Earthquakes and Structures, *Vol. 10, No. 6 (2016) 1273-1289*
DOI: http://dx.doi.org/10.12989/eas.2016.10.6.1273

Overstrength factors for SDOF and MDOF systems with soil structure interaction

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(Received July 30, 2015, Revised April 15, 2016, Accepted April 20, 2016)

Abstract. This paper addresses the concept of lateral overstrength; the ratio of actual lateral strength to design base shear force, for both SDOF and MDOF systems considering soil structure interaction. Overstrength factors are obtained with inelastic time history analysis for SDOF systems for period range of 0.1-3.0 s, five different aspect ratios (h/r=1, 2, 3, 4, 5) and five levels of ductility (μ=2, 3, 4, 5, 6) considering soil structure interaction. Structural overstrength for MDOF systems are obtained with inelastic time history collapse analysis for sample 1, 3, 6, 9, 12 and 15 storey RC frame systems. In analyses, 64 ground motions recorded on different site conditions such as rock, stiff soil, soft soil and very soft soil are used. Also lateral overstrength ratios considering soil structure interaction are compared with those calculated for fixed-base cases.

Keywords: seismic design; overstrength; soil structure interaction; SDOF systems; multi-storey structures

1. Introduction

Most of the current seismic design codes still sustain force based design criteria for new buildings despite the recent developments in displacement based design methods which aim at controlling earthquake damage to structural elements and many types of nonstructural 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 nonstructural elements, moderate earthquakes without structural damage, or with some nonstructural damage, and severe, infrequent earthquakes with damage to both the resisting systems and to nonstructural components. Hence, the conventional force based design method requires the usage of a reduction factor which leads a structure to be designed for a much less seismic force than the required one for structure to remain in elastic range. Post-earthquake investigations and seismic evaluations prove that the actual capacities of structures can be much higher than the design forces; this extra strength is called structural overstrength. Basically structural overstrength is expressed as the ratio of actual lateral

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http://www.techno-press.com/journals/eas&subpage=7 ISSN: 2092-7614 (Print), 2092-7622 (Online)

strength to design base shear force of the structure.

The main sources of structural overstrength are investigated in many previous studies and found to include (a) the difference between the actual and the design material strength; (b) conservatism of the design procedure and ductility requirements; (c) load factors and multiple load cases; (d) accidental torsion consideration; (e) serviceability limit state provisions; (f) participation of nonstructural elements; (g) effect of structural elements not considered in predicting the lateral load capacity (e.g., actual slab width); (h) minimum reinforcement and member sizes that exceed the design requirements; (i) redundancy; (j) strain hardening; (k) actual confinement effect; and (l) utilizing the elastic period to obtain the design forces. (Uang 1991, Mitchell and Paulter 1994, Humar and Ragozar 1996, Park 1996).

Jain and Navin (1995) studied on the seismic overstrength of multistory reinforced concrete frames by means of nonlinear pseudo static analysis on four-bay, three-, six-, and nine storey frames designed for seismic zones I to V as per Indian codes. They reported that the overstrength increases as the number of stories decreases; and interior frames have higher overstrength as compared to the exterior frames of the same building. Kappos (1999) focused on the evaluation of behavior factors for seismic design of structures, with due consideration to both ductility and overstrength and concluded that the overstrength-dependent part of the behavior factor is found to be higher in the case of low rise structures compared to medium- and high-rise structures for the buildings considered. Elnashai and Mwafy (2002) investigated the relationship between the lateral capacity, the design force reduction factor, the ductility level and the overstrength factor by means of inelastic static pushover as well as time-history collapse analysis for 12 RC buildings. They concluded that the conservative overstrength of medium and low period RC buildings is proposed and a new ratio between the overstrength factor and the force reduction factor is defined as the inherent overstrength. Stefano *et al*. (2006), studied on the effect of overstrength on the seismic behavior of multi-storey regularly asymmetric buildings. They reported that the analyzed multi-storey asymmetric system ductility demands may become larger at unexpected locations because of overstrength and in the upper floors of the asymmetric building, overstrength reaches very large values. In 2008, Annan *et al*. studied on the inelastic behavior of steel frames to assess the structural overstrength resulting from redistribution of internal forces in the inelastic range, design assumptions, and strain hardening behavior of steel and displacement ductility. In 2011, L. Sanchez-Ricart conducted a study which incorporates the development of a computer program to test the influence of the structural overstrength to calibrate seismic codes. More recently, Louzai and Abed (2015) conducted a comparative study on seismic behavior factors including overstrength factor for RC frame structures with non-linear static pushover and incremental dynamic analyses. Mohammadi *et al*. (2015) focused on the reliability index and the behavior factor of a numbers of three dimensional RC moment resisting frames with the same story area, equal lateral resistant as well as different redundancy using both deterministic and probabilistic overstrength approaches.

