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Modification Factors to Evaluate Elastic Seismic Demand of Soil-Foundation-Structure Systems

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ABSTRACT

A simplified procedure for practitioners is proposed to evaluate seismic demand of structures accounting for kinematic and inertial soil-foundation-structure-interaction (SFSI). Typically, in design codes seismic demand is calculated by the intersection of the free-field demand spectrum with the fixed-base structure's period. More detailed approximations use the intersection of the free-field with the effective period of the coupled system. FEMA440 goes one step further and proposes use of the "foundation input motion" instead of the free-field motion, accounting for kinematic interaction, whereas inertial interaction is considered in a subsequent step. After numerical investigation using the direct method, an alternative approximation will be presented herein based on appropriate modification factors of the free-field demand, to evaluate the actual demand considering SFSI. The results show that for some cases acceleration demand in most cases decreases compared to the free-field, a conclusion that is in accordance to state-of-practice analytical models suggested by other researchers.

Introduction

Available methodologies for design and assessment of structures are based on response spectrum approach (Stewart et al., 2003). The design acceleration response spectra consist of four branches (CEN, 1994): (ii) an ascending linear branch, (ii) a flat branch, (iii) an exponentially descending branch and (iv) a second exponentially descending branch. Besides, SFSI increases the structural period and the damping of the system. Most building structures and bridge piers respond between 0.5s and 2s. Regarding the spectral response, this means that the structure's acceleration response is on the flat or the descending branches and thus the SFSI effects are favorable for the structure, reducing acceleration demands and thus design forces. However, response spectra from actual earthquake recordings are not smooth in shape, like the ones proposed in building codes and have spikes at the predominant response periods (Freeman 1998). Aviles and Perez Rocha, (2003) suggested that for structures with fixed-base period lower than the soil resonant period, a detailed study of the SFSI effects is needed. In the present study, we perform thousands of viscoelastic time history analyses of simplified coupled soil- foundation- structure- systems (SFSS), to estimate seismic demand of the examined reinforced concrete structures. Properties of the soil-foundation-structure-systems are selected such as to cover a wide range of meaningful geometries and materials for engineering practice. Ten judiciously-chosen earthquake records are used to excite the soil-foundation-structure- system. The scope of the parametric analysis is to investigate the effects of SFSI on seismic elastic demand of systems ideally founded on ground

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surface, and to propose appropriate modification factors of free-field demand to evaluate the actual seismic demand considering SFSI effects.

Proposed methodology

The proposed methodology is described in the following steps: (i) Estimation of the dynamic and geometric characteristics of the simplified coupled SFSI system, namely the structure's height h and mass m, soil profile's shear wave velocity Vs, foundation's width 2B and soil free-field motion(FFM). FFM can be easily estimated through one or two dimensional elastic time history analysis for the soil. (ii) Evaluation of relative soil to structure's stiffness ratio 1/σ (Equation 1), normalized mass m_{norm} (Equation 2) and fixed-base fundamental period of structure T_{FIX} (iii). Estimation of the flexible to fixed base periods ratio T_{SFSI}/T_{FIX} in terms of 1/σ and m_{norm}, (iv) estimation of the proposed modification factors in terms of relative soil to stiffness ratio, from proposed graphs, (v) calculation of acceleration demand of the structure based on the modification factor calculated at step iv. The resulting values, when applying the proposed methodology, are in very good agreement with the ones from the time history analyses, as both inertial and kinematic interaction effects are considered The main advantage of the proposed methodology is that, in order to calculate acceleration demand considering SFSI effects, one needs to know only the main characteristics of the system, the ground response at free-field conditions and the proposed modification factors.

$$1/\sigma = \frac{h}{T_{FIX} \cdot V_s} \tag{1}$$

$$m_{norm} = \frac{m}{B^3 \cdot \rho} \tag{2}$$

Model Configuration and Parametric Analyses

A single-degree-of-freedom structure (SDOF) is used for simplicity, the degree of freedom being the horizontal displacement of the structural mass, m_s. The structure is founded on a rigid surface foundation of width equal to 2B resting on the soil surface. Between foundation's and soil nodes full connection is assumed. Four distinct geometries of structures are chosen to demonstrate the effects of soil-foundation-structure interaction on seismic demand. Details on the system properties are given in Table 1. Soil and structural properties are judiciously chosen, such as to represent a large variety of actual soil-foundation-structure- systems. Material and geometrical properties of the systems vary based on the governing parameters of soil-foundation-structure interaction problems, notably the relative structure-to-soil stiffness ratio 1/σ and the slenderness h/B (B=half-width of foundation) of the structure (Veletsos and Meek, 1974). The SDOF can represent any structure, but in this study represents typical single-column bridge piers having cylindrical cross section, a common typology of bridges.

