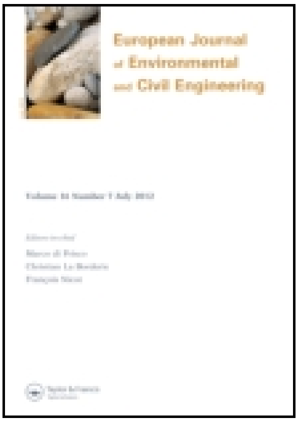

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Soil–structure interaction effects on seismic behaviour of multistorey structures

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This paper addresses the behaviour of multistorey structures considering soil–structure interaction under earthquake excitation. To this end, samples 1, 3, 6, 9, 12 and 15 storey RC plane frames corresponding to aspect ratios (h/ℓ) of 1/3, 1, 2, 3, 4 and 5 are designed based on Turkish Seismic Design Code and analysed in time domain with incremental dynamic analysis. Inelastic displacement ratios and strength reduction factors are investigated for designed sample plane frames for 64 earthquake motions recorded on different site conditions such as rock, stiff soil, soft soil and very soft soil. According to the analysis result, strength reduction factors of sample buildings considering soil–structure interaction are found to be smaller than design strength reduction factors, given the current seismic design codes. This condition leads to an unsafe design and non-conservative design forces. Besides, inelastic displacement ratios of fixed-base and interacting cases are found to be quite different especially for periods shorter than .5 s.

Keywords: soil-structure interaction; inelastic displacement ratio; strength reduction factor; incremental dynamic analysis

Introduction

Performance-based seismic design methodologies aim at controlling earthquake damage to structural elements and many types of non-structural elements by limiting lateral deformations on structures. Generally accepted standpoints of seismic design methodologies establish that structures should be capable of resisting relatively frequent, minor intensity earthquakes without structural damage or damage to non-structural elements, moderate earthquakes without structural damage or with some non-structural damage, and severe, infrequent earthquakes with damage to both the resisting systems and non-structural components. For a more rational design procedure, it is important to estimate the lateral structural displacement demands.

The inelastic displacement ratio, (C), is defined as the maximum lateral inelastic displacement demand ($\Delta_{\text{inelastic}}$) divided by the maximum lateral elastic displacement demand (Δ_{elastic}) on a system with the same mass and initial stiffness (i.e. same period of vibration) when subjected to the same earthquake ground motion. Inelastic displacement ratios have been the topic of several investigations so far. The first well-known studies were conducted by Veletsos and Newmark (1960), Veletsos, Newmark, and Chelapati (1965) using the response of SDOF systems having elastoplastic hysteretic behaviour and predefined levels of displacement ductility, μ, when subjected to a

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limited range of earthquake ground motions and periods of vibration. Since then, several researchers have performed statistical studies to evaluate constant-ductility inelastic displacement ratios using larger sets of ground motions and for wider range of periods than those pioneer studies. Recently, Miranda et al. (2000), Ruiz-Garcia and Miranda (2004, 2006), Decanini, Liberatore and Mollaioli (2003) and Chopra and Chintanapakdee (2004) studied the inelastic displacement ratio and presented a series of new functions based on statistical studies to obtain the ratio of the maximum inelastic to the maximum elastic displacement for SDOF systems.

Strength reduction factor, (R), is defined as the ratio of elastic base shear required to avoid yielding in the system (V_e) to design strength of a building (V_d). The ductility part of strength reduction factor (R_μ) can be defined as the ratio of elastic base shear (V_e) to actual structural strength of a building (V_y). The relationships between the strength reduction factor, R, structural overstrength, Ω, and ductility part of strength reduction factor, R_μ, can be seen in Figure 1.

The first well-known studies on strength reduction factors were conducted by Veletsos and Newmark (1960) and Newmark and Hall (1973). They proposed formulas for strength reduction factors as functions of structural period and displacement ductility to be used in the short-, medium- and long-period regions. Alternative formulas were proposed by Lai and Biggs (1980) and Riddell, Hidalgo and Cruz (1989). Riddell and Newmark proposed new formulas for strength reduction factors considering the effect of stiffness degrading on strength reduction factors. Similar to the previous study by Newmark, these formulas not only depend on structural period and displacement ductility but also on the damping ratio, β (1979). The first study that considered the effects of soil conditions on the strength reduction factors was conducted by Elghadamsi and Mohraz (1987). Strength reduction factors were computed using the ground motions recorded on rock and alluvium. Another study which considered the site effects on the strength reduction factors was conducted by Nassar and Krawinkler, also considering

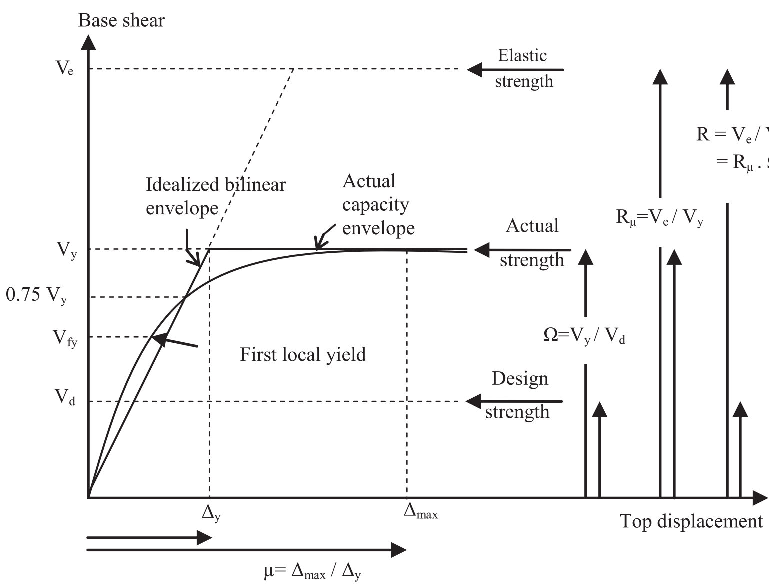

Figure 1. The relationships between the strength reduction factor, R, structural overstrength, Ω, and ductility part of strength reduction factor, R_μ (Elnashai & Mwafy, 2002).

the effects of yield level, strain hardening ratio and the type of inelastic material behaviour (1991). The effect of stiffness degrading was also studied by Vidic, Fajfar and Fischinger (1992). The effect of different hysteretic models on strength reduction factors was studied by Lee, Han and Oh (1999). Miranda (1993) studied the influence of local site conditions on strength reduction factors, using a group of 124 ground motions classified into three groups as; ground motions recorded on rock, alluvium and very soft soil. Afterwards, mean strength reduction factors were computed for each soil group. As a consequence of site effects, the formulas for strength reduction factors on soft soil depend on the ratio of structural period to predominant period of ground motion, whereas strength reduction factors on rock and alluvium depend on the structural period.

