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MODULE 3

Modelling of Non-Linear Phenomena

Soil-Structure Interaction in Alpine Valleys

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Soil-Structure Interaction in Alpine Valleys

Introduction

Certain aspects related to the possibility of modelling soil-structure interaction using widely available software like PLAXIS 2010 (Brinkgreve et al., 2010) were approached and studied within the framework of the Module 3a, Task 3a.4.3 – Modelling of non-linear phenomena.

The motivation to use PLAXIS has been driven by the increasing need of practitioners to use simple, clear and efficient ways to deal with soil-structure interaction in earthquake engineering. This is more challenging nowadays as serviceability limit states are often restrictive for use and there is an increasing tendency to incorporate performance-based design methods in ultimate limit state analyses in the area of geotechnical earthquake engineering.

The object of the analysis is represented by an industrial building, located in the area of Visp, with three foundation types: shallow raft, deep basement and a pile group foundation (Figure 1). Aspects such as modeling of the ground structure, calibration of the soil parameters and a simplified modeling of the structure were considered. The aim of the calculation is to explore the reliability of results such as accelerations at foundation level, displacements of the soil-structure system or earth pressure on foundation walls.

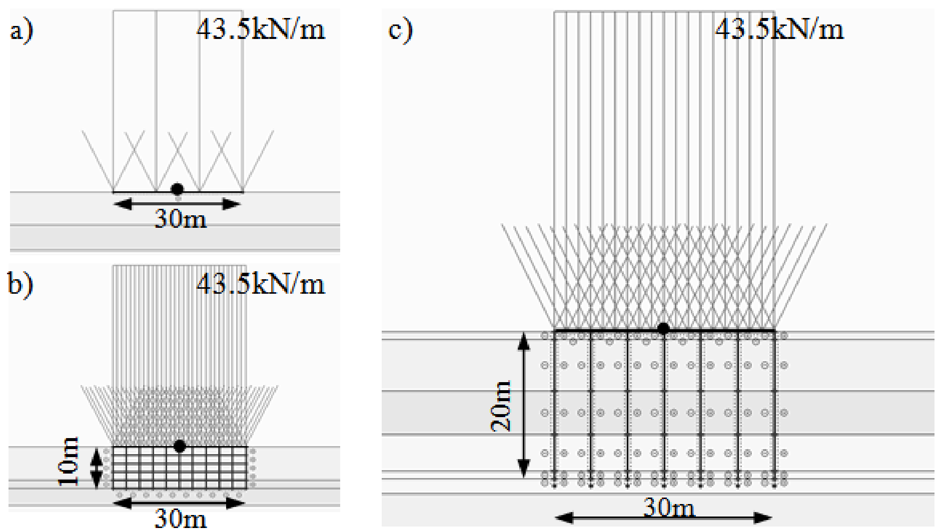


Figure 1. Analysed foundation types: a) shallow foundation, b) deep basement foundation, c) piled foundation.

Modeling of the ground structure – absorbent boundaries

One of the current encountered issues in modelling of the soil structure for dynamic analises is the problem of absorbent boundaries. The use of such boundary conditions is possible with PLAXIS 2D 2010, but the calibration of the relaxation coefficients to be incorporated into the calculation tends to move to the empirical range. In this situation, a possible symplifying solution, which provides good results, is to find an optimal width of the model (Figure 2), for which the influence of the boundary conditions on the surface accelerations is minimal. This optimal width of the model will allow for the travelling waves to be dissipated within the soil mass and produce no disturbing effects at the surface.