Material properties of the structure are kept constant (modulus of elasticity E=32GPa, corresponding to concrete type C30/37), while the fictitious circular cross-section diameter d of the pier ranges from 0.6m to 3.0m. The height is 3m, 6m, 5m and 10m, so as to cover typical bridge piers that could potentially be founded on shallow footings. For the 3m-and the 6m-tall pier, square footing width is 6m, while for the 5m- and the 10m-tall pier the footing is 10m wide.

The slenderness h/B varies between, 1 and 2. The mass of superstructure m_s is 100Mg, 200Mg, 400Mg and 800Mg, standing for the concentrated mass of the two adjacent half spans of the bridge deck and half of the pier. Damping ratio is assumed to be 5% in the superstructure.

Table 1. Characteristics of the four distinct elastic soil-foundation-structure systems.

h (m)	**2B (m)**	**m_s (Mg)**	**V_s (m/s)**	**T_{soil} (s)**	**T_{fix}(s)**	**T_{SFSI}(s)**
3	6	100/200/400/800	100/200/300/400	2.0/1.0/0.67/0.5	0.10-1.18	0.12-1.78
5	10	100/200/400/800	100/200/300/400	2.0/1.0/0.67/0.5	0.10-0.92	0.12-1.59
6	6	100/200/400/800	100/200/300/400	2.0/1.0/0.67/0.5	0.10-1.88	0.20-2.70
10	10	100/200/400/800	100/200/300/400	2.0/1.0/0.67/0.5	0.10-1.32	0.20-2.32

The soil profile is simplified to a homogeneous halfspace, with mass density ρ=2Mg/m^3 and Poisson's ratio ν=1/3. For the shear wave velocity Vs of the homogeneous soil profile we assumed values of 400m/s, 300m/s, 200m/s and 100m/s, classifying the soil profile in soil class B, C and D according to Eurocode 8, Part I (CEN, 2004), to examine a wide range of shear wave velocity values that are frequent in practice. The modification of Vs with depth is not considered in this study, as the goal herein was to propose a methodology easy to use in engineering practice. Therefore, relative stiffness 1/σ varies between 0.01 and 0.98. Soil material damping is kept constant at 4% in all analyses, in order to make meaningful the comparison of peak acceleration demand between different systems. The fundamental period of soil profiles is varying from 0.5s to 2.0s (Table 1).From all configurations, we did not retain the ones that gave very low, non-realistic values for fixed-base period T_{FIX} (lower than 0.1s), as well as the combinations for which safety factor for bearing capacity under earthquake loading, according to Eurocode 8 (CEN, 1998), was lower than unity. Static safety factor for bearing capacity was evaluated according to Eurocode 7 (CEN, 1998) and was kept well above unity for all cases. In order to evaluate the safety factors, typical values for friction angle and for the shear strength of soil were assumed. It is important to mention that all analyses are conducted in the elastic range and therefore soil strength values were only used to exclude cases were the safety factor for bearing capacity was lower than or close to unity.