As all of these mentioned studies focus on the overstrength in fixed base systems, a similar study involving the effects of soil structure interaction on the structural overstrength has not been carried out as far as the authors' knowledge. Thus, the effect of foundation flexibility on structural overstrength for SDOF and MDOF systems is aimed to be studied comprehensively.

Soil-structure interaction effects on inelastic behavior have been the topic of some investigations (Ciampoli and Pinto 1995, Rodriguez and 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. (Lin and Miranda 2008). During last decade, Aviles and Perez-Rocha studied on soil-structure interaction phenomenon widely (2005, 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. Also, Ghannad and co-workers studied on soil–structure interaction effects on strength reduction factors and ductility demands (Ghannad and Jahankhah 2004, 2007). They showed that both ductility and strength demanded by the structure may experience considerable variations under the effect of SSI. 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 behaviour of structures has been studied by Eser *et al*. (2012).They proposed new equations for inelastic displacement ratio of interacting system, as a function of structural period of interacting system, ductility and period lengthening ratio. 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 and Trifunac 1991). 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 and Hao 2012a, 2012b). More recently, a study to estimate higher mode effects of multistorey structures with considering soil structure interaction under near fault ground motions is conducted (Khoshnoudian *et al*. 2014). Another study focusing on the effects of soil structure interaction on the strength reduction factors of multistorey buildings is completed and a new formula to estimate strength reduction factors for MDOF structure-soil systems is derived in (Nik and Khoshnoudian 2014). The objective of this study is to present the results of an investigation conducted to provide more information on the structural overstrength for interacting systems compared to fixed base systems. To this purpose, structural overstrength is first investigated for SDOF systems with period range of 0.1-3.0 s with elastoplastic behavior for five different aspect ratios (h/r=1, 2, 3, 4, 5) and five levels of ductility (μ=2, 3, 4, 5, 6) considering soil structure interaction. Later, inelastic time history analyses are conducted for sample 1, 3, 6, 9, 12 and 15 storey RC frame systems. 64 ground motions recorded on different site conditions such as rock, stiff soil, soft soil and very soft soil are used for the analyses. Results are compared with those calculated for fixed-base cases.

2. Analysis method

The soil structure analysis may be conducted either in the frequency domain using harmonic impedance functions or in the time domain using impulsive impedance functions. However, the frequency-domain analysis is not practical for structures that behave nonlinearly. On the other hand, the time-domain analysis can be conducted by using constant springs and dampers regardless of frequency to represent the soil. With this simplification, the convolution integral describing the soil interaction forces is avoided, and thus the integration procedure of the equilibrium equations is carried out as for the fixed-base case. In the present study, the described soil-structure model is analyzed in time domain. The dynamic equation of motion of a SDOF system is given by

$$m\ddot{u} + c\dot{u} + ku = -m\ddot{u}_g \tag{1}$$

where u is the relative displacement and $\ddot{u}_g$ is the acceleration of ground motion. For SDOF systems, Newmark method for step by step time integration was adapted in an in-house computer program for inelastic time history analyses. Besides, for inelastic time history analyses of MDOF systems, 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).

Since the yield point can easily be obtained for SDOF systems, definition of response parameters for different limit states are needed to obtain the seismic performance and overstrength factors of sample MDOF buildings for both fixed-base and interacting cases. Two important limit states in the response of the buildings are yielding and collapse. In this study, yield point of building where elastic behavior 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 strain of cover concrete reaches the strain corresponding to the compressive stress of concrete at ground floor column sections. The material strains corresponding to these situations are 0.002 for cover concrete (ε_{co}) and 0.0021 for reinforcing steel (ε_{sy}), respectively. For global criteria, the yield capacity of the structure is defined as the point where the incremental dynamic analysis curve leaves the linear path. There are many previous studies using the local yield criteria as response parameters such as Elnashai and Mwafy (2002), Mwafy *et al*. (2010), Di Sarno *et al*. (2003) and Aksoylar *et al*. (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 and Mwafy 2002, Penelis and 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 *et al*. 1998, FEMA 355E 2000).

In this study a total of 64 earthquake acceleration time-histories recorded on different soil types are used. Ground motions are selected to represent far-field earthquakes based on far field definition in ATC documents (1996 and 2008). Near-field records are deliberately excluded in the present study. Details of selected ground motions are listed in Table 1. These accelerograms are downloaded from the strong motion database of the Pacific Earthquake Engineering Research (PEER) Center. Site classes given in the Table 1 are in accordance with United States Geological Survey site classification system (Boore 1993) which correspond to shear wave velocity value higher than 750 m/s for site class A, between 360-750 m/s for site class B, 180-360 m/s for site class C and lower than 180 m/s for site class D.

