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Reduction factors to evaluate acceleration demand of soil-foundation-structure systems

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ABSTRACT

The seismic acceleration loading of structures founded on compliant soil is investigated through numerical elastic time history analyses of coupled soil-foundation-structure (SFS) systems and appropriate reduction factors of acceleration demand for free-field to evaluate acceleration demand for the SFS systems are proposed. The proposed reduction factors are the division of the acceleration demand for the coupled SFS system over the acceleration demand for the free-field, and propose an alternative method to calculate the actual acceleration loading considering interaction effects. The advantages of the proposed methodology are i) its accuracy, as the reduction factors result from coupled SFS numerical finite element analyses and consider both inertial and kinematic interaction effects and ii) its practicality, as it can be applied by the user performing no finite element numerical analysis. Additionally, the presented methodology can be applied to systems with important mass (e.g. bridge structures). The proposed acceleration reduction factors are presented in terms of dimensionless engineering parameters such as soil to structure stiffness ratio and the structure's aspect ratio. The accuracy, efficiency, and practicality of the proposed methodology are highlighted through an application to a typical bridge structure. Because structures with surface foundations are examined, inertial interaction mainly affects the acceleration demand. Therefore, the proposed reduction factors clearly demonstrate and quantify the beneficial effect of damping on buildings and bridges, as the maximum average acceleration at the top of the actual SFS system can reduce to about 55–85% of the acceleration demand for the free-field motion.

1. Introduction

The earthquake acceleration loading of any structure depends on its *dynamic properties*, as well as on the *foundation motion* which is the input motion for the structure. To evaluate the seismic acceleration loading of structures having surface foundations, the available methods are the following:

i) Assume a *fixed-base structure* subjected to the *free-field* motion. In this case, SSI effects are totally ignored. Nevertheless, this methodology is conventionally used in seismic design practice.
ii) Assume a *flexible-base structure* subjected to *free-field motion*. In this way, inertial interaction effects are considered only on the system's dynamic properties (modification of fundamental period and damping), while the effects of inertial and kinematic interaction on foundation motion are ignored [e.g. [21]].
iii) Assume a *flexible-base structure* subjected to *foundation input motion* (*FIM*). FIM is different from the actual foundation motion, as it considers only kinematic interaction effects. More specifically, this framework considers inertial interaction effects only on the system's dynamic properties (modification of fundamental period and damping) and the effects of kinematic interaction are considered only on foundation motion. This approach is based on sub-structure method, which decomposes the soil-foundation-structure system into several subdomains [4,17]. Kinematic interaction effects are interpreted in the abovementioned methodology in an approximate manner from variations between free-field and foundation ground motion indices, neglecting inertial interaction because inertial interaction effects are concentrated in a narrow frequency range around the first-mode frequency [11]. However, foundation motion is affected by both inertial and kinematic interaction as stated also in Stewart et al. [22]. Additionally, the frequency range around the first mode mainly affects the acceleration demand, and consequently the response at foundation [10].
iv) Simulate the complete soil-foundation-system with continuum numerical simulations and calculate its response in one step with direct analysis. In this case, inertial and kinematic interaction effects are considered simultaneously [e.g. [24,7]]. This procedure is the

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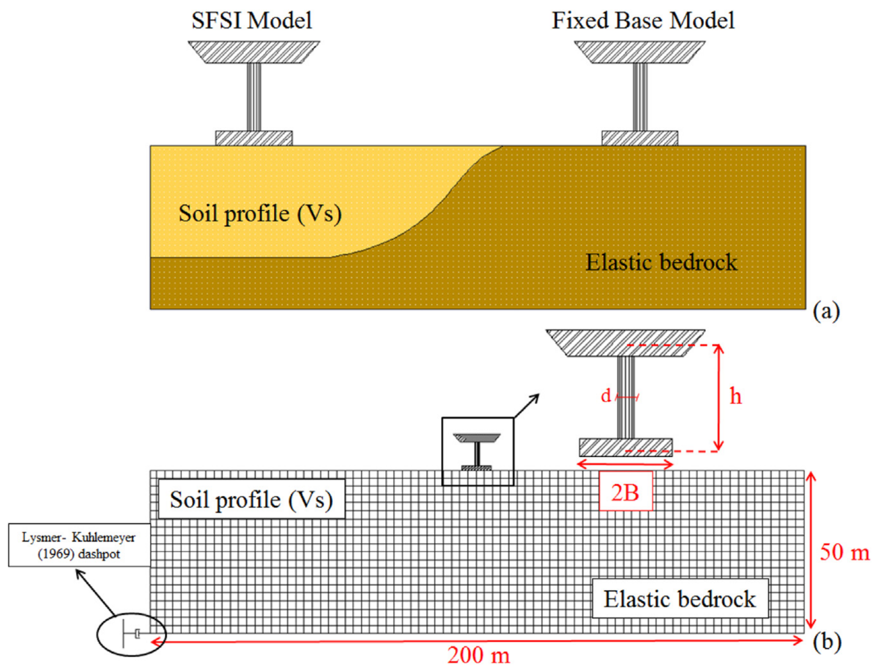

Fig. 1. Finite element modelling of the soil-foundation-structure-systems studied, (a) description of the fixed base and SSI models and (b) numerical SSI model adopted in the present study [10].

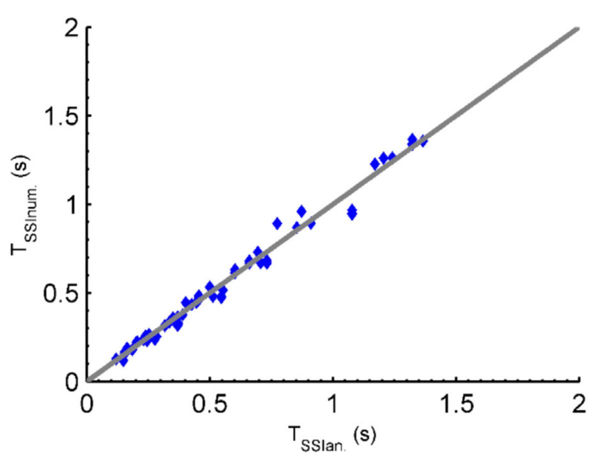

Fig. 2. Comparison between the FE-based ($T_{SSInum,}$) and analytical formulae of TSSI ($T_{SSIan.}$) for the selected SSI systems.

one adopted in the present paper.

