard Penetration Test blow-count,

Table 1 Ground types defined in the TEC, UBC, IBC, FEMA 368 and EC8

	TEC		UBC, IBC & FEMA 368		EC8	
	Ground types	Description	Ground types	Description	Ground types	Description
Z1	–	Massive volcanic rocks, unweathered sound metamorphic rocks, stiff cemented sedimentary rocks $V_s >$ 1000 m/s; Very dense sand, gravel V_s >700 m/s; Hard clay, silty clay $V_s >$ 700 m/s	S_A^*	Hard rock V_s >1500 m/s		
	$h_1^+ \leq 15$ m	Soft volcanic rocks such as tuff and agglomerate, weathered cemented sedimentary rocks with planes of discontinuity $V_s \approx 700 – 1000$; Dense sand, gravel $V_s \approx 400 – 700$; Very stiff clay, silty clay $V_s \approx 300–700$	S_B	Rock $V_s \approx 760 – 1500$	A	Rock or rock-like geological formation including most 5 m weaker material at the surface $V_{s,30}$ >800 m/s
Z2	$h_1 > 15$ m	Soft volcanic rocks such as tuff and agglomerate, weathered cemented sedimentary rocks with planes of discontinuity $V_s \approx 700 – 1000$; Dense sand, gravel $V_s \approx 400 – 700$ Very stiff clay, silty clay $V_s \approx 300 – 700$	S_C	Very dense soil or soft rock $V_s \approx 360 – 760$	B	Deposit of very dense sand, gravel or very stiff clay, at least several tens of m in thicknesses, characterized by a gradual increase of mechanical properties with depth $V_{s,30} \approx$ 360 – 800
	$h_1 \leq 15$ m	Highly weathered soft metamorphic rocks and cemented sedimentary rocks with planes of discontinuity $V_s \approx 400 – 700$; Medium dense sand and gravel $V_s \approx$ 200 – 400; Stiff clay, silty clay $V_s \approx$ 200 – 300				
Z3	15 m $< h_1 \leq 50$ m	Highly weathered soft metamorphic rocks and cemented sedimentary rocks with planes of discontinuity $V_s \approx 400 – 700$; Medium dense sand and gravel $V_s \approx$ 200 – 400; Stiff clay, silty clay $V_s \approx$ 200 – 300	S_D	Stiff soil $V_{s,30} \approx$ 180 – 360	C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of m $V_s \approx 180 – 360$
	$h_1 \leq 10$ m	Soft, deep alluvial layers with high water table $V_s < 200$; Loose sand $V_s \approx 200$**;** Soft clay, silty clay $V_s < 200$				

(*Continued on next page*)

Table 1 (*Continued*)

Z4	h_1>50 m	Highly weathered soft metamorphic rocks and cemented sedimentary rocks with planes of discontinuity $V_s \approx 400-700$; Medium dense sand and gravel $V_s \approx 200-400$; Stiff clay, silty clay $V_s \approx 200-300$	S_E	Soft soil $V_s <$ 180	D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil. $V_{s,30} < 180$
	$H_1 > 10$ m	Soft, deep alluvial layers with high water table $V_s < 200$; Loose sand $V_s < 200$; Soft clay, silty clay $V_s < 200$				
-		In all seismic zones, soft, deep alluvial layers with high water table $V_s < 200$, loose sand $V_s < 200$ and soft clay, silty clay $V_s < 200$ with water table less than 10 m from the soil surface shall be investigated and the results shall be documented to identify whether the *Liquefaction Potential* exists, by using appropriate analytical methods based on in-situ and laboratory tests.	S_F	Soil requiring site specific evaluation. It is more detailed defined in the IBC	E	A soil profile consisting of a surface alluvium layer with $V_{s,30}$ values of class C or D and thick-ness varying between about 5 and 20 m, underlain by stiffer material with $V_{s,30} >$ 800 m/s
					S_1	Deposits consisting or containing a layer at least 10 m thick of soft clays/ silts with high plasticity index (PI > 40) and height water content, $V_{s,30} < 100$ m/s
					S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types *A–E* or S_1

*S_A, S_B, S_C, S_D, S_E and S_F given in the UBC are symbolized with A, B, C, D, E, and F, respectively, in the IBC and FEMA 368
+h_1 is the topmost layer thickness for subsoil

unconfined compression strength, relative density, etc. in some earthquake codes like TEC. However, based on empirical studies by Borcherdt (1994), recommended shear wave velocity V_{S-30} as a means of classifying sites for building codes, and similar site categories were selected for the FEMA seismic design provisions for new buildings (Dobry et al., 2000). Boore et al. (1994) indicate that the ideal parameter would be the average shear-wave velocity to a depth of one-quarter wavelength for the period of interest, as was used by Joyner and Fumal (1984). By the quarter-wavelength rule, 30 m is the appropriate depth for period of 0.16 s for stiff soil and period values tend to increase as the soil gets softer (Boore et al., 1994). It should be noted that code defined spectra depending on ground types are provided only for cases where the 30 m of soil immediately below the site dominates the frequency content of the design motions. The average shear wave velocity of the upper 30 m of soil (V_{S-30}) is also considered in EC8 and the velocity bounds of 360 m/s and 180 m/s for types B, C and D, make the velocity values consistent with the N_{SPT} values (Sabetta and Bommer, 2002).

Elastic and inelastic response and design spectra

Various seismological and geophysical parameters affect the shape of response spectra. Ambraseys et al. (1996) and Bommer and Acevedo (2004) presented and discussed the effects of earthquake magnitude, source-to-site distance, site classification, and style-of-faulting on the strong-motion accelerograms and consequently response spectra. As known, the damping ratio and structural vibration period are other parameters affecting the response spectra. The earthquake-induced ground shaking is generally represented in the form of acceleration response spectra or displacement response spectra.

Acceleration response spectra

In all current seismic codes, the earthquake actions are represented in the form of a spectrum of absolute acceleration. But code acceleration spectra tend to be conservative at longer periods with the result that the long-period ordinates of the displacement spectra are unnecessarily high (Bommer et al., 2000). This has

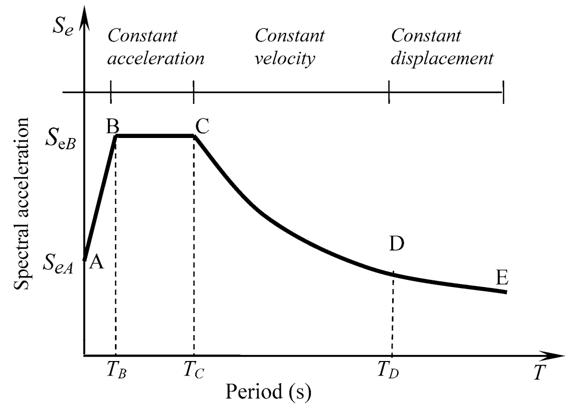


Fig. 1 Typical shape of elastic design spectra.

been shown to be case for EC8 by Tolis and Faccioli (1999).

A typical shape of horizontal elastic design spectrum can be drawn as seen in Fig. 1. In this figure, T shows periods of structure, S_{eA} and S_{eB} show the ordinate values at points A and B of the elastic design spectra, T_B and T_C show the lower and the upper limits of the period of the constant spectral acceleration branch, and T_D shows the value defining the beginning of the constant displacement response range of the spectrum. There are some differences in spectral shapes recommended by the earthquake codes. Therefore, the differences and similarities in the spectra used by considered seismic codes are mentioned below:

EC8 has a note starting that, if deep geology is not accounted for, the recommended choice is the use of two types of spectra: Type 1 and Type 2. If the earthquakes that contribute most to the seismic hazard defined for the site for the purpose of probabilistic hazard assessment have a surface-wave magnitude, M_s, not greater than 5.5, it is recommended that the Type 2 spectrum is adopted.

