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Effect of design spectral shape on inelastic response of RC frames subjected to spectrum matched ground motions

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Abstract. In current seismic design codes, various elastic design acceleration spectra are defined considering different seismological and soil characteristics and are widely used tool for calculation of seismic loads acting on structures. Response spectrum analyses directly use the elastic design acceleration spectra whereas time history analyses use acceleration records of earthquakes whose acceleration spectra fit the design spectra of seismic codes. Due to the fact that obtaining coherent structural response quantities with the seismic design code considerations is a desired circumstance in dynamic analyses, the response spectra of earthquake records used in time history analyses had better fit to the design acceleration spectra of seismic codes. This paper evaluates structural response distributions of multi-story reinforced concrete frames obtained from nonlinear time history analyses which are performed by using the scaled earthquake records compatible with various elastic design spectra. Time domain scaling procedure is used while processing the response spectrum of real accelerograms to fit the design acceleration spectra. The elastic acceleration design spectra of Turkish Seismic Design Code 2007, Uniform Building Code 1997 and Eurocode 8 are considered as target spectra in the scaling procedure. Soil classes in different seismic codes are appropriately matched up with each other according to V_{S30} values. The maximum roof displacements and the total base shears of considered frame structures are determined from nonlinear time history analyses using the scaled earthquake records and the results are presented by graphs and tables. Coherent structural response quantities reflecting the influence of elastic design spectra of various seismic codes are obtained.

Keywords: seismic design codes; elastic design acceleration spectra; time domain scaling procedure; nonlinear time history analyses; dynamic response quantities

1. Introduction

Seismic design codes are periodically revised guidelines depending upon the improvements in the assignation of strong ground motions, soils and structures. The design and seismic evaluation of new structures in earthquake-prone regions of the world have to satisfy the minimum requirements of seismic design codes in order to reduce structural damages, limit seismic risk and minimize the loss of life by preventing the collapse of structures. In reference to current seismic design codes, structures are designed to withstand the severe earthquake of a certain probability that is likely to occur. The provisions of current seismic design codes are intended to ensure that structures can sufficiently resist seismic forces during strong ground motion effects.

The seismic action in design codes is basically represented in the form of elastic design acceleration response spectrum. The elastic design spectrum is constructed in accordance with the response spectrum of the seismic design code, however, a site-specific spectrum may be developed as alternative to the code-based design acceleration spectrum in view of geological, seismological and soil characteristics of the specific site region. Additionally, the seismic motion may also be represented in terms of appropriate ground acceleration time histories and related quantities, such as velocity and displacement. The seismic records used in dynamic analyses may be real accelerograms as well as artificial accelerograms.

Ground motion time histories reflecting the site characteristics can be used in dynamic analyses in accordance with the requirements of seismic design codes. In time history analyses, using the strong ground motion records without scaling according to the specific code-based design acceleration spectrum is likely to be incoherent so, they have to be scaled by using an appropriate method to make them compatible with code-specific hazard levels. Accordingly, in nonlinear time history (NLTH) analyses of structures seismic input is usually defined in terms of acceleration time series whose response spectra are compatible with a specified target response spectrum (Hong and Xu 2007, Atik and Abrahamson 2010, Cacciola 2010, Iervolino *et al*. 2010, Cacciola and Deodatis 2011, Yazdani and Takada 2011, Gao *et al*. 2014, Kayhan 2016, Kayhan and Demir 2016). Therefore, coherent structural response estimations may be obtained by using the accelerograms having similar strong motion characteristics.

In previous studies, it is stated that there are many

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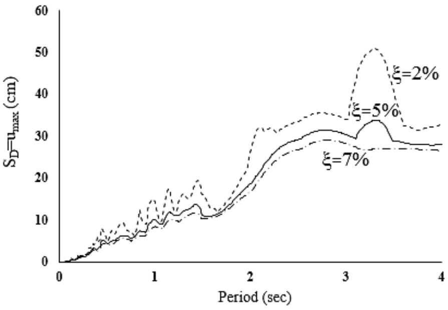


Fig. 1 Displacement response spectra for Superstition Hills-02 ground motion

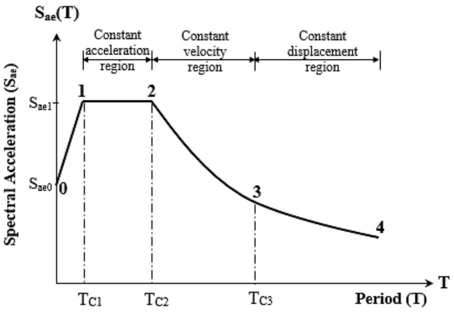
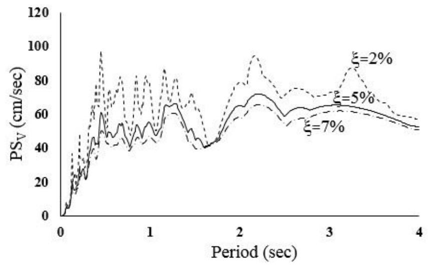
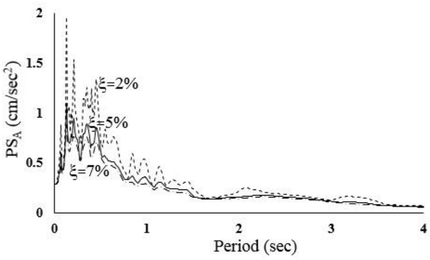


Fig. 3 Typical elastic design acceleration spectra



(a) Pseudo-velocity response spectra



(b) Pseudo-acceleration response spectra

Fig. 2 Response spectra (ξ =2%, 5% and 7%) for Superstition Hills-02 ground motion

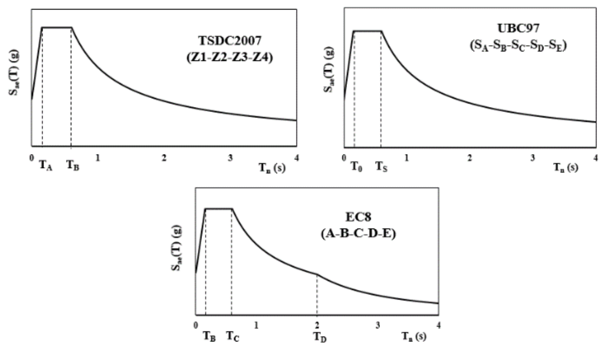


Fig. 4 Control periods of response spectra for TSDC2007, UBC97 and EC8

Table 1 Soil profile types in EC8 and UBC97

EC8		UBC97	
Soil Profile Type	V_{S30} (m/s)	Soil Profile Type	V_S (m/s)
A	> 800	S_A	> 1500
B	360 – 800	S_B	760 – 1500
C	180 – 360	S_C	360 – 760
D	< 180	S_D	180 – 360
E	–	S_E	< 180

differences in the response parameters of dynamic analyses performed by considering different accelerograms and seismic design codes (Bommer and Acevedo 2004, Dogangun and Livaoglu 2006, Iervolino *et al*. 2008, Atkinson 2009, Giaralisa and Spanos 2009, NEHRP Consultants Joint Venture 2011, Giaralis and Spanos 2012, Gascot and Montejo 2016). In these studies, it has also been indicated that the scaling procedure of ground motions has an importance so that the structural response quantities from the analyses can be obtained more compatible. There are many distinct studies in literature that have discussed the subject of scaling on the response of structures (Nau and Hall 1984, Bommer and Acevedo 2004, Iervolino and Cornell 2005, Watson-Lamprey and Abrahamson 2006, Luco and Bazzurro 2007, Kalkan and Chopra 2010, Ay and Akkar 2012, Khoshnoud and Marsono 2012, Wood and Hutchinson 2012, Zekkos *et al*. 2012, Ozdemir and Gulkan 2015, Pant and Maharjan 2016).

Elastic acceleration response spectra of three different seismic design codes, Turkish Seismic Design Code 2007 (TSDC 2007), Uniform Building Code 1997 (UBC 1997) and Eurocode 8 (EC8 2004), are constructed. Seven recorded accelerograms are assembled and scaled in time-domain to match elastic design spectra of three different seismic codes. NLTH analyses of 6-, 8- and 10-story 4-bay reinforced concrete (RC) frames are performed by using time histories of the scaled ground motions. The maximum roof displacements and base shear forces are considered as response parameters within the present study and the average of these parameters obtained from NLTH analyses are compared with each other. Additionally, the average base shear forces are compared with different code based modal base shears. Owing to the fact that the earthquake records are scaled in time domain compatible with different elastic design acceleration spectra, consistent response parameters are obtained from nonlinear time history analyses of the selected multi-story RC frames. The paper presents a comparative study towards understanding the seismic design philosophy of different design codes.