Current design codes [17,4,5] treat the SSI using the sub-structure approach and appropriate foundation impedances [16,18,6], or via simplified discrete systems [14]. Therefore, they account for inertial or kinematic interaction separately and are unable to consider their combined effect. The present study aims at filling this gap by proposing a simple methodology for the evaluation of seismic acceleration demand for coupled soil-foundation-structure systems based on a comprehensive set of direct linear numerical analyses of coupled SFS systems subjected to earthquake motions at the base of the soil model (bedrock level). We propose appropriate reduction factors (RFs) to account for SSI in the seismic loading of soil-structure systems, by properly modifying the seismic loading of the simplest case of an SDOF structure which is fixed at its base and subjected to free field motion. The RFs describe the combined inertial and kinematic effects and can be very easily used in engineering practice for the estimation of seismic acceleration demand considering SSI effects in a single step.

2. Configuration and numerical modelling

To calculate the seismic acceleration demand of the studied SFS systems, we conducted 2D linear elastic time history analyses of coupled SFS systems using two-dimensional plane strain models in Opensees [15].

The superstructure is a single-degree-of-freedom structure (SDOF), the degree of freedom being the translational displacement of the structural mass, m_s. Single-degree-of-freedom structures are commonly used in SSI analyses because inertial interaction effects are most pronounced in the first mode [22]. The SDOF structure is characterized by its stiffness k_s, its mass m_s, its damping c_s and its height h. The structure is founded on a massless rigid surface foundation of width equal to 2B resting on the ground surface. Both the structure and the foundation are modelled with elastic beam column elements. A full connection is assumed between the foundation and the soil nodes. The entire superstructure's mass is lumped at the top of the superstructure without any contribution from the massless column. This SDOF can be interpreted as an equivalent representation of the fundamental mode of vibration of a multi-storey structure which is dominated by first-mode response or,

Table 1
Characteristics of the four distinct soil-foundation-structure systems [10].

V_s (m/s)	h (m)	2B (m)	m_s (mg)	T_{FIX} (s)	T_{SSI}/T_{FIX}	$h/T_{FIX} * V_s$	h/B
100/200/300/400	3	6	100/200/400/800	0.10–1.18	1.04–5.20	0.006–0.28	1
100/200/300/400	5	10	100/200/400/800	0.10–0.92	1.06–6.50	0.01–0.49	1
100/200/300/400	6	6	100/200/400/800	0.10–1.88	1.03–7.90	0.008–0.48	2
100/200/300/400	10	10	100/200/400/800	0.10–1.32	1.06–6.73	0.02–0.98	2

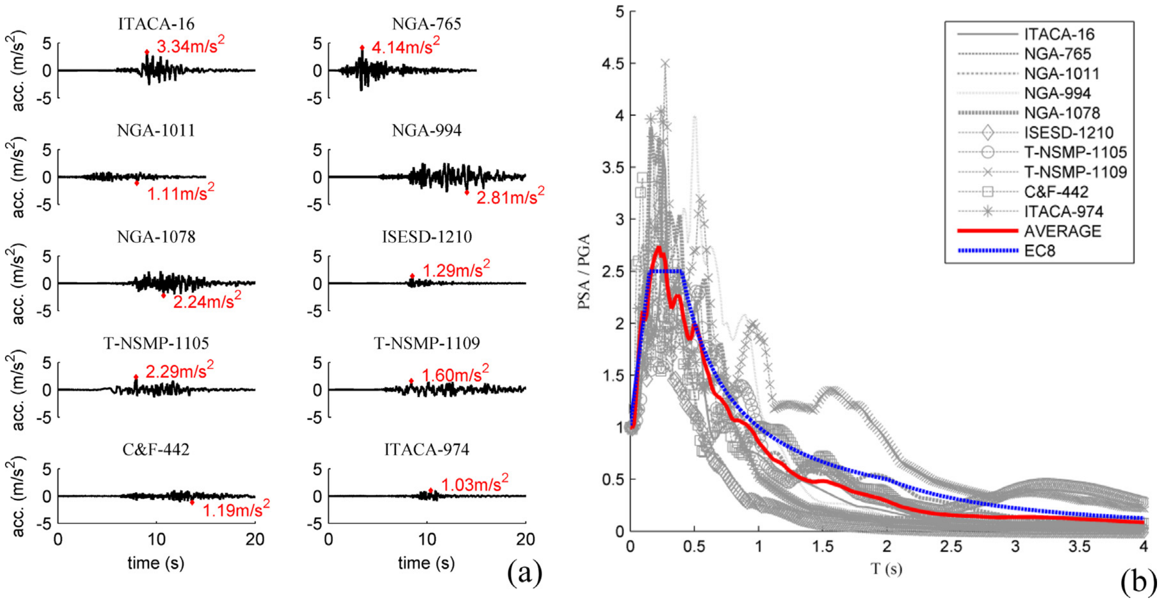

Fig. 3. (a) Time series and (b) comparison between average normalized elastic acceleration response spectrum of the ten earthquakes used in the parametric analyses with the EC8 normalized acceleration response spectrum (Soil Class A-Type A) [10].

Table 2
Earthquake records used in the parametric analyses [10].

No.	Location	Station	Epicentral_Dist. (km)	M_w	Rec. PGA (m/s^2)	$V_{s,30}$ (m/s)	Soil type (EC8)	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.

under specific assumptions, as a bridge system.

The modelling of soil profile was performed using the four-node plane strain formulation available in Opensees with appropriate bilinear isoparametric quadrilateral elements of two degrees-of-freedom at each node. The soil remains in the linear elastic range and the use of absorbing elements proposed by Zienkiewicz et al. [25] provides a relatively simple and easy to use in engineering practice way, to model the unbounded soil domain. Elastic bedrock is modelled using Lysmer-Kuhlemeyer [13] dashpots at the base of the soil. Fig. 1 shows a detailed view of the finite element modelling of the soil-foundation-structure-systems studied.

