Kinematic interaction forces in rigid inclusions under seismic loading

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Abstract: The reinforcement technique with rigid inclusions is considered as a practical, economical, and time-saving foundation solution. Several studies have recently focused on the static and cyclic behaviour of this reinforced foundation system. However, its seismic behaviour has not yet been widely studied. This paper presents a state of the art in the dynamic soil-structure interaction studies on foundation reinforced with rigid inclusions. Special attention is given to the estimation of the kinematic bending moments of inclusions in different soil deposits comprising several stiffness contrasts. The kinematic bending moments calculated by a transient dynamic analysis are compared with those obtained by means of pseudo-static Winkler model approach. The results show that, compared to the dynamic approach, the pseudo-static approach is able to predict the location where the maximum bending moment develops but underestimates its amplitude for important values of the stiffness contrast at the corresponding interface between soil layers.

Keywords: Kinematic bending moment, rigid inclusion, group effect, ASIRI+ project

1. Introduction

The technique of soil reinforcement by rigid inclusions (see Fig. 1) is a foundation solution that has significant technical and economical advantages compared to a conventional pile group foundation. This technique has been the focus of several research studies over the past twenty years, resulting in practical recommendations such as those of the ASIRI French National Project (2012). Those studies were mainly concerned with the static behaviour of this type of foundations, but their behaviour under vibratory and seismic loading has not yet received the same attention. This paper proposes to quantify the kinematic interaction effects on rigid inclusions under seismic loading with two different calculation approaches. The work is part of task 8 of the ASIRI+ French National Project (ASIRI+, 2019).

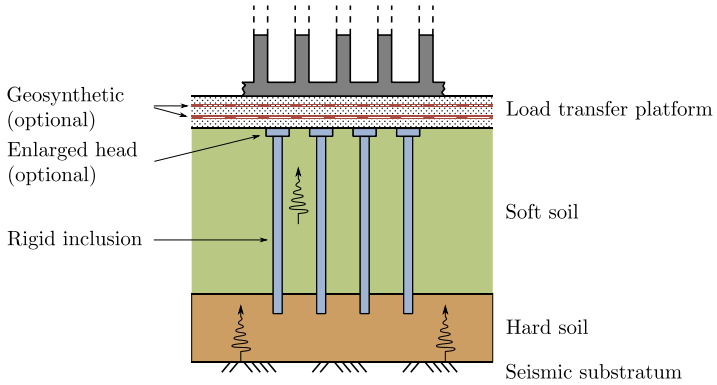

Fig. 1 - Scheme of a foundation on rigid inclusions under seismic loading

The kinematic interaction phenomenon is a dynamic soil-structure interaction effect related to the stiffness contrast between the soil and the foundation elements. In the case of a foundation on a reinforced soil by rigid inclusions, the presence of these interactions may induce additional stresses (bending moment, shear effort, etc.) in the rigid inclusions besides of those related to the vibration of the superstructure, so called inertial interaction effect. To apprehend this phenomenon, the current practice in design offices is mainly based on the pseudo-static approach using Winkler-type models. The present study is based on a series of dynamic models covering different soil deposits (inspired by real projects) and is aimed at verifying the usual calculation approach used in design offices.

2. State of the art

The seismic design of structures requires that the dynamic effects of soil-structure interaction are considered in a rigorous manner. Although design codes such as Eurocode 8 (EN 1998) have recognized the importance of taking into account soil-structure interaction effects, the available approaches for foundation design are still limited (Pérez-Herreros et al., 2017). Moreover, the behaviour of the foundation on rigid inclusions under seismic loading has not yet been studied intensively.

Many experimental studies found in the literature have focused on the static and cyclic behaviour of inclusion systems. The experimental research under seismic loading is still rare. Centrifuge tests were carried out by Garnier and Pecker (1999) to validate a macro-element foundation design concept used for the design of the Rion-Antirion bridge, which then demonstrated a good aseismic performance under a real earthquake. EDF (Électricité de France) conducted a series of vibration tests on two experimental plots obtaining the vertical and horizontal responses of a shallow slab foundation and a shallow foundation on rigid inclusions in the framework of ICEDA project. The objective of these tests was to justify the practical calculation methodology for inclusion-reinforced soil (Vandeputte et al., 2010). Santruckova (2012) attempted to investigate the seismic behaviour of rigid inclusions reinforced soil through a tri-dimensional small-scale experimental study under seismic loading. The response of the reinforced soil was measured to analyse the energy dissipation mechanism.