To show differences and similarities, important points of elastic design spectra shown in Fig. 1 and requirements related to these points defined in the TEC, UBC and EC8 are shown comparatively in Table 2. In this table, S is the soil factor defined in EC8 depending on ground types and η is the damping correction factor with a reference value of $\eta = 1$ for 5% viscous damping.

The ordinates of elastic design spectra S_e and inelastic design spectra S_d for the reference return period defined by the earthquake codes except IBC can be determined using the expressions given in Table 3. In

Table 2 Comment on the important periods of elastic design spectra defined in the TEC, UBC and EC8

	TEC	UBC	EC 8 Two types of spectrum (Type-1 and Type-2) defined depending on the magnitude of earthquakes
T_B	Defined depending only subsoil classes. The values of T_B corresponding to Z1, Z2, Z3 and Z4 ground types can be taken 0.10, 0.15, 0.15 and 0.20, respectively.	Defined depending on ground types and earthquake zone. In addition, for seismic zone 4 it also depends on the near source factors. $T_B = 0.2 \frac{C_v}{2.5 \cdot C_a}$ The value of T_B is changed from 0.1 s to 0.22 s	• For Type-1 the values of T_B correspond-ding to A, B, C, D and E ground types can be taken 0.15, 0.15, 0.20, 0.20 and 0.15, respectively. • For Type-2 the values of T_B correspond-ding to A, B, C, D and E ground types can be taken 0.05, 0.05, 0.10, 0.10 and 0.05, respectively.
T_C	Defined depending only subsoil classes. The values of T_C corresponding to Z1, Z2, Z3 and Z4 ground types can be taken 0.3, 0.4, 0.6 and 0.9, respectively.	Defined depending on ground types and earthquake zone. In addition, for seismic zone 4 it depends on the near source factors. $T_C = \frac{C_v}{2.5 \cdot C_a}$ The value of T_C is changed from 0.4 s to 1.07 s	•,For Type-1 The values of T_C corresponding to A, B, C, D and E ground types can be taken 0.4, 0.5, 0.6, 0.8 and 0.5, respectively. • For Type-2 The values of T_C corresponding to A, B, C, D and E ground types can be taken 0.25, 0.25, 0.25, 0.3 and 0.25, respectively.
T_D	–(not defined)	–(not defined)	• It is equal to 2 for Type-1 It is equal to 1.2 for Type-2
S_{eA} S_{eB}	$1.0 \cdot a_g$ $2.5 \cdot a_g$	$1.0 \cdot C_a \cdot g$ $2.5 \cdot C_a \cdot g$ Where C_a and C_v are soil modified ground motion parameters	$S\ a_g$ $2.5 \cdot Sg \cdot \eta\ a_g$ • For Type-1 considering 5% damping ratio, S_e corresponding to A, B, C, D and E ground types can be taken a_g times 2.5, 3, 2.875, 3.375 and 3.5, respectively. • For Type-2 considering 5% damping ratio, S_e corresponding to A, B, C, D and E ground types can be taken a_g times 2.5, 3.375, 3.75, 4.5 and 4, respectively.

Table 3 Ordinates of elastic and inelastic design spectra (S_e and S_d) for TEC, UBC and EC8

	$T \leq T_B$	$T_B \leq T \leq T_C$	$T \geq T_C$
TEC	$S_e = a_{gR}[1 + 1.5\frac{T}{T_B}]$ $S_d = \frac{a_g}{R_a}[1 + 1.5\frac{T}{T_B}]$	$S_e = 2.5 \cdot a_{gR}$ $S_d = \frac{2.5 \cdot a_g}{R_a}$	$S_e = 2.5 \cdot a_{gR}[\frac{T_C}{T}]^{0.8}$ $S_d = \frac{2.5a_g}{R_a}[\frac{T_C}{T}]^{0.8}$
UBC	$S_e = [C_a + \frac{1.5 \cdot C_a \cdot T}{T_B}] \cdot g$ $S_d = [C_a + \frac{1.5 \cdot C_a \cdot T}{T_B}] \cdot g \cdot \frac{\gamma_I}{R}$	$S_e = 2.5 \cdot C_a \cdot g$ $S_d = 2.5 \cdot C_a \cdot g \cdot \frac{\gamma_I}{R}$	$S_e = \frac{C_v}{T} g$ $S_d = \frac{C_v}{T} g \cdot \frac{\gamma_I}{R}$
EC8	$S_e = a_g \cdot S[1 + \frac{T}{T_B}(\eta 2.5 - 1)]$ $S_d = a_g S[\frac{2}{3} + \frac{T}{T_B}(\frac{2.5}{q} - \frac{2}{3})]$	$S_e = 2.5 \cdot a_g \cdot S \cdot \eta$ $S_d = \frac{2.5}{q} \cdot a_g \cdot \mathrm{S}$	$T_C \leq T \leq T_D \rightarrow S_e = 2.5a_g \cdot S \cdot \eta \cdot [\frac{T_C}{T}]$ $T_C \leq T \leq T_D \rightarrow S_d \begin{cases} = \frac{2.5}{q} a_g \cdot S \cdot [\frac{T_C}{T}] \\ \geq \beta \cdot a_g \end{cases}$ $T_D \leq T \leq 4s \rightarrow S_e = 2.5a_g \cdot S \cdot \eta \cdot [\frac{T_C T_D}{T^2}]$ $T \geq T_D \rightarrow S_d = \frac{2.5}{q} a_g \cdot S \cdot [\frac{T_C T_D}{T^2}] \geq \beta \cdot a_g$

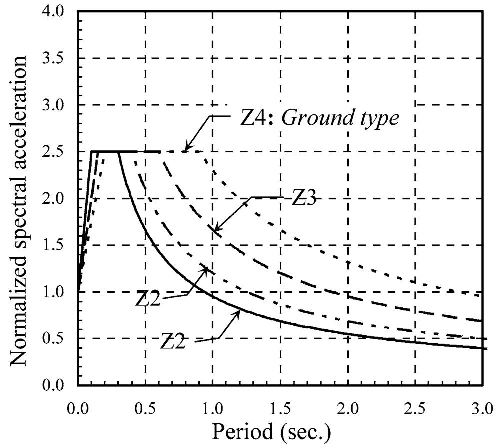
this table, β shows lower bound factor for the horizontal design spectrum, recommended value for β is 0.2 and γ_I shows importance factor. The design spectra defined by IBC are not directly compared with the others (TEC, UBC, EC8) due to significant differences for seismic design provisions. The IBC does not use seismic zones to establish design earthquake ground motion or to impose additional design requirements and structural limitations. Rather than seismic zone, the IBC uses a parameter called Seismic Design Category as the mechanism for imposing design restrictions, detailing requirements, and structural limitations. The seismic design category assigned to a building is important in that it affects the permissible analysis procedures, applicability of structural redundancy, method of lateral load distribution, limitations on structural systems, applicability of special load combinations, and ductile detailing requirements. The design ground motion parameters fall into are now S_{DS} and S_{D1} rather than seismic zone factor. S_{DS} and S_{D1} are the ordinate values that equal to five-percent-damped design spectral response accelerations at short periods and 1 second period respectively. S_{DS} determines the upper-bound design base shear (the "flat-top" of the design spectrum) used in seismic design. S_{D1} defines the descending branch or the period-dependent part of the design spectrum. The values of S_{DS} and S_{D1} are estimated depending on the mapped spectral response accelerations prepared for USA as explained below.