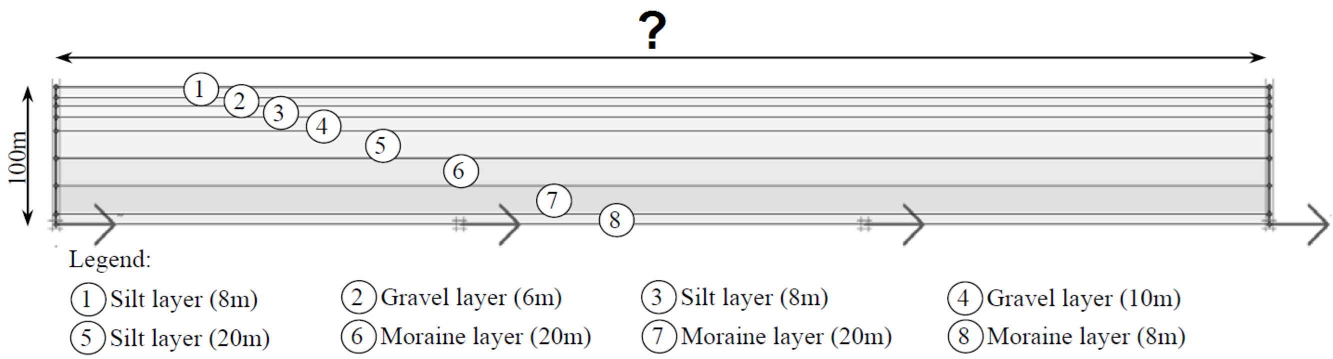


Figure 2. Simplified soil model.

Figure 3 shows the change of spectral accelerations at the surface of the soil model, for a change of the boundary relaxation coefficients. It becomes obvious that for a soil model width of 900m, the variation of the spectral accelerations caused by the change of the relaxation coefficients is minimal.

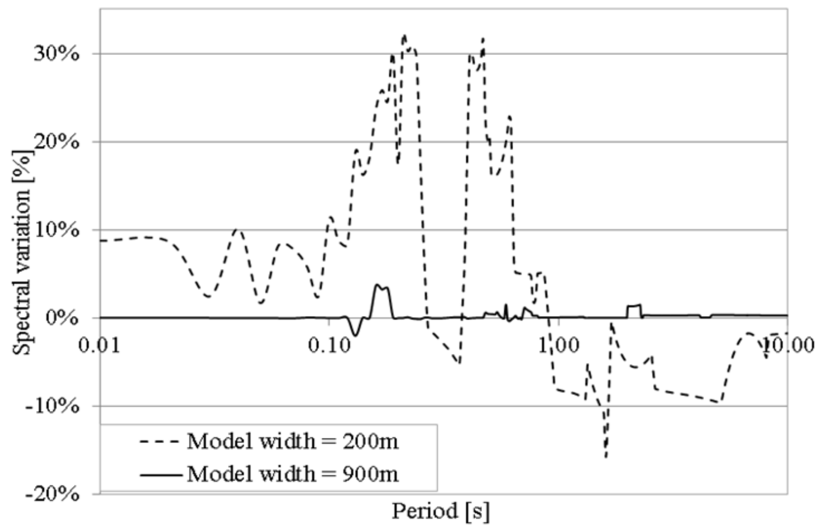


Figure 3. Variation of spectral accelerations with the change of relaxation coefficients, for different widths of the soil model.

Considering the general simplifying method mentioned above, the optimal width of the soil model for this special case of the current analysis is 900m. Further possibilities, such as infinite boundary elements may also be used but this depends on the FE-software employed.

Calibration of the soil parameters – Rayleigh damping coefficients

Damping represents a very important material parameter in the time domain dynamic analyses. The formulation of material damping has a significant impact on the final results and therefore a correct approach ensures reliability of the calculations.

The well-known formulation of Rayleigh damping (Park & Hadash, 2004, Figure 4), against its simplicity, seems to be difficult in application, as the influence on the significant natural frequencies of the geotechnical structure is very strong.

$$\begin{bmatrix} \xi_m \\ \xi_n \end{bmatrix} = \frac{1}{4\pi} \cdot \begin{bmatrix} \frac{1}{f_m} & f_m \\ \frac{1}{f_n} & f_n \end{bmatrix} \cdot \begin{Bmatrix} \alpha_R \\ \beta_R \end{Bmatrix} \quad (1)$$

where f_m and f_n are two significant natural frequencies and ξ_m and ξ_m are the corresponding damping ratios.

Figure 4. Rayleigh damping formulation (Park & Hashash, 2004).