Concerning the numerical studies, different types of numerical models have already been used to calculate the response of this type of foundation, such as finite element method (FEM) by Rangel et al. (2008) and Mánica et al. (2016), finite differential method (FDM) by Jiménez et al. (2019) and boundary element method (BEM) as well as hybrid BEM-FEM method.

Most of the numerical studies mentioned above have used a complete model of the system, with an explicit modelling of the soil, the foundation and the superstructure. However, it is pragmatic and interesting to study separately the impact of different interaction phenomena, such as: the kinematic and inertial interaction effects. For this purpose, computational approaches based on the superposition theorem can be used.

On the basis of a superposition approach, certain research focused for example on the inertial foundation responses under harmonic loading in terms of flexibility. Okyay et al. (2012) studied the foundation system consisting of a soil-platform on a soft soil deposit reinforced by rigid inclusions. The vertical and horizontal flexibilities for the slab foundation with concrete inclusions obtained from in-situ tests were compared with the numerical results from a FEM model. Messioud et al. (2016) used a tri-dimensional FEM dynamic analysis to interpret the influence of the load transfer platform stiffness, the geometrical parameters, and

the head/tip connection condition to the inertial responses of foundation, still in terms of flexibility.

Shen et al. (2021, 2022a, 2022b) investigated the inertial et kinematic effects for soil reinforced by rigid inclusions under seismic loading and explored several aspects controlling the response of the foundation and the differences between this foundation solution versus shallow and piled solutions. Different configurations were studied and compared by means of an extensive numerical dynamic analysis. The inertial behaviour was analysed separately for each direction in terms of dynamic impedances and an equivalent soil deposit approach was proposed. The kinematic behaviour was studied by means of kinematic interaction factors. The study also compared the kinematic bending moment obtained by pseudo-static and dynamic approaches for a simple soil deposit but highlighted the need to conduct more extensive analyses of this topic.

This brief state of the art review shows that there is still a limited amount of research regarding the response under dynamic and seismic loading. Therefore, more research is necessary to better apprehend the main phenomena controlling the linear and nonlinear dynamic soil-structure response of this foundation solution. In addition, practical engineering methods should also be implemented.

3. Numerical study

3.1. Input motion

The seismic input motion used in this study is based on the elastic design response spectrum defined in Eurocode 8 (EN 1998) for a seismicity zone IV, an importance category III building, and a type E soil deposit. Five artificial accelerograms compatible with the design spectrum have been generated using the formulation proposed by Vanmarcke et al. (1976), which allows to establish a relation between the response spectrum and the spectral density function. A strong phase duration of 20 seconds is used. Figure 2 shows the response spectra at 5% damping for each of the calculated accelerograms (dashed curves), the average spectrum of the five accelerograms (red curve), and the target spectrum (black curve). The five accelerograms are used as the input solicitations and the average of the system response under five different accelerograms is presented in the following results.

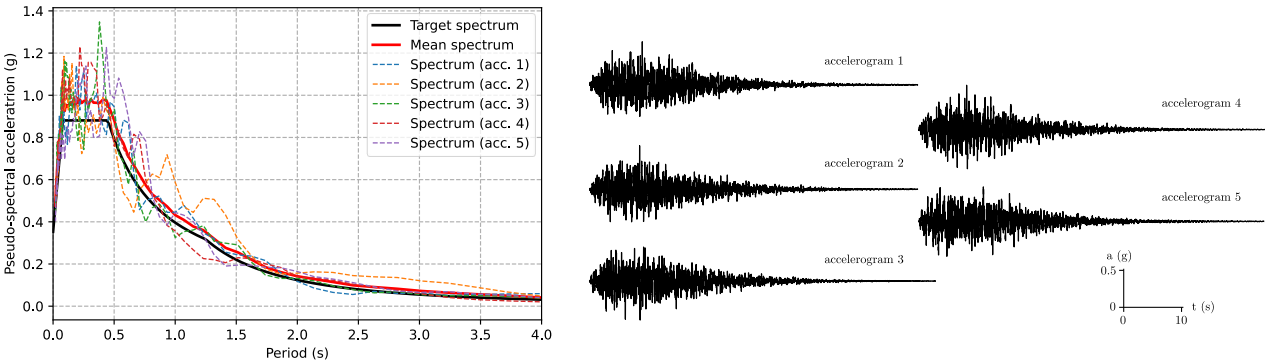

Fig. 2 - Comparison of the 5% average spectrum of the calculated accelerograms and the target spectrum

