

Shear Wave Velocity as a Key Parameter that Steers Seismic Structural Design

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***Abstract.** Seismic design of safe and economically justified structures is a multidisciplinary and challenging task. Due to complexity in modelling and high computational cost, soil is usually modelled using discrete impedance functions. On the other end, many code-based methods for design of earthquake resistant structures introduce the response spectrum (RS). Albeit both impedance and RS functions may govern design of e.g. nuclear facilities or hospital buildings, they seldom recognize stress induced from structure to foundation soil. This paper shows how this stress change the shear wave velocity distribution within foundation soil, a major soil property that steers the selection and shape of the mentioned functions and thus redirect structural design. A study was conducted: on a set of real soil profiles collected by the authors; for both low and high structural loading and by using different methods for correction of free-field measurements of shear wave velocity within a soil profile to account for the overburden pressure.*

1 Motivation

Seismic design of safe and economically justified structures is a multidisciplinary and challenging task. Most of its complexity lies in soil, an infinite medium that provides support for structures but also hazard entrenched in ground motions. Due to the complexity in modeling and high computational cost, soil is usually modeled using

discrete impedance functions [1]. On the other end, today many different code-based methods for the design of earthquake resistant structures exist [1]: equivalent lateral force method, modal response spectrum analysis and nonlinear static pushover method, among others. All the mentioned methods introduce response spectrum (RS) and are widely used in structural design.

Albeit both impedance and RS functions may govern design of important structures (e.g. nuclear facilities or hospitals) they rarely recognize stress induced from structure to foundation soil. Indeed, this stress is of major importance here as it may change the shear wave velocity distribution within foundation soil, a major soil property that steers the both selection of RS and shape of impedance functions.

Within the seismic design of structures foundation soil is generally described by the average shear wave velocity in the upper 30 m of the soil profile with free-field conditions $v_{s,30}$ (e.g. [1] – [9]). In this light, soils classified within norms (e.g. [3], [6], [10], [11]) are defined according to the distribution of shear waves within the first 30 m of the foundation soil profile since this depth represents a typical drilling depth for the purposes of sampling and determination of soil characteristics (e.g. [7], [12], [13]). The first 30 m depth description of soil conditions has been defined by Borchardt in 1994 [5]. Nevertheless, it is clear that the parameter $v_{s,30}$ is one of the key parameters in code based design that govern earthquake loading on structures.

Engineering practice often assumes that foundation soils within a coded soil type respond similarly to a particular earthquake. On the other end, it is well known that even soils with the same value of shear wave velocity within the upper 30 m do not always have the same fundamental period of vibration [1], as it is a function of deeper soil layers [13], [14]. This is important to bear in mind as the fundamental period of vibration of soil may be a strong indicator of the predominant earthquake period, and thus strong indicator of the frequency content of an earthquake (e.g. [15] – [17]). Moreover, it is also clear that building activities may alter soil conditions, which in turn may lead to input motions that differ from the design motions used in structural analysis [18]. Structural analyses are mainly conducted using the earthquake records obtained in free-field conditions.

This paper shows how the presence of additional weight from a structure may influence shear wave velocity distribution within a foundation soil profile and thus redirect structural design. A study was conducted: on a set of real soil profiles collected by the authors; for both low and high structural loading and by using different methods for correction of shear wave velocity profiles to account for the overburden pressure.

2 Calculation of average shear wave velocity of a soil profile

Average shear wave velocity within the upper 30 m of a soil profile may be determined by the following expression (e.g. [3], [7], [9], [11]):

$$v_{s,30} = \frac{30}{\sum_{i=1,N} \frac{h_i}{v_{s,i}}} \quad (1)$$

where h_i is thickness of the i -th layer of a deposit, $v_{s,i}$ shear wave velocity at a shear strain level of 10^{-5} or less of the i -th layer of a deposit, in a total of N layers within the

upper 30 m of a deposit. Also, in literature (e.g. [5], [7]) the following expression for the estimation of the average shear wave velocity in seismically active regions may be found:

$$v_{s,30} = \frac{\sum_{i=1,N} h_i}{\sum_{i=1,N} \frac{h_i}{v_{s,i}}} \quad (2)$$