S_{DS} and S_{D1} are two-thirds of S_{MS} and S_{M1}, which are the soil modified spectral response accelerations at short period and 1.0 second period, respectively. S_{MS} is obtained by multiplying the mapped spectral acceleration S_S by F_a, the acceleration-related soil factor. S_{M1} is similarly obtained by multiplying the mapped spectral acceleration S_1 by F_v, the velocity-related soil factor. The specific factor F_a is defined over a low-period range ($T = 0.1$–0.5 sec) and F_v which is defined over a mid-period range ($T = 0.4$–2 sec). These site factors derived using both observational and analysis-based approaches (Dobry et al., 2000) are analogous to C_a/Z and C_v/Z of the UBC 97. Response acceleration at any period in the high frequency range is equal to the design spectral response acceleration at short periods and any point in the constant velocity range can be obtained from the relationship of S_{D1}/T.

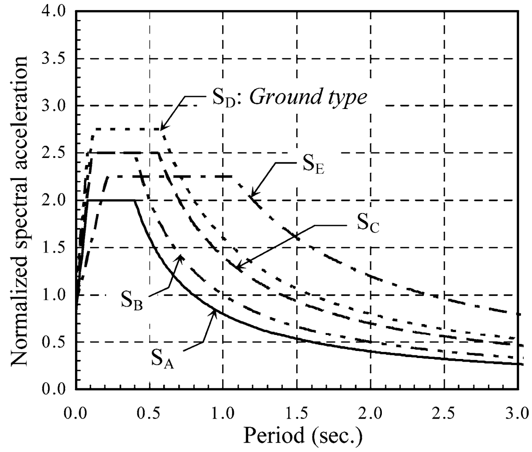
Elastic design spectra were drawn as shown in Fig. 2 using the expressions shown in Table 1–3 for all ground types defined in the codes. As seen from Fig. 2, only TEC considers the same peak values for all ground types. EC8 gives the maximum peak values for ground types other than ground type A. The shapes of the elastic response spectrum of Type-2 are more peaking for short period structures except for ground type A.

The concept of dividing the elastic response spectra by a single factor to arrive at the inelastic design spectra is a practical one and has been adopted by most earthquake codes. The factor used for reducing the elastic response spectrum is called behaviour factor (q) in EC8, response modification coefficient (R) in FEMA 368 (2001), R coefficient in UBC and the seismic load reduction factor R_a in the TEC.

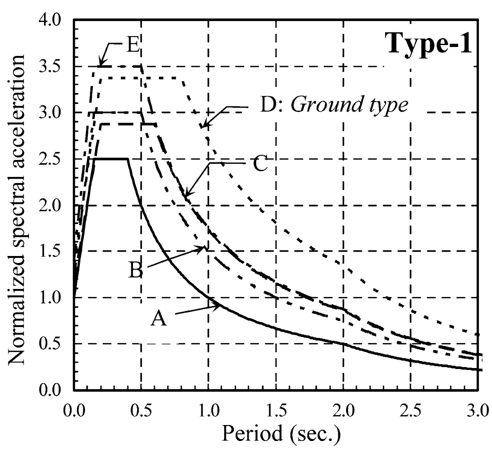
Earthquake codes describe different behaviour factors. The values of the maximum allowable behaviour factor are taken considering the type of structural system, regularity in elevation and prevailing failure mode in the system with walls in EC8, whereas, TEC specifies

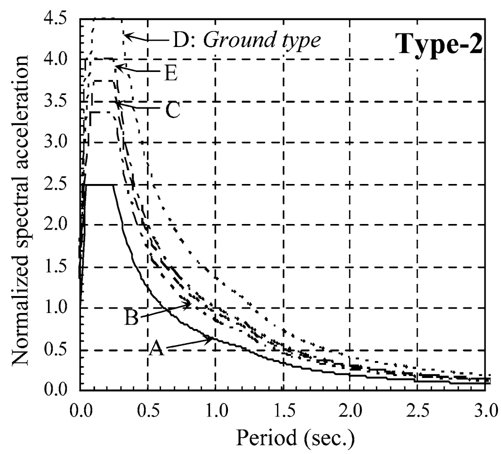


(a) Elastic design spectra for TEC



(b) Elastic design spectra for UBC





(c) Elastic design spectra of Type-1 and Type-2 for EC8 (η were taken to be 1.0)

Fig. 2 Normalized elastic design spectra drawn for ground types described in TEC, UBC and EC8 (they normalized by the design ground acceleration)

periods (T and T_B) dependent values of behaviour factor in addition to structural system.

Figure 3 illustrates inelastic design spectra that were obtained considering the structure importance factor equal to 1, the behaviour factor equal to 4 and the reference peak ground acceleration equal to 0.4 g for sample structures and soil conditions. As seen from this figure, the trend of the graph for UBC with the period values from zero to $T_{B,}$ the first limit of the constant spectral acceleration branch, is different from those drawn for TEC and EC8. Eurocode 8 gives the maximum peak values for ground types other than ground type A. The values of S_d dramatically decrease from 0 to T_B for EC8. The maximum peak values obtained for the EC8 Type-2 spectrum and the large values obtained for this spectrum are within the small period range. The ordinates of Type-1 and Type-2 are constant after period T_D, whereas there is not a lower bound constant value for the other codes.

Displacement response spectra

Sözen and his associates developed the substructure concept that enabled the use of an elastic displacement spectrum in design by using the displacement capacity of an inelastic system in 1970s (Gülkan and Sözen, 1974). But force-based seismic design remains, in spite of its shortcomings, the method most widely used in codes up to recent years. However, the recognition of the poor correlation between transient inertial forces