3. Soil-structure interaction model

For fixed-base case, there is no need to define foundation beneath the structure. For interacting

Table 1 Earthquake ground motions used in analyses

Earthquake	M	Station	Station no	Dist. (km)	Comp. 1	PGA (g)	PGV (cm/s)	Comp. 2	PGA (g)	PGV (cm/s)	Site 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
Northridge 17/01/94	6.7	Montebello	90011	86.8	BLF206	0.179	9.4	BLF296	0.128	5.9	D

Table 1 Continued

Earthquake	M	Station	Station no	Dist. (km)	Comp. 1	PGA (g)	PGV (cm/s)	Comp. 2	PGA (g)	PGV (cm/s)	Site class
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

case, the foundation is modeled as a circular disk of radius r. The soil under the foundation is characterized by shear wave velocity V_s, mass density ρ and Poisson's ratio υ. The soil related parameters defined based on the concept of Cone Models (Wolf 1994) are given as follows

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

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

$$C_x = \rho \cdot V_s \cdot \pi \cdot r^2 \tag{4}$$

$$C_\theta = \rho \cdot V_p \cdot \pi \cdot \frac{r^4}{4} \tag{5}$$

Shear wave velocities and calculated properties for different soil types are given in Table 2.

For SDOF systems, the most common approach to consider soil structure interaction effects is to use a single degree of freedom replacement oscillator with effective period and damping of the system. The first well-known studies on the use of replacement oscillator were conducted by Veletsos and his co-workers (Veletsos *et al*. 1974, 1975, 1977). Effective period and damping of the system are denoted by $\tilde{T}$ and $\tilde{\beta}$, respectively, as they are used in current U.S. codes (ATC 3-06/1984, FEMA 450/2003). Effective period of the interacting system is given by the equation below

$$\tilde{T} = T\sqrt{1 + \frac{k}{K_x}\left(1 + \frac{K_x h^2}{K_\theta}\right)} \tag{6}$$

Rearranging this equation gives the equivalent stiffness of the interacting system as follows

$$\frac{1}{k_{eq}} = \frac{1}{k} + \frac{1}{K_x} + \frac{h^2}{K_\theta} \tag{7}$$

Effective damping for the interacting system is given by the equation below

$$\tilde{\beta} = \beta_0 + \frac{0.05}{\left(\frac{\tilde{T}}{T}\right)^3} \tag{8}$$

Table 2 Dynamic properties of different soil types

Soil properties	Soil class			
	A	B	C	D
Shear wave velocity (m/s)	750	400	250	150
Horizontal stiffness of soil medium, K_x (kN/m)	1.35×10^6	4.4×10^5	1.72×10^5	6.6×10^4
Rocking stiffness of soil medium, K_θ (kN/m)	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

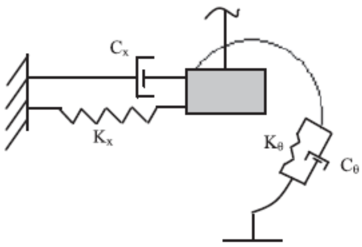

Fig. 1 Mathematical model of supports with soil-structure interaction

where β_0 denotes the foundation damping factor and values for this factor should be read from the figure given in current U.S. codes (ATC 3-06/1984, FEMA 450/2003).

For MDOF systems, 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). Realizing 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. 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 characterized 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. A schematical view considering soil-structure interaction modelling of supports is shown in Fig. 1.

4. Results and discussion

4.1 SDOF systems

In Fig. 2, variations of overstrength factor (Ω) against period are shown for fixed base and interacting systems. The results are presented for two ductility demands (μ=2 and 6), two aspect ratios (h/r=1 and 5) and considered soil classes. It can be seen from the figure that, as the ductility demand increases, the overstrength factors for both fixed base and interacting cases increase for all soil classes. Fixed base overstrength factors are almost always smaller than the corresponding ones of interacting systems. Although the variation of overstrength factors for fixed base and interacting cases are similar for low aspect ratios (h/r=1), this tendency is reversed for increasing aspect ratios. Besides, it should be noted that overstrength factor variation is strictly based on structural period for all parameters and considered soil classes. From a certain period point, say 1.0 s, the variation in overstrength factors vanishes and the mentioned factor remains approximately constant. As the soil class varies from A to D, the overstrength factors tend to increase especially for short period range.