In this case the proposed solution was a calibration of the Rayleigh coefficients, in the linear-equivalent range, using a frequency domain analysis of the problem. The employed software is EERA (Bardet et al., 2000) and the method of calibration was based on an iterative process. Using a down-scaled seismic signal, the frequency domain analysis was performed in the linear-elastic range (values of the damping profile within the soil model smaller as 5%). Using the damping values obtained for the different material layers together with 2 significant frequencies of the soil profile, the values of the Rayleigh coefficients were calculated. These values were subsequently plugged in the time domain analysis (PLAXIS) with the same input signal. Finally the convergence of the frequency domain and the time domain analyses was checked. In the case of poor convergence of results, the process was iteratively repeated, by changing the 2 natural frequencies. In the case of convergence, the Rayleigh coefficients were declared as valid and used in further calculations. A visual representation of the entire process can be observed in Figure 5.

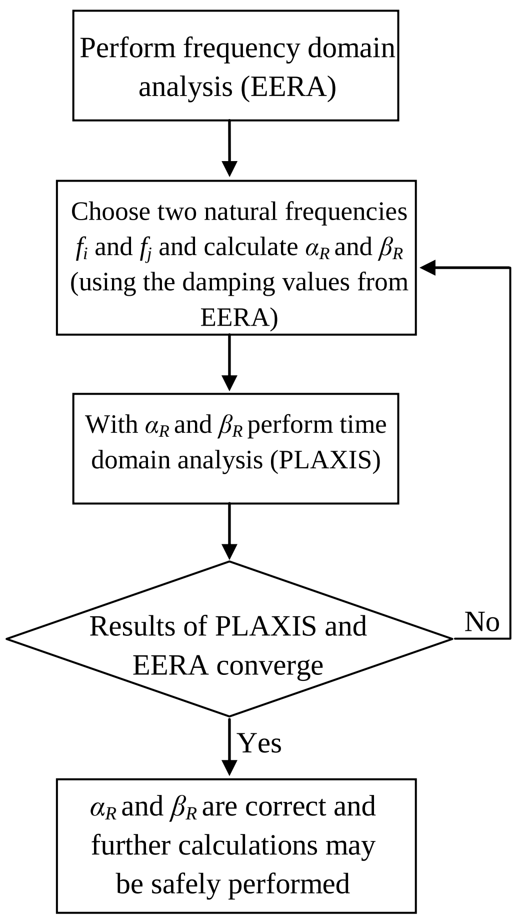
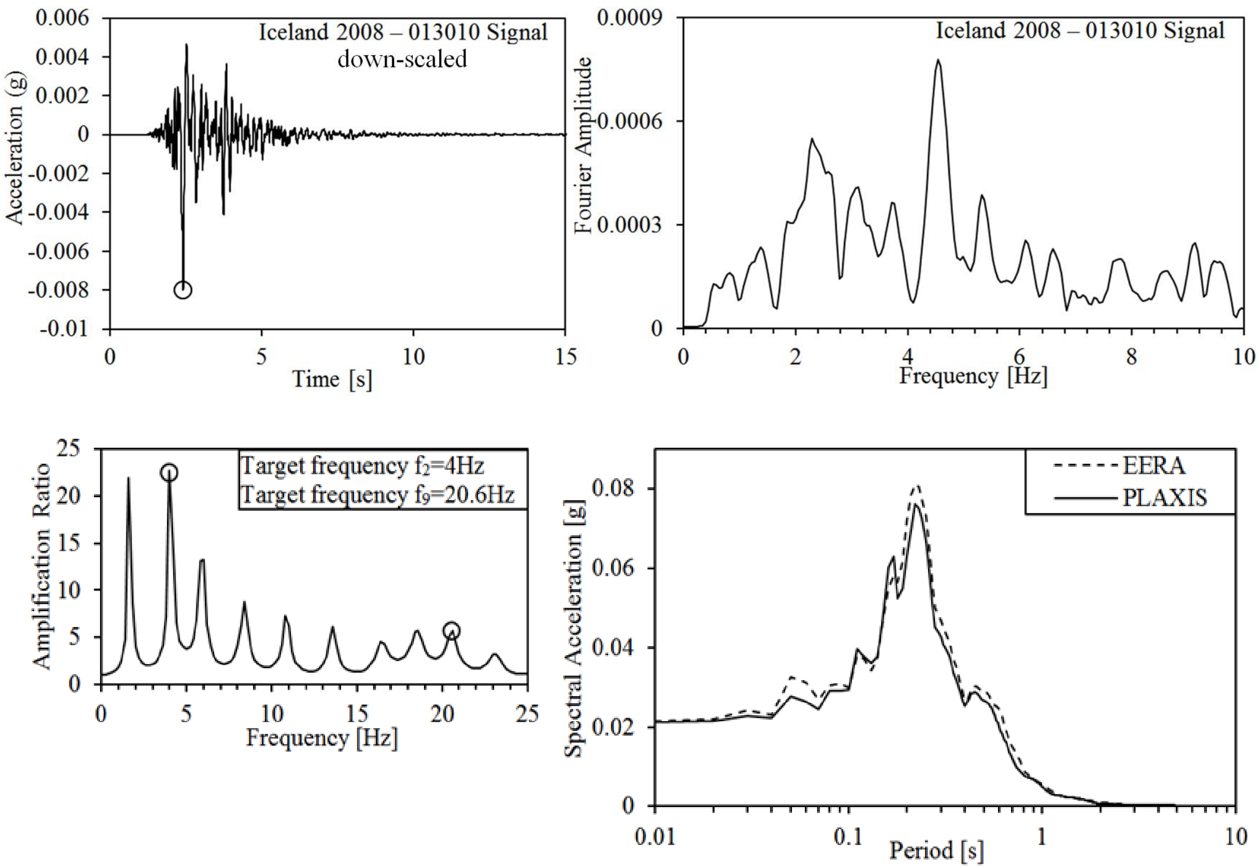