SSI varies based on the relative structure-to-soil stiffness ratio $h/(T_{FIX} \cdot V_s)$ and the aspect ratio h/B, where h = distance from the base to the centroid of the inertial mass, T_{FIX} is the resonant period of the fixed-base structure with $(2\pi/T_{FIX})^2 = m_s/k_s$, V_s = shear wave velocity of soil, and B = half-width of foundation of the structure. Of course, an equivalent linear, or nonlinear soil model would be expected to render a nonlinear relationship due to the introduction of added damping, and due to softening of the soil profile, but this is not the case of the present study where the soil behaves linearly.

The seismic acceleration demand of the structure is calculated directly from time-history analyses and is plotted on the same graph with the acceleration demand of the flexible-base structure subjected to free-field motion (maximum acceleration at the top of a fixed-base structure with period T_{SSI} equal to the period of the flexible-base structure, subjected to the free-field motion). The flexible-base period T_{SSI} was selected as the contributions of inertial interaction effects are more pronounced around this period. The proposed dimensionless reduction factors (RF) are the ratio of the acceleration demand of the coupled SFS system to the acceleration demand of the flexible-base structure resting on free-field.

A reduction factor that is not equal to one implies that the actual acceleration demand of the system differs from the one for free field conditions which is typically used in engineering practice. Considering that, for surface foundations on a homogeneous soil layer subjected to vertically propagating shear waves, kinematic interaction effects are not important [22], the proposed reduction factors capture and quantify in a simple way the effect of radiation and material damping in the system. Radiation damping has been a matter of intense research in SSI studies providing analytical formulas that allow the computation of this quantity for a particular SSI system [14]. The present study proposes an alternative methodology to the ones proposed in the literature for damping quantification. The damping quantification proposed herein results from an FE parametric study and is expressed in terms of soil to structure stiffness ratio $1/\sigma = h/(T_{FIX} \cdot V_s)$ (where h is the structure's height, T_{FIX} the fundamental period of fixed-base structure and Vs the shear wave velocity of the soil) and aspect ratio h/B (where h is the structure's height and B the half width of foundation).

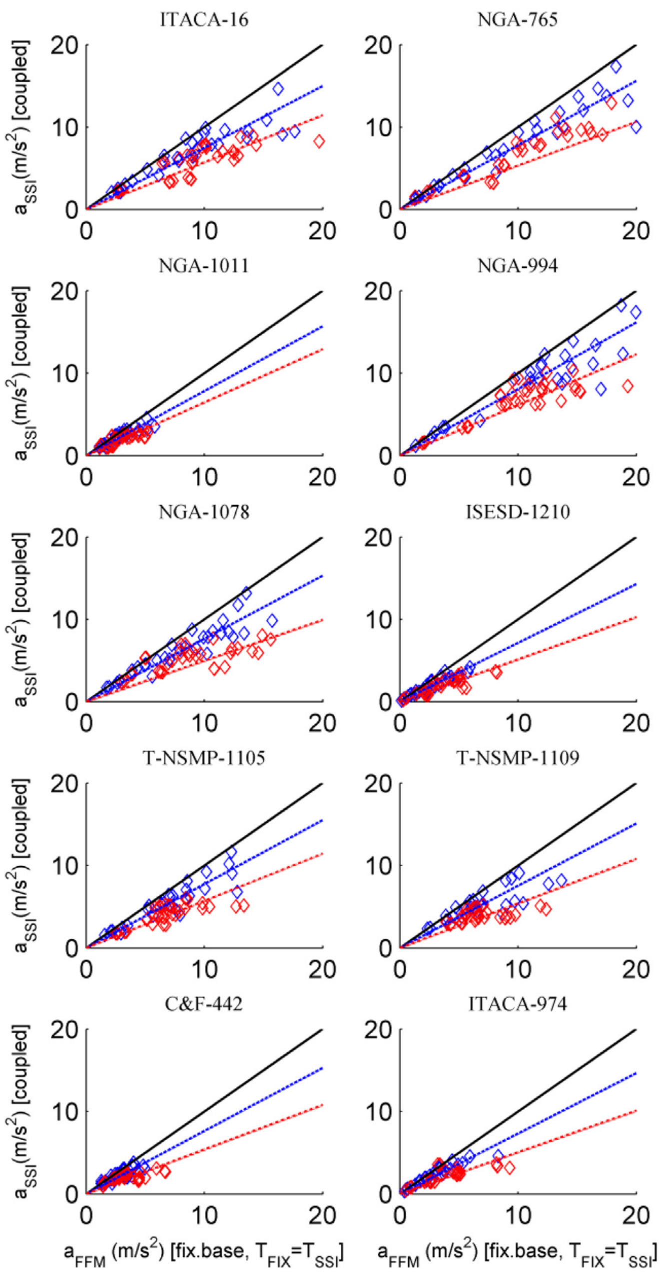

Fig. 4. Acceleration demand for the SSI system (a_{SSI}) and acceleration demand at the top of a fixed base system with fundamental period equal to T_{SSI} subjected to the acceleration time history in free field conditions (a_{FFM}) for $1/\sigma < 0.1$ (blue radial line) and $1/\sigma > 0.1$ (red radial line), for the ten studied records and for SDOF system of height equal to **3 m** and width of foundation equal to **6 m**. The black continuous line represents the 1:1 radial, while the blue and red radials resulted after regression analysis of the blue and red points, respectively.

3. Parametric analyses

Four distinct geometries of structures are chosen to highlight the effects of SSI on seismic acceleration demand. The mass of the superstructure m_s is 100 mg, 200 mg, 400 mg, 800 mg and selected as described below. According to Stewart et al. [23], for ordinary structures, normalized mass ($m_n = m/\rho r^2 h$) varies between 0.4 and 0.6, resulting (for the examined foundation geometries and soil properties) to masses between 40 mg and 300 mg. In this range we selected two masses, 100 mg and 200 mg. For the bridges, the unit weight of the deck is taken equal to 200 kN/m, which is a realistic value according to Ciampoli and Pinto [3]. In these parametric analyses, we selected bridge spans equal to 20 m and 40 m which result in masses 400 mg and 800 mg, respectively and simulate the concentrated mass of two adjacent half spans of the bridge deck considering a massless pier. In this study, the mass and stiffness of the 2D SDOF are considered to be equal to the actual 3D values.