3.2. Soil deposit characterization

The reinforcement of soil by rigid inclusions is generally used to increase the bearing capacity and to reduce settlements under static load. The configurations where this type of foundation is usually adapted, identify in most cases as class D or E according to Eurocode 8

(EN 1998). Therefore, the soil profiles considered in this study intend to reproduce several configurations close to those found in real projects:

- Profile I: soft soil 1 between 0 and 10 m,
- Profile II: based on the profile I, the first 3 m of soft soil layer is replaced by soft soil 2,
- Profile III: based on the profile I, a 2 m layer of soft soil 3 is added in the middle of the soft soil layer.

In all tested cases, a 5 m thickness hard soil layer resting over the semi-infinite bedrock is considered. A scheme representing the different soil profiles is shown in the fig. 3 with the mechanical properties of the model shown in table 1. Table 2 resumes the geometric details for the three proposed profiles. The soft soil 1 shear wave velocity is also variable between 100 m/s, 150 m/s and 200 m/s for all three profiles, named a, b, and c, respectively.

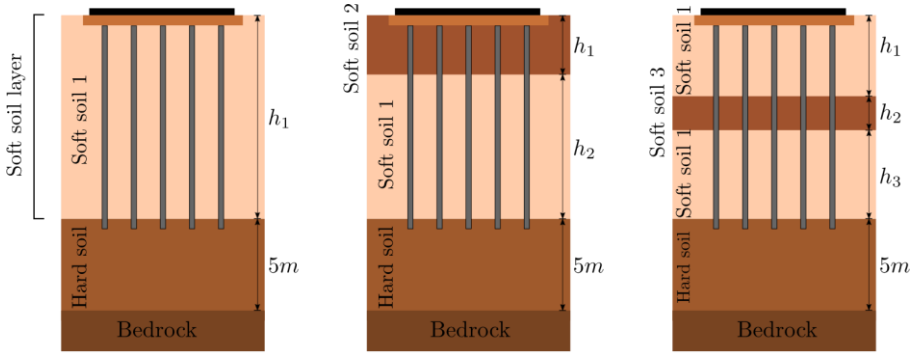

Fig. 3 - Schemes of proposed soil profiles

Table 1. Mechanical properties of soil and foundation used in the analyses

	Soft soil 1 (a/b/c)	Soft soil 2	Soft soil 3	Hard soil	Bedrock	LTP	Concrete inclusions
Shear modulus G (MPa)	20/45/80	80	180	320	2 500	125	12 500
Young's modulus E (MPa)	58/130.5/232	232	522	928	5800	362.5	30 000
Shear wave velocity Vs (m/s)	100/150 /200	200	300	400	1 000	250	2 236
Poisson ratio ν (-)	0.45	0.45	0.45	0.45	0.45	0.35	0.2
Mass density ρ (t/m^3)	2.0	2.0	2.0	2.0	2.5	2.0	2.5
Material damping ratio ξ (-)	0.05	0.05	0.05	0.05	0.05	0.05	0

Table 2. Soil profiles and the corresponding fundamental frequency of the soil column

Profile	Case	$h_1(m)$	$h_2(m)$	$h_3(m)$	f_1*(Hz)
I	a	10	-	-	2.42
	b	"	-	-	3.50
	c	"	-	-	4.43
II	a	3	7	-	2.46
	b	"	"	-	3.52
	c	"	"	-	4.43
III	a	4	2	4	2.64
	b	"	"	"	3.73
	c	"	"	"	4.61

* Fundamental frequency of the soil column considering the soft and compact soil layers on an infinitely rigid bedrock.

3.3. Foundation configuration

The studied configuration consists of a 10 m × 10 m rigid slab installed on a load transfer platform with a thickness of 0.5 m. The load transfer platform exceeds of 0.5 m from the

slab edges. The rigid inclusions have a length of 10 m and are embedded of 0.5 m in the hard soil layer. They have a diameter of 0.42 m (this corresponds to a typical dimension used in France for inclusions) and a centre-to-centre spacing of 2 m is considered. It should be noted that rigid inclusions having this diameter can be considered flexible with respect to their transverse response, i.e., the total length is greater than 3 times the transfer length l_0 (Brûlé and Cuira, 2017).