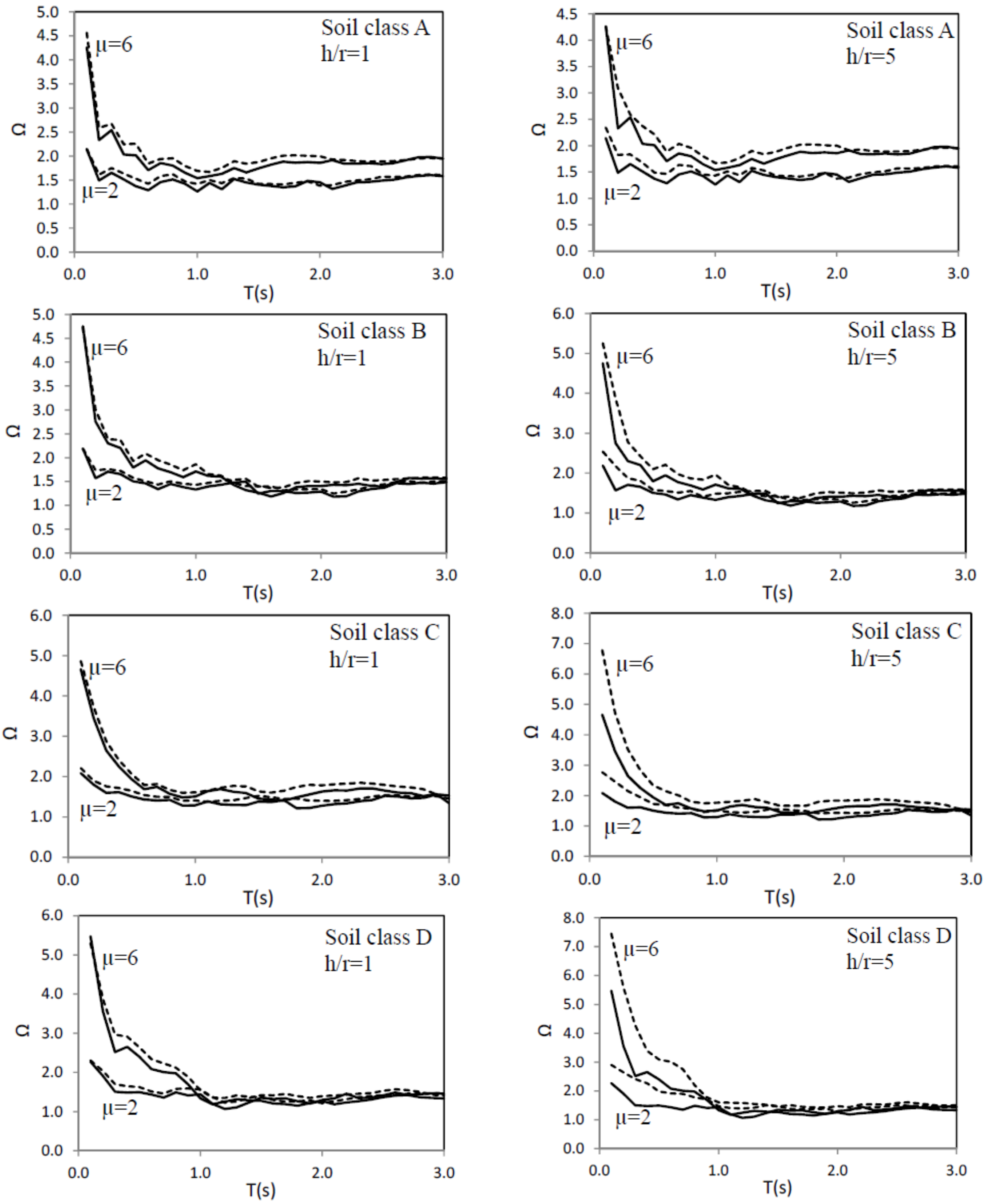

Fig. 2 Variations of overstrength factors against period with (dashed line) and without (solid line) interaction

Fig. 3 shows the variations of overstrength factor (Ω) for an interacting system with an aspect ratio of 3. Results are presented for all ductility demand considered. The top line shows the result of a system with ductility demand of 6 whereas the bottom line presents the ductility demand of 2.

As the ductility demand increases, the overstrength factors tend to increase and increase amount is much larger for soil class D compared to soil class A. The maximum overstrength factors for varying ductility demands are found to be 4.4 for soil class A, 4.8 for soil class B, 5.2 for soil class C and for soil class D 5.6, respectively.