All selected earthquakes were recorded on rock or very stiff sites with $V_{s,30}$ larger than 600m/s. The selected records have peak acceleration amplitude varying form 1.03m/s^2 to 4.14m/s^2. Figure 1 shows the comparison between the average elastic acceleration response spectrum of the ten selected records with the elastic response spectrum proposed by Eurocode 8 (CEN, 2004) soil classification scheme for soil class A and earthquake Type A. All combinations of the input parameters result in 5120 free-field and coupled soil-foundation-structure interaction analyses. All analyses are performed in 2D (plain strain conditions for the soil profile). Dynamic analyses were performed using Opensees (PEER, 2000). The soil domain is considered homogeneous with thickness of H=50m. The soil deposit is simulated by 4-node linear elastic elements. The elastic bedrock is simulated using Lysmer-Kuhlemeyer (1969) dashpots at the base of the soil profile, and has shear wave velocity equal to Vs=1500m/s. Plane strain conditions are assumed for both soil and bedrock. The foundation is a surface, rigid foundation and simulated by 4-node linear elastic elements, and its width is equal to 2B. The structure is h meters high and is simulated by linear elastic beam elements. The structural mass is assumed to be lumped at the top of the structure.

Table 2. Earthquake records used in the parametric analyses.

No	Location	Station	Distance (km)	M_w	Rec. PGA (m/s^2)	$V_{s,30}$ (m/s)	Soil type	Fault mech. [a]
1	Friuli/Italy	ITACA_16	21.70	6.4	3.34	1029	A	RV
2	Loma Prieta/USA	NGA_765	28.64	6.93	4.14	1428	A	RV-OB
3	Northridge/USA	NGA_1011	18.99	6.69	1.11	1274	A	RV
4	Northridge/USA	NGA_994	25.42	6.69	2.81	971	A	RV
5	Northridge/USA	NGA_1078	14.66	6.69	2.24	715	B	RV
6	Kozani/Greece	ISESD_1210	16.00	5.3	1.29	623	B	NM
7	Izmit/Turkey	T-NSMP_1105	42.77	7.6	2.29	700	B	SS
8	Izmit/Turkey	T-NSMP_1109	3.40	7.6	1.60	827	A	SS
9	Kyushu/Japan	C&F_442	36.00	6.6	1.19	819	A	SS
10	L Aquila/Italy	ITACA_974	15.10	5.6	1.03	684	B	NM

a RV: Reverse, OB: Oblique, SS: Strike-Slip, NM: Normal

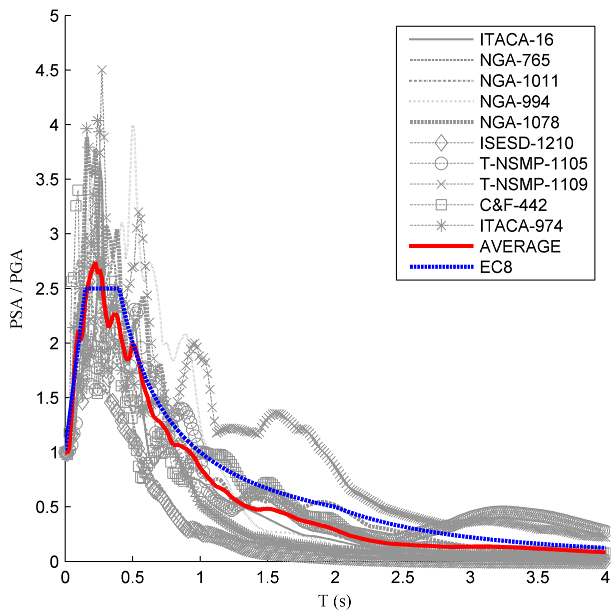

Figure 1. Comparison between average elastic acceleration response spectrum of the earthquakes used in the parametric analyses with EC8 acceleration response spectrum (Soil Class A- Type A)

Fundamental Period Calculation

The fundamental period of structures considering soil-foundation-structure interaction is usually calculated using closed form solutions, like the equation proposed by Veletsos and Meek (1974). For the estimation of horizontal and rocking stiffness of the foundation soil, analytical expressions for several combinations of footing geometries and soil profiles are presented in

Mylonakis et al. (2006). On the other hand, the structural period can be calculated directly from numerical analyses by simple division of the Fourier spectra at the top of the structure and the free-field. It can be proved that the numerically evaluated effective period $T_{SFSInum}$ is directly comparable to the analytically evaluated effective period ($T_{SFSIan.}$) from the expression proposed by Veletsos and Meek (1974), as seen in Figure 2 (concerns the numerical period values of one out of ten selected earthquake records). Discrepancy in numerical and analytical results is 1%-6%. Consequently, for the selected cases one can use the period values calculated directly from theoretical expressions proposed in literature, where K_y and K_θ are the swaying and rocking stiffness values and $\bar{k}$ and $\bar{h}$ are the stiffness and effective height of the fixed-base structure respectively.