Figure 5. Flowchart for the calibration of the Rayleigh coefficients.

Figure 6 shows the application of the calibration procedure and the obtained Rayleigh coefficients for the Iceland 2008 – 013010 seismic signal (USGS, 2008) in the case of the optimised (width = 900m) soil model presented above (Figure 2).



Determined Rayleigh Coefficients:

Layer	1 (Silt)	2 (Gravel)	3 (Silt)	4 (Gravel)	5 (Silt)	6 (Moraine)	7 (Moraine)	8 (Moraine)
α_R [-]	1.447	0.272	1.084	0.272	0.910	0.226	0.227	0.215
β_R [-]	0.000445	0.000084	0.000333	0.000084	0.000280	0.000070	0.000070	0.000066

Figure 6. Input signal (top) and results of Rayleigh coefficients calibration for the Iceland 2008 – 013010 Signal.

Modelling procedure – simplifying assumptions

Several of the assumptions made within the framework of the current analysis will be presented together with their advantages and disadvantages:

- The soil model was considered as rectangular and uniform, with horizontal soil layers. This assumption simplifies the computational effort and approaches very well the uncertainties in the layered structure, but cannot account for eventual topographical effects. More complicated and close to reality geometrical features can be considered in future analyses.

- The material model used in the calculations was the simple Mohr-Coulomb model. This assumption regarding the behaviour of the materials represents a robust method to approach the problem, which was proved in time to be very simple and effective in FE-simulations. The main disadvantage is that Mohr-Coulomb represents a theoretical, highly approximated approach, which in some cases can be far from the real behaviour of soils. In future analyses, more advanced user defined or already available models can be used, at the cost of an increased number of parameters to be defined. Nevertheless, using an elasto-plastic model is a huge improvement in dynamic analyses of soil structure interaction problems.

- For the case of the piled foundation, the piles were considered as standard plate elements, with reduced stiffness, in order account for the 3D character of this particular situation. According to already previous literature (Randolph, 1981, Naylor, 1982), this assumption manages to capture in a reasonable manner the behaviour of the designed elements, considering the limitations of a 2D analysis. The main disadvantage is that the effects of the real 3D interaction between the soil and piles are lost. A future analysis may consider a 3D FE-environment.

- The superstructure was approximated as a uniformly distributed load on the foundation system. This severely simplifying assumption has the main advantage of reducing the computational effort and eventual errors due to limitations of the employed software in predicting the behaviour of the superstructure. On the other hand the inertial effects from the superstructure on the entire systems, which are not to be neglected, are totally lost. Future calculations could consider the possibility of using more a general FE-platform, able to model structural elements in a more detailed manner.

Considering the fact that the main purpose of the current contribution is to rather explore a simple, efficient and clear way of dealing with the problem of soil-structure interaction, the simplifying assumptions are well founded and reasonable. There are clearly more accurate methods to approach the problem, but the cost of employing these methods is always translated into more complicated, detailed and sophisticated models and procedures, which tend to give to the stated purpose less importance in the favour of accuracy.