The most common approach to consider elastic SSI effects has not changed over the years. This approach involves the usage of a replacement oscillator represented by the effective period and damping of the system ($\tilde{T}_e$, $\tilde{\beta}_e$). The mass of this equivalent oscillator is taken to be equal to that of the actual structure. Under harmonic base excitation, it is imposed that the resonant period and peak response of the interacting system be equal to those of the replacement oscillator. Most of the seismic design codes currently applied in structural design (Applied Technology Council [ATC], 1978 and FEMA-450, 2003) use the same procedure to take SSI effects into account. However, it is noted by Crouse (2002) that the current SSI provisions in the ATC and NEHRP codes have a significant shortcoming, which is the lack of a link between the strength reduction factors and the effects of SSI. Besides, Eurocode 8 obligates to take the effects of dynamic soil-structure interaction into account for structures where P-δ (2nd order) effects play a significant role; structures with massive or deep-seated foundations, such as bridge piers, offshore caissons, and silos; slender tall structures, such as towers and chimneys; and structures supported on very soft soils, with average shear wave velocity less than 100 m/s (EC8, 1994).

It has been known for many years that SSI affects the elastic strength demand of structures and, generally, elastic strength demand is reduced due to SSI. This is mainly because the soil–structure system has longer period and, usually, higher damping ratio in comparison to the fixed base structure (Veletsos, 1977). In 1970s, many researchers put effort into estimating the SSI effect on elastic response of structures (Chopra & Gutierrez, 1974; Novak, 1974). Besides, soil–structure interaction effects on inelastic behaviour have been the topic of some investigations (Ciampoli & Pinto, 1995; Rodriguez & Montes, 2000). Lin and Miranda conducted a statistical study of the kinematic soil-foundation–structure interaction effects on the maximum inelastic deformation demands of structures. They found that kinematic interaction will reduce the maximum inelastic displacement demands of structures, especially for systems with short periods of vibration, and the larger the foundation size, the smaller the maximum inelastic displacement becomes. In addition, the inelastic displacement ratio is nearly not affected by the strength ratio of structures for systems with periods of vibration greater than about .3 s and with strength ratios smaller than about 3.0 (Lin & Miranda, 2008). During last decade, Aviles and Perez-Rocha studied on soil–structure interaction phenomenon widely (2003, 2005a, 2005b, 2011). They concluded that for soft/deep soil deposits, the SSI effects in yielding structures may result in either increase or decrease of the fixed-base strengths and displacements, depending primarily on the period ratio of the structure and site. The higher the structural ductility, the smaller becomes these effects. Also, Ghannad and co-workers studied on soil–structure interaction effects on strength reduction factors and ductility demands

(Ghannad & Ahmadnia, 2002, 2006; Ghannad & Jahankhah, 2004, 2007). They showed that both ductility and strength demanded by the structure may experience considerable variations under the effect of SSI. It has been shown that the interaction between the soil and structure also affects the hysteretic energy dissipation of the structure under earthquake loading. Both Ghannad and co-workers, and Aviles and Perez-Rocha concluded that generally SSI reduces strength reduction factors of SDOF systems, especially for the case of short-period structures located on relatively soft soils. The effect of soil–structure interaction on inelastic displacement ratio of structures has been studied by Eser Aydemir and Aydemir (2011). They proposed a new equation for inelastic displacement ratio of interacting system, as a function of structural period of interacting system, strength reduction factor and period lengthening ratio. Besides, there are some other researches on earthquake-induced behaviour of structures considering soil–structure interaction phenomenon (Doo & Yun, 2003; Sarkani, Lutes, Jin, & Chan, 1999).

In addition to studies carried out using SDOF systems, there are some other researches conducted using MDOF analytical models of buildings. Gupta and Trifunac have shown that it is possible to include the SSI effects in the analysis of multistorey buildings' response via response spectrum superposition method by incorporating a few modifications in the input excitation (Gupta & Trifunac, 1991). Barcena and Esteva have analysed a set of multi-degree-of-freedom non-linear systems, designed for earthquake resistance in accordance with current criteria and methods, to study the influence of the dynamic interaction on the seismic structural response, ductility demands and reliability levels (Barcena & Esteva, 2007). Another study conducted by Grange et al. has focused on soil–structure interaction effects on a reinforced concrete viaduct by means of using multifibre beam elements and constitutive laws (Grange, Botrugno, Kotronis, & Tamagnini, 2011). Roy and Dutta studied the inelastic seismic response of low-rise buildings through adequate idealisation of structure and sub-soil medium. They concluded that buildings depict that inelastic response of the asymmetric structure relative to its symmetric counterpart is not appreciably influenced due to soil–structure interaction (Roy & Dutta, 2010). The effect of foundation non-linearity on the structural response of low-rise steel moment-resisting frame buildings in terms of base moment, base shear, storey drift and ductility demand was investigated (Raychowdhury, 2011). Ganjavi and Hao studied on soil–structure interaction effects on MDOF systems in recent years (Ganjavi & Hao, 2012a, 2012b). They have concluded that generally SSI reduces the strength reduction factor of both MDOF and more intensively, SDOF systems (Ganjavi & Hao, 2012b).