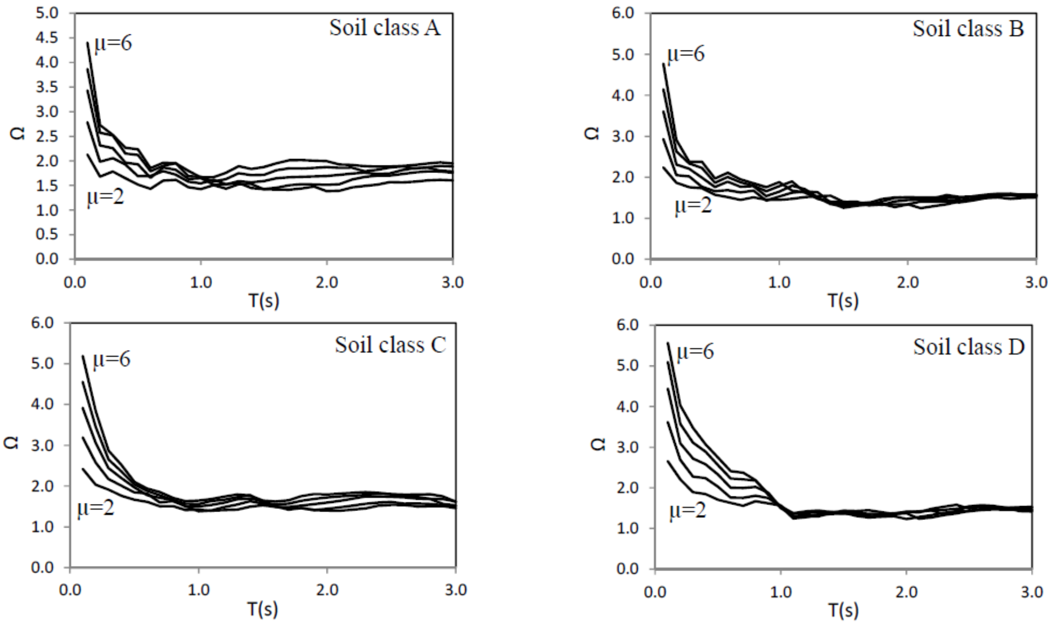

Fig. 3 Variations of overstrength factors with ductility for an interacting system with aspect ratio (h/r) of 3

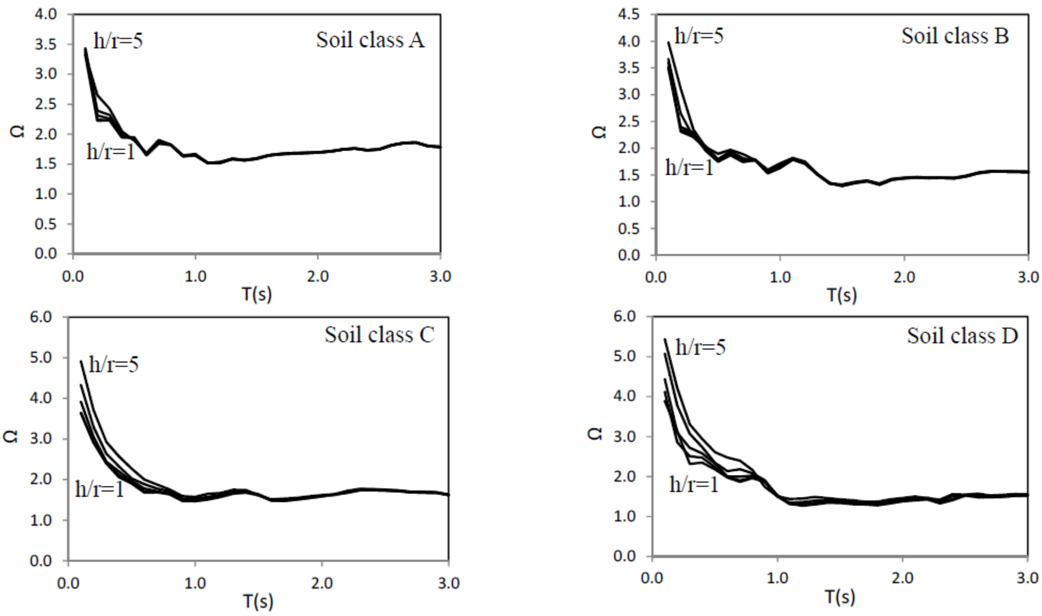

Fig. 4 Variations of overstrength factors with aspect ratio for an interacting system with ductility demand (μ) of 4

Fig. 4 shows the variations of overstrength factor (Ω) for an interacting system with ductility demand of 4. Results are presented for varying aspect ratios of 1 to 5. The top line shows the result of a system with aspect ratio of 5 whereas the bottom line presents the aspect ratio of 1. It is seen from the figure that aspect ratio is an effective parameter on overstrength factor especially in high frequency region. The variation of overstrength factor due to aspect ratio is obvious especially for soil class D and period range shorter than 1.0s. As the aspect ratio increases, the overstrength factors tend to increase and increase amount is much larger for soil class D compared to soil class A. The maximum overstrength factors for varying aspect ratios are found to be 3.4 for soil class A, 4.0 for soil class B, 4.9 for soil class C and for soil class D 5.4, respectively.