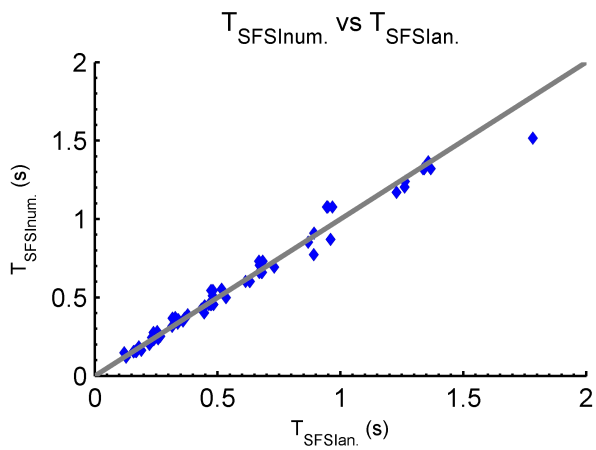

Figure2. Fundamental periods calculated by numerical and analytical procedures

Modification Factors

In the proposed approach, factors are used to estimate acceleration response at the top of a flexible structure subjected to an earthquake motion, from the acceleration response from response spectrum analysis of the flexibly-supported structure subjected to the free-field motion. These modification factors are to be used subsequently in a response spectrum framework for the estimation of seismic demand. In the proposed graphs in Figure 3, two different approximations of the structural acceleration demand are presented: In the horizontal axis is plotted the acceleration at the top of a fixed-base structure having fundamental period T_{SFSI} (calculated from the analytical expression presented above) and subjected to the free-field motion. More specifically, this acceleration is calculated as the intersection of the free-field response spectrum and the elastic period of the flexibly–supported structure. On the vertical axis is plotted the acceleration at the top of the structure, as calculated directly by the numerical analyses. Using this rationale, values on the horizontal axis represent the free-field acceleration demand, whereas the on the vertical axis the actual demand, considering for FSI. It is important to note that as both acceleration values refer to the same period (the effective period of the flexibly-supported structure), any difference between the acceleration demand simply differentiation of the input motion to the structure, and more specifically modification of the actual acceleration response at foundation level (effective foundation motion) compared to the free field acceleration motion. The modification factors concern surface foundations but the proposed methodology could be extended to various foundation types.