In this study, differently from previous SSI studies, generally focusing on inelastic displacement ratios and strength reduction factors of SDOF systems, the seismic behaviour of multistorey structures considering soil–structure interaction effects is investigated. To this purpose, samples 1, 3, 6, 9, 12 and 15 storey plane frames corresponding to aspect ratios of 1/3, 1, 2, 3, 4 and 5 were designed and detailed according to Turkish Seismic Design Code (2007). Incremental dynamic analyses were performed for those sample buildings using 64 ground motions recorded on different site conditions such as rock, stiff soil, soft soil and very soft soil to determine the yielding and collapse capacity of each sample building. Analyses were repeated both for fixed-base and soil–structure interacting cases. Structural parameters such as strength reduction factors and inelastic displacement ratios of sample frames were calculated for fixed-base and interacting case and results were compared with each other.

Methodology

To obtain the seismic performance and considered structural parameters such as strength reduction factors and inelastic displacement ratios of sample buildings for both fixed-base and interacting cases from inelastic dynamic analysis results, definition of response parameters for different limit states are needed. Two important limit states in the response of the buildings are yielding and collapse. In this study, yield point of building where elastic behaviour is not valid anymore is obtained at the time when either the local or global yielding criterion occurs first. The criteria used for defining yielding are classified into two groups: local and global criteria. The local yield criterion is defined as the first point when the strain in the longitudinal tensile reinforcement exceeds the yield strain of steel or the cover concrete crushes at ground floor column sections. The material strains corresponding to these situations are .002 for cover concrete ($\varepsilon_{\mathrm{co}}$) and .0021 for reinforcing steel ($\varepsilon_{\mathrm{sy}}$), respectively. For global criteria, the yield capacity of the structure is defined as the point where the incremental dynamic analysis (IDA) curve leaves the linear path. During the analyses, local yielding at ground floor column sections and global yielding occurred simultaneously. Consequently, scale factors (SFs) corresponding to local yield state and global yield state have been found to be approximately equal within a very slight difference. Besides, there are many previous studies using the local yield criteria as response parameters such as Elnashai and Mwafy (2002), Mwafy Kwonb, and Elnashai (2010), Di Sarno, Elnashai, and Nethercot (2003) and Aksoylar, Elnashai, and Mahmoud (2011).

For collapse limit state, maximum interstorey drift (ID) ratio is considered as the primary and most important global collapse criterion and limited to 3% in this study (Elnashai & Mwafy, 2002; Penelis & Kappos, 1997). Also, local collapse or failure criteria such as rupture of longitudinal reinforcement, beam failure, column failure, beam–column connection failure, exceeding the shear strength or the ultimate curvature in any structural member can be used for collapse limit state. However, recent analytical and experimental works have shown that the ID (global criterion) is more suitable for certain construction types than local (member) failure (Elnashai, Elghazouli, & Denesh-Ashtiani, 1998; FEMA 355E, 2000). Several values for the ID collapse limit have been suggested in the literature (Broderick & Elnashai, 1996; FEMA, 1997; Structural Engineers Association of California [SEAOC], 1995). The upper limit of ID ratio should be sufficient to restrict second-order (P–Δ) effects and to express the damage in structural and non-structural elements. This limit is adopted over other conservative limits to reflect the ability of structural frame systems to sustain relatively large deformations, especially those designed for modern seismic codes such as the Uniform Building Code (UBC, 1997) or EC8 (1994).

The procedure used in this study to obtain the strength reduction factor of multistorey structures can be summarised as:

(1) For each ground motion, SF is obtained from IDA as the factor which causes the first yield point of building where elastic behaviour is not valid anymore. This yield point is obtained at the time when either the local or global yielding criterion occurs first.
(2) Elastic base shear (V_{e}) is calculated as the product of mass times spectral acceleration at predominant period of vibration and SF. ($V_e = m \times \mathrm{SA} \times \mathrm{SF}$).
(3) Strength reduction factors are obtained as the ratio of elastic base shear to design base shear of buildings ($V_{\mathrm{e}}/V_{\mathrm{d}}$).

Description of sample frames and soil structure modelling

In order to achieve the aforementioned objectives, samples 1, 3, 6, 9, 12 and 15 storey RC frames are designed and detailed according to Turkish Seismic Design Code (2007). All frames are designed to be a moment-resisting frame having three bays. Total building height of sample buildings is between 3 and 45 m, whereas aspect ratios (h/ℓ) of sample buildings are 1/3, 1, 2, 3, 4 and 5, respectively. The span lengths and storey heights of the investigated frames are selected to be equal to each other and 3.0 m to be able study on buildings with high aspect ratios. Typical elevation view for sample buildings can be seen in Figure 2. The cross-section capacities have been computed by considering characteristic cylinder strength of 25 N/mm^2 for concrete and characteristic yield strength of 420 N/mm^2 for both longitudinal and transverse steel. Concrete behaviour is modelled by a uniaxial Mander model without consideration of tensile strength. For the confined concrete, the strength and strain values have been increased according to the formulae developed by Mander, Priestley, and Park (1988). Softening beyond the maximum compressive strength is taken into consideration as a linear function. Steel behaviour is represented by a bi-linear steel model with kinematic strain hardening. Two dimensional non-linear dynamic analyses were performed for each sample building. Aspect ratios, number of stories and initial periods of sample buildings are given in Table 1. More details regarding member cross-section sizes and reinforcements are given elsewhere (Eser Aydemir, 2011).

In this study, the span lengths and storey heights of the investigated frames are selected to be equal to each other and 3.0 m to be able to study on buildings with high aspect ratios. Realising that no two structures are the same, and that the dynamic behaviours of real structures depend on so many parameters, it is decided to focus on simplified MDOF models in order to gain insight into seismic response with soil–structure interaction. For this purpose, although it is known that the typical buildings may have wider spans, the frames are selected to be two-dimensional regular type with mid-length spans for simplicity. Besides, the periods of vibrations of sample buildings are relatively longer than typical RC buildings with the same heights because, especially, the column