The modulus of elasticity is E = 32 GPa for the structural elements, while the cross-section diameter d of the idealized SFOF circular column ranges from 0.6 m to 3.0 m. The structure's height is 3 m, 5 m, 6 m, and 10 m. For the 3 m- and the 6 m-tall structures the footing width is 6 m, while for the 5 m and the 10 m tall structures the footing is 10 m wide. The foundation width was chosen to allow for a safety factor against bearing capacity well above 1, according to Eurocode 7 [1]. For the structure, the Rayleigh damping is equal to 5% at frequencies f_{SSI} and f_{fix}. Relative structure-to-soil stiffness varies between 0.01 and 0.98 and aspect ratio h/B varies between 1 and 2.

The soil profile is simplified to a homogeneous soil layer over elastic bedrock, with mass density $\rho = 2\,\mathrm{mg/m^3}$ and Poisson's ratio $\nu = 1/3$. The elastic bedrock ($V_{s,bedrock}$ = 1500 m/s) lies at 50 m beneath the ground surface. We assumed shear wave velocity V_s of the homogeneous soil layer equal to 400 m/s, 300 m/s, 200 m/s and 100 m/s, classifying the soil profile in soil class B, C and D according to Eurocode 8 [2]. For the soil material, we used Rayleigh damping equal to 4% at two target frequencies, i.e. the first and the third resonant mode of the soil profile [12]. Considering that the soil material damping is kept constant for the target frequencies in all analyses independently of the earthquake motion characteristics or the reference depth in the soil, the main source of modification on the accelerations is related to the radiation type of damping.

The fundamental period of the soil profiles varies from 0.5 s to 2.0 s, whereas the resonant period T_{SSI} of the flexible-base system is in the range of 0.12–2.7 s, spreading over a wide range of civil engineering applications. T_{SSI} is calculated directly from the numerical analyses, from the ratio of the Fourier spectra of the response at the top of the structure and at the free-field. Comparison between the FE-based and analytical formulae of T_{SSI} that have been proposed in the literature [e.g. [20], Eq. (1)] for the selected SSI systems showed that the periods with the two approaches (analytical and numerical) are directly comparable (Fig. 2). Details on the system properties are given in Table 1.

$$T_{SSI} = T_{FIX}\sqrt{1+\frac{k_{str}}{kx}\left(1+\frac{k_x h^2}{k_\theta}\right)} \tag{1}$$

Where k_{str}, h and T_{FIX} denote the stiffness, height and fixed-base natural period of the structure, respectively.

From all possible soil-foundation-structure configurations, we retained those where the fixed-base period T_{FIX} was greater than 0.1 s in order for the systems to be realistic for civil engineer practice, as well as the combinations where the safety factor against bearing capacity under earthquake loading, according to Eurocode 8 [2] was greater than unity. The soil strength values considered for the bearing capacity evaluation stem from Eurocode 8 – Part 1 [2] and depend on the soil classification scheme determined by shear wave velocity. Static safety factor against bearing capacity was evaluated according to Eurocode 7 [1] and was kept well above one for all cases. Besides, evaluation of the soil bearing capacity revealed that it was impossible to choose a realistic surface foundation for aspect ratio larger than 2.

4. Earthquake records

The soil-foundation-structure system is subjected to ten earthquakes records covering a significant range in magnitudes (M_w = 5.6–7.6), epicentral distances (R = 3.4–43 km) and predominant periods (T_p = 0.10–0.50 s). The predominant period of each earthquake record is calculated according to Rathje et al. [19] by $T_p = T(max(PSA))$, as the period where the peak spectral acceleration of the recording at outcropping bedrock is maximized.

Table 3
Reduction factors for the selected earthquake records and the geometries where aspect ratio h/B is equal to unity, categorized according to the 1/σ ratio. In red the average (AVE) values are depicted, while the standard deviation is noted with SDEV.

	h3, h/B=1		**h5, h/B=1**	
Record name	1/σ<0.1	1/σ>=0.1	1/σ<0.1	1/σ>=0.1
ITACA_16	0.749	0.571	0.753	0.56
NGA_765	0.78	0.532	0.829	0.518
NGA_1011	0.785	0.645	0.818	0.632
NGA_994	0.809	0.614	0.748	0.609
NGA_1078	0.765	0.496	0.753	0.507
ISESD_1210	0.714	0.512	0.751	0.518
T-NSMP_1105	0.775	0.571	0.756	0.565
T-NSMP_1109	0.755	0.54	0.741	0.54
C&F_442	0.764	0.539	0.774	0.551
ITACA_974	0.733	0.504	0.752	0.51
AVE	0.7629	0.5524	0.7675	0.551
SDEV	0.027	0.048	0.031	0.042
AVE+SDEV	0.790	0.601	0.798	0.593
AVE+SDEV	0.736	0.504	0.737	0.509

Table 4
Reduction factors for the selected earthquake records and the geometries where aspect ratio h/B is equal to two, categorized according to the 1/σ ratio. In red the average (AVE) values are depicted, while the standard deviation is noted with SDEV.

	h6, h/B=2		**h10, h/B=2**	
Record name	1/σ<0.1	1/σ>=0.1	1/σ<0.1	1/σ>=0.1
ITACA_16	0.792	0.702	0.818	0.764
NGA_765	0.867	0.639	0.894	0.699
NGA_1011	0.855	0.74	0.849	0.795
NGA_994	0.799	0.728	0.839	0.789
NGA_1078	0.787	0.622	0.845	0.677
ISESD_1210	0.824	0.655	0.852	0.692
T-NSMP_1105	0.786	0.67	0.84	0.748
T-NSMP_1109	0.758	0.601	0.811	0.653
C&F_442	0.813	0.605	0.849	0.672
ITACA_974	0.856	0.654	0.9	0.732
AVE	0.8137	0.6616	0.8497	0.7221
SDEV	0.036	0.049	0.028	0.051
AVE+SDEV	0.850	0.710	0.878	0.773
AVE+SDEV	0.778	0.613	0.821	0.671

