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Rigid inclusions combined with stone columns as ground improvement technique in a high seismicity area prone to liquefaction

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ABSTRACT: The rapid urban expansion and the increasing demands for usable space have posed significant challenges in design and construction processes. This is primarily because many important and complex structures are planned to be constructed in difficult areas from the geotechnical point of view. In such conditions ground improvement techniques are systematically being used by engineers to address issues associated to soft soil deposits. Throughout the years, in literature and practical applications a number of such techniques have been developed and extensively used. The aim of such techniques is to enhance the behavior of the weak soil deposits by either modifying strength and deformability parameters or by the means of reinforcement. The latter representing a composite mass in which the applied loads are distributed between the soil and the reinforcement elements. One of such techniques is the usage of rigid inclusions. The principle is that of the reinforcement of the soil by the introduction of pile elements of much greater rigidity than the surrounding soil that considerably reduces the settlement. The performance of this technique in real complex conditions such as, varying raft thickness with different load distributions due to the differences in building height and non-horizontal soil layers prone to liquefaction has yet to be carefully studied and documented. This paper shows the design aspects of the implementation of rigid inclusions in the aforementioned situations in a high seismic area prone to liquefaction in combination with stone columns to meet both ultimate and serviceability limit states.

KEYWORDS: ground improvement, liquefaction, rigid inclusions, settlement, stone columns

1 INTRODUCTION

The traditional approach of geotechnical engineers when dealing with the design of a foundations is focused on adapting solutions with existing soil conditions as provided by investigation and characterization processes. However, the selection of deep foundations as an alternative to shallow foundations for specific site conditions may result in added costs (Coduto, 2016). There are many cases in which shallow foundation solution is not acceptable, but the alternative deep foundation is overdesigned (ASIRI, 2012).

The aforementioned conditions in combination with rapid urban expansion, the increasing demands for usable space together with land scarcity and the increased recognition of seismic hazards imply the usage of techniques that improve soil conditions. The process of improving soils for civil engineering purposes is generally known as *ground improvement* (ASIRI, 2012) (Bowels, 1996).

There is a wide variety of ground improvement methods with the aim of increasing soil strength and/or decreasing compressibility. The selection of the method to be implemented is tightly related to the soil conditions and project requirements and in specific cases there is a mixing of technique. In the context of ground improvement, it is important to make a distinction between methods of compaction and densification and methods of soil reinforcement by introducing additional materials or elements into the ground (Moseley & Kirsch, 2004).

One of the most used reinforcement-based techniques are Rigid Inclusions. Such interventions are defined by (ASIRI, 2012) as slender elements, often cylindrical in shape, mechanically continuous and typical vertical that are laid out according to a regular mesh pattern. The main target of Rigid Inclusions is to increase bearing capacity and reduce the settlements by creating a mix system with the surrounding soil.

The aim of the paper is to analyze the performance of this reinforcement technique used in combination with Stone Columns in a high seismic area prone to liquefaction in Albania. As previously noted, the stone columns utilized in the project were excluded from the settlement analysis but were incorporated into the liquefaction assessment. This approach was based on the prevailing subsurface conditions, which included soft cohesive soils with undrained shear strengths (S_u) below 15 kPa in certain strata. These conditions, combined with the potential for insufficient lateral confinement—particularly within the upper third of the column length—contributed to a high likelihood of performance limitations. Additionally, the area replacement ratios, ranging between 15% and 40%, were deemed insufficient to achieve the desired settlement control when used in isolation. As a result, stone columns were employed in conjunction with rigid inclusions to enhance overall ground performance under both static and seismic loading conditions. Ultimate and Serviceability verifications are performed in the case of non-horizontal soil layers with different load distributions due to differences in building height.

2 SITE AND SOIL CONDITIONS

The site is located in the western part of Albania along Adriatic coast in the city of Durrës. Albania belongs to the Alpine-Mediterranean seismic belt, which is one of the most seismic regions in Europe (Aliaj, et al., 2010). The latest probabilistic seismic hazard map of Albania shows values of the reference peak ground acceleration (PGA) in bedrock for Durrës varying from 0.259g to 0.274g (IGEO, 2021). On the other hand, the site is part of the Holocene marine deposits which consist of sands and clayey sands with a maximum thickness of 20m as depicted in Figure 1.