It is clear that the parameter $v_{s,30}$ significantly lacks information when compared to the whole shear wave velocity profile. This is also stressed by other researchers (e.g. [2], [19], [20]). In this light, recent studies (e.g. [7], [19]) have shown that the shear wave velocities can considerably differ with the depth of a profile, even when the values of $v_{s,30}$ are similar. Moreover, these studies suggest that the velocity profiles cannot be sufficiently described if only upper 30 m of the foundation soil are observed, but also that the parameter $v_{s,30}$ is insufficient to describe the soil response. Recent studies suggested (e.g. [9], [19]) that foundation soils would be better described if profiles up to depths where shear wave velocity reach 800 m/s are known. But such profiles may reach great depths. Obviously, it is always preferable to use the entire shear wave velocity profile in analyses, yet this is often impossible due to economical reasons.

3 Effect of vertical pressure from structure on shear wave velocity distribution in soil

American guidelines for design of earthquake resistant structures [22] stress that the classification of soil types with regard to the shear wave velocity distribution within the upper 30 m of deposit is justified for analysis of shallow founded structures. Additionally, National Institute of Standards and Technology (NIST) [18] recommends that the shear wave velocity should be calculated for conditions when the soil is loaded by a structure, using the following expression:

$$v_{s,F} \approx v_s(z) \cdot \left(\frac{\sigma'_v(z) + \Delta\sigma'_v(z)}{\sigma'_v(z)} \right)^{n/2} \quad (3)$$

where $v_s(z)$ is shear wave velocity in the free-field at depth z , $\sigma'_v(z)$ effective stress from the soil self-weight at the depth z , $\Delta\sigma'_v(z)$ increment of vertical stress due to weight of the structure at the depth z , n coefficient that varies from approximately 0.5 for granular soils to 1.0 for cohesive soils. Additional vertical stress in the soil, due to weight of the structure, has the greatest influence on the distribution of the shear wave velocity at depths that corresponds 50 to 100 % of the foundation width (Figure 1). This is also confirmed by others (e.g. [1], [18], [23], [24]).

Furthermore, NIST suggests that the average shear wave velocity for the soil profile under a structure should be calculated using the following expression:

$$v_s = \frac{h_{s,eff}}{\sum_{i=1,N} \frac{h_i}{v_{s,F,i}}} \quad (4)$$

where $h_{s,eff}$ is effective depth of the soil profile affected by weight of the structure, $h_{i,eff}$ thickness of the i -th layer within the effective depth of the soil, $v_{s,F,i}$ effective value of a shear wave velocity for the i -th layer of the soil under the structure. This approach is assumed to be valid for structures with rigid foundations [18].

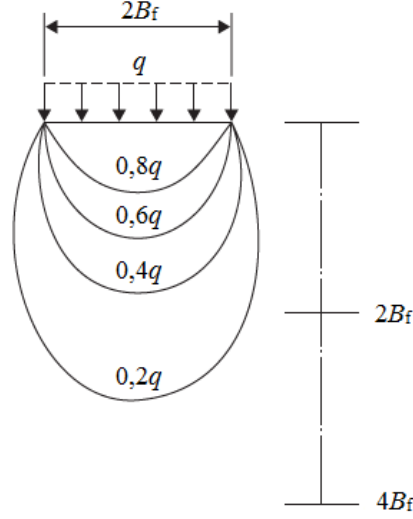


Figure 1: Stress bulb in soil under the foundation [24] (edited by the authors)

The effective vertical stress in the soil due to self-weight may be estimated using the following expression [24]:

$$\sigma'_v(z) = (\rho - \rho_w) \cdot g \cdot z \quad (5)$$

where ρ is soil mass density, ρ_w water mass density, g gravitational acceleration, z observed depth in soil profile. In the case of dry soils, water density in expression (5) should be ignored.