Simulation results

The results of the analysis proved to be in compliance with already existing results, observed phenomena and expected behaviour of the system.

As a first component of the results set, the acceleration at foundation level are presented in Figure 7.

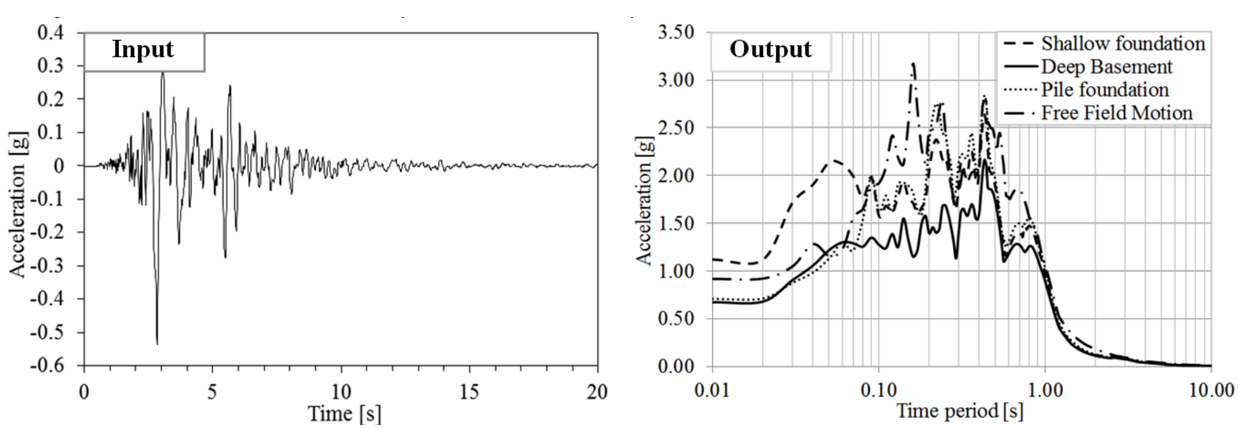


Figure 7. Input motion (Iceland 2008 – 01310) and output spectral accelerations at foundation level.

The compliance with expected and already observed behaviour becomes obvious. The shallow foundation is characterised by the highest accelerations across the entire spectrum. The most favourable case in terms of acceleration of the superstructure is the one of the deep basement foundations, where the minimal accelerations are recorded. For the piled foundation, the first part of the spectrum (up to a period of 7 – 8s) exhibits low acceleration values, comparable with the deep basement case. For the rest of the spectrum (periods longer than 8s) the accelerations increase significantly up to the levels observed in the case of the shallow foundation. This effect may be explained by the fact that the raft of the piled foundation system is subjected to the combined action of waves travelling through the soil and through the concrete piles.

The displacements of the foundation systems are also of great importance, especially in the case of a performance-based design. The results obtained for this particular case, where the inertial effects of the superstructure were totally neglected, are presented in Figure 8.