All selected earthquakes were recorded on a rock or very stiff sites with $V_{s,30}$ larger than 600 m/s, specified as types A and B according to Eurocode 8 soil classification scheme [2]. No scaling was applied, whereas a second order Butterworth band-pass filter with corner frequencies 0.25 Hz and 25 Hz was used. The selected earthquake records were chosen to have relatively low peak accelerations values varying from 1.03 m/s^2 to 4.14 m/s^2. The average elastic acceleration response spectrum of the ten selected records is compatible with the elastic response spectrum proposed by Eurocode 8 [2] soil classification scheme for soil class A and earthquake Type A (Fig. 3). Therefore the average values reported herein for acceleration demands can be generalized and the results are statistically meaningful. Details on the earthquake records are given in Table 2, while the time series are plotted in Fig. 3.

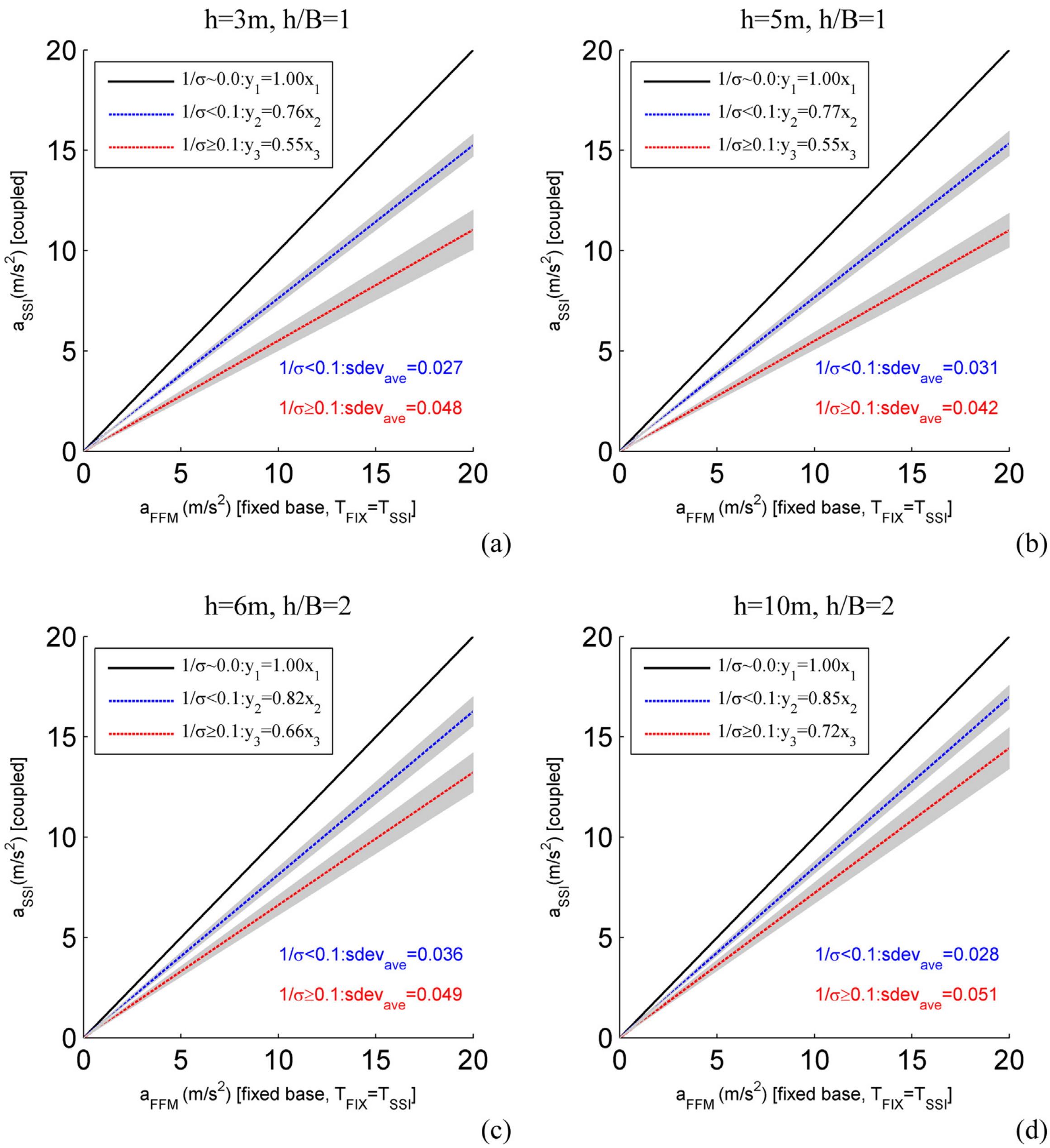

Fig. 5. Reduction factors for $1/\sigma < 0.1$ (blue radial line) and $1/\sigma \geq 0.1$ (red radial line), for aspect ratio being (a) h = 3 m and 2B = 6 m (b) h = 5 m and 2B = 10 m (c) h = 6 m and 2B = 6 m and (d) h = 10 m and 2B = 10 m. The black continuous line is the 1:1 line and in grey is the range between the average line plus and minus the standard deviation.