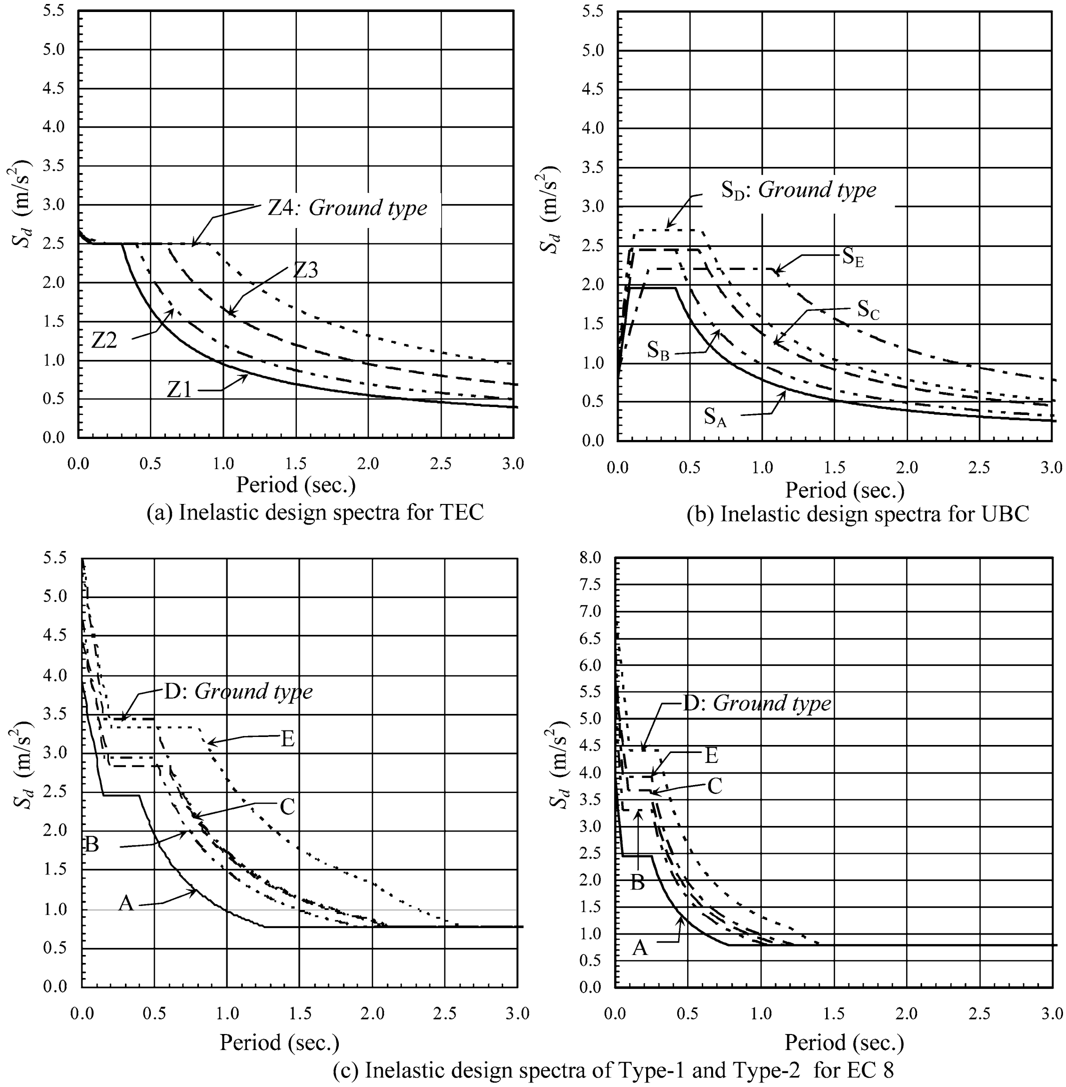


Fig. 3 Inelastic design spectra drawn for ground types described in TEC, UBC and EC 8 for a reference peak ground acceleration of 4 m/s^2

induced by earthquake shaking and damage to structures has led to the development of displacement-based approaches, which utilize displacement spectra. The first and most obvious way to obtain elastic displacement spectra (S_{De}) for design would be to convert the elastic design spectrum of absolute acceleration (S_e) defined in the code, via the pseudo-spectral relationship:

$$S_{De}(T) = \frac{S_e(T)}{\omega^2} \tag{1}$$

The resulting displacement spectra obtained in this way are used in more than 20 seismic design codes from around the world (Bommer et al., 2001). The transformation of the acceleration spectra in current seismic codes to displacement spectra will generally not produce reliable displacement ordinates at the longer periods relevant to displacement-based design (Bommer and Elnashai, 1999; Faccioli et al., 2004). This transformation shall normally be applied for vibration periods not exceeding 4 seconds. For structures with vibration periods greater than 4 sec, a more complete definition of the elastic spectrum is presented in EC8. There is currently a significant level of disagreement regarding appropriate values for the control periods of the displacement spectra (Bommer and Mendis, 2005).

Faccioli et al. (1998) derived relationship between damping and ductility for displacement spectra depending on European earthquakes and Borzi et al. (2001) described derivation of inelastic displacement spectra for displacement-based design in detail. In seismic design codes such as EC8, the displacement spectra for damping ratios other than 5% are obtained by applying scaling factor to the ordinates of the 5% damped spectrum as below;

$$\eta = \sqrt{\frac{10}{5+\xi}} \quad (2)$$

where ξ is the viscous damping ratio of the structure, expressed as a percentage. Bommer and Mendis (2005) have recently reviewed several different proposals for these scaling factors. In the EC8, the displacement spectra were defined for various damping ratios. Since such displacement spectra are not included in the TEC, it is not possible to make a comparison for these spectra.

Structural data

Sample buildings described herein were selected as typical (not a template project) 6 and 12 story reinforced concrete buildings. The buildings have three different floor plans that are symmetric (SB), monosymmetric (MB), and unsymmetric (UB). Six buildings are considered and they are henceforth referred to as; 6-SB, 6-MB, 6-UB, 12-SB, 12-MB and 12-UB. The plan dimensions of buildings, typical at all floors, are 22.7 m by 13.75 m, with a story height of 3 m (Fig. 4). The structural systems of the buildings are selected as structural systems consisting of structural walls and moment resisting frames in both directions. It is assumed that the structural systems have nominal ductility level. In this case, the value of 4 is recommended by TEC for structural behaviour factor (R). Seismic load reduction factor (R_a) can be determined in terms of R according to TEC. If natural vibration period (T) is smaller or equal to the lower limit of the period of the constant spectral acceleration branch (T_B), R_a will be equal to $1.5 + (R\text{-}1.5)T/T_B$. If T is greater than T_B, R_a will be equal to R. As the fundamental periods obtained for sample buildings considered in this study are greater than T_B, R_a is taken equal to R.

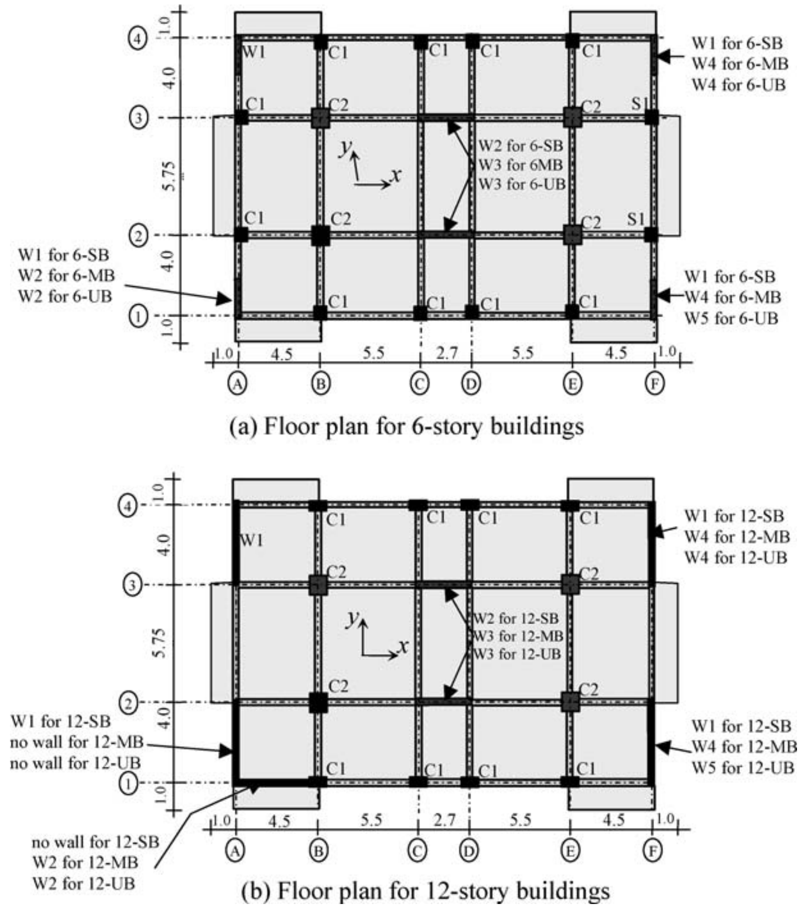
Columns, beams, structural walls and slabs are sized considering the requirements given in TEC. The dimensions of columns and structural walls for x and y directions, the thickness of slabs, the width and height of beams are given in Table 4. As seen from this table, the cross-sections of columns have been changed after the 3rd story for 6-story buildings, and changed after the 4th and 8th story for 12-story buildings.

Flexural rigidities for longitudinal and transverse directions are different for each building. Total moments of inertia of vertical structural elements can be determined using dimensions given in Table 4 for x and y directions. It should be noted that values used for rigidities are gross values and they are not reduced to consider cracking. Minimum and maximum values of Torsional Irregularity Factors (η_b) (which are defined for any of the two orthogonal earthquake directions as the ratio of the maximum storey drift at any story to the average story drift at the same story in the same direction) for sample buildings are also estimated. This factor reach as its maximum values as; 1.12 for 6-SB, 1.55 for 6-MB, 1,39 for 6-UB, 1,22 for 12-SB, 1.34 for 12-MB and 1,89 for 12-UB. According to TEC, torsional irregularity occurs in buildings when η_b is greater than 1.2. No other structural irregularities occurred for sample buildings.