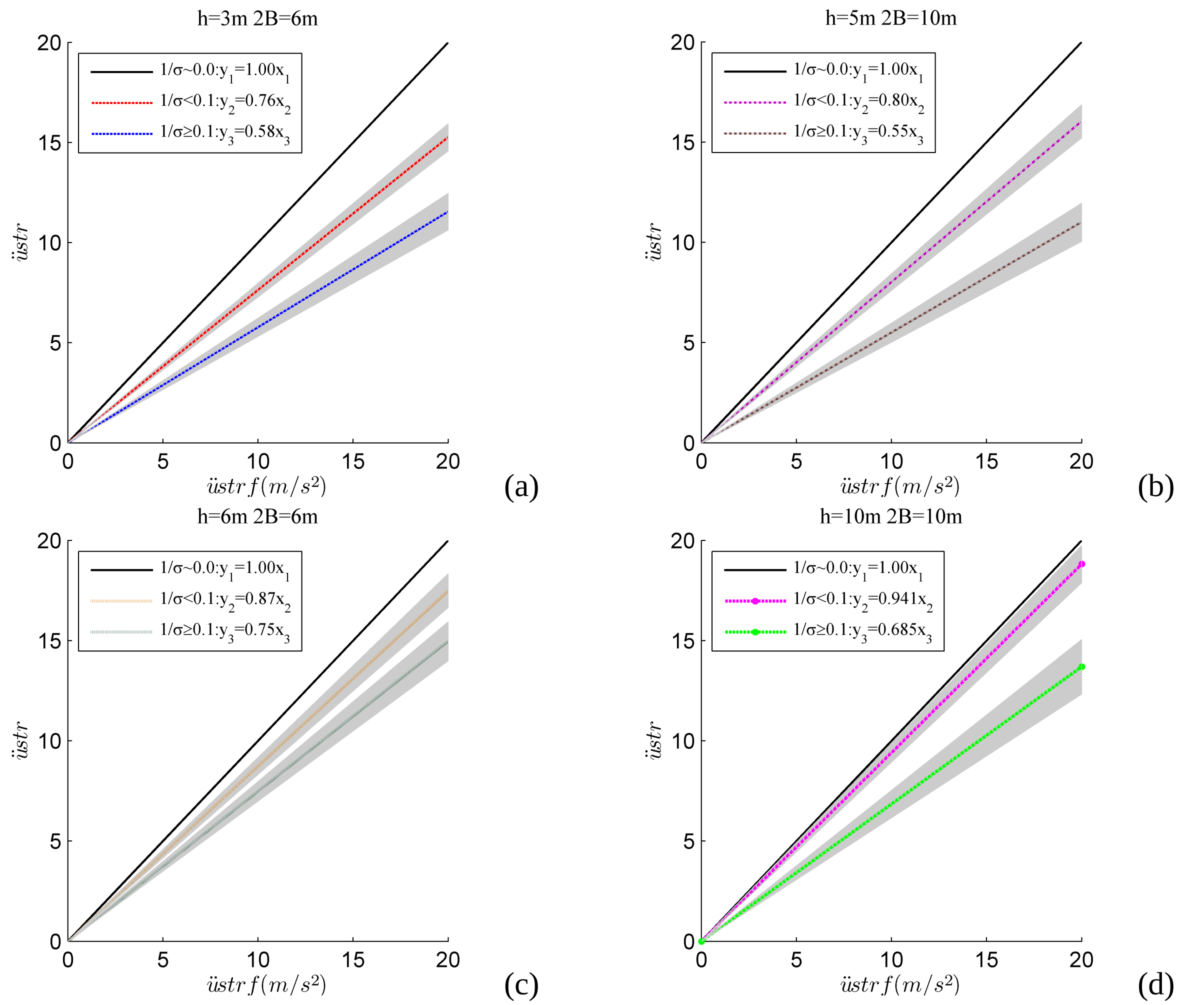

Figure 3. Radial lines that show the $\ddot{u}_{str}$ to $\ddot{u}_{str\,f}$ ratio for SDOF with (a) h=3m and 2B=6 (b) h=5m and 2B=10 (c) h=6m and 2B=6 and (d) h=10m and 2B=10. The black continuous line is the 1:1 radial line and in grey is the average line plus and minus the standard deviation.

Figure 3a presents the acceleration of a SDOF system with height h= 3mand foundation width 2B=6m (h/B=1). Figure 3b concerns a SDOF system with h/B=1, having, however, a column height h=5m and foundation width 2B=10m. Figures 3c and 3d present the results for a SDOF system of slenderness h/B=2, and height and foundation width 6m and 6m (Figure 3c) and 10m and 10m (Figure 3d) respectively. Deviation from the 1:1 radial line suggests modification of the effective foundation motion from the free-field motion. The slope of the resulting radial line, shows the value of the appropriate modification factor that is needed to multiply the free-field demand in order to calculate the actual soil-foundation-structure system demand. For all examined cases when the soil-to-structure stiffness ratio $1/\sigma$ is greater than or equal to 0.1, shifting of the resulting radial line from the 1:1 is more intense. Irrespectively the SDOF system's geometry, the average value is more or less similar for same slenderness ratio. More specifically, for h/B =1and $1/\sigma < 0.1$, the acceleration ratio slope is 0.76 for the SDOF system of 3m height and 6mwide and 0.80 for the 5m height and 10m wide, and thus their difference is about 5%. The same conclusion could be reached for soil-to-structure stiffness ratio $1/\sigma > 0.1$, with the difference being equal to 5.5%.