2. Response spectrum concept

A response spectrum, which may be described as a plot of the peak dynamic response parameter of many linear single-degree-of-freedom (SDOF) oscillators having a

Table 2 Variation of control periods in different seismic codes in terms of soil profile types

TSDC2007			UBC97					EC8			
Soil Profile Type	Control Periods T_A (s)	Control Periods T_B (s)	Soil Profile Type	C_a (Z=0.4)	C_v (Z=0.4)	Control Periods T_0 (s)	Control Periods T_s (s)	Soil Profile Type	Control Periods T_B (s)	Control Periods T_C (s)	Control Periods T_D (s)
			S_A	$0.32N_a$	$0.32N_v$			A	0.15	0.40	2.0
Z1	0.10	0.30	S_B	$0.40N_a$	$0.40N_v$			B	0.15	0.50	2.0
Z2	0.15	0.40	S_C	$0.40N_a$	$0.56N_v$	$0.2T_s$	$\frac{C_v}{2.5C_a}$	C	0.20	0.60	2.0
Z3	0.15	0.60	S_D	$0.44N_a$	$0.64N_v$			D	0.20	0.80	2.0
Z4	0.20	0.90	S_E	$0.36N_a$	$0.96N_v$			E	0.20	0.50	2.0

Table 3 Seismic coefficients C_a and C_v defined in UBC97

Soil Profile Type	*Coefficient C_a* Z=0.075	Z=0.15	Z=0.2	Z=0.3	Z=0.4
S_A	0.06	0.12	0.16	0.24	$0.32N_a$
S_B	0.08	0.15	0.20	0.30	$0.40N_a$
S_C	0.09	0.18	0.24	0.33	$0.40N_a$
S_D	0.12	0.22	0.28	0.36	$0.44N_a$
S_E	0.19	0.30	0.34	0.36	$0.36N_a$
	Coefficient C_V Z=0.075	Z=0.15	Z=0.2	Z=0.3	Z=0.4
S_A	0.06	0.12	0.16	0.24	$0.32N_v$
S_B	0.08	0.15	0.20	0.30	$0.40N_v$
S_C	0.13	0.25	0.32	0.45	$0.56N_v$
S_D	0.18	0.32	0.40	0.54	$0.64N_v$
S_E	0.26	0.50	0.64	0.84	$0.96N_v$

range of periods and viscous damping ratio, to a particular component of ground motion, is one of the useful tools of earthquake engineering since it provides a simple way of evaluating the response of structures to strong ground motion and to quantify the demands of earthquake on the capacity of structures. The concept of a response spectrum was first introduced by Biot and extended by Housner to engineering applications (Kalkan and Gulkan 2004). Having recognized the importance of the response spectrum approach in the seismic design of structures, the concept of response spectra was incorporated into the United States building codes in the late 1950's. Over the decades, response spectra have been playing an increasing role in the development of earthquake design criteria (Freeman 2007).

Elastic design response spectra, which are envelopes of peak dynamic response parameters for SDOF systems having a certain range of periods, form basis for developing design lateral forces in nonlinear structural systems. For a given ground motion and viscous damping ratio, general equation of motion of a SDOF system may be written as

$$m \cdot \ddot{u} + c \cdot \dot{u} + f_s(u) = -m \cdot \ddot{u}_g(t) \tag{1}$$

where u: is the relative displacement of SDOF system, m: is the mass, c: is the viscous damping coefficient, $f_s(u)$ is the resisting force and $\ddot{u}_g(t)$: is the acceleration of strong ground motion (Chopra 1995). For a certain ground motion and different damping ratios (ξ), displacement responses of SDOF systems ($u_{\max}$) having different period values may be obtained by solving the differential equation of motion indicated in Eq. (1). A plot of displacement response spectra of SDOF systems with ξ = 2%, 5% and 7% for Superstition Hills-02 ground motion (PEER 2018), is given for illustration in Fig. 1.

The pseudo-velocity and the pseudo-acceleration response spectra for ground motions can be obtained by properly using the computed displacement response spectrum. The relation between the displacement response spectrum (S_D) and the pseudo-velocity (PS_V) and the pseudo-acceleration (PS_A) response spectrum can be interrelated by using the equality of structural dynamics in terms of natural frequency (ω_n) (Chopra 1995) as

$$S_D = \frac{PS_V}{\omega_n} = \frac{PS_A}{\omega_n^2} \tag{2}$$

The pseudo-velocity and the pseudo-acceleration response spectra for Superstition Hills-02 ground motion is shown in Fig. 2. Figs. 1-2 may provide an insight into general shape of the pseudo-velocity and the pseudo-acceleration response spectra.

The elastic design acceleration response spectra in seismic codes characterizing ground motions, show the envelop values of the computed peak dynamic responses

Table 4 Spectral ordinate values of different elastic design spectra

TSDC2007	UBC97	EC8
$0 \le T \le T_A$	$T \le T_o$	$0 \le T \le T_B$
$S_{ae}(T) = A_0 \cdot I \cdot \left(1 + 1.5\dfrac{T}{T_A}\right) \cdot g$	$S_{ae}(T) = \left(C_a + \dfrac{1.5 \cdot C_a \cdot T}{T_o}\right) \cdot g$	$S_{ae}(T) = a_g \cdot S \cdot \left[1 + \dfrac{T}{T_B}(\eta \cdot 2.5 - 1)\right]$
		$T_B \le T \le T_C$
$T_A < T \le T_B$	$T_o \le T \le T_s$	$S_{ae}(T) = a_g \cdot S \cdot \eta \cdot 2.5$
$S_{ae}(T) = A_0 \cdot I \cdot 2.5 \cdot g$	$S_{ae}(T) = 2.5 \cdot C_a \cdot g$	$T_C \le T \le T_D$
$T_B < T$	$T_s \le T$	$S_{ae}(T) = a_g \cdot S \cdot \eta \cdot 2.5 \cdot \left(\dfrac{T_C}{T}\right)$
		$T_D \le T \le 4.0s$
$S_{ae}(T) = A_0 \cdot I \cdot \left[2.5\left(\dfrac{T_B}{T}\right)^{0.8}\right] \cdot g$	$S_{ae}(T) = \left(\dfrac{C_v}{T}\right) \cdot g$	$S_{ae}(T) = a_g \cdot S \cdot \eta \cdot 2.5 \cdot \left(\dfrac{T_C \cdot T_D}{T^2}\right)$

and assess the demands on the structures. It is clearly seen from the plots of the acceleration response spectra that the graphs tend to be constant generally between two control periods. Therefore, the design acceleration response spectra are generalized in seismic design codes worldwide as it is depicted in Fig. 3, where constant acceleration, constant velocity and constant displacement regions of the design spectra are indicated. Many seismic design codes accept two control periods as T_{C1} and T_{C2} whereas the others define three control periods as T_{C1}, T_{C2} and T_{C3} in Fig. 3, where T is the vibration period of the structure and S_{ae0} and S_{ae1} are the spectral ordinates of points 0 and 1, respectively.

3. Elastic design acceleration spectra of TSDC2007, UBC97 and EC8

The earthquake ground motion is generally represented in the form of design acceleration response spectrum in all current seismic codes. Different approaches are used in the definition of elastic acceleration design spectrum of Turkish Seismic Design Code (2007), Uniform Building Code (1997) and Eurocode 8 (2004). Classification of site conditions, expressions of spectral ordinates for the computation of design spectrum and control periods for spectrum are the major differences between these code specified spectra. The classification of soil profile types is based on definitions of site classes in terms of the average shear wave velocity, standard penetration test, undrained shear strength of soil, relative density, etc. More detailed classification depending on the topmost layer thickness of soil is given in TSDC (2007). Soil classes are named as Z1, Z2, Z3 and Z4 in TSDC (2007). According to EC8 (2004), the site should be classified according to the value of the average shear wave velocity, if it is available. The lower and upper limits of average shear wave velocity (denoted as V_{S30} in EC8 and as V_S in UBC97) for each soil profile type are given in Table 1. Site classification is defined as A, B, C, D and E in EC8 (2004), whereas it exists in the form of S_A, S_B, S_C, S_D and S_E profiles in UBC (1997).