5. Reduction factors to estimate acceleration demand

The proposed reduction factors (RFs) are produced as the ratio of the acceleration demand of fully-coupled SFS system (maximum acceleration at the top of the soil-foundation-structure system subjected to earthquake loading at the bedrock) to the acceleration demand of the flexible-base structure on free-field (maximum acceleration at the top of a structure fixed at its base, with period T_{SSI}, subjected to the free-field motion) (Eq. (2)). The RFs describe the combined inertial and kinematic effects and can be very easily used in engineering practice for the estimation of seismic acceleration demand considering SSI effects in a single step.

$$RF = \frac{a_{SSI}}{a_{FFM}} \tag{2}$$

For the system under investigation, one needs only to calculate T_{SSI} and the acceleration demand spectrum for the free-field motion. The period of the flexibly supported system T_{SSI} can be calculated from the proposed in literature closed form solutions [e.g. [20]] and the acceleration demand spectrum for free- field motion (FFM) from the available in engineering practice guidelines and codes [e.g. [2]]. Then, the acceleration demand for FFM for the T_{SSI} period can be easily evaluated. Multiplying the acceleration demand for FFM with the proposed reduction factor, the actual acceleration demand for the coupled SFS system is calculated.

The following figures and tables present the acceleration demand (maximum absolute accelerations) of the linear visco-elastic time history analyses of the studied soil-foundation-structure systems. More specifically, in Fig. 4 the acceleration demand for free-field (a_{FFM}) and for SFS systems (a_{SSI}) are plotted on the same graph for all ten selected earthquake records. Each of the ten plots in Fig. 4 refers to one earthquake record. Each point on Fig. 4 presents the acceleration demand for free-field and for the SFS system for a single system, with the blue points being the cases where $1/\sigma$ is lower than 0.1 and the red points

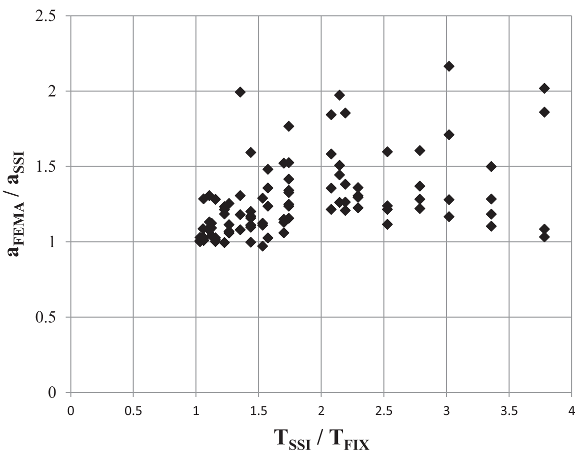

Fig. 6. Comparison of acceleration demands that result with FEMA440, 2005 (a_{FEMA}) with the proposed methodology (a_{SSI}) in terms of T_{SSI}/T_{FIX} ratio.

the cases where $1/\sigma$ is greater than or equal to 0.1. The blue and red radials result after regression of the blue and red points, respectively. The slope of these radials is the proposed RF. Fig. 4 shows one of the four studied SDOF systems, which is 3 m high and its foundation width is 6 m. Similar results are produced also for the other three soil-structure systems (see Table 1) for which we present here the proposed RFs in Tables 3, 4. More specifically, Tables 3, 4 present the RF for aspect ratio h/B equal to 1 and 2 respectively for each earthquake record, the average RF for all earthquake records, as well as the RF plus and minus one standard deviation. The black continuous line is the 1:1 radial line and in grey shadow is shown the range between the average line plus and minus the standard deviation (Fig. 5).

Fig. 4 clearly shows that independently of the earthquake record and the soil profile's characteristics, the acceleration demand for the coupled SFS system is reduced compared to the acceleration demand for free-field (both spectral accelerations refer to the T_{SSI} period). In Karatzetzou and Pitilakis [10], it was shown that the average effective foundation motion reduces from the free-field motion because of SSI. This highlighted the significance of damping, as for surface foundations inertial interaction defines the response. If larger foundation types were considered (i.e. piles, raft foundations etc.), the response would be more prone to kinematic interaction [14].

Fig. 4 shows the average acceleration RF for all ten earthquakes and the average plus and minus one standard deviation. It shows that for aspect ratio $h/B = 1$ and $1/\sigma < 0.1$, the average reduction factor is 0.76 for the SDOF system of 3 m height and 6 m foundation width and 0.77 for the 5 m height and 10 m foundation width system, and thus their difference is about 1.5%. Similarly, for structure-to-soil stiffness ratio $1/\sigma > 0.1$, the proposed RF is equal to 0.55 for both h = 3 m and h = 5 m structures. When the aspect ratio is equal to 2, the proposed reduction factor for $1/\sigma < 0.1$ is equal to 0.82 for the 6 m tall pier and equal to 0.85 for the 10 m tall pier (3.7% difference), while for the soil profiles with $1/\sigma > 0.1$, the average reduction factor is 0.66 and 0.72 for the 6 m and 10 m high column respectively (9.1% difference). These results are of great interest, as it seems that the acceleration demand for the actual SFS system, can be a percentage of about 55–85% of the response in case we consider the seismic demand for a flexible-base structure subjected to the free-field motion.

In other words, the results highlight and quantify the damping effect on the acceleration demand. For all examined cases when the structure-to-soil stiffness ratio $1/\sigma$ is equal or greater than 0.1, reduction of the resulting RF from 1 is more intense. Irrespectively the SDOF system's geometry, the reduction factor is more or less similar for same aspect ratios. An important conclusion is that earthquake record characteristics have a minor effect on the acceleration RF (Fig. 4, Tables 3, 4). For soil profiles with $1/\sigma \geq 0.1$ the correct estimation of damping is crucial (the RF for $1/\sigma \geq 0.1$ varies between 0.55 and 0.72), while on the other hand, for the same $1/\sigma$ and aspect ratio, the height of the structure h, as well as the frequency content of the earthquake record, are not that important. The aspect ratio characterizing short squatty or tall slender structures is related to the contribution of the translational or the rocking component of the foundation motion to the overall response which in turn are associated to different levels of SSI damping as presented in Tables 3, 4.