3.1 Boussinesq method

When the foundation soil is loaded with rectangular or square foundation, additional vertical stress in the soil profile under the middle of the foundation may be estimated using the Boussinesq solution for distribution of stresses, using the following expression [25]:

$$\Delta\sigma'_v(z) = \frac{2q}{\pi} \left(\frac{m \cdot n}{\sqrt{1+m^2+n^2}} \cdot \frac{1+m^2+2n^2}{(1+n^2) \cdot (m^2+n^2)} + \sin^{-1} \left(\frac{m}{\sqrt{m^2+n^2} \cdot \sqrt{1+n^2}} \right) \right) \quad (6)$$

where q is uniform vertical load per unit area, and m and n are parameters that take into account the foundation geometry and observed depth in the foundation soil. The two parameters can be calculated by using following expressions:

$$m = \frac{L_f}{B_f} \quad (7)$$

$$n = \frac{z}{B_f} \quad (8)$$

where L_f and B_f are half-length and half-width of the foundation respectively, z observed depth in the foundation soil, measured from the ground surface. For practical reasons, the calculation of additional stresses within the soil, using the expression (6) will be referred to as *them-n method* further in this paper.

3.2 2:1 method

Besides the above described Boussinesq method, the so-called 2:1 method is also widespread in engineering practice (see [18], [25], [26]), where the subsurface distribution of the stress is illustrated as shown in Figure 2.

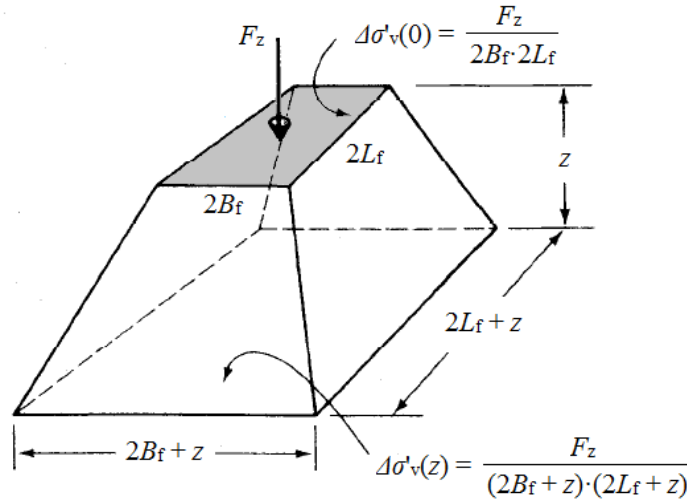


Figure 2: Approximate distribution of a vertical stress under the square foundation, according to the 2:1 method [26] (edited by the authors)

According to the 2:1 method, the stress at a certain depth below the foundation may be determined using the following expression [26]:

$$\Delta\sigma'_v(z) = \frac{F_z}{(2B_f + z) \cdot (2L_f + z)} \quad (9)$$

where F_z is point load on the foundation. A recently published reference manual [18], which provides detailed insight into soil-structure interaction (SSI), allows the application of the 2:1 method in evaluation of seismic response of structures founded on soft soils.

4 Study environment: selected soil profiles and structures

To demonstrate how the vertical loading from a structure affects the shear wave velocity distribution in soil the two above described methods are applied on ten real, randomly selected and well explored soil profiles from Croatia, Romania, Montenegro and Greece. A reference list for the used soil profiles is given in Table 1.

Table 1: Description and references for soil profiles observed in this study

Profile No.	City	Country	Source
1	Bar	Montenegro	[31]
2	Bucharest	Romania	[32]
3	Lefkada	Greece	[32]
4	Osijek	Croatia	[1]
5	Osijek	Croatia	[1]
6	Ploče	Croatia	[1]
7	Sirova Katalena	Croatia	[1]
8	Sisak	Croatia	[1]
9	Thessaloniki	Greece	[32]
10	Ulcinj	Montenegro	[31]

Every soil profile noted in Table 1 has been associated with a corresponding soil class as defined in Eurocode [3]. For the observed soil profiles it is assumed that the water table is very deep. Following the definition provided in Eurocode [3], soil class A include profiles whose average shear wave velocity exceeds 800 m/s, while the soil class B is characterised by the average shear wave velocities that range from 360 to 800 m/s. Soil class C includes profiles with average shear velocities between 180 and 360 m/s, while the upper limit of average shear velocity for soil class D corresponds to 180 m/s. Soil class E includes profiles from soil classes C and D but where the bedrock is located at a depth of 20 m below the ground surface. Apart from the soil classes mentioned here two special soil classes exist [3], but are not described here due to brevity.