Due the complexity of area, the investigation process consisted in numerous in situ and laboratory tests with disturbed and undisturbed samples taken in different depths in several boreholes. In addition, CPTU tests were performed and the entire information was analyzed to provide a detailed geotechnical model.

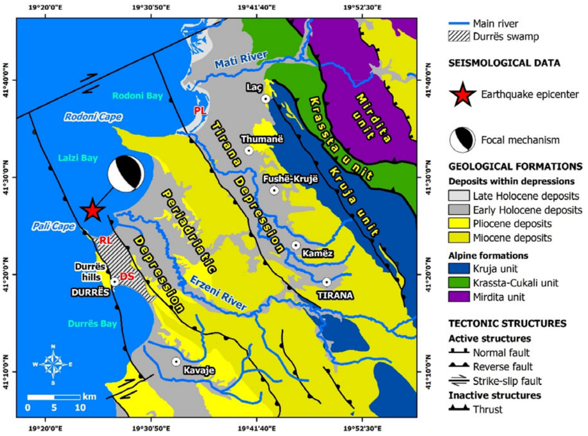

Figure 1. Geological Map of the area (Mavroulis, et al., 2021).

The entire investigation process was extensive, but the data provided showed significant discrepancies in the derived geotechnical parameters from the Boreholes, CPT and SPT interpretations. As a result, special attention was given to the determination of the soils strength deformation characteristics.

The soil profile primary consists of sand and silty sand, these layers are underlain by clayey layers at greater depths which exhibited low plasticity. For the sandy layers the elastic modulus ranged from 9.3 to 15 MPa, whereas for the clayey layers did not exceed 6.4 MPa. The undrained shear strength was estimated at approximately 38 kPa. A comprehensive description of the investigation and characterization process is beyond the scope of this paper.

3 MATERIALS AND METHOD

3.1 *Overview of Rigid Inclusions and Stone Columns*

In principle foundations on rigid inclusions represent aspects that are similar to piled raft foundations with the main difference being the lack of rigid mechanical connection between the slab element and the vertical slender elements. In practice this is reflected by a combined geometric and mechanical discontinuity often by the introduction of a thin granular mattress (often reinforced) known as the Load Transfer Platform (LTP).

In such conditions the system comprises various modes of interaction between (1) inclusions themselves, (2) the LTP and (3) the foundation soil between the inclusions. Due to the much higher stiffness of the inclusions compared to the surrounding soil, an arching effect develops within the LTP. This mechanism causes a redistribution of stresses onto the rigid inclusions leading to a reduction of the load transmitted to the surrounding soft soil (see Figure 2). Additionally, due to the relative settlement of the soft soil, negative skin friction is also mobilized along the upper part of the inclusions up to a certain depth. The additional stress at the rigid inclusions due to this phenomenon has to be taken into consideration in the design process (ASIRI, 2012) (Pham, 2018).

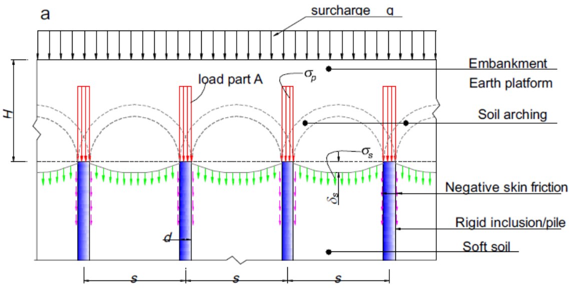

Figure 2. Load Transfer Mechanism of RI system (Pham, 2018)

In addition to rigid inclusions another ground improvement technique categorized under vibro-replacement is the use of stone columns. Their installation involves the partial replacement of the weak compressible soil with a compacted vertical column of granular material. Stone columns are used in cases where there is the need of increasing the bearing capacity, reducing total and differential settlements, reducing the time rate of settlement and also reducing the liquefaction (Mosley Kirsch and Das). The effectiveness of stone columns relies on the lateral support provided by the surrounding soil. Such aspect, constitutes the core difference between stone columns and rigid inclusions. Literature suggests that the minimum shear strength (s_u) where the applicability of such technique is feasible is 15 kPa (Das, 2014) (Moseley & Kirsch, 2004) (Barksdale & Bachus, 1983). A group of stone columns in soft soil undergoes a combined local bearing type failure and bulging with the latter being the dominant failure mechanism due to the insufficient lateral support (see Figure 3).