3.4. Numerical modelling and resolution

The problem is solved in the frequency domain by means of a hybrid transmitting boundary-FEM model built under the SASSI2010 program (Ostadan et al., 2000), which allows to consider appropriately the wave propagation in the soil and the group effect (dynamic pile-soil-pile interaction).

In this model, the soil is modelled by horizontal layers with viscoelastic behaviour on a semi-infinite bedrock. The slab is considered infinitely stiff and is represented by a tri-dimensional rigid beam grid without mass. The inclusions are modelled by two-node beam elements (not connected to the foundation slab) with six degrees of freedom per node (three translational and three rotational components). The beam axial and bending stiffness are calculated from the cross-section geometry, and the Young's modulus and Poisson's ratio of concrete (see table 1). According Mánica et al. (2016), modelling the inclusions by means of beam elements instead of volume elements has a negligible impact on the dynamic response of the groups of inclusions. The load transfer platform is modelled by eight-node hexahedral volume elements whose mechanical properties are shown in table 1. The soil and the foundation interact at the common nodes, so called interaction nodes, between the two sub-systems. Finally, a sufficiently fine mesh is used to ensure correct propagation of seismic waves over the frequency range of interest (the largest dimension is less than 1/8 of the shortest wavelength).

3.5. Dynamic approach and pseudo-static approach

Kinematic efforts in piles result from the deformation of the surrounding soil due to the passage of seismic waves. In this work, we are interested in comparing the estimation of kinematic bending moments in inclusions using a pseudo-static approach with Winkler model (p-y type model) and a dynamic approach (fig. 4).

In the pseudo-static approach, the kinematic soil deformation is introduced as a free-field displacement noted $\mathbf{g(z)}$ which is imposed on the free end of the springs representing the pile-soil interaction and not directly on the inclusion (Cuira, 2012; Hoang et al., 2020).

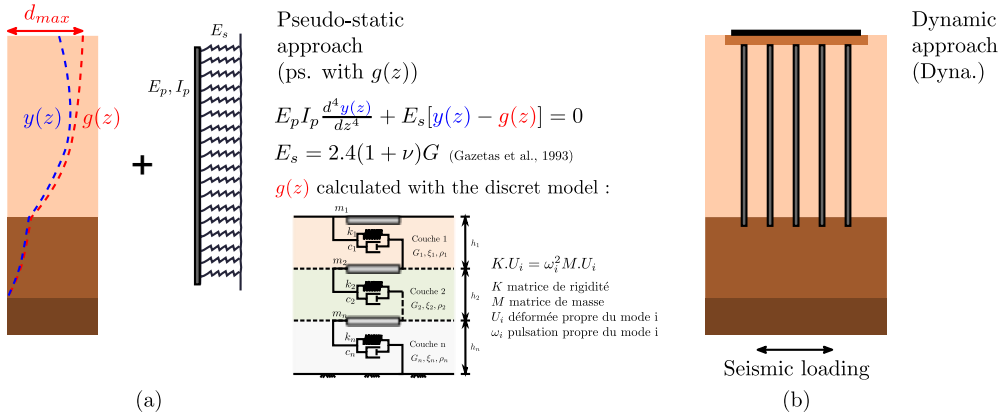

Fig. 4 - (a) Pseudo-static approach and (b) dynamic approach to calculate the kinematic bending moment

The soil displacement profile $\mathbf{g}(\mathbf{z})$ is supposed to have the same form as the fundamental eigenmode of the soil column with an amplitude equal to $\boldsymbol{d_{max}}$ at the soil surface (see fig. 4), which corresponds to the maximum free-field displacement at the ground surface and in this case is obtained as the average value of the pic ground displacements (PGD) of the five artificial accelerograms.

In the dynamic approach, the transient analysis is conducted with a complete model described in the above paragraphs.

4. Results

4.1. Kinematic bending moments of inclusions at different positions

A comparison of the kinematic bending moment profiles of inclusions at different positions is presented in the fig. 5 for the three soil deposits and three different shear wave velocities of the predominant soft soil. The objective of this comparison is to study the importance of group effects in terms of kinematic efforts applied to the inclusions. The inclusions at the centre, the edge and the corner are chosen in the analysis. It is to be noted that all the kinematic bending moment results presented in this article have been normalized by $\boldsymbol{E_p I_p / d}$ for comparison purposes.