Finite element modeling of buildings and analysis results

To evaluate the seismic response of the buildings, elastic analyses were performed by the response spectrum method using the computer program SAP2000 (2003). The seismic analyses of the buildings are carried out separately in the longitudinal and the transverse directions. However, seismic responses only for y direction are comparatively presented with graphs and tables in this paper for the sake of brevity. Sample finite element models of the six and twelve story buildings are shown in Fig. 5. Degrees of freedom at the base nodes are fixed, for other nodes are left free. Therefore, there is no finite element model for subsoil to consider soil-structure interaction. Columns and beams are modelled with frame elements, slabs and structural walls are modelled with shell elements. Slabs also have been considered as a rigid diaphragm in each story level. The masses of infill

Fig. 4 Floor plan for six and twelve story buildings (lengths of spans are in m)



walls are also taken into account in the model. In the analysis, Young's modulus and unit weight of concrete are taken to be 28000 MPa and 25 kN/m^3, respectively. The damping ratio is assumed as 5% in all modes.

The reference peak ground acceleration is taken to be 0.4 g that is recommended as seismic zone 1 in TEC and 4 in UBC. Thus, it is assumed that the buildings are sited in high seismicity zone. Seismic analysis of the buildings accounting for the influence of the local ground conditions is carried out with the help of the design spectra given in Fig. 3 for TEC, UBC and EC 8.

Periods for the analyzed buildings

The mode numbers taken into account for six and twelve-story buildings are 10 and 20, respectively. The first seven or eight modes with periods and participating mass ratios for the buildings are presented in Table 5. As seen from this table, the fundamental periods are in the range between 0.463 s and 1.043 s. In the first mode the 6-SB, 6-MB, 6-UB and 12-MB vibrate dominantly in the y direction; whereas 12-SB and 12-UB vibrate in the x direction. The third mode takes place as torsional modes for all buildings considered.

Base shears for the analyzed buildings

The base shear expressions defined in the codes are given in Table 6. The base shears of the buildings were acquired from seismic analysis using the design spectra corresponding to 5% critical damping and considering fixed base condition. Seismic analysis of buildings were carried out for four ground types defined in TEC, five ground types in UBC and in EC8. Therefore, fourteen ground types in total are considered for the site. Figs. 6 and 7 present the base shears and maximum differences obtained for 6 and 12-story buildings, respectively.

Table 4 Dimensions of structural elements for buildings considered with (mm) unit

		Six story buildings				Twelve story buildings					
		1–3 stories		4–6 stories		1–4 stories		5–8 stories		9–12 stories	
Buildings	Structural elements	b_x	b_y	b_x	b_y	b_x	b_y	b_x	b_y	b_x	b_y
Symmetric buildings	C1	600	600	500	500	900	300	700	300	400	300
	C2	900	900	700	700	900	900	700	700	400	400
	W1,W2,W4,W5	250	1750	250	1750	300	4300	300	4300	300	4300
	W3	3000	250	3000	250	3000	300	3000	300	3000	300
Mono-symmetric buildings	C1	600	600	500	500	900	300	700	300	400	300
	C2	900	900	700	700	900	900	700	700	400	400
	W1,W2	1750	250	1750	250	4800	300	4800	300	4300	300
	W3	3000	250	3000	250	3000	300	3000	300	3000	300
	W4,W5	250	1750	250	1750	300	4300	300	4300	300	4300
Unsymmetric buildings	C1	600	600	500	500	900	300	700	300	400	300
	C2	900	900	700	700	900	900	700	700	400	400
	W1	250	1750	250	1750	300	4300	300	4300	300	4300
	W2	3000	250	3000	250	4800	300	4800	300	4300	300
	W3	3000	250	3000	250	3000	300	3000	300	3000	300
	W4,W5	250	1750	250	1750	300	4300	300	4300	300	4300
Thickness of slabs	150					150					
Cross section of beams	250 × 500					300 × 600					

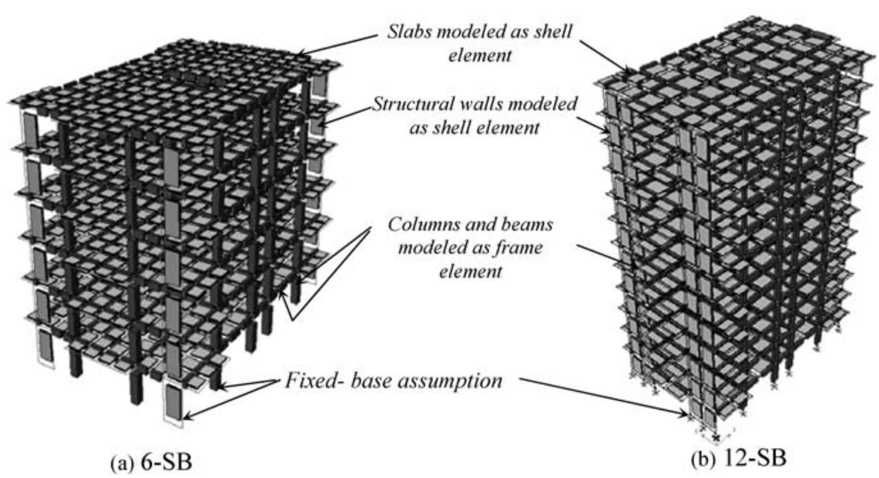


Fig. 5 The views of three-dimensional finite element models of six and twelve story buildings

As seen from Fig. 6, EC8 gives the maximum base shears for similar ground types defined in the TEC and UBC. TEC gives the maximum base shear for ground type Z4. But almost the same values are obtained for Z3 & Z4 and maximum difference reaches 53% between Z4 and Z1 for 6-MB. UBC gives the maximum base shear for ground types of S_D, maximum difference reaches 86% between S_D and S_A for 6-MB and this code gives smaller base shear values for S_E than for S_C and S_D. EC 8 gives the maximum base shear for ground types E & D and the maximum difference reaches 87% between D and A for 6-MB.

As seen from Fig. 7, although there is a dominant ground type D for maximum base shear in EC8, there

Table 5 First seven/eight periods (s) and modal properties for six different buildings considered

Buildings		Horizontal modes for the buildings						Torsional mode	
		x-direction			y-direction				
6-SB	Mode, period	2nd, 0.440	5th, 0.126	–	1st, 0.463	4th, 0.139	7th, 0.068	3rd, 0.371	6th 0.111
	Mass ratio	0.000	0.000		0.748	0.135	0.053	0.000	0.000
6-MB	Mode, period	2nd, 0.418	5th, 0.119	–	1st, 0.546	4th, 0.175	7th, 0.093	3rd, 0.388	6th, 0.117
	Mass ratio	0.000	0.003		0.658	0.103	0.040	0.104	0.021
6-UB	Mode, period	2nd, 0.430	5th, 0.123	–	1st, 0.493	4th, 0.151	7th, 0.076	3rd, 0.382	6th, 0.115
	Mass ratio	0.004	0.001		0.714	0.123	0.048	0.035	0.081
12-SB	Mode, period	1st, 0.871	4th, 0.298	7th, 0.156	2nd, 0.825	5th, 0.226	8th, 0.100	3rd, 0.622	6th, 0.174
	Mass ratio	0.000	0.000	0.000	0.687	0.157	0.000	0.000	0.000
12-MB	Mode, period	2nd, 0.769	5th, 0.229	8th, 0.115	1st, 1.043	4th, 0.348	6th, 0.182	3rd, 0.534	7th, 0.157
	Mass ratio	0.000	0.000	0.021	0.543	0.091	0.046	0.188	0.034
12-UB	Mode, period	1st, 0.924	4th, 0.273	7th, 0.130	2nd, 0.813	5th, 0.254	8th, 0.117	3rd, 0.648	6th, 0.180
	Mass ratio	0.013	0.049	0.016	0.634	0.088	0.016	0.022	0.061