A short study was conducted based on the assumption that a light ($q = 100$ kPa) and heavy ($q = 300$ kPa) structure will be founded on the soil profile. Selection of the light and heavy structures was done in line with studies carried out by well-known research teams [27] – [30]. The structure is assumed to be regular and shallow founded on a square foundation with side lengths of 20. The foundation length of 20 m corresponds to the maximum depth considered for the soil class E defined in Eurocode [3].

5 Results and discussion

This study shows that the structural loading may have big impact on the alteration of shear wave velocity in the soil. A leap from the lower to higher soil class was detected in 50% of the observed cases (Table 2), especially when the soil is loaded by a heavy structure. The leap is also evident for light structures, but only if the value of the average shear wave velocity for the observed soil profile is close to the average shear wave velocity value that separates two coded soil classes. For light and heavy structures an increase in average shear wave velocity of 11 % and 23 % respectively was observed (Table 2).

Table 2: Soil profiles observed in this study described by the average shear wave velocity for the first 30 m of deposit in free-field and when loaded by a structure

Profile No.	City	Soil class according to [3] ($v_{s,30}$ in m/s)				
		Free-field	m-n method		2:1 method	
			100 kPa	300 kPa	100 kPa	300 kPa
1	Bar	B (459)	B (508)	B (568)	B (498)	B (545)
2	Bucharest	D (165)	C (181)	C (203)	D (178)	C (195)
3	Lefkada	C (325)	B (365)	B (414)	C (357)	B (397)
4	Osijek	C (230)	C (266)	C (304)	C (258)	C (290)
5	Osijek	D (172)	C (200)	C (231)	C (194)	C (219)
6	Ploče	D (154)	C (182)	C (210)	D (177)	C (200)
7	Sirova Katalena	C (349)	B (410)	B (475)	B (401)	B (455)
8	Sisak	C (235)	C (280)	C (326)	C (271)	C (309)
9	Thessaloniki	C (288)	C (319)	C (355)	C (313)	C (342)
10	Ulcinj	B (400)	B (438)	B (483)	B (431)	B (467)

Furthermore, both methods (m-n and 2:1) result in very similar distributions of vertical stress in the foundation soil, and thus have similar impact on the shear wave velocity distribution in the soil profile below the foundation (Figure 3). Moreover, study results shows that the effect of vertical loading from a structure is almost negligible at a depth greater than approximately the half-length of the foundation (Figure 3).

Among others, Figure 3 shows soil profile Sirova Katalena for which the shear wave velocity distribution is not completely known over the entire depth. Instead, it may be assumed that the shear wave velocity distribution up to 30 m below the ground level is similar to the deepest measured velocity value in the section or one may assume existence of very stiff soil in deeper layers. Thus, there is a chance of assigning a wrong soil class to this profile.

Finally, it was noticed that soil classes C and D are much more sensitive to structural loading, compared to the class B soil. This mostly results from the fact that the soil class B covers a significantly broader area of average shear wave velocities, when compared to the soil classes C and D.

Next, the influence of contact pressure on shear wave velocity distribution within the light of the RS method is provided in this chapter. European code-based RS, dependant on soil class, are provided in Figure 4, where T , $S_e(T)$ and \ddot{u}_{gm} are the period, elastic horizontal ground acceleration RS and design horizontal ground acceleration on soil type A respectively.