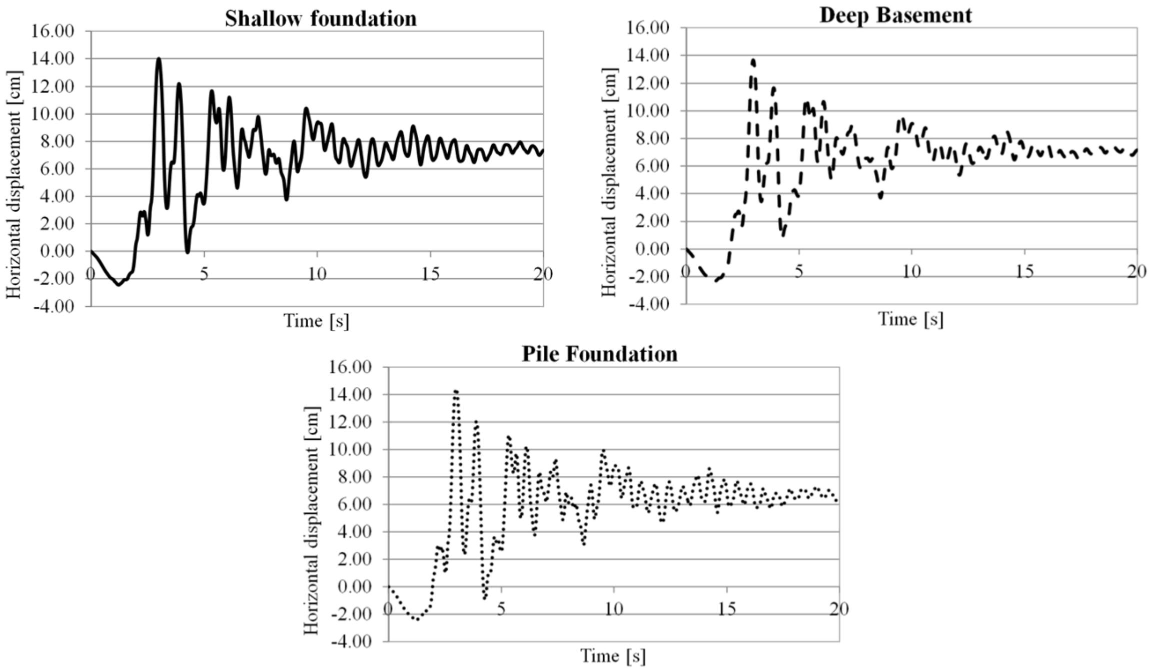


Figure 8. Lateral displacements of the 3 foundation systems.

The values are basically in the same range, with small differences for the shallow foundation, where the residual lateral displacements tend to be larger than in the other cases. In the same time, the amplitudes of displacement variation in time seem to be larger in the case of the piled foundation. These presented values are not relative displacements of the foundation system with respect to the soil but in fact, they are total absolute displacements with respect to the initial mesh generated by PLAXIS. The relative displacements between the foundation and the soil will be investigated in the future paragraphs.

During the shaking, besides the lateral displacements, vertical displacements of the foundation occur. Using these vertical displacements an average tilting of the foundation during the shaking can be approximated (Figure 9).

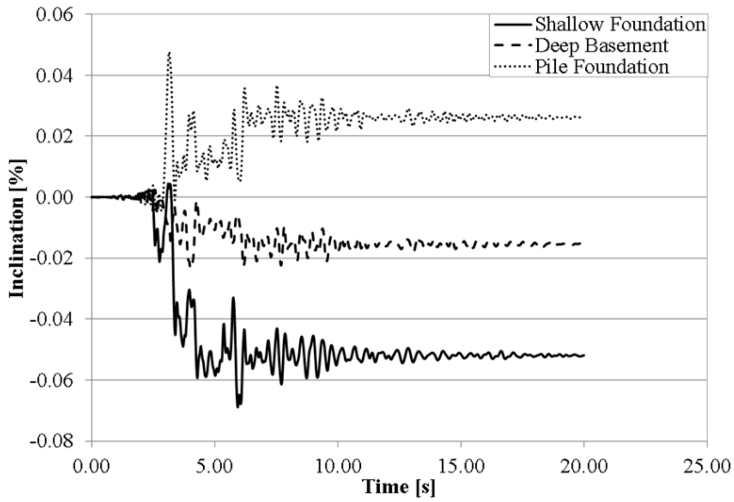


Figure 9. Average tilting of the foundations systems during shaking.

Neglecting the sense of the tilting, the first conclusion is that the most favourable case, with the smallest expected tilting is the deep basement followed by the piled foundation. The values for actual and residual tilting for these two foundations systems lay in the same range, even though they tend to displace in opposite directions. For the case of the shallow slab foundation, the inclination is up to 3 times larger.

The soil wall interaction may be observed from two perspectives: relative displacements and lateral earth pressures. For the relative displacements 4 different points were considered at the interface soil-wall, in order to obtain a good visualisation of the phenomena (Figure 10d).