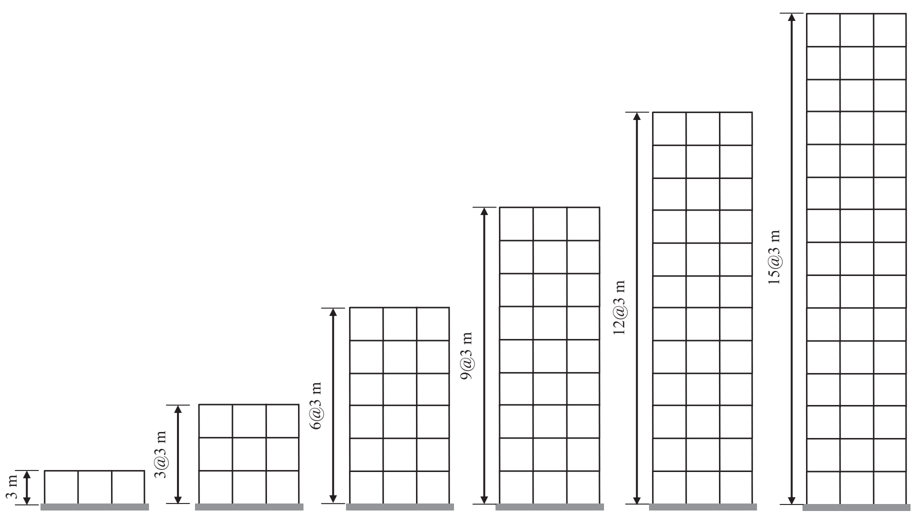

Figure 2. Geometry of the sample RC frames under investigation.

Table 1. Properties of sample buildings.

Aspect ratio (h/r)	1/3	1	2	3	4	5
Number of stories	1	3	6	9	12	15
Initial period (s)	0.23	0.54	0.91	1.25	1.56	1.88

sections of investigated frames have the minimum dimensions satisfying the ductile behaviour and design requirements such as strong column-weak beam principle.

For interacting case, soil–structure interaction can be modelled using the macro-element concept that lies in the fact that the movement of a foundation by a system of generalised variables (forces and displacements) defined at the foundation centre. The non-linear behaviour of the soil is reproduced using the classical theory of plasticity (Crémer, Pecker, & Davenne, 2002; Grange, Kotronis, & Mazars, 2009a, 2009b). In this study, the foundation is modelled as a circular rigid disk. The equivalent radius r of the circular foundation is obtained according to (Wolf, 1997). The soil under the foundation is considered a homogenous half-space and is characterised by shear wave velocity V_s, dilatational wave velocity V_p, mass density ρ and Poisson's ratio υ. The supporting soil is replaced with springs and dampers for the horizontal and rocking modes. The foundation is represented for all motions using a spring–dashpot-mass model with frequency-independent coefficients. The coefficients of springs and dashpots are calculated for circular rigid disk of radius r. Spring and dashpot elements are modelled individually under each column and the coefficient of each spring and dashpot element is obtained as springs in parallel, i.e. the sum of coefficients of all individual springs and dashpots are equal to the value calculated for circular rigid disk. The modelling of the foundation on deformable soil is performed in the same way as that of the structure and is coupled to perform a dynamic SSI analysis (Wolf, 1997). Realising that generally deep or pile foundations are used for tall buildings on soft soils, it is decided to focus on shallow foundations in order to gain insight into seismic response with soil structure interaction. A schematical view considering soil–structure interaction modelling of supports is shown in Figure 3.

The stiffness and damping coefficients for the horizontal (K_x, C_x) and rocking modes (K_θ, C_θ) of soil medium are defined as follows (Wolf, 1994):

$$K_x = \frac{8 \cdot \rho \cdot V_s^2 \cdot r}{2 - \upsilon} \tag{1}$$

$$K_\theta = \frac{8 \cdot \rho \cdot V_s^2 \cdot r^3}{3 \cdot (1 - \upsilon)} \tag{2}$$

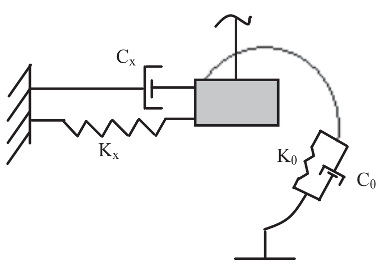

Figure 3. Mathematical model of supports with soil–structure interaction.

$$C_x = \rho \cdot V_s \cdot \pi \cdot r^2 \tag{3}$$

$$C_\theta = \rho \cdot V_p \cdot \pi \cdot \frac{r^4}{4} \tag{4}$$

Dynamic properties of different types of soil is presented in Table 2. Aspect ratio and shear wave velocity are the key parameters for soil–structure interacting case as used in previous studies (Ghannad & Jahankhah, 2007; Veletsos, 1977).

Ground motions

A total of 64 earthquake acceleration time histories recorded on different soil types are used in this study. Ground motions are selected to represent far-field earthquakes based on far-field definition in ATC documents (1996, 2008). Details of selected ground motions are listed in Table 3. Site classes given in tables are in accordance with United States Geological Survey site classification system (1993), which corresponds to shear wave velocity value higher than 750 m/s for site class A, between 360 and 750 m/s for site class B, 180–360 m/s for site class C and lower than 180 m/s for site class D. In analyses, soil – structure interacting systems are assumed to be located on soil profiles with shear velocities of 750 m/s for site class A, 400 m/s for site class B, 250 m/s for site class C and 150 m/s for site class D. The stiffness and damping coefficients mentioned above are calculated for these values of shear wave velocity.

Analysis platform

For IDA, the SeismoStruct computer package is used. SeismoStruct is a finite element structural analysis program developed for the non-linear analysis of two-dimensional and three-dimensional steel, reinforced concrete and composite structures under static and dynamic loading, taking into account the effects of geometric non-linearities and material inelasticity (Seismosoft, 2007). In SeismoStruct, use is made of the so-called fibre approach to represent the cross-section behaviour, where each fibre is associated with a uniaxial stress–strain relationship; the sectional stress–strain state of beam-column elements is then obtained through the integration of the non-linear uniaxial stress–strain response of the individual fibres (typically 300–400) in which the section has been subdivided and in our model, 200 section fibres are used for beam and column elements. It is widely accepted that this technique is more accurate than the point–hinge models mainly used in many other programs, especially when large axial force variations exist. Cubic shape function elements capable of representing the distribution

Table 2. Dynamic properties of different types of soil.