Table 6 Base shear defined in the TEC, UBC and EC8

Codes	Base shear
TEC	$V_t = S_d(T) \cdot \frac{W}{g} \geq 0.1 A_o \cdot \gamma_I \cdot W$
UBC	$V_s = S_d(T)\frac{W}{g} \geq \begin{cases} 0.11 C_a \cdot \gamma_I \cdot W \\ \frac{0.8 Z N_v \gamma_I}{R} W \ \text{(for zone 4)} \end{cases} \qquad \frac{2.5 C_a \gamma_I}{R} W \geq V_s$
	$T = \min\,(T_a,\ 1.3T_e)$ for zone 4
EC8	$F_b = S_d(T) \cdot \frac{W}{g} \cdot \lambda$ where $\lambda = 0.85$ if $T_1 \leq 2\ T_C$ or $\lambda = 1.00$ otherwise
	Note: The effective modal mass m_k, corresponding to a mode k, is determined so that the base shear force F_{bk}, acting in the direction of application of the seismic action, may be expressed as $F_{bk} = S_d(T_k)\ m_k$

Note. For S_d (T) given in this table, different expressions introduced in Table 4 depending on soil and structure properties including structural behaviour factor

is no a dominant ground type like D in TEC and UBC. TEC presents the maximum base shear for ground type of Z4 and maximum difference reaches 128% between Z4 and Z1 for 12-UB. UBC gives the maximum base shear for ground type S_E and maximum difference reaches 146% between S_E and S_A for 12-UB. EC8 gives the maximum base shear for ground type D and maximum difference reaches 154% between classes D and A for 12-UB.

The story number of story in which the maximum shear force occurred was investigated. Maximum shear force occurs at the 1st story for 6-SB and 6-MB, 12-SB and 12-UB, whereas it occurs at the 4th story for 6-UB and 12-MB.

As seen from Figs. 6 and 7, very different base shear values were obtained for different ground types and building structural systems. Although all ground types defined in the codes are included in this study, a general conclusion cannot be reached due to the limited number (6) of buildings. For a general conclusion, more buildings having different structural systems and period values should be investigated taking into account various design ground acceleration. Even now, it may be said mainly two things for considered buildings: Firstly differences for maximum base shears are tendency to increase for unsymmetrical buildings. Secondly when the number of story increase, in this case mass of the structure is also increase, the differences between base shears obtained for various ground types become larger. These increases for six to twelve story buildings are meanly 69% for TEC, 60% for UBC and 75% for EC 8.

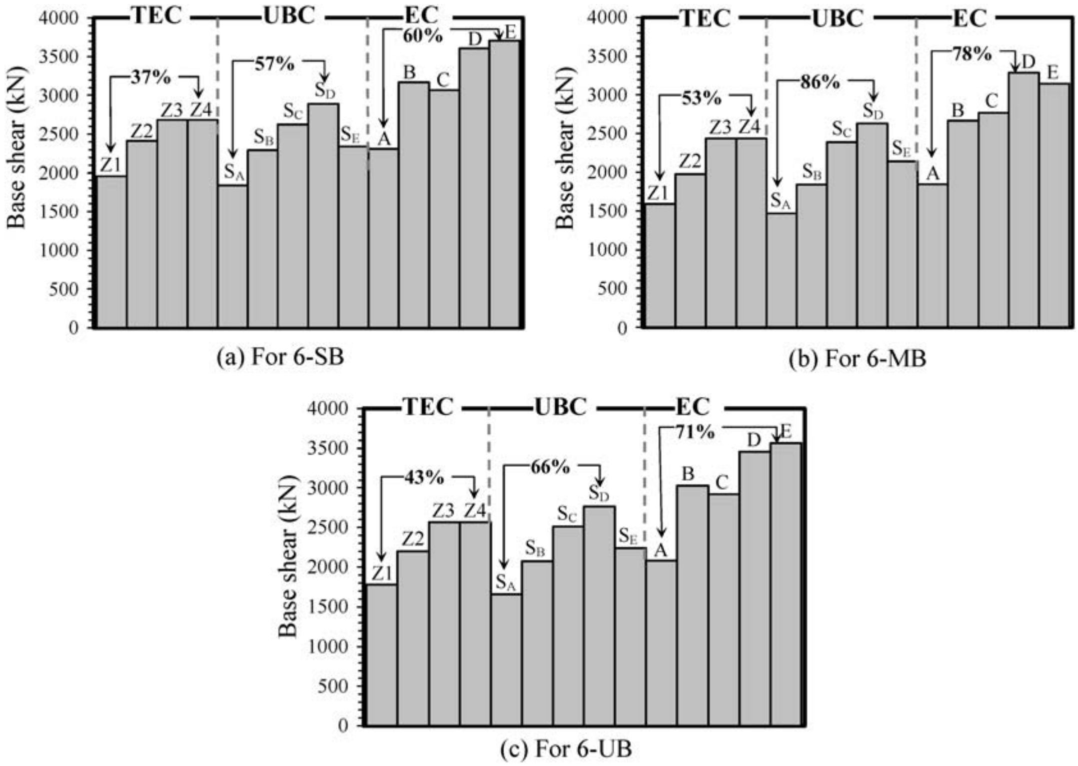


Fig. 6 Base shears for 6-story buildings considering fourteen ground types defined in the codes

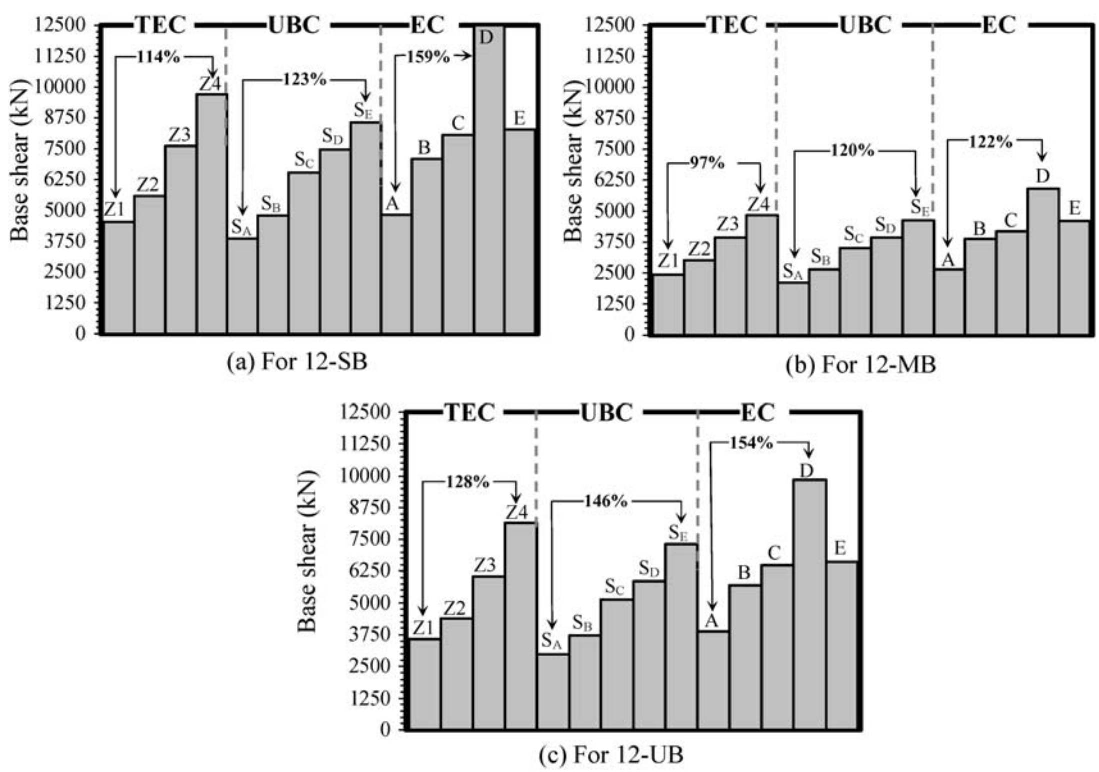


Fig. 7 Base shears for 12-story buildings considering fourteen ground types defined in the codes