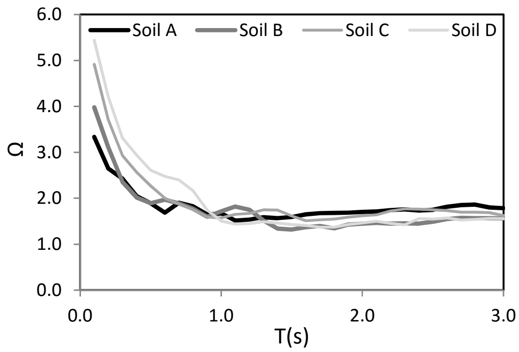

Fig. 5 Variation of overstrength factors with soil classes. Results correspond to an interacting system with aspect ratio (h/r) of 3 and ductility demand (μ) of 4

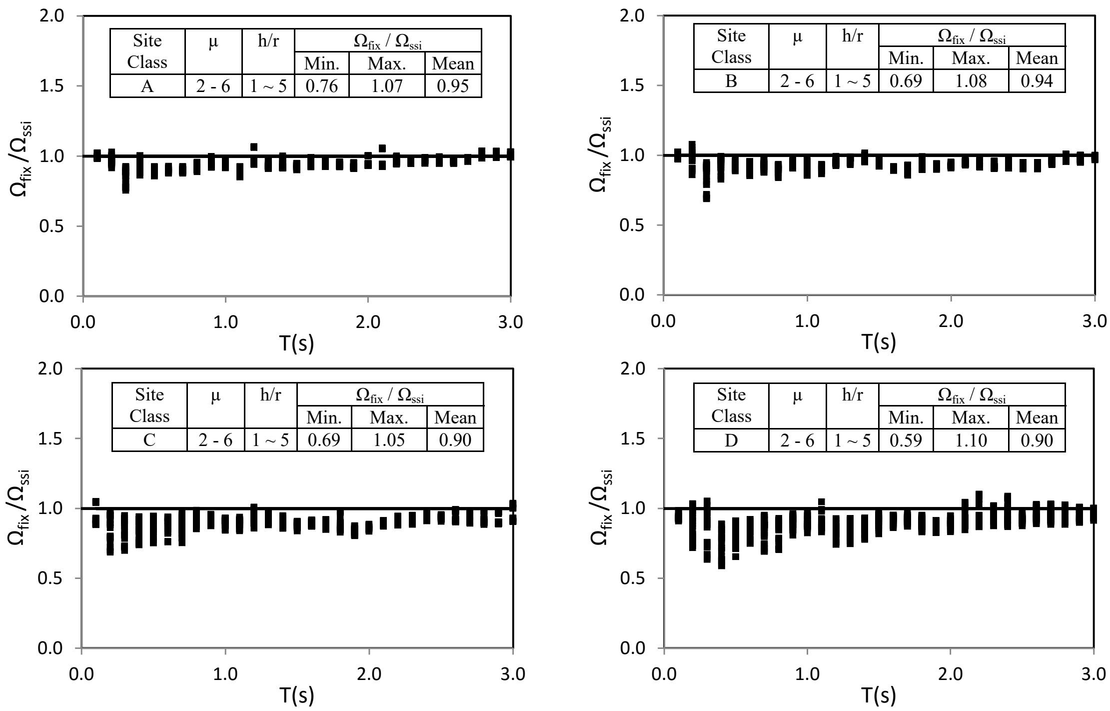

Fig. 6. Variation of the ratio of overstrength factors with and without interaction on different soil classes

The variations of overstrength factor (Ω) according to soil classes are shown in Fig. 5. It is seen from the figure that, although the overstrength factors of soil class A are almost always smaller than the corresponding ones of soil class D for short period range, it is not possible to mention about such a certain variation from this period point.

The ratios between the overstrength factors of fixed base and interacting systems are shown in Fig. 6 for different soil classes. It is seen from this figure that, as expected, aforementioned ratios decrease for lower values of shear wave velocity. Although the ratio of overstrength factors are very close to unity for T>1.0 s for site classes A and B, this ratio is much smaller than unity for site classes C and D. But for T<0.5 s, SSI exhibits an important variation for different site classes. The effect of interaction is clearer for site classes C and D. Therefore, it is an acceptable and reasonable approach not to consider soil structure interaction for shear wave velocities higher than approximately 250 m/s.

4.2 MDOF systems

As a second part of analyses to focus on the effects of soil structure interaction on the overstrength factors of multistorey structures sample 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. Realizing that no two structures are the same, and that the dynamic behaviors 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 and/or different story heights, the frames are selected to be two-dimensional regular type with mid-length spans for simplicity. Typical elevation view for sample buildings can be seen in Fig. 7. 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 behavior 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 *et al*. (1988). Softening beyond the maximum compressive strength is taken into consideration as a linear function. Steel behavior 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 3. More details regarding member cross-section sizes and reinforcements are given elsewhere (Eser Aydemir 2011). The periods of vibrations of sample buildings are relatively longer than typical RC buildings with the same heights because, especially, the column sections of investigated frames have the minimum dimensions satisfying the ductile behavior and design requirements such as strong column-weak beam principle.