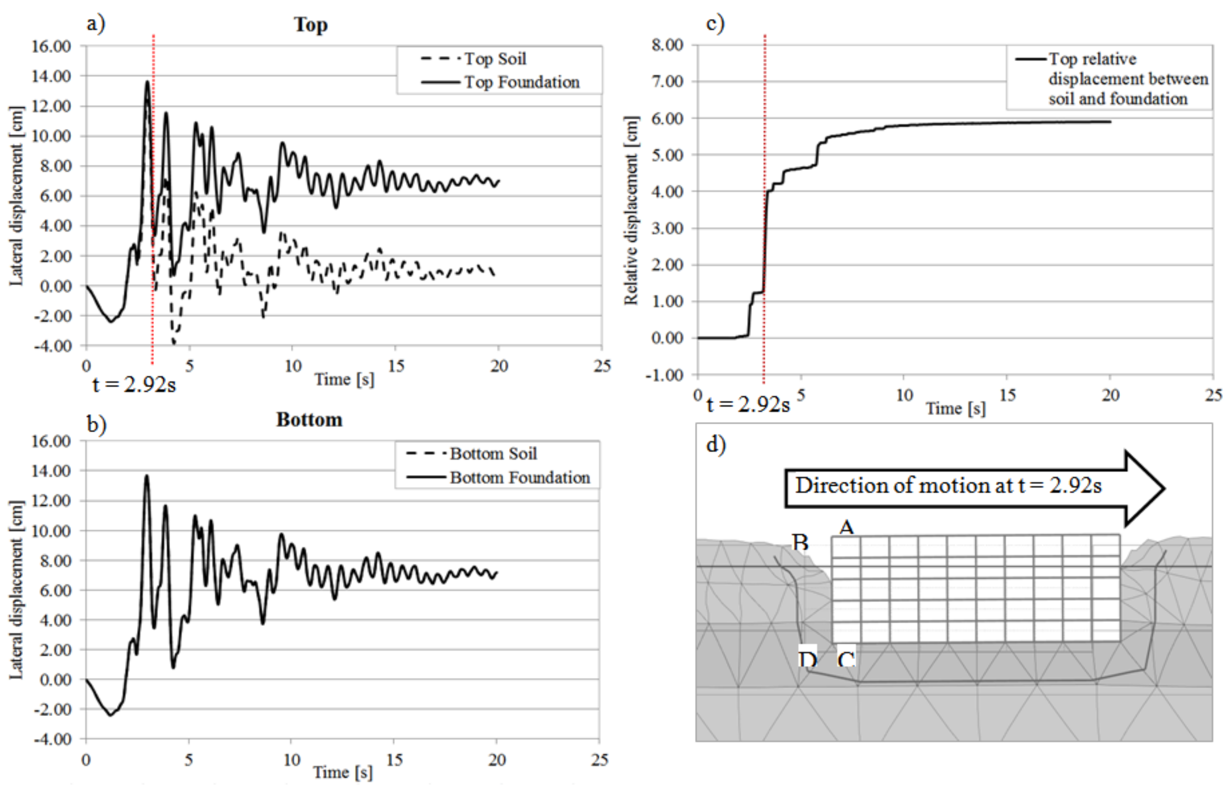


Figure 10. Lateral relative displacements for the soil-wall system.

Considering the lateral displacements in the interface, it becomes clear the points *C* and *D* move together, in other words, at the bottom of the interface the soil and the wall move together during the shaking (Figure 10b). At the top of the interface, the points *A* and *B* start to move together, but during the shaking, there is an increasing relative displacement between them (Figure 10a and c). This behaviour can be explained by a theoretical gap which repeatedly opens in the interface. When the motion occurs in the positive direction (from left to right), the foundation tends to move more than the soil mass and for the analysed left wall of the deep basement the gap is opening. When the motion is in the negative direction (from right to left), the foundation tends to remain behind and move less than the soil mass – the gap is opening again.

Taking into account the fact that the material is granular, this theoretical gap which repeatedly opens will constantly be filled – the material will flow in and will densify next to the wall. This interpretation of the variation of the relative displacements leads to the next way of observing the soil-wall interaction: lateral earth pressures.

If the lateral earth pressures on the same left wall of the basement are plotted at different moments of time an interesting effect will be observed (Figure 11).