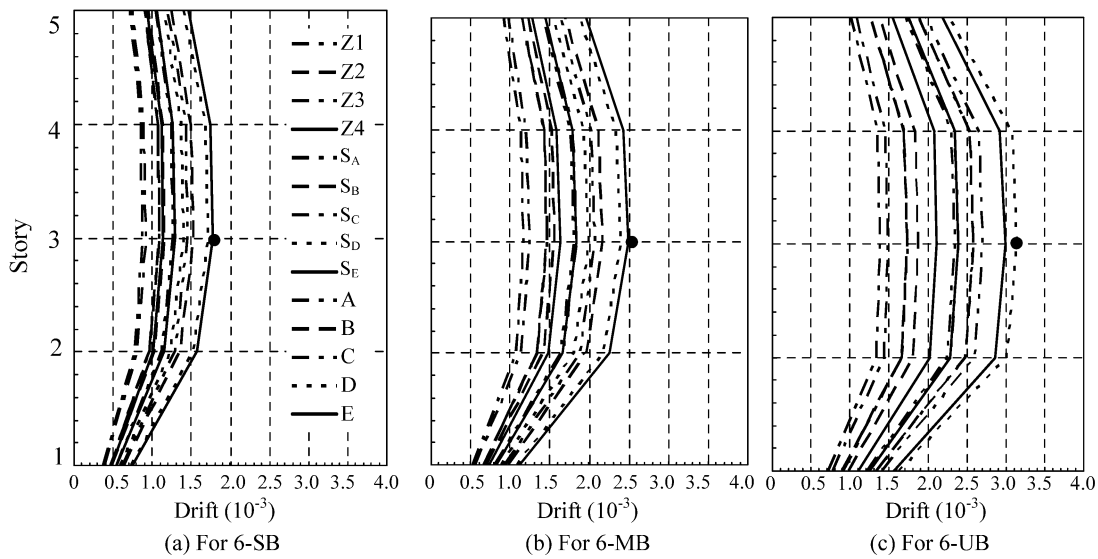


Fig. 8 Drift for the 6-story buildings considering ground types defined in the codes

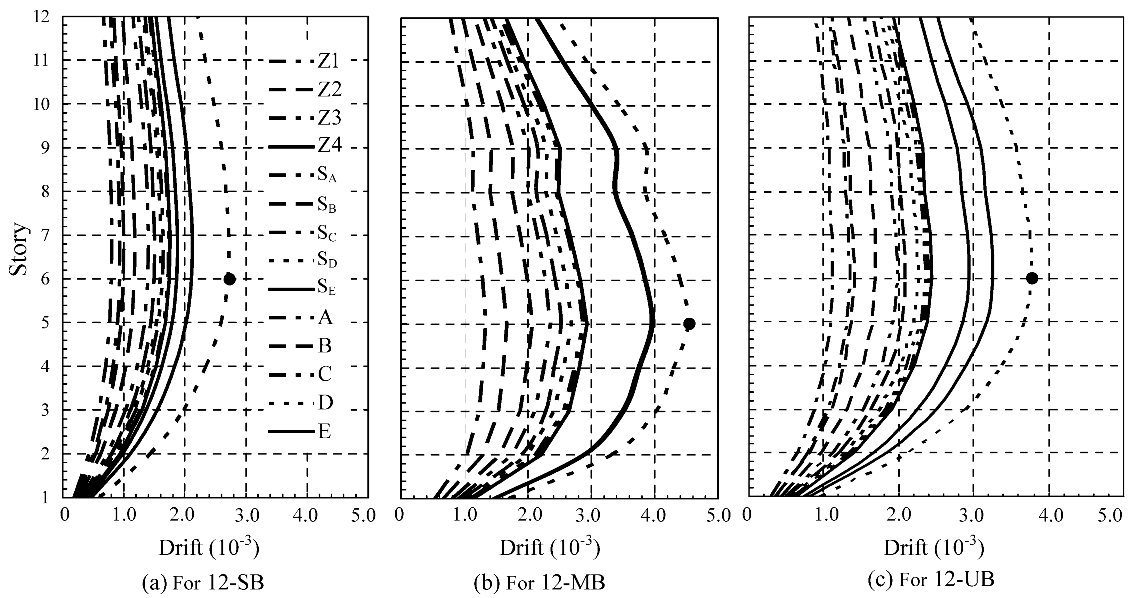


Fig. 9 Drifts for the 12-story buildings considering ground types defined in the codes

Lateral displacements and interstory drifts for the analyzed buildings

Minimum lateral displacements were estimated for all the buildings with ground types S_A (12 mm for 6-SB, 17 mm for 6-MB, 19 mm for 6-UB, 24 mm for 12-SB, 33 mm for 12-MB and 39 mm for 12-UB). EC8 gives the maximum, and UBC gives the minimum lateral displacement values for the buildings. Approximately the same displaced shapes are obtained for 6-SB ground types of S_E & Z2; A & S_B; Z1 & S_A; S_C & Z4. Similar cases occur for the ground types of A & S_B for 6-MB and 6-UB. For 12-story buildings, ground types of A & S_B for 12-SB and 12-UB give almost the same deformed shapes.

Figures 8 and 9 present drifts estimated from seismic analysis of the 6 and 12-story buildings, respectively. As seen from Fig. 8, the maximum value of story drifts

Table 7 Maximum and minimum displacements obtained for the buildings considering each code

Buildings		TEC			UBC			EC8		
		Min.	Max.	%	Min.	Max.	%	Min.	Max.	%
6-SB	Displacement	13 mm	18 mm	38	12 mm	20 mm	67	16 mm	25 mm	119
	Ground type	Z1	Z4		S_A	S_D		A	E	
6-MB	Displacement	21 mm	33 mm	57	19 mm	36 mm	89	24 mm	43 mm	79
	Ground type	Z1	Z4		S_A	S_D		A	D	
6-UB	Displacement	17 mm	26 mm	53	17 mm	28 mm	65	21 mm	35 mm	67
	Ground type	Z1	Z4		S_A	S_D		A	E	
12-SB	Displacement	28 mm	62 mm	121	24 mm	55 mm	129	30 mm	80 mm	167
	Ground type	Z1	Z4		S_A	S_E		A	D	
12-MB	Displacement	47 mm	113 mm	140	39 mm	112 mm	187	48 mm	129 mm	169
	Ground type	Z1	Z4		S_A	S_E		A	D	
12-UB	Displacement	40 mm	96 mm	140	33 mm	86 mm	161	42 mm	112 mm	167
	Ground type	Z1	Z4		S_A	S_E		A	D	

within a story, $(\Delta_i)_{\max,}$ for columns and structural walls of the ith story of a building for each earthquake direction satisfies the conditions given by $(\Delta_i)_{\max}/h_i \leq 0.0035$ (defined in the TEC) for the 6-story building. However, as seen from Fig. 9, the maximum values of the story drift exceed the condition for 12-MB supported on ground types of D, Z4, S_E, and for 12-UB supported on ground type of D.