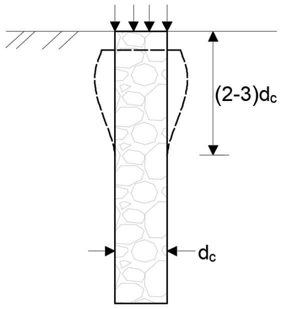

Figure 3. Combined failure mechanism of stone column groups.

3.2 *Design Methodology for Rigid Inclusions system*

The focus of this paper is specifically on the rigid inclusions and since they are installed in the conditions where stone columns with a diameter of 0.8m and an axial distance of 2.5m are present the liquefaction is not considered in the calculations.

The selection of the diameter, length and configuration of the rigid inclusions is tightly depended on the geotechnical model, the overlying structure and the available construction methodology. The eventual verification, performance, efficiency is characterized by iterative processes to find the optimal configuration.

The proposed solution consists of 800mm diameter Continuous Flight Auger (CFA) bored piles arranged in a square grid with a 2.5m axial spacing. A LTP made of granular fill, with a thickness ranging from 0.5 to 1.0m, is placed above the RIs heads to ensure the arching effect and proper load distribution between the RIs and the surrounding weak soil. The maximum distributed load is 182kPa and is transmitted through a raft foundation with a maximum thickness of 1.4m.

3.2.1 *Load Transfer Behavior*

This section presents the verification process of the maximum allowable stress value at the inclusions head for the given external load transmitted by the foundation. The equilibrium of the system is dependent on the geometry and the nature of loading and as shown by (ASIRI, 2012) two failure mechanisms modes can be possible; Prandtl's failure mechanism or punching shear (see Figure 4).

For the given external load q_0, both types of limit equilibrium serve to define the maximum load that can be concentrated at the inclusion head. The maximum load taken by the rigid inclusion (q_p^+) is dependent on (1) the thickness of Load Transfer Platform LTP (H_M) and its geotechnical parameters, (2) the presence of a rigid structural element above the LTP, (3) the applied load, (4) the inclusions grid size and (5) the diameter of the inclusions (Varaksin, et al., 2016). The calculations for the maximum applied load at the head of RIs are carried out for the Ultimate Limit State conditions as per Eurocode 7 using the Design Approach 2.

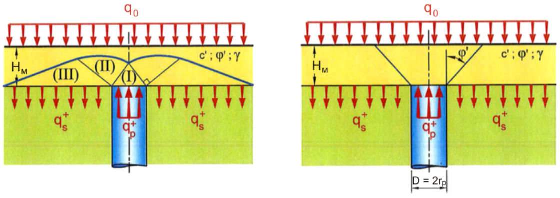

Figure 4. Behavior of LTP and failure mechanisms (ASIRI, 2012).

The combination of the conditions given by the failure mechanisms and the load limitations due to load bearing capacity of the inclusion itself can be represented in the q_s^+; q_p^+ plane. With the load taken by the soil noted as q_s^+. The graphic representation clearly shows the limiting values on the stresses at the base of the LTP considering also the load conservation equation, which is presented below:

$$\left(q_0 + \gamma_G \frac{\gamma}{\gamma_\gamma} H_M\right) \times s^2 = q_s^+ \times \left(s^2 - \pi r_p^2\right) + q_p^+ \times \pi r_p^2 \qquad (1)$$

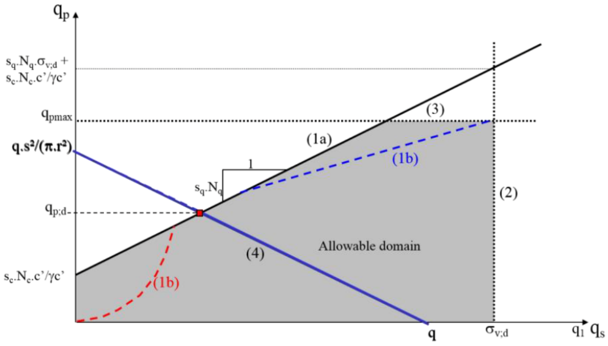

Figure 5. Domain of the allowable stresses at the base of the LTP (ASIRI, 2012).