Two control periods are proposed by TSDC (2007) (T_A and T_B) and UBC (1997) (T_0 and T_S), whereas three control periods (T_B, T_C and T_D) delineating the constant-acceleration, constant-velocity and constant-displacement spectral regions are available in EC8 (2004) as can be seen from Fig. 4. The variations of control periods in different seismic design codes in terms of soil profile types are summarized in Table 2, where C_a and C_v are seismic coefficients, N_a and N_v are near-source factors and Z is seismic zone factor (UBC 1997).

Table 3 list the seismic coefficients (C_a and C_v) used in determination of control periods of UBC97 spectrum. Z is the seismic zone factor in UBC97 and defined as 0.075, 0.15, 0.2, 0.3 and 0.4.

The horizontal design spectrum expressions proposed in TSDC (2007), UBC (1997) and EC8 (2004) are given in Table 4, where $S_{ae}(T)$ is the elastic response spectrum, T is the vibration period of a linear SDOF system, A_0 is effective ground acceleration coefficient, g is the acceleration of gravity, a_g is the design ground acceleration on soil profile type A, S is the soil factor, η is the damping correction factor with a reference value of η=1 for 5% viscous damping. The analytical expressions of spectral ordinates are defined between control periods of seismic design codes. For elastic design spectra of TSDC (2007) and UBC (1997), there are three spectral regions to be formulated. These are the regions whose period values are between zero and T_A ($0 \le T \le T_A$), between T_A and T_B ($T_A < T \le T_B$) and larger than T_B ($T_B < T$) in TSDC (2007). For UBC97, regions of spectra are divided into three parts considering periods as $T \le T_0$, $T_0 < T \le T_S$ and $T_S \le T$. There are three control periods in EC8 (2004) as indicated in Fig. 4 and accordingly four regions are constituted between the three corner periods as $0 \le T \le T_B$, $T_B < T \le T_C$, $T_C < T \le T_D$ and $\mathrm{T_D} < \mathrm{T} \le 4.0\mathrm{s}$.

Fig. 5 shows graphs of spectral ordinates of horizontal design spectra which are defined in Table 4. Design acceleration spectra are given for TSDC (2007), UBC (1997) and EC8 (2004), respectively. Code-based design

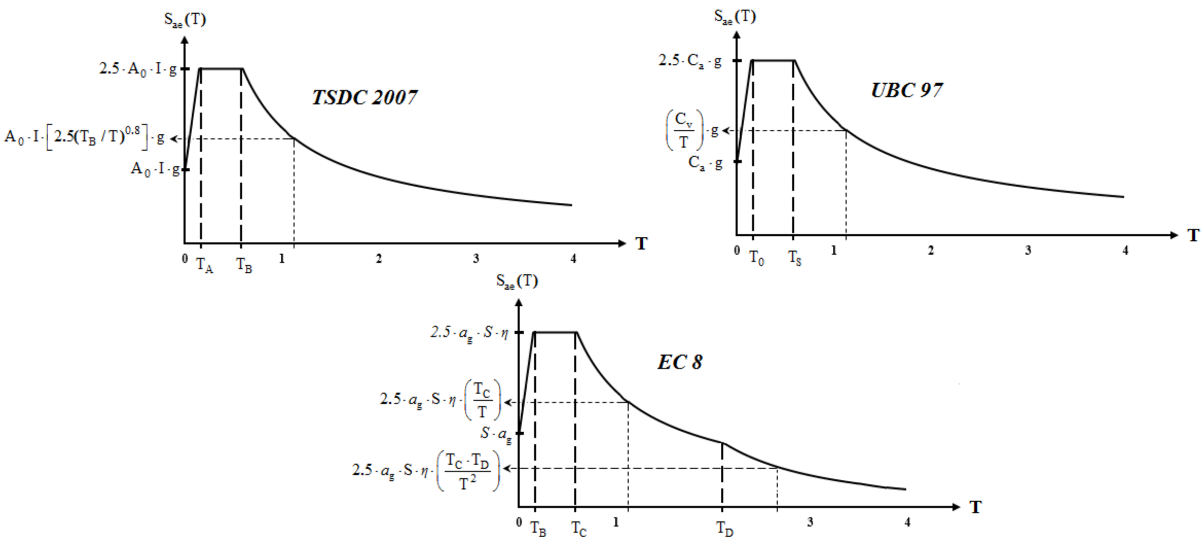


Fig. 5 Spectral ordinates of horizontal design spectra of TSDC2007, UBC97 and EC8

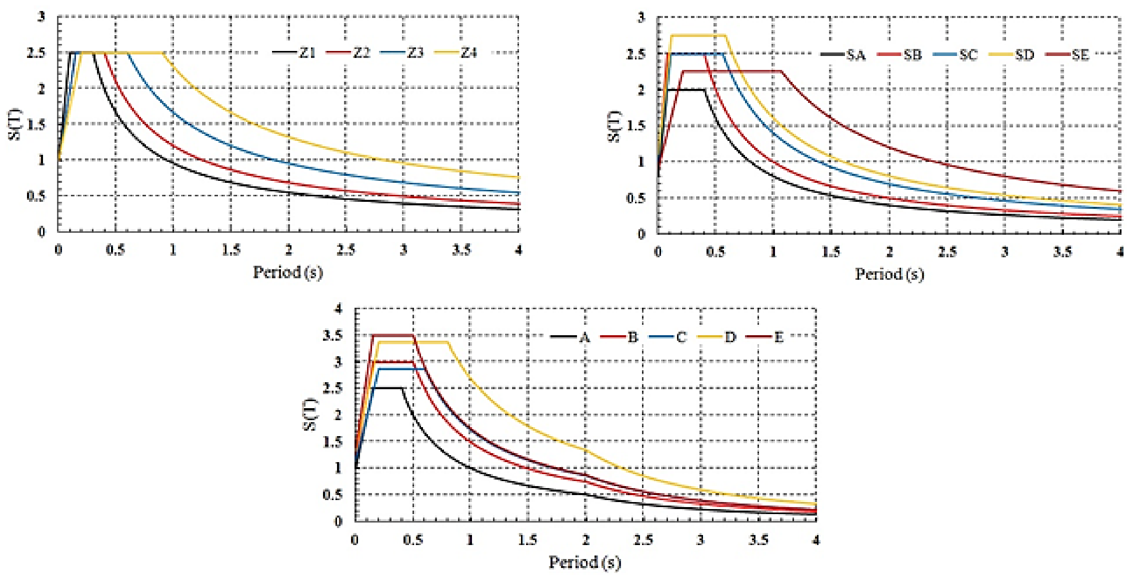


Fig. 6 Design acceleration spectra for TSDC2007, UBC97 and EC8, respectively

spectra considered in the present study can be seen from Fig. 6 for all soil profile types. The graphs are shown in terms of spectrum coefficient, $S(T)$.

4. Scaling procedure of ground motion records

It is of great importance that the acceleration spectra of ground motion records used in nonlinear analyses have to fit to the code-based design acceleration spectra because of having coherent structural responses. Due to major difficulty of assembling records fulfilling duration and amplitude related strict requirements of seismic design codes, scaling of properly selected ground motion time histories so that they match the target design spectrum within a period range of interest can be used (Naeim *et al*. 2004, Fahjan and Özdemir 2008, Kalkan and Chopra 2010, Ay and Akkar 2014). This way, ground motion records are modified appropriately to make them compatible with the code-specific hazard levels and coherent structural response estimations can be obtained by using the accelerograms having similar strong-motion characteristics. Unfortunately, there is currently no consensus on how to appropriately scale earthquake ground motions and several methods of scaling time histories have been proposed. These include time-domain methods, where the spectral acceleration values of the selected time history are simply scaled up or down by a constant scale factor, and frequency-domain methods, where the frequency content of the ground motions are manipulated (Naeim *et al*. 2004, Fahjan and Özdemir 2008).

Scaling process of ground motion to fit the response spectra may be in time domain or frequency domain (Fahjan 2008). The actual acceleration response spectrum of an earthquake is scaled up or down to best match the target spectrum in time domain ground motion scaling procedure.