All maximum and minimum displacement values determined for each code are given in Table 7. As seen from this table, the smallest differences between maximum and minimum displacement values for the 6-story buildings are obtained as to be 38% between Z1 and Z4 in TEC, 65% between S_A and S_D in UBC and 67% between A and E in EC8. The largest differences between maximum and minimum displacement values for the 12-story buildings are obtained as to be 140% between Z1 and Z4 in TEC, 187% between S_A and S_E in UBC and 169% between A and D in EC8. It should be noted here that when the soil gets softer, as mentioned above, the lateral displacements are increase.

Conclusions

The differences among the code defined response spectra may be summarized as: (a) The near source factors are considered only in the UBC. Such near source factors are not defined in EC8 and TEC. EC8 has a footnote such that the Type 2 Spectrum is adopted when a surface-wave magnitude is not greater than 5.5. (b) The ordinate value of design spectra increases with T for UBC, decreases for the TEC and EC8 for small values of vibration period ($T < T_B$). (c) The TEC specifies the same peak values for all ground types, whereas UBC and EC8 specify different peak values. (d) EC8 specifies the values of the maximum allowable behaviour factor depending on type of structural system, regularity in elevation and prevailing failure mode in the system with walls, whereas TEC specifies periods for structure and ground class (T and T_B) dependent values of behaviour factor in addition to structural system. (e) Although all domains of the response spectrum are defined differently in the EC8, the constant displacement and constant velocity domains are not defined differently in UBC and TEC.

There are significant differences between the three codes (TEC, UBC, EC8) and IBC&FEMA seismic design provisions. The biggest change related to the design spectra from the codes to the IBC is in the design ground motion parameters, now S_{DS} and S_{D1}, rather than seismic zone factor.

EC8 presents an annex for elastic displacement spectrum for periods of long vibration period. As the current trend in seismic design is displacement-based, it is expected that the displacement design spectra and for different peak values of the separate ground types are also included in the new versions of the TEC.

For the buildings, EC8 gives the maximum and UBC the minimum displacement values. EC8 generally gives the larger base shear for similar ground types defined in the other codes. The maximum base shears occurred for ground types of D or E defined in EC8. The number of the story in which maximum shear force is occurring changes depending on the ground types.

Some engineers share the view that internal forces decrease and lateral displacements increase from the first ground type to the last ground type code defined, that is, as if the ground becomes softer. This view is verified in the analysis carried out considering all ground types and the requirements defined in TEC, while it is valid only for the first two ground types defined in EC8 and UBC when analyzing the sample buildings. However, the view loses its meaning especially for the last two ground types given in EC8 and UBC, because larger internal forces are obtained for the 4th ground types (D in EC8, S_D in UBC) than that for the 5th ground type (E in EC8, S_E in UBC). Therefore, this may lead to some mistakes for design for soft soils. It should be noted that ground class E does not always identify a soil profile softer than class D.

Acknowledgement Authors wish to tank Ezio Faccioli for his helpful comments and suggestions on earlier versions of this manuscript.

References

Akkar S, Gülkan P (2002) A critical examination of near-field accelerograms from the Sea of Marmara region earthquakes. Bull Seism Soc Am 92:428–477

Ambraseys N, Simpson K, Bommer J (1996) Prediction of horizontal response spectra in Europe. Earthq Engng Struct Dyn 25:371–400

Bommer JJ, Elnashai AS (1999) Displacement spectra for seismic design. J Earthq Engng 3:1–32

Bommer JJ, Elnashai AS, Weir AG (2000) Compatible acceleration and displacement spectra for seismic design codes. Proc 12th World Conf Earthq Engng, Auckland, Paper no. 207

Bommer JJ, Borzi B, Chlimintzas G, Elnashai AS, Lee D, Faccioli E, Tolis S (2001) Definition of displacement response spectra for design. In: Faccioli E, Paolucci R (eds.) Seismic Actions. ECOEST2/ICONS Report No.1, pp 3–34

Bommer JJ, Acevedo AB (2004) The use of real earthquake accelerograms as input to dynamic analysis. J Earthq Engng 8:43–91

Bommer JJ, Mendis R (2005) Scaling of spectral displacement ordinates with damping ratios. Earthq Engng Struct Dyn 34:145–165

Boore DM, Joyner WB, Fumal TE (1994) Estimation of response spectra and peak accelerations from western North American earthquakes, an interim report, USGS Open-File Report 94–127

Borcherdt RD (1994) Estimates of site-dependent response spectra for design (methodology and justification). Earthq Spectra 10:617–653

Borzi B, Calvi GM, Elnashai AS, Faccioli E, Bommer JJ (2001) Inelastic spectra for displacement-based seismic design. Soil Dyn Earthq Engng 21:47–61

Dobry R, Borcherdt RD, Crouse CB, Idriss IM, Joyner WB, Martin GR, Power MS, Rinne EE, Seed RB (2000) New site coefficients and site classification system used in recent building seismic code provisions. Earthq Spectra 16:41–67

EC 8 (2004) Eurocode 8: design of structures for earthquake resistance Part 1: general rules, seismic actions and rules for buildings, European Norm. European Committee for Standardisation, European Committee for Standardisation Central Secretariat, rue de Stassart 36, B-1050 Brussels

Erdik M (2004) Report on 1999 Kocaeli and Düzce (Turkey) earthquakes. Kandilli Obsarvotary and Earthquake Research Institute, Turkey. http://www.koeri.boun.edu.tr/ depremmuh/eqspecials/kocaeli/Kocaelireport.pdf

Faccioli E, Tolis SV, Borzi B, Elnashai AS, Bommer JJ (1998) Recent developments in the definition of the design seismic action in Europe. Proc 11th European Conf Earthq Engng, ISBN 90 54 10 982 3, Balkema, Rotterdam, pp 1–10

Faccioli E, Paolucci R, Pessina V (2002) Engineering assessment of seismic hazard and long period ground motions at the Bolu Viaduct Site following the November 1999 earthquake. J Seism 6:307–327

Faccioli E, Paolucci R, Rey J (2004) Displacement spectra for long periods Earthq Spectra 20:347–376

FEMA 368–369 (2001) NEHPR-recommended provisions for seismic regulations for new buildings. Buildings Seismic Safety Council, Washington, DC, USA

Garcia-Mayordomo J, Faccioli E, Paolucci R (2004) Comparative study of the seismic hazard assessments in European national seismic codes. Bull Earthq Engng 2:51–73

Gülkan P, Sözen MA (1974) Inelastic response of reinforced concrete structures to earthquake motions. ACI Journal 71:604–610

IBC (2003) The International Building Code. International Code Council, Virginia, USA

Joyner WB, Fumal TE (1984) Use of measured shear wave velocity for predicting geologic site effects on strong ground motions. Proc 8th World Conf Earthq Engng San Francisco 2:777–783

Sabetta F, Bommer JJ (2002) Modification of the spectral shapes and subsoil conditions in Eurocode 8. Proc 12th European Conf Earthq Engng London, Paper No 518

SAP2000 (2003) Integrated Software for Structural Analysis and Design: Computers and Struct. Inc. Berkeley, California

TEC (1998) Specification for Structures to be Built in Disaster Areas, Ministry of Public Works and Settlement Government of Republic of Turkey, 85 pp

Tolis SV, Faccioli E (1999) Displacement design spectra. J Earthq Engng 3:107–125

UBC (1997) Uniform Building Code, International Conference of Building Officials. Whittier, California, USA