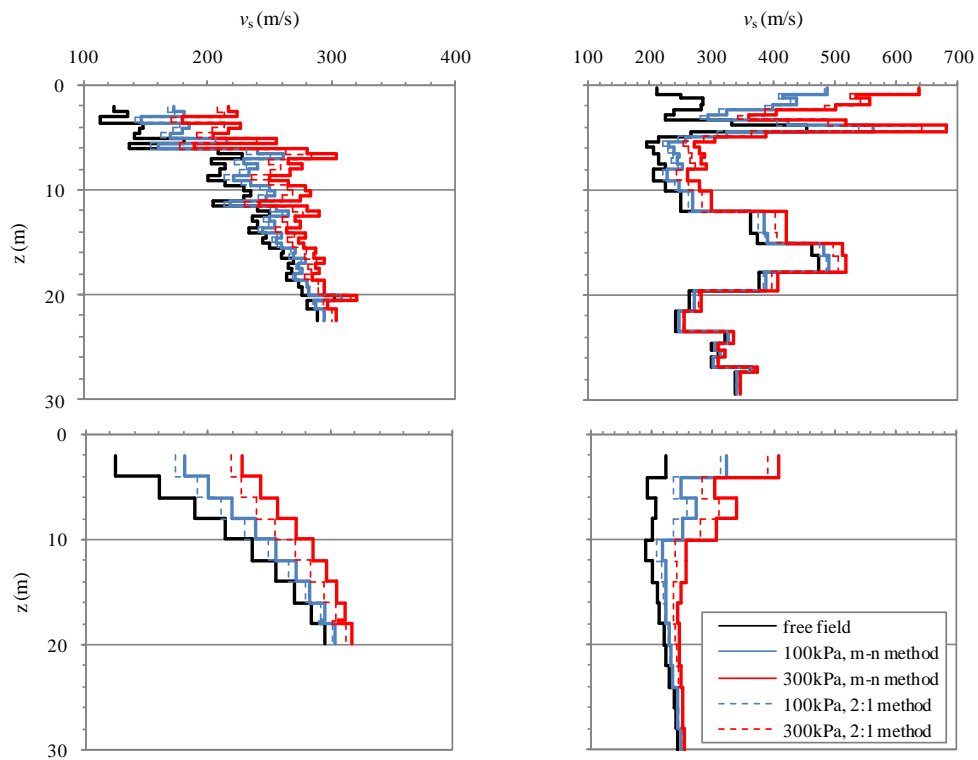


Figure 3: Shear wave velocity distribution along the depth of the section in a load free field and under the structure with a foundation of 20x20 m for following towns: Lefkada (*top left*), Thessaloniki (*top right*), Sirova Katalena (*bottom left*) and Osijek (*bottom right*)

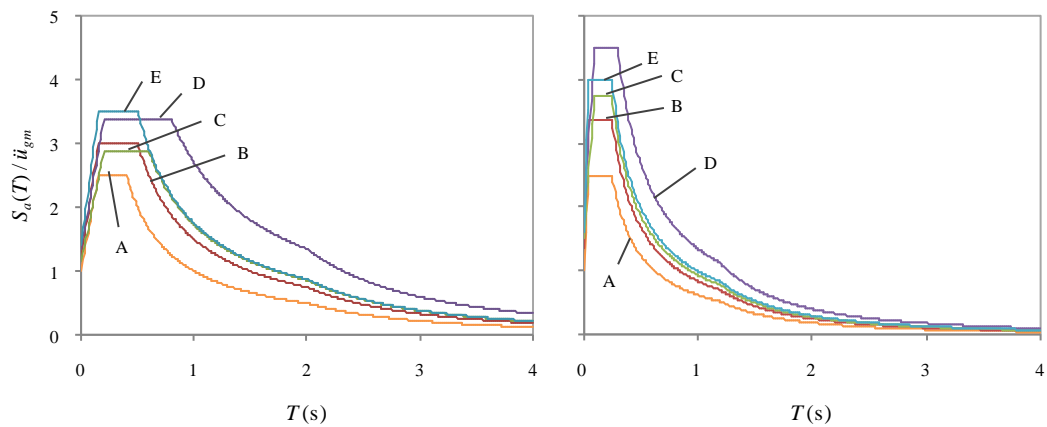


Figure 4: European code-based RS for 5 % damping: Type 1 (*left*), Type 2 (*right*)