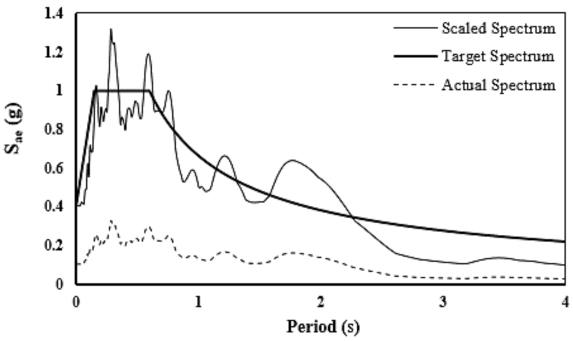


Fig. 7 The scaled, the target and the actual spectrum

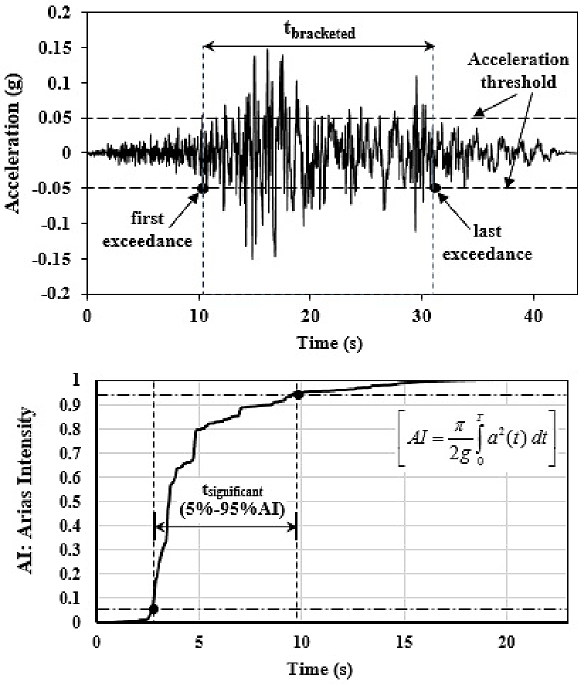


Fig. 8 Bracketed and significant durations

In the frequency domain scaling method, the actual ground motion is filtered in frequency domain to match the target spectrum. In this study, time domain scaling method is used to scale ground motion records.

Fig. 7 shows the actual acceleration spectrum of Big Bear ground motion (PEER 2018), the target spectrum for local site class Z3 in TSDC (2007) and the time domain scaled spectrum of the actual spectrum according to the Z3 spectrum. On account of the fact that the actual spectrum of the earthquake in Fig. 7 has smaller values than the target spectrum, the earthquake motion is scaled up to match with the target spectrum for local site class Z3 (TSDC 2007).

The scaling procedure used herein is based on minimizing the differences between the scaled response spectrum ($S_{ae}^{\text{actual}}(T)$) and target spectrum ($S_{ae}^{\text{target}}(T)$) by using the method of least-squares.

$$\left|\text{Difference}\right| = \int_{T_A}^{T_B} \left[\alpha_s \cdot S_{ae}^{\text{actual}}(T) - S_{ae}^{\text{target}}(T) \right]^2 dT \tag{3}$$

In Eq. (3), $S_{ae}^{\text{actual}}(T)$ and $S_{ae}^{\text{target}}(T)$ are target acceleration response spectrum and acceleration spectrum of the selected time history, respectively, α_s is the constant scale factor and T_A and T_B are lower and upper period of scaling, respectively. The first derivative of the difference function with respect to the scaling factor is equated to zero, i.e., the difference is minimized, and scale factor of each record is computed. Whereas upper limit of scale factor is accepted as 4 for linear elastic analyses, scaling factors in the range of 0.5 to 2 are advised for nonlinear analyses (Fahjan 2008).

Strong shaking duration is usually defined as "bracketed duration" (Bolt 1973, Bommer *et al*. 2009, Lee *et al*. 2015). In this study, the duration of strong ground motion considered as bracketed duration, which is specified as the length of the time interval between the first and last occurrence of a ground acceleration exceeding a fixed threshold value (an absolute 0.05 *g*). Another definition about the duration of strong ground motion is "significant duration". Significant duration of records is computed as the length of the time interval between the two time points in Husid plot when the Arias intensity exceeds 5% and 95% threshold values. The definition of bracketed and significant duration of an earthquake can be clearly understood from Fig. 8. *AI* shows Arias intensity, *g* is the acceleration of gravity and $a(t)$ is the acceleration time history.

5. Ground motion records used in time history analyses

In seismic design codes the earthquake ground motion is generally represented by an elastic acceleration response spectrum. Alternatively, the seismic motion may also be represented in terms of ground acceleration time-histories (TSDC 2007, UBC 1997, EC8 2004). Depending on the nature of the application and on the information actually available, the description of the strong ground motion may be made by using artificial accelerograms and recorded or simulated accelerograms. The attraction of using recorded accelerograms is due to the increase of available strong ground motion databases (Fahjan and Özdemir 2008) and real records ensure realistic information on seismological characteristics such as phasing (Ay and Akkar 2012).

A total of seven recorded accelerograms are assembled according to the moment magnitude, distance, fault type, and soil profile type information. The accelerograms with a magnitude range of and source-to-site distances (R_{JB}) less than 100 km are compiled from the PEER-NGA strong-motion database, which is used as the main source (PEER 2018) in the study. According to the average shear wave velocity (V_{S30}) values, soil profile type definitions of Z3 may be considered as the counterparts of soil profile types S_D in UBC (1997) and C in EC8 (2004) (Ay and Akkar 2012). The selected ground motions have all strike-slip fault mechanism and effects of near fault are not considered. Pulse-like records affected by forward directivity are also not included in the employed set of ground motion records. The list of ground motion records and the overall characteristics of accelerograms are presented in Table 5, where M_w is the moment magnitude of earthquake, R_{JB} is the Joyner-Boore distance, V_{S30} is the average of shear wave velocity in the first 30 m of the soil, *PGA* is the peak ground

Table 5 Major seismological parameters of records

Record Name	Earthquake Name	Recording Station	M_w	R_{JB} (km)	V_{S30} (m/s)	PGA (g)	PGV (cm/s)	PGD (cm)
BIGBEAR_HOS180	Big Bear-01, 1992	San Bernandino-E & Hospitality	6.46	34.98	296.97	0.101	11.85	3.36
BORREGO_A-ELC180	Borrego Mtn, 1968	El Centro Array #9	6.63	45.12	213.44	0.133	26.71	14.56
SUPER.B_B-POE360	Superstition Hills-02, 1987	Poe Road	6.54	11.16	316.64	0.286	29.02	11.56
KOCAELI_DZC180	Kocaeli, Turkey, 1999	Duzce	7.51	13.60	281.86	0.312	58.85	44.05
LANDERS_YER360	Landers, 1992	Yermo Fire Station	7.28	23.62	353.63	0.152	29.60	24.83
KOBE_KAK000	Kobe, 1995	Kakogawa	6.90	22.50	312.00	0.240	20.80	6.39
TRINIDAD.B_B-RDL270	Trinidad, 1980	Rio Dell Overpass-FF	7.20	76.06	311.75	0.151	8.88	3.63