It can be found that the dynamic responses of the inclusions at different positions under seismic loading are nearly identical. As a result, in the inclusions system, the horizontal group effect does not play an important role and does not impact the maximum kinematic bending moment. In the following analysis, the results of the centre inclusion are just presented since it is representative enough for all other inclusions.

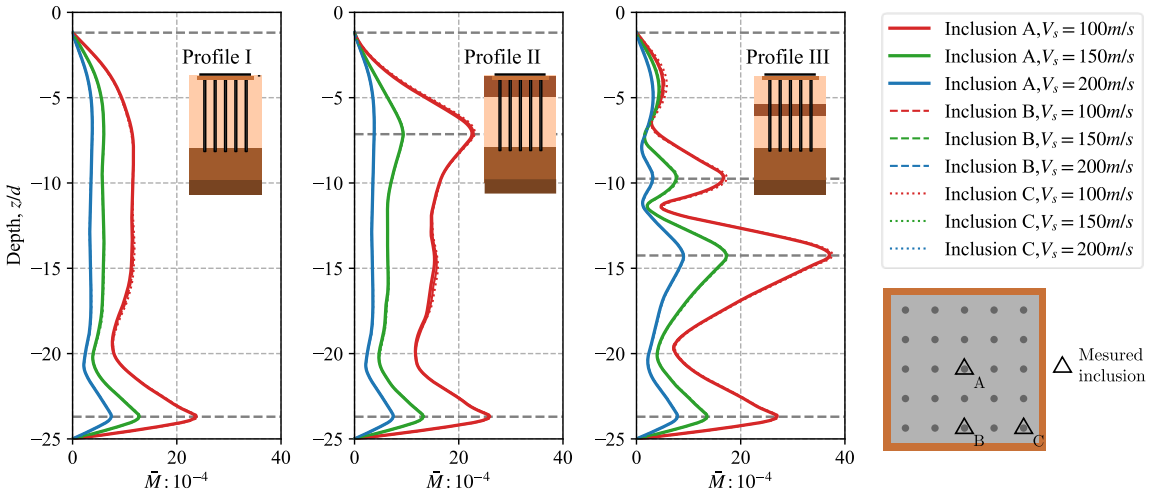

Fig. 5 - Kinematic bending moments of three profiles with different shear wave velocity Vs of soft soil 1: comparison of inclusions at different positions

4.2. Comparison between the dynamic approach and the pseudo-static approach

The kinematic bending moments calculated with two calculation approaches for three soil profiles and three shear wave velocities of the soft soil (thus different stiffness contrasts at the interface of different soil layers) are shown in the fig. 6.

The two approaches can provide the same tendencies of the moment variation with depth. The comparison shows the capacity of the pseudo-static approach to capture the location of

the interface where the maximum kinematic bending moment develops. However, it fails to capture its amplitude when an important stiffness contrast between layers is present.