With the predefined square grid, the maximum pressure that can be transmitted at the head of the inclusion is evaluated to be 631kN/m^2, or a load equal to 316kN, whilst the pressure that is transmitted to the surrounding weak soil is 54 kN/m^2

3.2.2 *Settlement evaluation and negative skin friction*

In weak soil deposits, for shallow depth the settlement of the soil along the rigid inclusion is greater than the vertical displacement of the rigid inclusion itself. This difference in the settlements results in the generation of a negative skin friction along the shaft of the rigid inclusion that has to be considered during the design process.

For this case study, an evaluation of the settlement of the reinforced soil with 2.5x2.5m Stone Columns is carried out using the Schmertmann's method (Schmertmann, et al., 1978) as per equation (2). The equivalent modulus of elasticity (E_s) of the soil layer is determined based on CPTU results for a foundation 1.7x1.7m, corresponding to the grid space between the inclusions

$$\delta = C_1 C_2 C_3 (q - \sigma'_{zD}) \sum \frac{I_\varepsilon H}{E_S} \qquad (2)$$

For the determination of rigid inclusion settlement, an analysis for a 35.3m pile below the LTP is performed. Due to soil conditions, a conservative approach is followed, by not considering the friction along the first 14m of the pile.

The analysis is performed based on the method proposed by (Fleming, 1992). According to this method hyperbolic functions are used to describe the individual shaft and pile base performance. In addition to them, an elastic pile shortening is also considered. The settlement analysis, results in a negative skin friction up to a length of 3m below the LTP (see Figure 6).

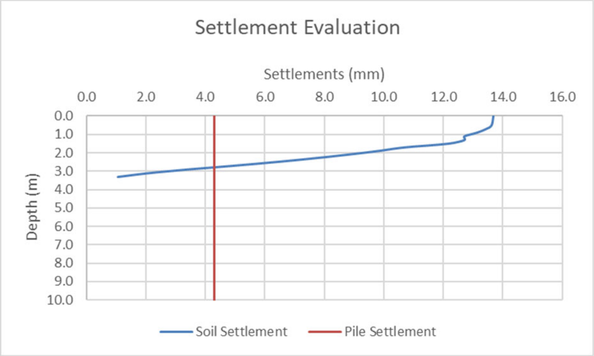

Figure 6. Determination of the negative skin friction.

3.2.3 *Bearing capacity of the Rigid Inclusion*

As mentioned in 3.2.1 an important part in limiting the load transmitted to the Rigid Inclusion is related to the bearing capacity which is calculated as the sum of side friction and base resistance.

For the purpose of this paper the bearing capacity of piles is evaluated by the analytical methods for total stress analysis (α method) or effective stress analysis (β method) based on the soil conditions. Also, these methods could be classified as drained and undrained conditions. In our case the consolidation process could be considered long term and β method is considered to evaluate the bearing capacity of the pile. According to (Burland, 1973) the resistance along the shaft can be evaluated as:

$$f_s = \beta \sigma'_z \qquad (3)$$

Several authors by the means of back calculation of load test results have proposed values of β like (Meyerhof, 1976), (Bhushan, 1982). Due to soil conditions for the first 12m the value of β is taken as 0.5 for sandy silts, and below 12m is taken as 0.2 for soft to medium clays as per (Meyerhof, 1976). The selection of β is important and the results of friction are sensitive this value, therefore a comparative study for total stress analysis using CPTU is done and verified that 0.2 is a good conservative value to be used for the design process. For foundations in clay the base resistance is evaluated as a function of the undrained resistance.