Also, real parts of the impedance functions are introduced but only for horizontal translation and rotation in vertical plane, due to brevity. These two functions influence

the most the natural period of the soil-structure system, and may be estimated using the following expressions respectively [1], [18], [33]:

$$k_x = \frac{8 \cdot G_s \cdot B_f}{2 - \eta_s} \quad (10)$$

$$k_{yy} = \frac{8 \cdot G_s \cdot B_f^3}{3 \cdot (1 - \eta_s)} \quad (11)$$

where G_s is average value of soil shear modulus, B_f half-width of the foundation in direction of loads acting on structure and η_s is Poisson's ratio for foundation soil. Value for average soil shear modulus for foundation soil may be estimated by using the following expression [18]:

$$G_s = \rho_s \cdot v_s^2 \quad (12)$$

where ρ_s is soil density and v_s is average shear wave velocity for the foundation soil profile. Values for v_s are provided in Table 2 for the observed set of soil profiles, for free-field and when the soil is loaded by a light and heavy structure. For the sake of this study, the soil density and Poisson's ratio for the soil are assumed to be constant over the whole depth of the soil profile and equal to 2000 kg/m³ and 0.30 respectively [1]. As stressed earlier in this chapter, one of the main dynamic properties of the soil-structure system is the natural period. This period may be assessed using the following expression, under the assumption that every regular structure may be represented with a fixed-base inverted pendulum with stiffness \underline{k} and height of the centre of mass \underline{H} [1], [4], [33]:

$$T_{ssi} = T_1 \sqrt{1 + \frac{k}{k_x} + \frac{k \cdot H^2}{k_{yy}}} \quad (13)$$

where T_1 is first natural period of vibration of a fixed-base structure, i.e. fixed-base inverted pendulum. Well estimated natural period of the system is the key parameter that governs design of the structure when using the RS method. This interplay between foundation soil compliance effects on structural natural period and possible influence of contact pressure on the design RS shift is often omitted in engineering practice. Moreover, the RS method does not recognize effects of the contact pressure [3], [6]. In this study the first natural period of vibration of fixed-base building was estimated using well known empirical expression [1]:

$$T_1 = 0.1 N_{floor} \quad (14)$$

where N_{floor} is number of floors, where each floor is about 3 m high. It is assumed that the centre of mass of the building is located at 70 % of the total height. Weight of the superstructure W_s is calculated by assuming that the weight of the superstructure equals three times the weight of the substructure and that the building produces 100 kPa of bearing pressure on the soil. Stiffness of the inverted pendulum is then calculated from the well known expression:

$$T_1 = 2\pi \sqrt{\frac{m}{k}} \quad (15)$$

where m is mass of the superstructure. In Tables 3 to 5 $S_e(T_1)$ and $S_e(T_{ssi})$ are elastic spectral acceleration for fixed-base structure and the soil-structure system respectively, calculated according to [3] for the design ground acceleration a_g equal to 1 m/s^2 and damping correction factor η equal to 1. Also, in Tables 3 to 5 values of δ show the percentile modification of the spectral acceleration values when observing fixed-base structure or the soil-structure system, which is calculated as follows:

$$\delta = \frac{S_e(T_{ssi}) - S_e(T_1)}{S_e(T_1)} \quad (16)$$

Tables 3 to 5 show that incorporating soil-structure interaction effects in the RS method may result with up to 50 % higher forces in very stiff structures. This brings to conclusion that existing shallow founded low-rise buildings on soft soils, analyzed using conventional design, may be severely underdesigned and unsafe (in this particular point of view).

Table 3: The percentile modification of the spectral acceleration values for a set of buildings founded in Bucharest

N_{floor}	T_1 (s)	\underline{H} (m)	\underline{K} (kN/m)	T_{ssi} (s)	$S_e(T_1)$ (m/s ²)	$S_e(T_{ssi})$ (m/s ²)	δ
1	0.1	2.1	11843525	0.23	2.36	2.88	22%
2	0.2	4.2	2960881	0.30	3.38	2.88	-15%
3	0.3	6.3	1315947	0.39	3.38	2.88	-15%
4	0.4	8.4	740220	0.49	3.38	2.88	-15%
5	0.5	10.5	473741	0.59	3.38	2.88	-15%
6	0.6	12.6	328986	0.69	3.38	2.50	-26%
7	0.7	14.7	241704	0.80	3.38	2.16	-36%