When the slenderness ratio is equal to two, the proposed modification factor for $1/\sigma < 0.1$ is equal to 0.87 for the 6m tall pier and equal to 0.941 for the 10m tall pier (8% difference). For the softer soil profiles of $1/\sigma > 0.1$, the modification factor results 0.75 and 0.685 for the 6m and 10m high column respectively (9% difference). These results are of great interest as it seems that the maximum acceleration at the top of the actual SFSS, and thus the design acceleration, can be a percentage of 55% - 94% of the response in case we consider the free field demand. Another important conclusion is that earthquake record characteristics have minor effect on the acceleration modification factor, and additionally that scatter is not important as presented in Figure 3.

Application example

The pier of an idealized two-span bridge is analyzed for demonstration of the methodology proposed. The bridge has total deck length of 60m and each span is 30m long. The pier is a idealized version of an actual bridge pier. It involves a single column of h=5m free height and d_p=2.2m section diameter, clamped on a surface rigid square caisson with width B_f=10m. The supported mass of the decks is assumed concentrated at the top of the pier, along with half of the pier mass. The total point mass at the top is m_s=400Mg. Damping ratio ζ=5% is assumed for the pier. The soil profile consists of a soil stratum over homogeneous halfspace, with shear wave velocity Vs=300m/s, density ρ=2Mg/m^3 and Poisson's ratio ν=1/3. According to Eurocode 7 (CEN, 1998), the static factor of safety for the pier footing is well above 1 for all cases. Both structure and soil are assumed to be linear elastic.

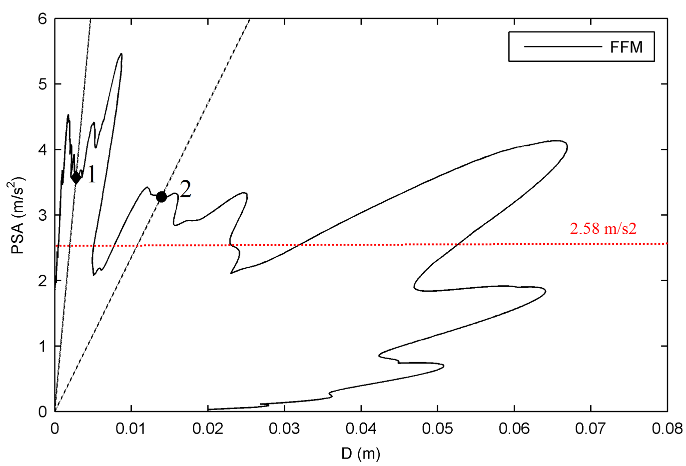

Figure 4. Free field demand curve and performance points (1 and 2) using different approximations. 2.58 m/s^2 is the absolute acceleration value at the top of the coupled SFSI system.

The elastic response spectrum from the obtained free-field motion from analysis of the soil profile only is shown in Figure 4. The demand spectrum is depicted graphically by an elastic spectrum with equivalent viscous damping ratio equal to 5%. Figure 4 presents the spectral points which are the intersection of the following curves: (i) point 1 is the intersection of the

FFM demand spectrum with fixed-base structural period T_{FIX}, (ii) point 2 is the intersection of the FFM demand spectrum with the effective period T_{SFSI} of the SFSI system. The continuous line parallel to the horizontal axis is the maximum absolute acceleration value (2.58m/s^2) that resulted at the top of the structure from numerical analysis. For this specific case study, it is not accurate to assume that the FFM is identical to the EFM, as the maximum absolute acceleration at the top of the SFSI system (2.58m/s^2) differs from the acceleration at the top of a fixed base system with fundamental period equal to the effective input motion triggered by the FFM (3.26 m/s^2). The actual acceleration in this case study is reduced by 21%, compared with the response acceleration for the system triggered with the FFM.

Conclusions

Based on the results presented, the following conclusions could be deducted from the proposed methodology to account for SFSI in seismic demand calculation. Firstly, SFSI modifies the effective foundation motion, and reduces acceleration demand by 6% to 45% from the free-field demand, according to the proposed modification factors. Even though after linear regression of the results it seems that in all cases acceleration demand reduces when considering the SFSI effects, for some combinations of the input parameters the acceleration demand increases (not depicted in the figures presented in this paper), which may be detrimental for the structure.

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