Soil properties	Soil class			
	A	B	C	D
Shear wave velocity (m/s)	750	400	250	150
Horizontal stiffnes of soil medium, K_x (kN/m)	1.35×10^6	4.4×10^5	1.72×10^5	6.6×10^4
Rocking stiffnes of soil medium, K_θ (kNm/rad)	2.84×10^7	9.27×10^6	3.62×10^6	1.5×10^6
Horizontal damping coefficient, C_x (kNs/m)	6556	3746	2341	1405
Rocking damping coefficient, C_θ (kNsm/rad)	73082	42631	27406	18271

Table 3. Earthquake ground motions used in analyses.

Earthquake	M	Station	Station number	Distance (km)	Comp. 1	PGA (g)	PGV (cm/s)	Comp. 2	PGA (g)	PGV (cm/s)	Soil class
Loma Prieta 18/10/89	7.1	Coyote Lake Dam	57217	21.8	CYC195	0.151	16.2	CYC285	0.484	39.7	A
Loma Prieta 18/10/89	7.1	Monterey City Hall	47377	44.8	MCH000	0.073	3.5	MCH090	0.063	5.8	A
Loma Prieta 18/10/89	7.1	SC Pacific Heights	58131	80.5	PHT270	0.061	12.8	PHT360	0.047	9.2	A
Northridge 17/01/94	6.7	Lake Hughes 9	127	28.9	L09000	0.165	8.4	L09090	0.217	10.1	A
Northridge 17/01/94	6.7	Wrıghtwood – Jackson Flat	23590	68.4	WWJ090	0.056	10	WWJ180	0.037	7	A
Northridge 17/01/94	6.7	Sandberg Bald Mtn	24644	43.4	SAN090	0.091	12.2	SAN180	0.098	8.9	A
Kocaeli 17/08/99	7.8	Gebze	–	17	GBZ000	0.244	50.3	GBZ270	0.137	29.7	A
Northridge 17/01/94	6.7	MT Wılson-Cıt Sta.	24399	36.1	MTW000	0.234	7.4	MTW090	0.134	5.8	A
Loma Prieta 18/10/89	7.1	Anderson Dam Downstream	1652	20	AND270	0.244	20.3	AND360	0.24	18.4	B
Northridge 17/01/94	6.7	Castaic Old Ridge	24278	25.4	ORR090	0.568	52.1	ORR360	0.514	52.2	B
Northridge 17/01/94	6.7	LA Century City North	24389	18.3	CCN090	0.256	21.1	CCN360	0.222	25.2	B
Kocaeli 17/08/99	7.8	Arçelik	–	17	ARC000	0.218	17.7	ARC090	0.149	39.5	B
Loma Prieta 18/10/89	7.1	Golden Gate Bridge	1678	85.1	GGB270	0.233	38.1	GGB360	0.123	17.8	B
Northridge 17/01/94	6.7	Ucla Grounds	24688	16.8	UCL090	0.278	22	UCL360	0.474	22.2	B
Northridge 17/01/94	6.7	LA Univ. Hospital	24605	34.6	UNI005	0.493	31.1	UNI095	0.214	10.8	B
Düzce 12/11/99	7.3	Lamont 1061	1061	15.6	1061-E	0.107	11.5	1061-N	0.134	13.7	B
Landers 28/06/92	7.4	Yermo Fire Station	22074	26.3	YER270	0.245	51.5	YER360	0.152	29.7	C
Loma Prieta 18/10/89	7.1	Hollister - South & Pine	47524	28.8	HSP000	0.371	62.4	HSP090	0.177	29.1	C
Northridge 17/01/94	6.7	Downey-Birchdale	90079	40.7	BIR090	0.165	12.1	BIR180	0.171	8.1	C
Northridge 17/01/94	6.7	LA-Centinela	90054	30.9	CEN155	0.465	19.3	CEN245	0.322	22.9	C
Imperial Valley 15/10/79	6.9	Chihuahua	6621	28.7	CHI012	0.27	24.9	CHI282	0.254	30.1	C
Imperial Valley 15/10/79	6.9	Delta	6605	32.7	DLT262	0.238	26	DLT352	0.351	33	C
Loma Prieta 18/10/89	7.1	Gilroy Array #4	57382	16.1	G04000	0.417	38.8	G04090	0.212	37.9	C
Düzce 12/11/99	7.3	Bolu	Bolu	17.6	BOL000	0.728	56.4	BOL090	0.822	62.1	C
Loma Prieta 18/10/89	7.1	Appel 2 Redwood City	1002	47.9	A02043	0.274	53.6	A02133	0.22	34.3	D

(*Continued*)

Table 3. (*Continued*).

Earthquake	M	Station	Station number	Distance (km)	Comp. 1	PGA (g)	PGV (cm/s)	Comp. 2	PGA (g)	PGV (cm/s)	Soil class
Northridge 17/01/94	6.7	Montebello	90011	86.8	BLF206	0.179	9.4	BLF296	0.128	5.9	D
Superstition Hills 24/11/87	6.6	Salton Sea Wildlife Refuge	5062	27.1	WLF225	0.119	7.9	WLF315	0.167	18.3	D
Loma Prieta 18/10/89	7.1	Treasure Island	58117	82.9	TRI000	0.1	15.6	TRI090	0.159	32.8	D
Kocaeli 17/08/99	7.8	Ambarli	–	78.9	ATS000	0.249	40	ATS090	0.184	33.2	D
Morgan Hill 24/04/84	6.1	Appel 1 Redwood City	58375	54.1	A01040	0.046	3.4	A01310	0.068	3.9	D
Düzce 12/11/99	7.3	Ambarlı	–	193.3	ATS030	0.038	7.4	ATS300	0.025	7.1	D
Kobe 16/01/95	6.9	Kakogawa	0	26.4	KAK000	0.251	18.7	KAK090	0.345	27.6	D

of inelasticity are used to model the horizontal and vertical structural members. For this type of element, SeismoStruct performs the numerical integration over two Gauss sections. Each Gauss section is subdivided into a number of fibres where stresses and strains are calculated by applying the inelastic cyclic constitutive relationships for each of the considered materials. The link elements and damping devices for translational and rotational motions are defined at the base for soil–structure interacting case. Two-dimensional analyses are undertaken in one direction only. The analysis is both inelastic and geometrically non-linear.