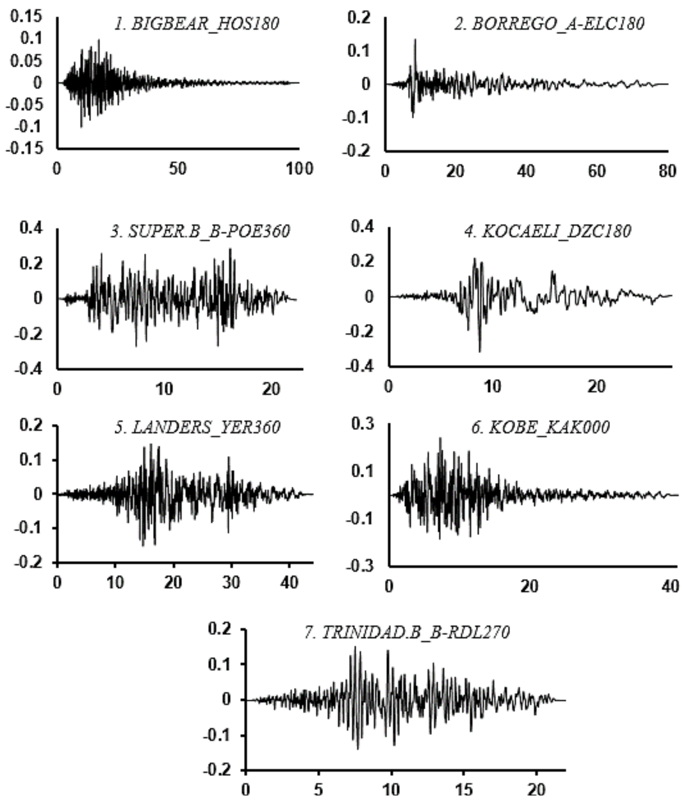


Fig. 9 Acceleration time histories of selected earthquakes

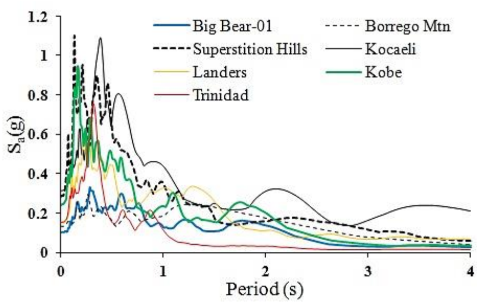


Fig. 10 Acceleration response spectra of earthquakes

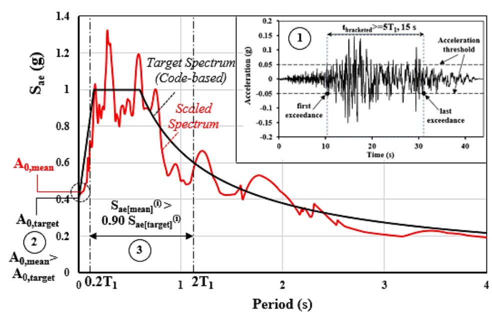
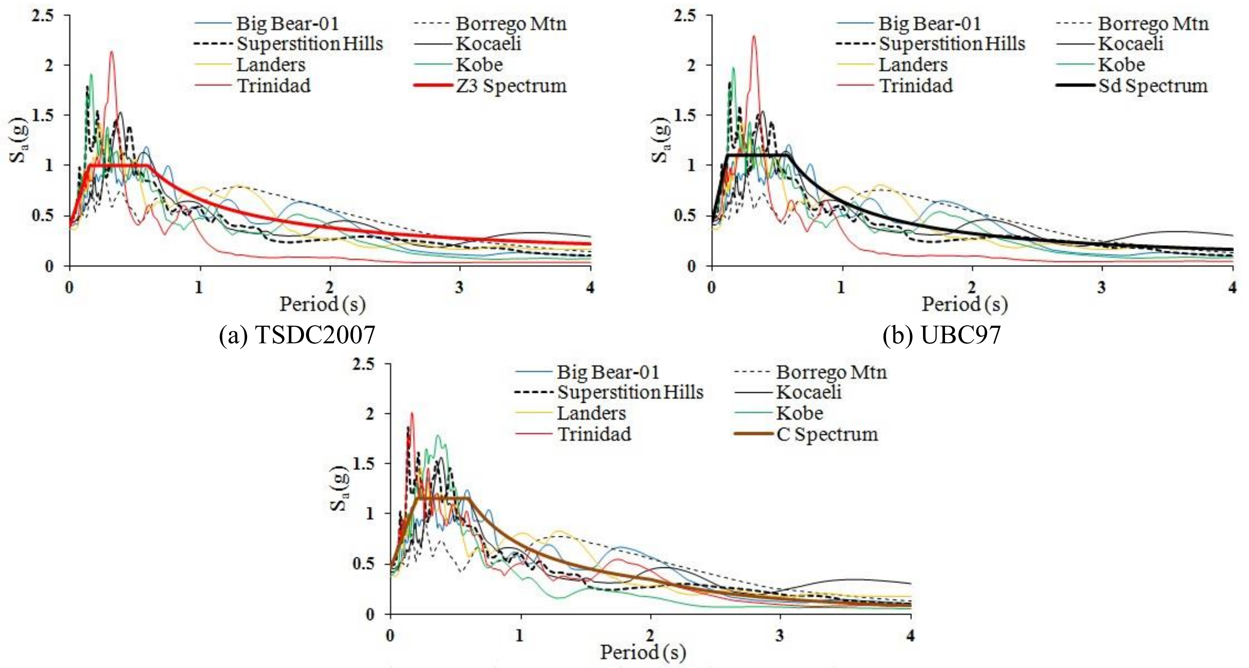


Fig. 11 Requirements of TSDC (2007) for scaled ground motions

acceleration, PGV is the peak ground velocity and PGD is the peak ground displacement. Acceleration time histories of the selected ground motions is presented in Fig. 9.

Shown in Fig. 10 is the 5% damped non-scaled linear-elastic acceleration response spectrum for ground motion records in Table 5. Response spectra are constructed by using SeismoSpect software (SeismoSpect 2018).

Fig. 11 shows three provisions of TSDC (2007) for scaled ground motion records to be used in time history analysis. First, the duration of earthquake has to be bigger than $5T_1$ and 15 sec. As the second provision of TSDC (2007); the mean of the spectral acceleration corresponding



(a) TSDC2007 (b) UBC97

(c) EC8

Fig. 12 Acceleration response spectra of scaled records

Table 6 Durations and scale factors

Earthquake Name	Duration (s)	Significant Duration (s)	Scale Factor α_s			Bracketed Duration (s)			$S_{ae}(T_0)$ (g)		
			TSDC 2007	UBC 97	EC8	TSDC 2007	UBC 97	EC8	TSDC 2007	UBC 97	EC8
Big Bear-01	99.990	23.650	3.99	4.03	4.14	38.59	38.59	38.59	0.403	0.408	0.418
Borrego Mountain	79.980	41.180	3.19	3.10	3.16	42.09	39.73	39.74	0.424	0.412	0.420
Superstition Hills-02	22.290	13.650	1.63	1.68	1.70	20.13	20.14	20.14	0.466	0.480	0.486
Kocaeli	27.180	11.790	1.41	1.42	1.44	19.92	19.92	20.31	0.440	0.443	0.449
Landers	43.980	18.860	2.38	2.40	2.46	31.50	31.52	31.64	0.362	0.365	0.374
Kobe	40.950	13.160	2.02	2.08	2.12	25.71	25.71	25.71	0.485	0.501	0.509
Trinidad	21.995	10.000	2.84	3.03	3.10	18.205	18.915	18.92	0.429	0.458	0.468

to zero period value ($A_{0,\mathrm{mean}}$) has to be bigger than the spectral acceleration of the target spectrum corresponding to zero period ($A_{0,\mathrm{target}}$). And the last one is that the mean spectral acceleration of the scaled spectrum has to be bigger than 90 per cent of target spectral acceleration in the period range of $0.2T_1$ and $2T_1$. Generally, similar requirements related with scaling of accelerograms to make them compatible with the code-specific hazard levels are provided in above-mentioned seismic design codes.

Ground motion records are scaled in time domain considering Z3 design spectrum for TSDC2007, S_d design spectrum for UBC97 and C design spectrum for EC8. The duration of the selected earthquakes and scale factors as a result of time domain scaling procedure can be seen from Table 6 for TSDC2007, UBC97 and EC8. $S_{ae}(T_0)$ is the spectral acceleration of the scaled spectra corresponding to zero period value. The mean of $S_{ae}(T_0)$ values of TSDC2007, UBC97 and EC8 elastic design spectrum compatible ground motions are 0.43 g, 0.44 g and 0.46 g, respectively. The scaled accelerograms satisfy duration and amplitude related requirements of considered seismic design codes.

Acceleration spectra of the scaled accelerograms developed for a damping ratio of 5% are shown together with 5 percent-damped elastic design acceleration spectrum of different seismic design codes in Fig. 12.

6. Description of structural models

6-, 8- and 10-story 4-bay RC frames, denoted as RCF_6.4, RCF_8.4 and RCF_10.4, respectively, are designed and detailed to satisfy the requirements of TSDC (2007) considering both gravity and seismic loads and as well as TS500 (2000). Material properties are assumed to be 25 MPa for the concrete compressive strength and 420 MPa for the yield strength of reinforcement. All frames are considered on Seismic Zone 1, designed for high ductility level and the local site class is taken as Z3 according to TSDC (2007). Typical plan and formwork of representative RC frames can be seen in Fig. 13.