For reliability purposes an additional verification was done by calculating the bearing capacity directly from the results of CPTU tests based on the LCPC method (Bustamante & Gianeselli, 1982). The bearing capacity following the LCPC has a difference of 11% compared to classic analytic method due to the limitation of the side friction to a maximum value of 0.015MPa. Nevertheless, the results show that the bearing capacity of the pile ensures that the maximum load transmitted from the LTP is within the permissible stress envelope.

3.3 *Finite Element Analysis*

For the evaluation of the performance and serviceability checks, a detailed finite element analysis was performed using Plaxis 2D and 3D. For the case of this study the structure contribution was modelled primarily as a distributed load based on the domain of the stresses at the base of the LTP, as it can be seen in Figure 7. On the other hand, the soils material behavior is taken as Soft Soil model with C_c and C_s values given by the geological study corrected to give the same values for immediate settlement as the Hardening Soil model material. The correction was done by using the axisymmetric model for both material behavior models.

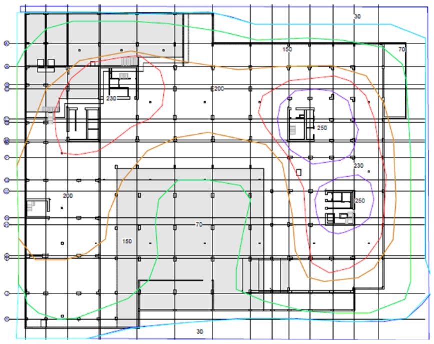

Figure 7. Domain of the allowable stresses at the base of the LTP.

Two types of models have been used:

- axisymmetric for individual capacity and 2D model calibration
- plane deformations 2D and 3D for the capacity of a section.

The 3D models are used for partial geometry of the foundations due to considerable complexity for SC modeling. Figure 8, gives the axisymmetric model that has been used for the calibration of the 2D model. In order to perform the calibration, the introduction of a transfer embedded beam element in LTP layer with higher soil friction before the RI pile embedded beam was needed. The reason is to obtain consistent settlement values for both models for each RI length. Without this calibration the equivalent embedded beam for the RI would pierce into the LTP and therefore resulting in unreasonable deformation patterns and values for the settlement. A representation of the final 2D model can be seen in Figure 9

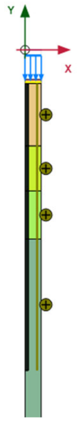

Figure 8. Axisymetrical model for RI calibration.

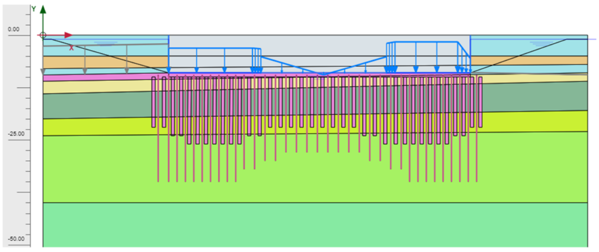

Figure 9. Final corresponding 2D Plaxis Model

In the 3D model, rigid inclusions were modeled as embedded beams while the stone columns as volumetric soil clusters with a corresponding Mohr Coulomb model for granular material having a modulus E_{ref}=80MPa. Figure 10 shows the corresponding 3D model benath the most loaded area of the foundation raft.

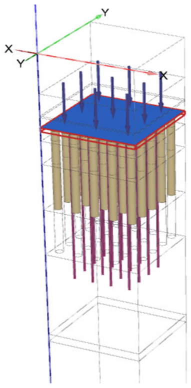

Figure 10. 3D Plaxis model for SC and RI soil reinforcement.

4 RESULTS

The determination of SC and RI lengths was done by the means of an iterative process. Several combinations were evaluated until an acceptable differential settlement of 0.8cm was achieved. In Figure 11 and Figure 12 the settlement results for the 2D and 3D model are given. In addition, ULS verifications based on the results confirm the final configuration of the RI.