Incremental dynamic analysis

IDA as a procedure developed for accurate estimation of seismic demand and capacity of structures, requires non-linear response history analysis of the structure for an ensemble of ground motions, each scaled to many intensity levels to obtain the whole range of structural response from elastic behaviour to global dynamic instability. This method is first proposed by Bertero (1977) and later developed by Vamvatsikos and Cornell (2002). At the end of an IDA, the IDA curve is produced using an intensity measure vs. a damage measure. Intensity measure which means scaling of the ground motion can be based on peak ground acceleration or spectral acceleration at first mode, whereas maximum base shear, node rotations, peak roof drift, peak storey ductilities or maximum ID ratio can be chosen as damage measure. In this study, scaling is applied according to peak ground acceleration and maximum ID ratio is selected as damage measure.

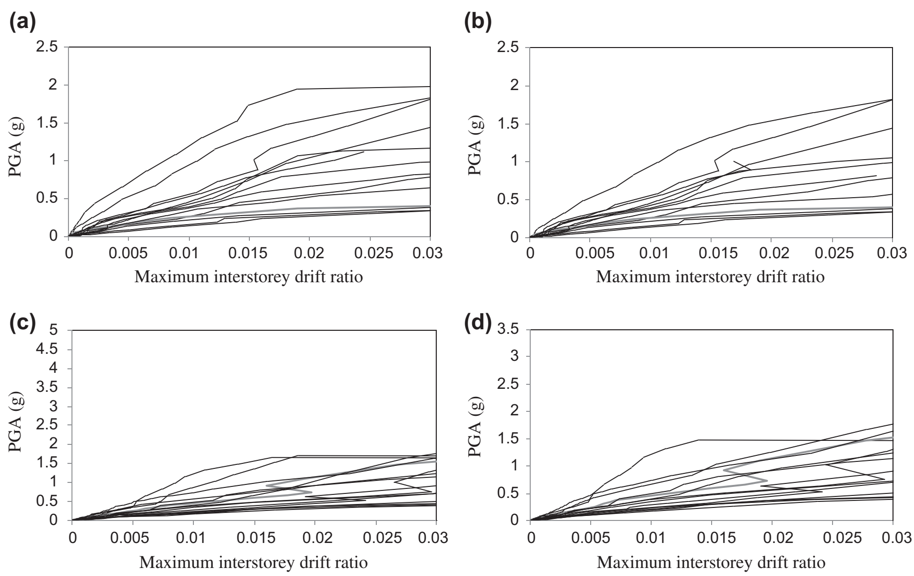

Figure 4. IDA curves for sample buildings (a) nine building on site class A – fixed base; (b) Nine storey building on site class A – interacting case; (c) 12 storey building on site class C – fixed base; (d) 12 storey building on site class C – interacting case.

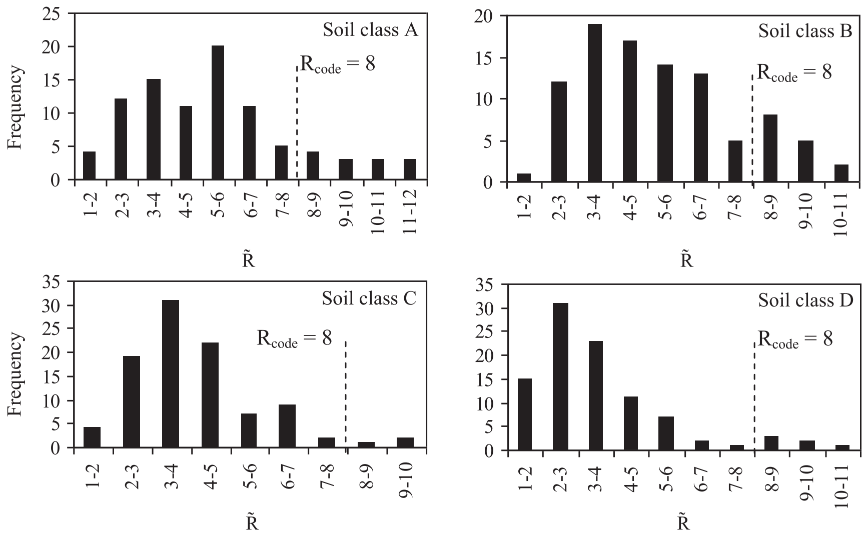

Figure 5. Histograms of R factor considering soil–structure interaction.

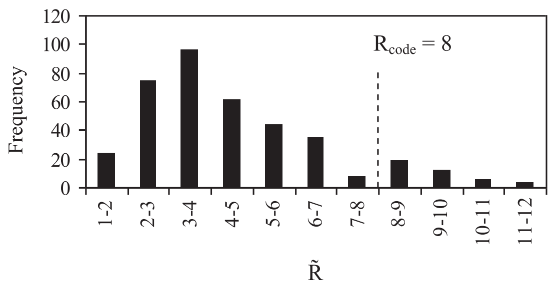

Figure 6. Histograms of R factor considering soil–structure interaction for all soil classes.

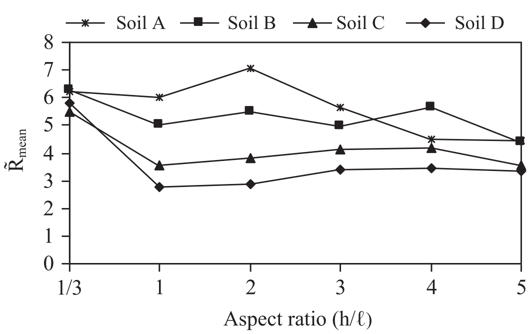

Figure 7. Variation of mean strength reduction factors of interacting case with aspect ratio for different soil classes.