The overstrength factors of fixed base and interacting systems for considered multistorey structures are shown in Fig. 8. Besides, Fig. 9 presents the overstrength factors of interacting case for different soil classes. It can be seen from the figures that, the overstrength factors of interacting case are almost always greater than the corresponding ones of fixed base case for all sample buildings. Especially for one storey structure -which is representative of a SDOF system- the factors with interaction are much greater than the fixed base case. It is also worth noting that, although the overstrength factors of soil class D are almost always greater than the corresponding ones of soil class A for low-rise buildings (i.e., for short period systems), this tendency is reversed for high-rise buildings (i.e., for long period systems), as it is observed for SDOF systems in Fig. 5.

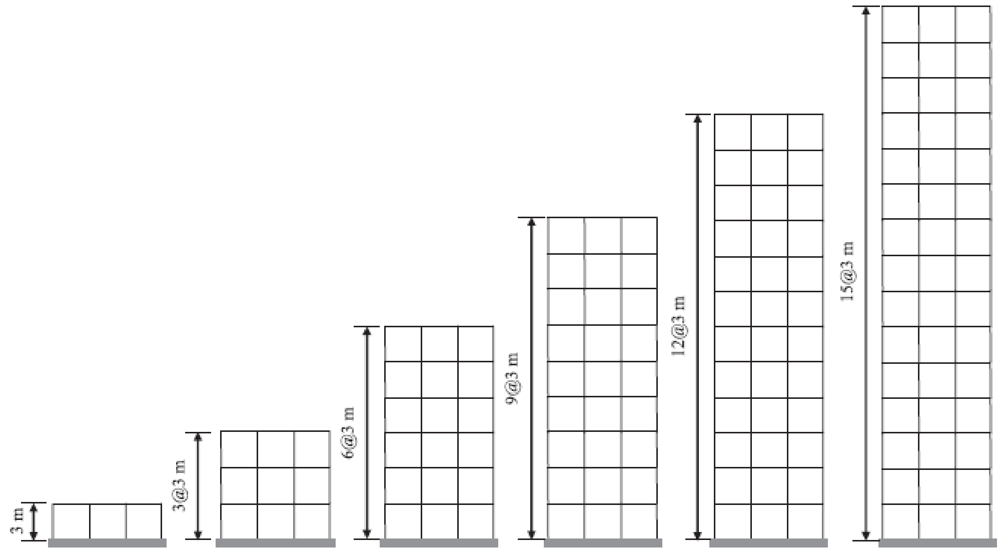

Fig. 7 Geometry of the considered sample RC frames

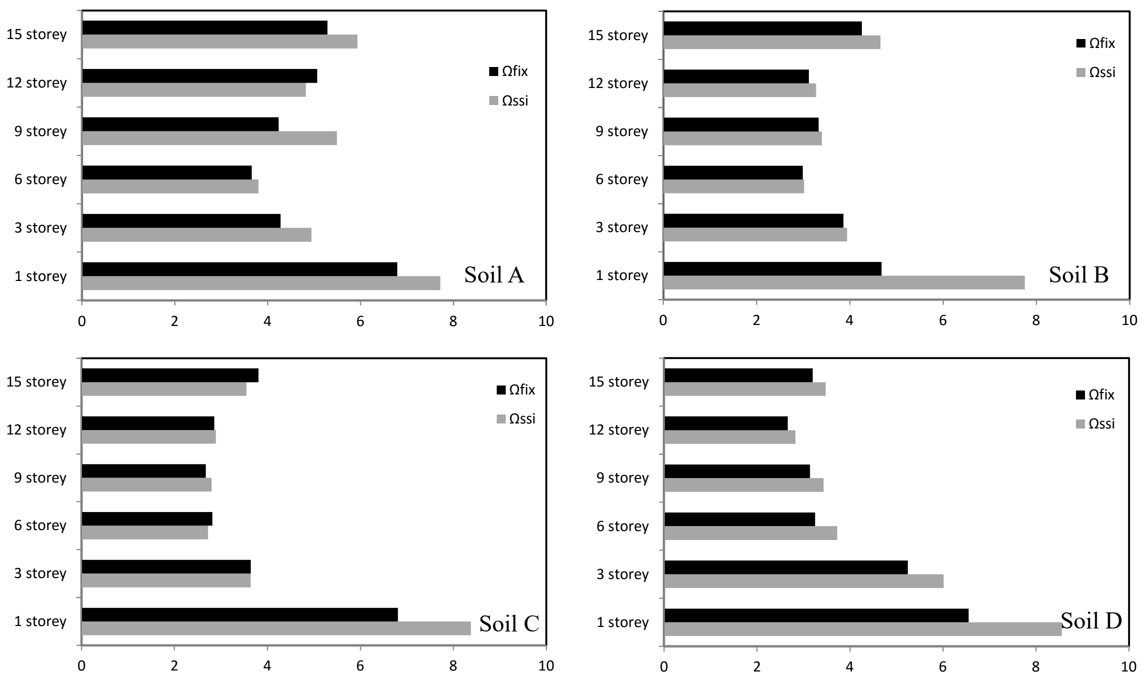

Fig. 8 Variation of overstrength factors for multistorey structures

Table 3 Properties of the reference buildings

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

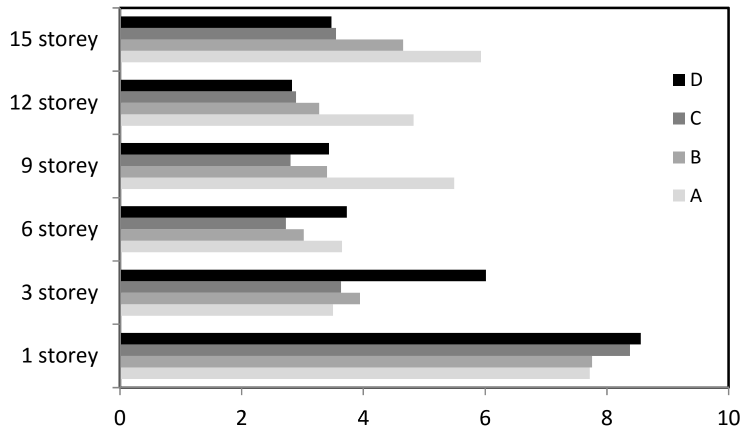

Fig. 9 Variation of overstrength factors with soil classes for multistorey structures

Fig. 9 also shows that as the number of stories increases, the overstrength factors tend to decrease. This is because in low-rise buildings the gravity loads play a more prominent role in the design of members than in high-rise buildings located in the same seismic zone. In other words, seismic forces generally play a less important role in the determination of cross-sectional sizes and reinforcements than do gravity loads, which govern the design of those buildings. Variation in overstrength with the number of stories is significant approximately for all soil classes. This is partially in agreement with the earlier findings by other researchers (Jain and Navin 1995, Elnashai and Mwafy 2002).

5. Conclusions

In this study, overstrength factors are investigated for both SDOF and MDOF systems with soil structure interaction considering ground motions recorded on different site conditions such as rock, stiff soil, soft soil and very soft soil. For this purpose, SDOF systems with period range of 0.1-3.0 s with elastoplastic behavior for five different aspect ratios (h/r=1, 2, 3, 4, 5) and five levels of ductility (μ=2, 3, 4, 5, 6) are considered. Subsequently, inelastic time history analyses are conducted for sample 1, 3, 6, 9, 12 and 15 storey RC frame systems representing MDOF systems considering soil structure interaction. The following conclusions can be drawn from the results of this study.