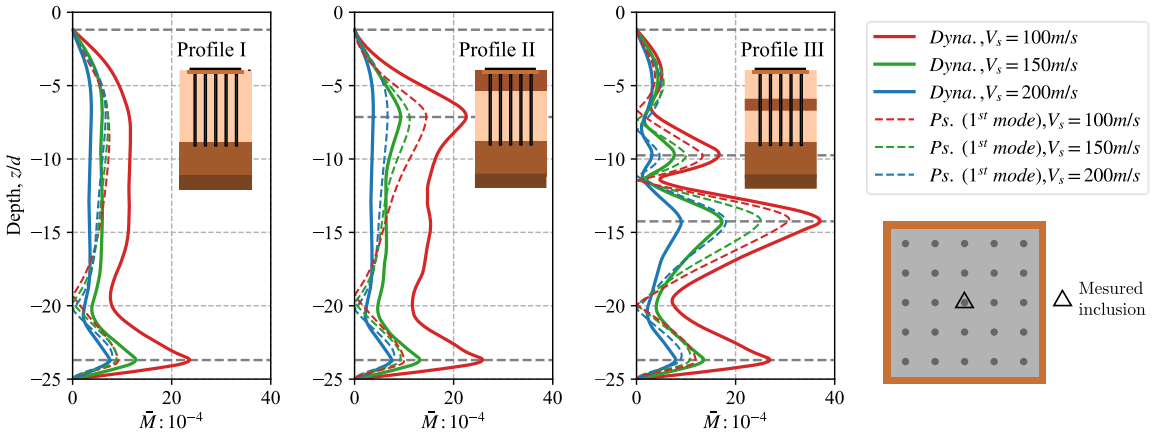

Fig. 6 - Kinematic bending moment profiles of three soil profiles with three different values of shear wave velocity Vs of soft soil 1: comparison of different calculation approaches

The relative error between kinematic bending moments calculated with both approaches at each interface are presented in fig. 7 for different values of the contrasts between the hard and the soft soil. A positive error means that the kinematic bending moment calculated with the dynamic approach is greater than the pseudo-static approach. Only at the interfaces with a small rigidity contrast (shear wave velocity ratio smaller than 3), the pseudo-static approach is found to be conservative.

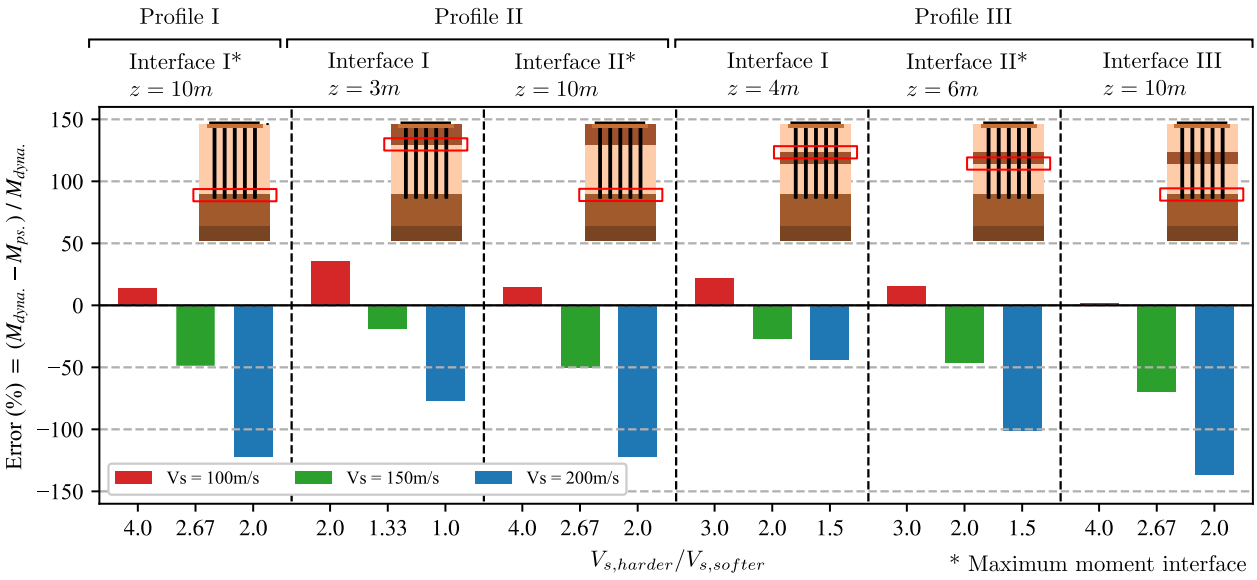

Fig. 7 - Relative error between the kinematic bending moments calculated with two approaches at the soil interfaces for the three soil profiles

5. Conclusions

Following a brief state of the art, a numerical work was presented to evaluate the kinematic bending moment of a foundation on a soil reinforced by rigid inclusions and submitted to the passage of vertical shear seismic waves. The calculations were carried out using a hybrid 3D model for the dynamic approach and a Winkler model for the pseudo-static approach. Three soil profiles with different soil stiffness contrast at several interfaces between soil

layers (inspired by the real projects) were studied. The results highlight that the pseudo-static approach is in general able to provide the correct trend of the response but under-estimates the amplitude of the maximum bending moment at the soil interfaces when important stiffness contrasts are present in the soil profile. The dynamic approach is thus recommended in the case of a complex stratigraphy and an important soil rigidity contrast profile (shear wave velocity ratio greater than 3).

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