Internal axes are examined in detailed as frame

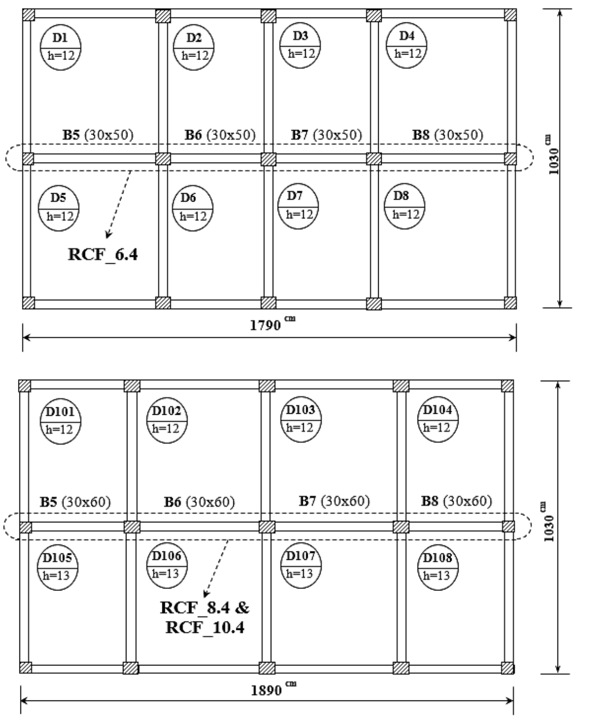


Fig. 13 Typical plan and formwork of representative RC frames

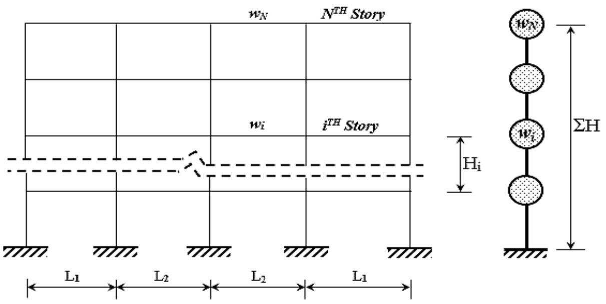


Fig. 14 Typical figure of RC frames (w_i: i^{th} story weight)

structures from the formwork plan of RC buildings. The considered axes are indicated in Fig. 13 for frame structures. Rectangular beams and square columns are considered in RC design of frames which is performed using the structural analysis program SAP2000 (2018). In order to ensure a suitable plastic mechanism and avoid brittle failure modes, the structural design of the frames is implemented on the basis of capacity design considerations. Beam dimensions are 30×50 cm for RCF_6.4 and 30×60 cm for RCF_8.4 and RCF_10.4. Column dimensions are 55×55 cm (1st and 2nd stories) - 50×50 cm (other stories) for RCF_6.4, 70×70 cm (1st and 2nd stories) - 55×55 cm (other stories) for RCF_8.4 and 80×80 cm (1st story) - 70×70 cm (other stories) for RCF_10.4. Typical figure of RC frames is shown in Fig. 14. Typical story height is 3 m in all frames whereas beam spans are L_1=5 m and L_2=3.8 m in RCF_6.4, L_1=4 m and L_2=5.3 m in RCF_8.4 and RCF_10.4.

Concentrated seismic masses consisted of dead loads plus 30% of live loads are 921.4 tons, 1321.9 tons and 1660.7 tons for RCF_6.4, RCF_8.4 and RCF_10.4, respectively. Eigenvalue analysis yields the natural vibration periods as 0.83 s, 0.89 s and 1.02 s for RCF_6.4, RCF_8.4 and RCF_10.4, respectively.

6.1 Nonlinear time history analyses

NLTH analyses of RC frames are performed by using the time histories of the scaled ground motions (Table 5) compatible with elastic design acceleration spectrum of three different seismic design codes (TSDC2007, UBC97 and EC8). Since seven recorded accelerograms are selected to be scaled, totally 63 time history analyses are performed considering three different nonlinear structural models created in SAP2000 environment. Modal damping ratio is taken as 5% and Rayleigh damping model, which assumes that the damping is proportional to the stiffness and mass (Chopra 1995), is used in dynamic analyses (for the first two modes). Rayleigh damping graphs for frame structures are given in Fig. 15.

Beams and columns are modeled as nonlinear frame elements by assigning plastic hinges at both ends of these elements. In nonlinear analyses, bending moment-axial force relation for column plastic hinges is idealized as in Fig. 16. Initial effective stiffness values of RC components are reduced according to TSDC (2007) in order to account for cracking of concrete during the inelastic response of frames. Accordingly, the effective flexural stiffness of beams is taken as 40% of the uncracked stiffness of the section. For columns, the axial force is obtained from interaction diagram and the effective flexural stiffness of columns is taken between 40% and 80% of the uncracked stiffness according to the level of axial load.

Bilinear hysteretic model for beam and column plastic hinge elements are taken as a basis in nonlinear dynamic analyses. The stress-train relations of confined-unconfined concrete and steel reinforcement given in TSDC (2007) are used while modelling the nonlinear behavior of RC sections. Direct integration method is used in NLTH analyses. Nonlinear base shear force-top displacement graphs of the frames for Big Bear, Kocaeli and Superstition ground motions are shown in Fig. 17 with their backbone (envelope) curves. The results are given considering the earthquake records scaled compatible with the spectra of TSDC (2007), UBC (1997) and EC8 (2004).

Fig. 18 shows the cyclic moment-rotation (M–θ) graphs for base story columns (base plastic hinges) of frames under the effect of Trinidad, Kobe and Big Bear ground motions. It is observed that plastic hinges generally occur at beam ends and column bases of frames under the effect of considered scaled earthquake records. The cyclic relation obtained by using the scaled earthquake record compatible with the Z3 design spectrum of TSDC (2007) is indicated by black curve in Fig. 18. The green curves show the cyclic moment–rotation relations obtained by using the scaled ground motion compatible with the acceleration design spectrum of UBC (1997), while the blue curves indicate the values obtained for the ground motion scaled compatible with the design spectrum for C soil profile type of EC8 (2004). The moment and rotation values are obtained quite close to each other for three seismic codes but, the EC8 always gives the maximum results. TSDC2007 gives the

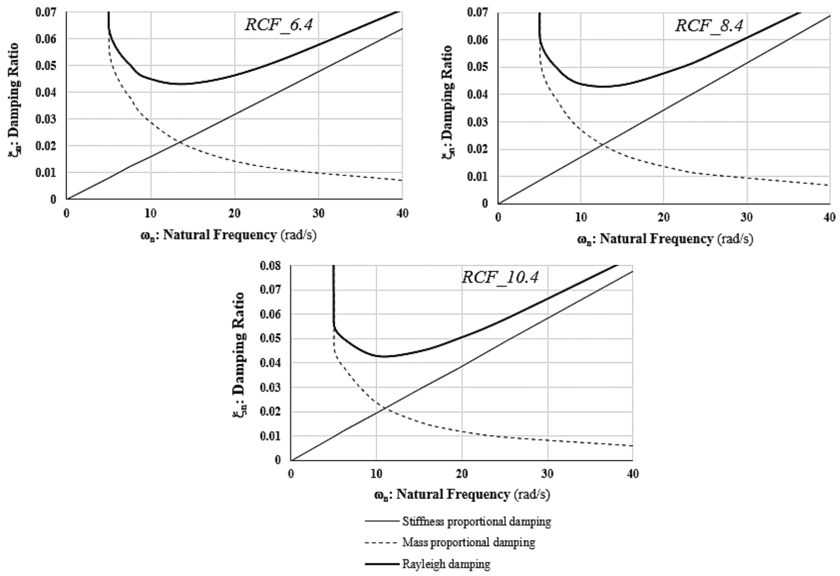


Fig. 15 Rayleigh damping graphs for frame structures

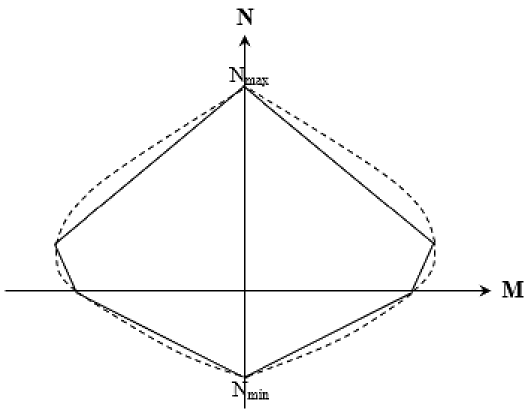


Fig. 16 Bending moment-axial force relation for column plastic hinges

minimum results in comparison to the other considered seismic design codes within this study.