• Fixed base overstrength factors are almost always smaller than the corresponding ones of interacting systems. Although the variation of overstrength factors for fixed base and interacting cases are similar for low aspect ratios (h/r=1), this tendency is reversed for increasing aspect ratios.

• Overstrength factor variation is strictly based on structural period for all parameters and considered soil classes. From a certain period point, say 1.0 s, the variation in overstrength factors vanishes and the mentioned factor remains approximately constant.

• As the ductility demand increases, the overstrength factors tend to increase and increase amount is much larger for soil class D compared to soil class A.

• Aspect ratio is an effective parameter on overstrength factor for interacting systems especially in high frequency region. The variation of overstrength factor due to aspect ratio is obvious especially for soil class D. This condition is also valid for multistorey structures.

• The effect of soil structure interaction on overstrength factors is clearly evident for site classes C and D. Therefore, it is an acceptable and reasonable approach not to consider soil structure interaction for shear wave velocities higher than approximately 250 m/s.

• In MDOF analyses, the overstrength factors of Soil class D are found to be almost always greater than the corresponding ones of Soil class A for low-rise buildings, but for high-rise buildings this trend is reversed. As the number of stories increases, the overstrength factors tend to decrease because of gravity loads are more effective in the design of members than the seismic forces.

• The results of the analyses are valid for the considered earthquake database where the near field effects are not investigated, thus in future researches near field effects on the overstrength ratios can be investigated. Besides, this study focuses on very regular and symmetric MDOF systems with frame elements. As most of the existing building stock consist of irregular buildings and / or buildings with shear walls, the effects of the mentioned properties on the overstrength ratios need to be investigated for further researches.

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