Table 4: The percentile modification of the spectral acceleration values for a set of buildings founded in Lefkada

N_{floor}	T_1 (s)	\underline{H} (m)	\underline{K} (kN/m)	T_{ssi} (s)	$S_e(T_1)$ (m/s ²)	$S_e(T_{ssi})$ (m/s ²)	δ
1	0.1	2.1	11843525	0.15	2.01	3.00	49%
2	0.2	4.2	2960881	0.23	2.88	3.00	4%
3	0.3	6.3	1315947	0.33	2.88	3.00	4%
4	0.4	8.4	740220	0.43	2.88	3.00	4%
5	0.5	10.5	473741	0.53	2.88	2.83	-2%
6	0.6	12.6	328986	0.63	2.88	2.38	-17%
7	0.7	14.7	241704	0.73	2.46	2.05	-17%

Table 5: The percentile modification of the spectral acceleration values for a set of buildings founded in Osijek

N_{floor}	T_1 (s)	\underline{H} (m)	\underline{K} (kN/m)	T_{ssi} (s)	$S_e(T_1)$ (m/s ²)	$S_e(T_{ssi})$ (m/s ²)	δ
1	0.1	2.1	11843525	0.21	2.36	2.88	22%
2	0.2	4.2	2960881	0.28	3.38	2.88	-15%
3	0.3	6.3	1315947	0.37	3.38	2.88	-15%
4	0.4	8.4	740220	0.47	3.38	2.88	-15%
5	0.5	10.5	473741	0.57	3.38	2.88	-15%
6	0.6	12.6	328986	0.68	3.38	2.54	-25%
7	0.7	14.7	241704	0.78	3.38	2.21	-35%

On the other end, the same tables show that medium- to high-rise structures may be overdesigned (refers the same as in the above brackets) if one omits to include soil-structure interaction effects when analyzing structures using the RS method, which may lead to uneconomical design.

Finally, vertical loading from a structure may change the resonant frequency of the soil profile and thus alter its filtering capabilities (e.g. [7], [19], [33], [34]). Consequently, vertical loading from a structure may alter the frequency composition of an earthquake that will attack the structure. Evidence of this is provided in several studies by well-known experts ([16], [35] – [38]). It is clear that there is a need for a more precise inclusion of the structural loading effects on the foundation soil in the coded RS. Also, deeper investigation is still needed since damping in foundation soil is not considered in this paper. Also, this study did not explore soil-structure effects on velocity and displacement RS nor the rocking or sliding of the building on the foundation soil were observed.

6 Summary and conclusion

In this paper influence of weight from a structure on shear wave velocity distribution within a foundation soil profile is investigated within the light of the RS method. A study was conducted: on a set of real soil profiles collected by the authors; for both low and high structural loading and by using different methods for correction of shear wave velocity profiles to account for the overburden pressure. Highlights of the research conducted are provided as follows:

- a) The two used methods for correction of shear wave velocity profiles to account for the vertical loading from a structure give very similar results.
- b) The RS method does not recognize effects of the contact pressure
- c) Structural loading may have big impact on the alteration of average shear wave velocity in the soil, a parameter that governs selection of RS function.
- d) European coded soil classes C and D are much more sensitive to structural loading when compared to the soil class B.
- e) For light and heavy structures an increase in average shear wave velocity of 11 % and 23 % respectively was observed.

- f) Incorporating soil-structure interaction effects in RS method may result with up to 50 % higher forces in very stiff structures.
- g) Medium- to high-rise structures may be overdimensioned if one omits to include soil-structure interaction effects when conducting analysis by using the RS method.
- h) Vertical loading from a structure may change resonant frequency of the soil profile and thus it may alter filtering capabilities of the soil.

Further researches in this field will include the structural loading effects on the foundation soil in the coded response spectra, as well as the damping in the foundation soil and the SSI effects on velocity and displacement response spectrum. Also rocking and sliding effects of the building on the foundation soil will be observed.

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