6.2 Maximum base shear forces and roof displacements

The maximum nonlinear base shear force results obtained using the scaled ground motions compatible with elastic design spectrum of TSDC (2007), UBC (1997) and EC8 (2004) are presented as bar charts in Fig. 19 for the considered frames. EC8 (the design acceleration spectrum for C soil profile type) generally gives the maximum nonlinear base shear forces as the results of NLTH analyses. Borrego Earthquake gives the maximum base shear forces for RCF_6.4, whereas Landers Earthquake gives the maximum for RCF_8.4 and RCF_10.4. For RCF_6.4, the maximum base shear force is obtained as 1921 kN for Borrego Mountain Earthquake which is scaled according to the Z3 spectrum of TSDC (2007). For the same frame, the maximum base shear forces are obtained as 1925 kN and 1928 kN for the S_d and C soil profile types spectrum, respectively. When the nonlinear base shear force is investigated from NLTH analyses for RCF_8.4, the maximum value is obtained as 1618 kN, 1620 kN and 1622 kN using the scaled Landers ground motion compatible with Z3, S_d and C soil profile types spectra, respectively. The values of 1918 kN, 1924 kN and 1933 kN are obtained for RCF_10.4 as the maximum base shear force of Landers Earthquake which is scaled to match Z3, S_d and C design spectrum of TSDC (2007), UBC (1997) and EC8 (2004), respectively.

For the six-story frame (RCF_6.4), the ground motions which are scaled according to the acceleration design spectrum of EC8 (2004) always gives the maximum base shear forces. For RCF_8.4 and RCF_10.4, Borrego Mountain ground motion scaled to match the elastic design spectrum of TSDC (2007) gives the maximum base shear forces. Otherwise, the maximum base shear results for RCF_8.4 and RCF_10.4 are obtained by using the earthquake records scaled to be compatible with the elastic design spectrum of EC8 (2004), as it is also obtained for RCF_6.4.

Generally, Trinidad Earthquake gives the minimum base shear force results for all RC frames when it is compared to the other earthquake records. The mean values of the maximum base shear forces ($V_{\text{max-mean}}$) calculated from NLTH analyses of RCF_6.4 are $V_{\text{max-mean}}$ = 1784.4 kN for TSDC2007 (Z3 Spectrum), $V_{\text{max-mean}}$ = 1793.6 kN for UBC97 (S_d Spectrum) and $V_{\text{max-mean}}$ is 1799.9 kN for EC8 (C Spectrum). The mean of the maximum base shear force values for RCF_8.4 are $V_{\text{max-mean}}$ = 1485.7 kN for TSDC2007 (Z3 Spectrum), $V_{\text{max-mean}}$ = 1500.2 kN for UBC97 (S_d Spectrum) and $V_{\text{max-mean}}$ = 1511.3 kN for EC8 (C Spectrum). Finally, for the ten-story frame (RCF_10.4) the mean of the maximum base shear forces are obtained as; $V_{\text{max-mean}}$ = 1739.3 kN for TSDC2007 (Z3 Spectrum), $V_{\text{max-mean}}$ = 1756.7 kN for UBC97 (S_d Spectrum) and $V_{\text{max-mean}}$ = 1774.3 kN for EC8 (C Spectrum).

The biggest NLTH analysis based base shear coefficient (V_{max}/W: the maximum nonlinear base shear force/seismic weight of the frame) are obtained for RCF_6.4. The value

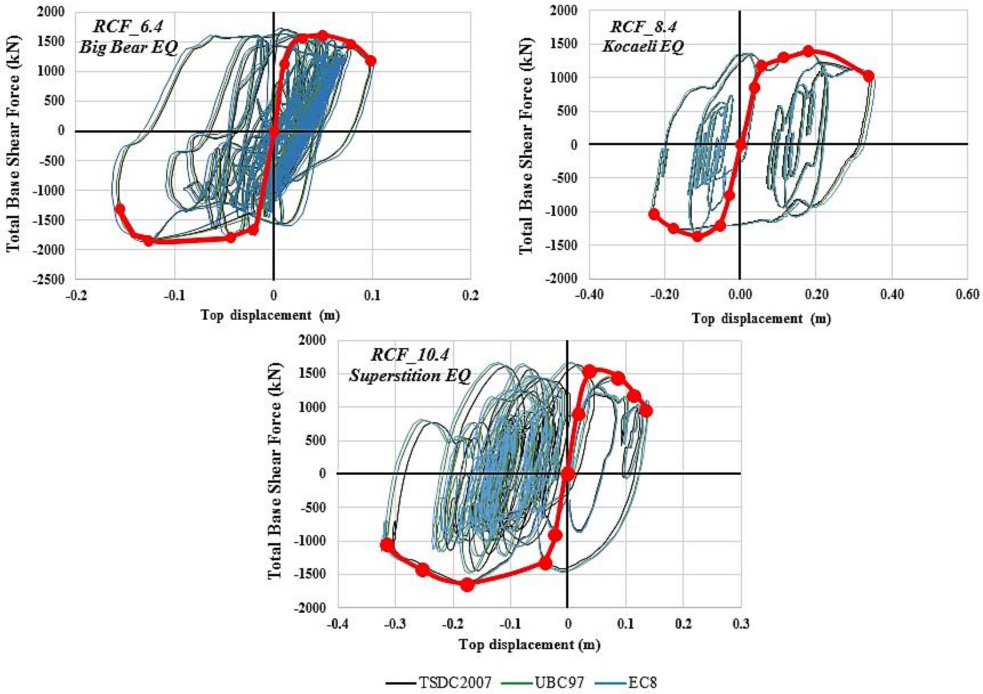


Fig. 17 Nonlinear base shear-top displacement relations of frames

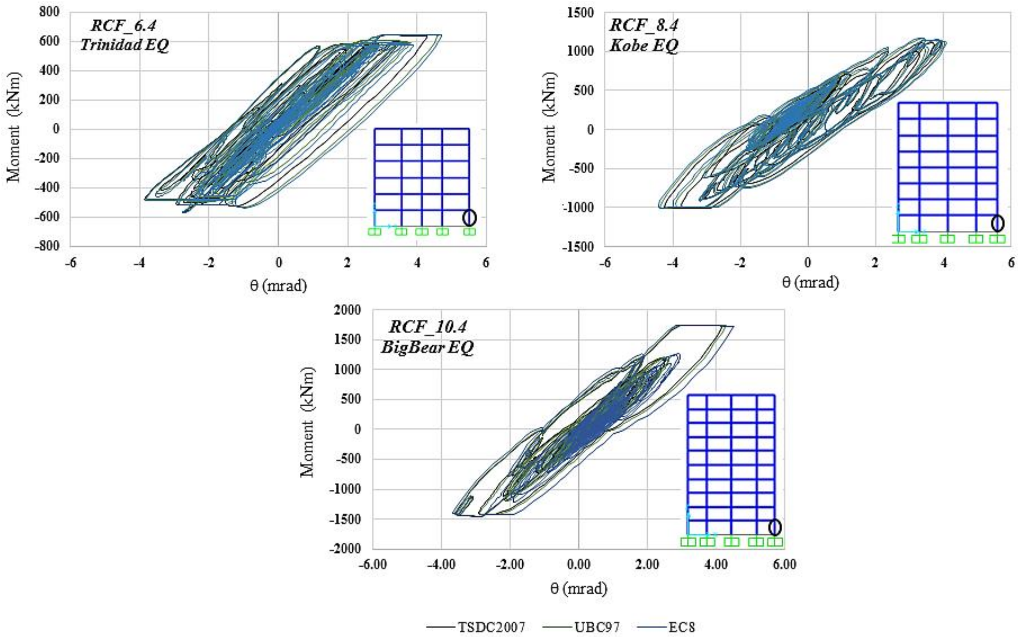


Fig. 18 M–θ graphs for the marked base story columns of frames

of base shear coefficient (V_{max}/W) obtained from the nonlinear dynamic analyses by using the accelerograms scaled compatible with Z3 soil profile type spectrum of TSDC2007, are 0.197, 0.115 and 0.106 for RCF_6.4, RCF_8.4 and RCF_10.4, respectively. Using the ground motions records scaled to match the elastic design spectrum of UBC (1997), base shear coefficients are obtained as 0.198, 0.116 and 0.108 for RCF_6.4, RCF_8.4 and RCF_10.4, respectively. For the same frames, these coefficients are obtained as 0.200, 0.117 and 0.110 from the NLTH analyses where acceleration time series scaled to fit the elastic design spectrum of EC8 (2004) are used as seismic input.