6. Comparison of acceleration demands with FEMA440 methodology

A comparison of acceleration demands resulting from the present numerical SSI analyses with the accelerations resulting by FEMA440 [4] methodology a_{FEMA}/a_{SSI} is shown in Fig. 6 in terms of T_{SSI}/T_{FIX} ratio. Fig. 6 shows the comparison between the two approaches for all examined SFS systems (256 in total) and one out of the ten earthquakes (ITACA-16) for T_{SSI}/T_{FIX} ratios up to 4. When T_{SSI}/T_{FIX} ratio is lower than 1.3, a_{FEMA}/a_{SSI} ratios are between 1 and 1.3. When T_{SSI}/T_{FIX} ratio is greater than 1.3, FEMA 440 [4] method seems to be conservative (resulting in higher acceleration demands with a_{FEMA}/a_{SSI} ratios up to 2.2). FEMA 440 confirms this conclusion and proposes that the equations for including SSI effects in accelerations evaluation are most applicable for low T_{SSI}/T_{FIX} ratios and that they generally provide low damping estimates for high T_{SSI}/T_{FIX} ratios.

7. Application to a bridge pier

In order to investigate the accuracy, efficiency, and the practicality of the proposed methodology, we applied the proposed RF to a typical bridge structure. The selected bridge structure is quite common in modern motorway construction in Europe (for example T7 Overpass, Greece, [9]). The three-span bridge has a total length equal to 99 m (Fig. 7). The two piers have a cylindrical cross section (section diameter d = 2 m), a common choice for bridges in Europe, while both pier heights are considered equal to 6 m (clear column height). The deck is monolithically connected to the two piers. The selected bridge has a square surface foundation and it rests on a soil having shear wave velocity equal to 300 m/s (soil class C according to EC8 soil classification scheme, [2]). We consider that the selected bridge structure is subjected to ITACA_16 input motion at bedrock level (Table 2).

The first step of the proposed methodology is the calculation of the dynamic and geometrical characteristics of the soil-bridge system, notably the pier height h (h = 6 m), mass m (m = 1030 mg at the top of each pier), soil profile's shear wave velocity V_s (V_s = 300 m/s) and foundation's width 2B (2B = 6 m). Secondly, we calculate structure-to-soil stiffness ratio $1/\sigma$, which for the examined case is equal to 0.06, the fixed-base fundamental period of the piers T_{FIX} (T_{FIX} = 0.345 s), the aspect ratio h/B (h/B = 2) and the effective period value T_{SSI} (T_{SSI} = 0.81 s) from the proposed in [20]] closed form solution. Afterwards, we evaluate the soil response (when the soil profile is subjected to ITACA_16 motion), at free-field conditions (FFM) in terms of acceleration time series (Fig. 7). Next, acceleration demand for free-field a_{FFM} for the T_{SSI} period (a_{FFM} = 10.85 m/s^2) is calculated. The proposed reduction factor RF (average value) of acceleration demand for free-field to evaluate acceleration demand for the SFS system for $1/\sigma < 0.1$ and h/B = 2 is considered equal to 0.82 (Fig. 5). Therefore, the average acceleration demand for the soil-foundation-bridge system is $a_{SSI} = a_{FFM} * RF = 8.9$ m/s^2.

The traditionally evaluated acceleration demand is equal to 10.85 m/s^2, while the actual acceleration demand considering SSI is equal to 8.9 m/s^2 (± 0.4 m/s^2). This means that for this examined case the actual acceleration demand is on average 22% lower compared to the traditionally evaluated acceleration. For softer soil profiles the reduction of acceleration loading for coupled SFS system compared to the demand for free-field is even larger.