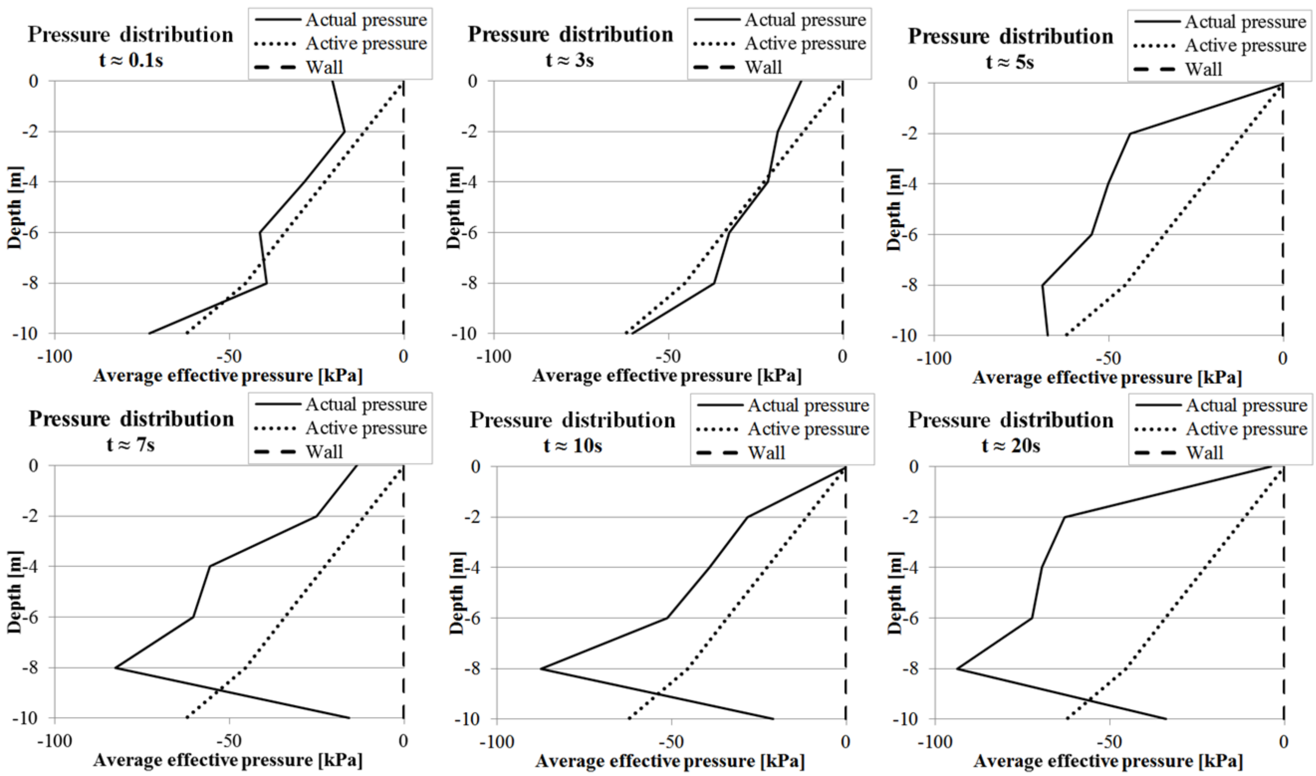


Figure 11. Variation of lateral earth pressure on the side wall of the basement during shaking.

During shaking, the pressure on the wall decreases in the top and bottom sections of the interface. This phenomenon was already observed in previous contributions dealing with integral bridge abutments (Ng et al. 1998, Tsang et al., 2002) and piles (Chari & Meyerhof, 1983) under cyclic loading. As mentioned above, the material tends to accumulate in the mid-height area of the wall, increasing the stiffness of the system in this region, and the top and bottom parts are subsequently unloaded, due to a decreased stiffness. In this way, pressures, which act on the wall, are higher than intuitively expected.

Conclusions

Observing the results of this simple FE-analysis applied to the 3 types of foundation systems for a general industrial building, the first conclusion is that the widely available PLAXIS software can provide a reasonable approach to the non-linear phenomena associated with soil-structure interaction problems.

Against the multiple simplifying assumptions, parameters approximations and partially empirical calibration methods, the final results are in compliance with already observed phenomena and previous contributions in the area of dynamic analyses.

Finally, as already stated, the present analysis attempts to address the permanently increasing need of approaching problems such as soil-structure interaction and non-linear behaviour of systems, with limited but widely available means. The results are, in this sense, very encouraging and open perspectives for better opportunities of incorporating similar approaches in current design practice.

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