The maximum roof displacements of RC frames from NLTH analyses performed by the scaled ground motions are graphically shown in Fig. 20. The maximum roof displacements of frames are obtained from NLTH analyses using the ground motions scaled to fit the design spectrum of EC8 (2004) for C soil profile type. The minimum displacement values are generally observed from NLTH analysis of frames using accelerograms scaled to be compatible with the elastic acceleration design spectra of

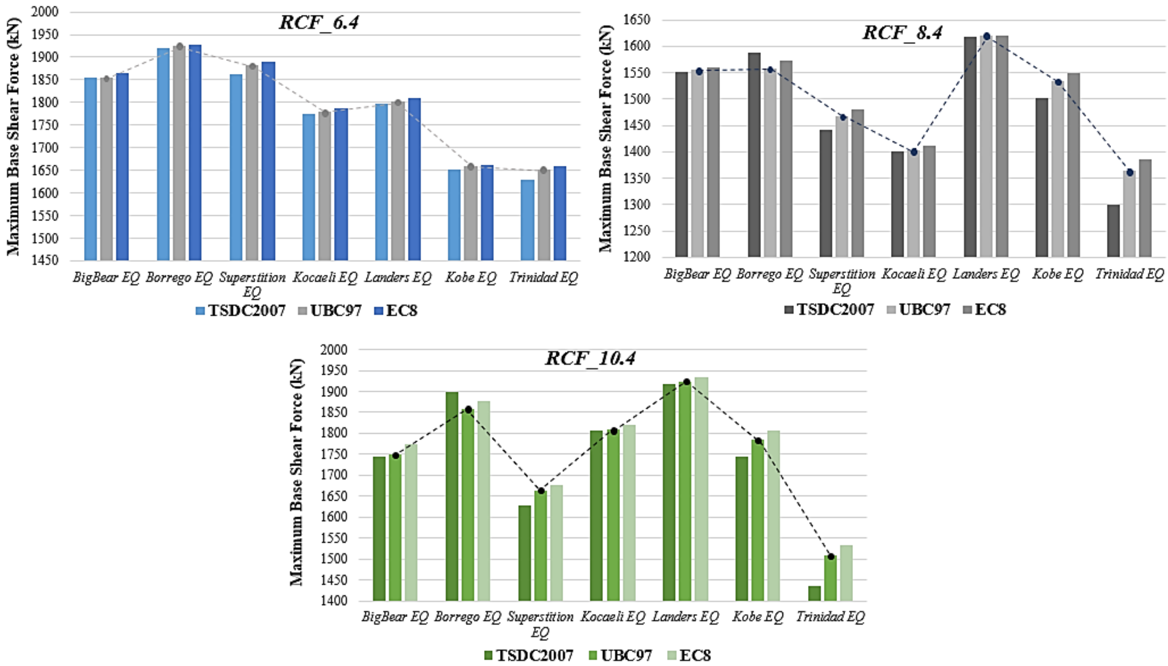


Fig. 19 Maximum nonlinear base shear forces of the frames

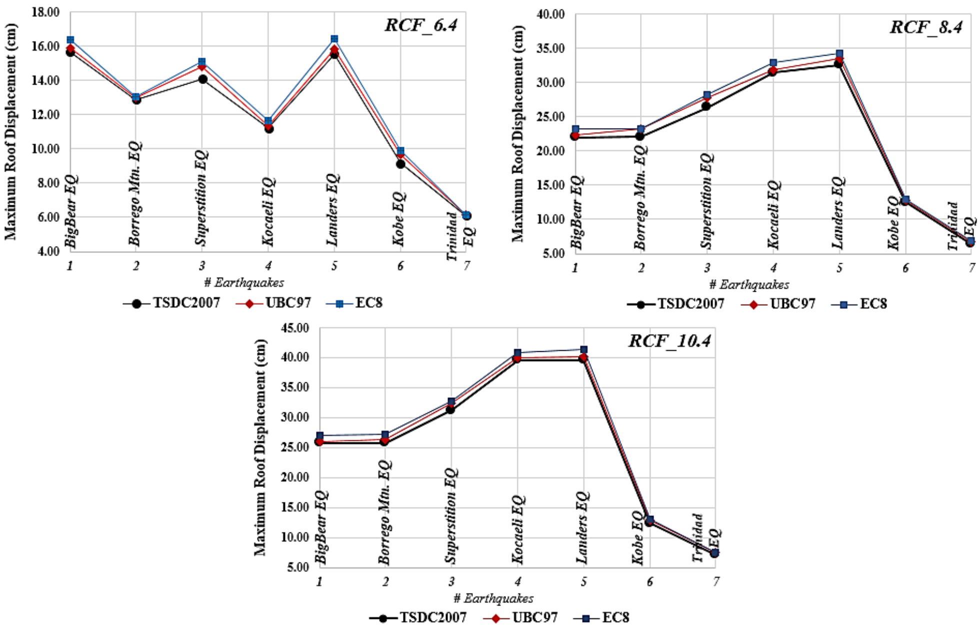


Fig. 20 M–θ graphs for the marked base story columns of frames

TSDC for Z3 soil profile type. However, the displacement values are very close to each other for all three design spectra. There is 4.33% difference for the maximum displacement values of RCF_6.4 when accelerograms scaled to fit the elastic design spectrum of TSDC and EC8 are used as seismic input. The difference ratio of the maximum displacements obtained from NLTH analyses with acceleration time series scaled to fit design spectrum of the same two codes, is 5.50 for RCF_8.4 and 5.01 for RCF_10.4. Whereas the maximum roof displacement of RCF_6.4 is 15.67 cm for Big Bear ground motion, the maximum roof displacement is obtained as 6.10 cm for Trinidad ground motion. Big Bear Earthquake for RCF_6.4 and Landers Earthquake for RCF_8.4 and RCF_10.4 gives the maximum roof displacements. Trinidad ground motion gives the minimum roof displacements for all structures. From Landers to Trinidad ground motions, the displacement graphs tend to decrease for all frames. The maximum displacement profiles of RCF_8.4 and RCF_10.4 are very similar to each other from the first to the last earthquake, whereas the displacement profile of RCF_6.4 shows fluctuations from an earthquake to another earthquake as shown in Fig. 20.

7. Conclusions

Elastic design acceleration spectra of TSDC2007, UBC97 and EC8 are constructed and each one is used as a target spectrum in scaling of the selected earthquake records. Seven real earthquake records are selected and accordingly scaled in time domain as to satisfy duration and amplitude related requirements of the considered seismic design codes. Six-, eight- and ten-story RC frames are considered as a case study and NLTH analyses are conducted for these frames by using the scaled ground motion time histories. Structural response quantities such as nonlinear base shear force and the maximum roof displacement, are investigated for frame structures in order to find out the influence of code-based design spectra to dynamic analyses.

The main findings of the study indicate that there are no major differences in quantities of considered response parameters obtained from NLTH analyses of RC frames using the recorded accelerograms scaled to match the elastic design spectra of TSDC2007, UBC97 and EC8. Generally, the maximum base shear forces and the roof displacements results of NLTH analyses are obtained as the biggest when the response spectrum of EC8 is considered as target in time domain scaling. However, the differences in base shear forces and the displacement values obtained as a result of NLTH analyses performed by the scaled ground motions in view of TSDC2007, UBC97 and EC8 are not so significant. So, it may be concluded that, the use of various code-based design spectra as target spectra in time domain scaling is not of capital importance.

When the nonlinear base shear forces and maximum roof displacements of the frames are sorted in ascending order within the study, the values obtained using the scaled accelerograms to fit the response spectra of TSDC2007, UBC97 and EC8, respectively, can be arrayed from the smallest to the biggest value. Ground motion time histories scaled to be compatible with the elastic acceleration design spectrum of TSDC2007 for Z3 soil profile type always lead the smallest response quantities while the biggest values are always obtained from NLTH analyses using the accelerograms scaled to fit the design spectrum of EC8. Since all earthquake records reflect their own characteristics to the results of NLTH analyses, the range of base shear forces and roof displacements differ from each other.

It is observed that scaling ground motion records in view of various elastic design spectra of different seismic codes does not dramatically affect the total base shear forces and the maximum displacement values. The results are found to be very close to each other for the considered code-based design spectra. Therefore, it can be inferred from the present study that the use of elastic design spectra of TSDC2007, UBC97 and EC8 as target spectra in time domain scaling procedure has not a considerable effect on quantities of some structural response parameters such as base shear forces and lateral displacements.

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