Results of IDAs

Strength reduction factor (*R*) and inelastic displacement ratio (*C*) of sample buildings for both fixed-base and interacting cases are calculated from incremental dynamic analyses. Some of the IDA curves generated for sample buildings for fixed-base and interacting cases are shown in Figure 4. Figure 5 shows the histograms of *R* factor considering soil–structure interaction for different soil classes. Besides, Figure 6 shows the histogram of *R* factor for all sample buildings regardless of difference in soil classes. The strength reduction factor given in the code for the considered sample frame type is also shown in figures with dashed line. It can be seen from the figures that the strength reduction factors calculated considering soil–structure interaction are generally smaller than the one given in the code, especially for soil classes C and D. However, there is a limited similar tendency for soil classes A and B. Variation of mean strength reduction factors of interacting case with aspect ratio for different soil classes is shown in Figure 7. The top line shows the mean strength reduction factors of soil class A, whereas the bottom line shows the factors of soil class D. Strength reduction factors of soil class B and C are between these lines. It can be seen from this figure that strength reduction factors decrease for decreasing values of shear wave velocity.

Figure 8 shows the variation of ratio of strength reduction factor calculated considering soil–structure interaction to design strength reduction factor against period for soil classes A to D, respectively. The mean values of these ratios are given in Figure 9 for all soil classes. It can be seen from the figures that strength reduction factors of interacting systems are almost always smaller than the design strength reduction factors given in codes.

In Figure 8, the ratio of strength reduction factor calculated considering soil–structure interaction to design strength reduction factor for each earthquake record is shown. As it

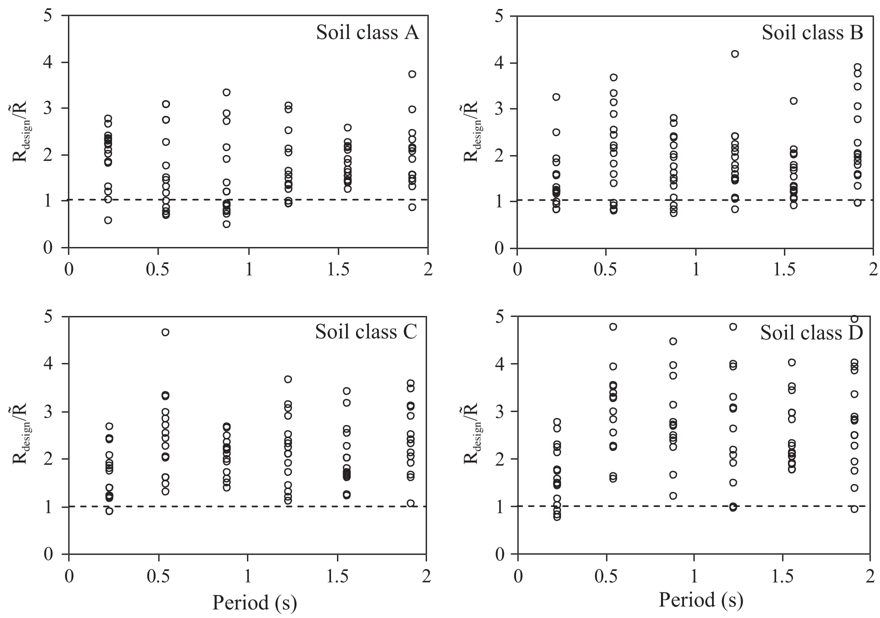

Figure 8. Ratio of strength reduction factor calculated considering soil–structure interaction to design strength reduction factor for soil classes.

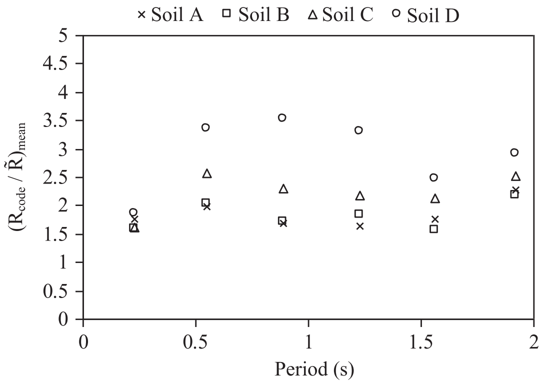

Figure 9. Ratio of design strength reduction factor to strength reduction factor calculated considering soil–structure interaction for all soil classes.

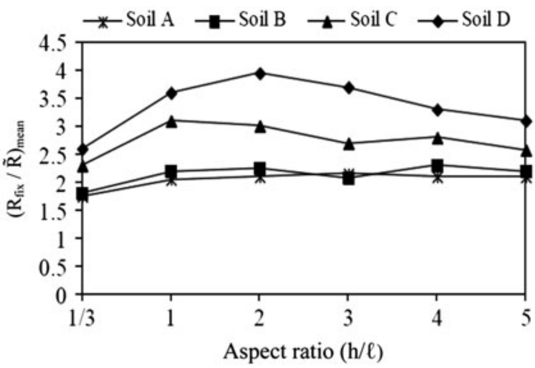

Figure 10. Variation of ratio of fixed-base strength reduction factor to interacting strength reduction factor with aspect ratio.

is seen in Figure 8, the dispersion in mentioned ratios is large for different earthquake records, so the mean values of these ratios are given in Figure 9. Also, it is worthy noting that, since the limited number of MDOF systems with different periods are mentioned in this study, results cannot be given for the whole period range. It can be said that the average value of the mentioned ratio is approximately equal to 1.75 for soil classes A and B, 2.5 for soil class C and 3.5 for soil class D, respectively. There are some previous studies in agreement with the findings of this study. One of the most important conclusions of these studies is that soil–structure interaction reduces strength reduction factor values, especially for a particular period range (Aviles & Perez-Rocha, 2005b) and for the entire period range (Ghannad & Jahankhah, 2004, 2007). The strength reduction factor graph plotted for SDOF systems on rock sites with and without interaction in the study of Ghannad and Jahankhah, strength reduction factors for fixed-base case is almost always larger than the corresponding values of interacting case.