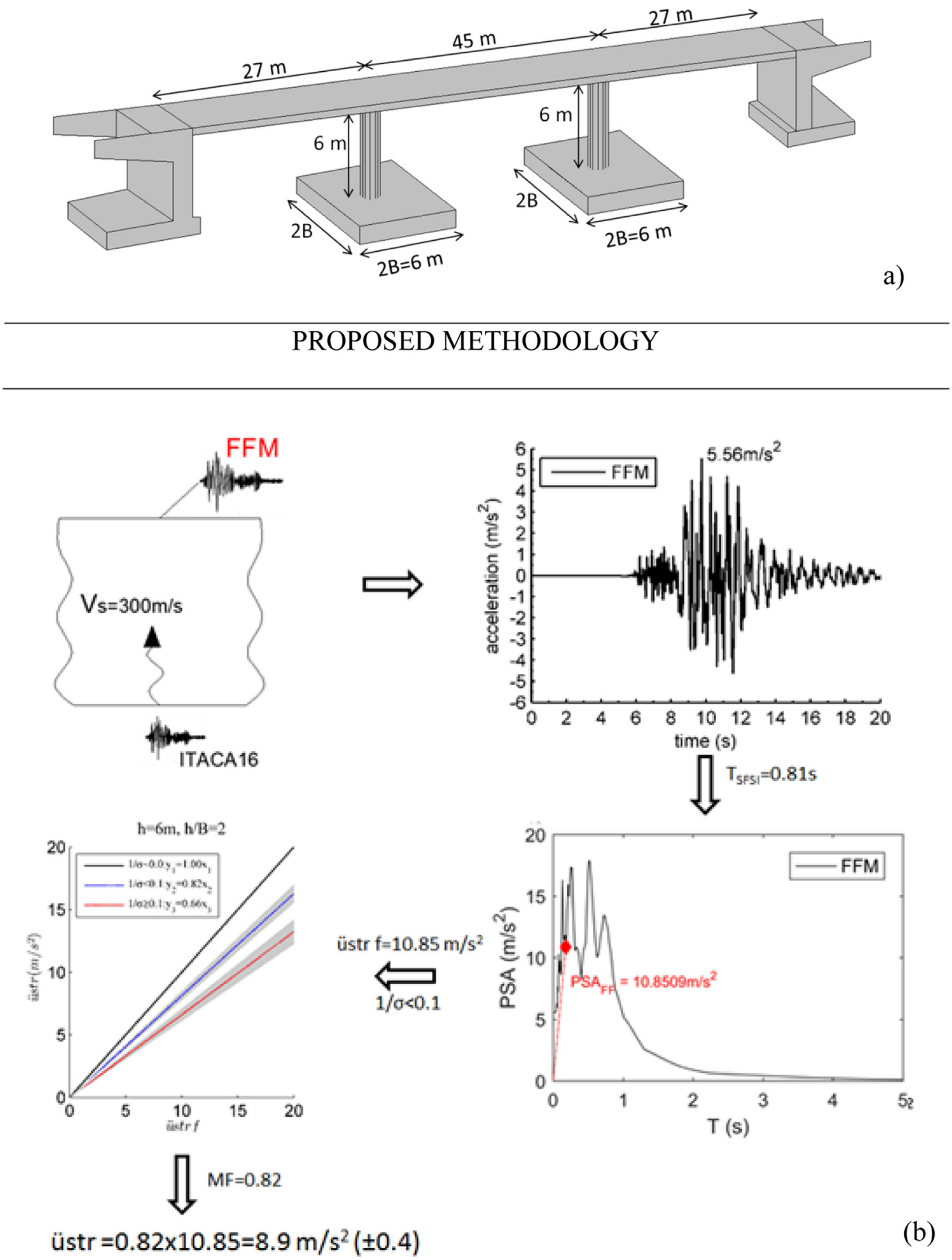

Fig. 7. Application of the proposed methodology. The steps of the proposed methodology from the evaluation of the free-field (FF) response to the evaluation of the actual acceleration demand for SFS system.

The advantages of the proposed methodology compared to the existing methodologies that consider interaction effects on structure's acceleration demand [4,17] are:

(i) The proposed RFs concern the response at the top of the structure. For the selected SFS systems and under the assumptions of the present study, the response at the top of the structure is reduced due to SSI effects. Similarly, for the same SFS systems, it has been shown that the maximum acceleration at the foundation is also reduced (in average) compared to the free-field motion [10]. This foundation motion, which includes both inertial and kinematic interaction effects, is the most appropriate index to express SSI effects in seismic input [8].

(ii) FEMA 440 [4] neglects the influence of inertial interaction effect on the foundation motions, as these effects are concentrated at narrow frequency range around the first mode frequency and do not affect the higher modes. Nevertheless, the first mode frequency of the soil-foundation-structure system, is the most important as for this frequency according to the codes we usually evaluate the inertial forces and thus for this frequency usually engineers design all structures. Utilizing the proposed RFs of acceleration demand for free-field at the effective period, the effect of inertial interaction on seismic loading are taken into consideration. Finally, this is the first time the effect of damping on acceleration demand is highlighted and quantitative results are presented, considering the fact that the proposed RFs concern surface foundations and, thus, they depict mainly the effect of damping at the period of the flexibly supported system.

8. Conclusions

The main goal of the present study was to investigate the seismic acceleration demand of reinforced concrete structures, accounting for soil-foundation-structure interaction and to propose a simple way to account these effects in practice. An efficient methodology for the acceleration demand evaluation of reinforced concrete structures is proposed and applied to a bridge structure. Reinforced concrete structures are represented by single degree of freedom (SDOF) systems, a common choice for structures with prevailing first mode of vibration. All studied structures are founded on rigid surface foundations and lay on well-defined soil conditions from the surface to the elastic half-space, while they are subjected to vertically propagating earthquake shear waves. The coupled soil-foundation-structure interaction system is analyzed numerically in a single step. The discussion is focused on the consideration of both inertial and kinematic interaction on the acceleration demand evaluation, using appropriate reduction factors resulting from the comparison between the accelerations for the coupled SFS system and the free-field. The proposed reduction factors clearly show the beneficial effect of SSI on seismic acceleration demand for the period of the flexibly supported system. The required parameters for the application of the practical and easy-to-use for earthquake engineering problems methodology are the geometrical properties of the structure, the dynamic characteristics of the soil-structure system and the earthquake input motion. With these parameters known, one can easily calculate acceleration demands considering SSI. For a structure founded on a surface foundation resting on an elastic homogeneous soil layer subjected to vertically propagating shear waves, the main conclusions that stem from the investigation of SSI effects on the selected RC structures soil-foundation-structure-systems are the following:

- From the numerous time history analyses conducted in the present study for both squatty and slender structures, it seems that the acceleration demand for the period of the flexible supported system decreases highlighting the beneficial effect of the increased damping due to SSI. This reduction is affected mainly by the soil conditions (the reduction factor is lower when $1/\sigma$ is greater than or equal to 0.1), whereas the effects of earthquake frequency content and foundation-structure slenderness ratio are not very important.
- The proposed reduction factors show that maximum acceleration at the top of the actual SFS system is about 55–85% of the response in case we consider the seismic acceleration for free-field (for the same T_{SSI} period).
- Earthquake record characteristics have a minor effect on the acceleration reduction factor and the scatter is not important.
- It seems that short structures on soft soils exhibit larger damping than high structures on stiff soils during an earthquake since squatty structures are dominated by the translational component of base motion that is related to higher levels of damping with respect to the rocking motion which in turn dominates the response of slender high-rise structures.
- The proposed reduction factors are an alternative to the currently available methodology with the impedance functions to account for inertial interaction effects. The impedance functions are complex valued and frequency dependent On the other hand, the proposed reduction factors are proposed at a single frequency, the predominant frequency of the soil-structure system and can be very easily calculated.
- The presented quantitative evaluation of damping stems from finite element time history analyses of the coupled SFS system.
- With increasing h/B except for the increase in period lengthening, the results also indicate a decrease in foundation damping factor, as the reduction factors are greater for larger h/B ratio. The same conclusion exists also in Stewart et al. [22], but in the present paper quantitative and ready for use in engineering practice results are shown.
- Inertial interaction effects were generally observed to be small for $1/\sigma < 0.1$ and for practical purposes could be neglected in such cases.
- Considering that the present study refers to surface foundations on a homogeneous soil layer over elastic bedrock subjected to vertically propagating shear waves and, thus, the kinematic interaction effects are not important the proposed reduction factors capture in a simple way the effect of total damping (viscous damping in the structure as well as radiation and hysteretic damping in the foundation) generated in the system on the acceleration demand. The herein proposed reduction factors can be very easily used in engineering practice overcoming the existing difficulties (an iterative process is needed) when utilizing the proposed in literature closed form solutions [e.g. [17]].

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