Figure 10 shows the variation of ratio of strength reduction factor of fixed-base case to interacting case with aspect ratio for soil classes A to D, respectively. The top line shows the mean strength reduction factor ratio of soil class D, whereas the bottom lines show the ratios of soil classes A and B. It can be seen from the figure that fixed-base strength reduction factors of all soil classes – but especially C and D – are almost always larger than the strength reduction factors of interacting case. The ratio of fixed-base strength reduction factor to interacting strength reduction factor can be nearly up to 4.0 for soil class D and 3.0 for soil class C. Thus, using the strength reduction factors of codes given for fixed-base case, in case of predominant soil–structure interaction effect–such as low shear wave velocity and considerable soil flexibility–leads to higher ductility demand in the structure and non-conservative design forces. This is partially in agreement with earlier findings by other researchers (Aviles & Perez-Rocha, 2005a; Ghannad & Jahankhah, 2004, 2007; Jarernprasert, 2005; Jarernprasert, Bazan-Zurita, & Bielak, 2001). Although previous studies such as Aviles and Perez-Rocha (2005a, 2005b) and Ghannad and Jahankhah (2004, 2007) are focused on SDOF systems, the results are in agreement with the findings of this study. For many reasons, SDOF systems may be viewed representative for complex MDOF systems; the consistency of the results for these systems is meaningful. Besides, the results of this study is in agreement with another study conducted by Ganjavi and Hao which concludes that generally SSI reduces the strength reduction factor of both MDOF and more intensively SDOF systems (Ganjavi & Hao, 2012b).

Figure 11 shows the variation of inelastic displacement ratios of fixed-base and interacting cases with period for soil classes A to D, respectively. It is seen from the figures that inelastic displacement ratios of fixed-base and interacting cases are very close to each other and approximately equal to unity, especially for periods longer than .5s. This behaviour is in accordance with well-known "equal displacement rule" for fixed-base systems at long period range. Besides, for periods shorter than .5s, there is a difference between inelastic displacement ratios of fixed-base and interacting cases, and the ratio can be up to 1.5.

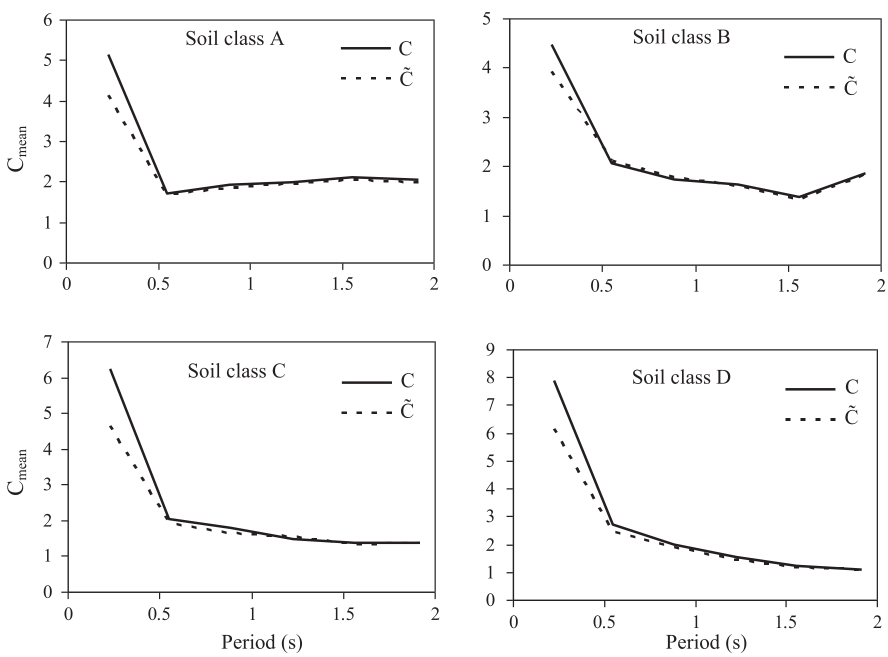

Figure 11. Inelastic displacement ratios of fixed-base and interacting cases for soil classes.

Conclusions

In this paper, the seismic behaviour of multistorey structures considering soil-structure interaction effects is investigated. For this purpose, samples 1, 3, 6, 9, 12 and 15 storey plane frames with aspect ratios of 1/3, 1, 2, 3, 4 and 5 were generated according to Turkish Seismic Design Code (2007). Sixty-four ground motions recorded on site conditions such as rock, stiff soil, soft soil and very soft soil classified based on USGS (United States Geological Survey) site classification system were used in analyses. Incremental dynamic analyses were performed for sample buildings to determine the yielding and collapse capacity of each sample building. The local and global criteria are used to define yielding point, whereas maximum ID ratio is considered as the primary and most important global collapse criterion and limited to 3%. Analyses were repeated both for fixed-base case and considering soil–structure interaction. Structural parameters such as strength reduction factors and inelastic displacement ratios of sample frames were calculated for fixed-base and interacting case. As the more complex models such as coupled springs and macro elements for non-linear soil behaviour are available, they have not been considered for simplicity. For this reason, it should be noted that the results are valid for analyses of interacting cases that conducted with simplified modelling using uncoupled linear springs and dashpots for horizontal and rocking modes. The following conclusions can be drawn from the results of this study.

The histograms for strength reduction factor values considering soil–structure interaction are given in Figures 5 and 6 for soil classes individually and all sample, respectively. It is seen that strength reduction factors calculated considering soil–structure interaction are generally smaller than the one given in the code, especially for soil classes C and D. However, there is a similar tendency also for soil classes A and B. This case leads an unsafe design in case of primary soil–structure interaction effects.

Variation of mean strength reduction factors of interacting case with aspect ratio for different soil classes is shown in Figure 7. It can be seen from this figure that strength reduction factors decrease from soil class A to D for considered aspect ratio values. The ratio between the cases of soil class A and D can be up to 2.0. The ratio of strength reduction factor value given in codes for design to the one calculated considering soil–structure interaction is almost always higher than unity for all sample buildings investigated. Especially for soil classes C and D, soil–structure interaction effects on strength reduction parameters cannot be neglected. Thus, using the fixed-base strength reduction factor for interacting case leads to higher ductility demand in the structure and non-conservative design forces. The considered ratio shows an increased tendency for lower shear wave velocities.

Inelastic displacement ratios of fixed-base and interacting cases are found to be quite different especially for periods shorter than .5 s and the ratio of these inelastic displacement ratios can be up to 1.5.

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