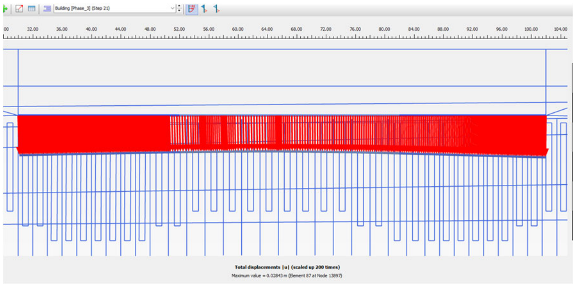

Figure 11. Settlement of the foundation raft for Plaxis 2D model.

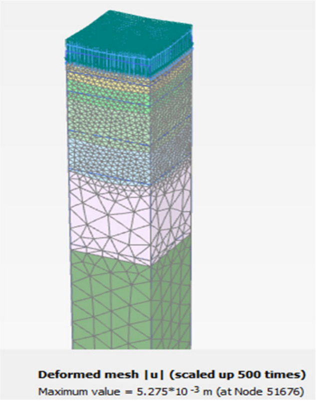

Figure 12. Settlements for Soil reinforcement SC& RI 3D model.

As previously mentioned in the paper, the liquefaction potential was studied based on the shear wave velocity (Vs) method proposed by (Andrus & Stokoe, 2004) . The analysis was performed using corrected values of soil layer stiffness in order to simulate the presence of the reinforcement. As it can be seen in Figure 13, after the introduction of the reinforcement the factor of safety (FoS) are more than 1.5.

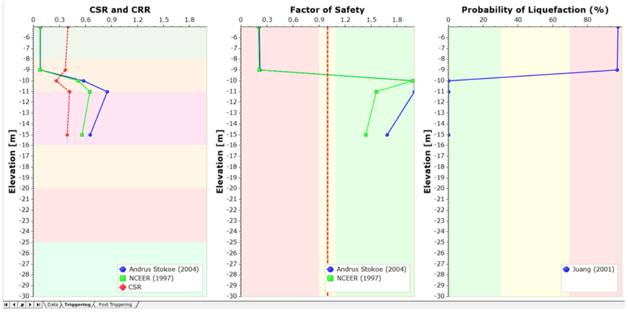

Figure 13. The results of the liquefaction analysis.

5 CONCLUSIONS

This study presents the analysis of the performance of two different ground improvement techniques: Rigid Inclusions and Stone Columns. Each one of the techniques has its own advantages and disadvantages and carefully analyzing such aspects could result in an efficient compound system that fulfills the performance criteria objects located above weak soil deposits.

The main fundamental difference is that Rigid Inclusions, in comparison to Stone Columns do not require any later confinement from the surrounding soil to achieve stability, thus providing an optimal solution to control the global and differential settlements, which constitute the main objective.

By the means of Rigid Inclusions, the load is partially transmitted to the RI and to the surrounding weak soil located beneath them. For this study roughly 74% of the total load is supported by the inclusions

Since part of the load is transmitted to the soil, the differences between the settlement of the soil and the RIs result in a negative skin friction for the first 3m of the RI below the LTP.

The LTP is reinforced with a geosynthetic layer and the load transfer mechanism is the key in the derivation of the load shared by the geosynthetic reinforcement and critical in the design of LTP.

Differential settlements, and therefore the shear, extend to the layers above the inclusion heads, between the column centerlines and the mesh centerline, meaning that some load transfer already takes place at a distance above the inclusion heads.

This scenario differentiates RI ground improvement from SC ground improvement. A stone column design generally assumes that settlements are uniform in any horizontal plane such that all planes are “equal settlement planes”.

Rigid Inclusions represent an advanced ground improvement technique characterized by several engineering advantages. These include;

- the decoupling of column axial strength from the mechanical properties of the in-situ soil, and the inherent lateral stability of the inclusions versus the stone column or other techniques.
- the possibility to achieve greater settlement reduction compared to alternative methods having an equivalent replacement ratio.
- The construction time period is significantly shorter in comparison to other widely used solutions, such as preloading with vertical drains.

Ground improvement techniques that incorporate mixing methods are employed to mitigate the risk of failure modes commonly associated with 'semi-rigid' stone columns, such as bulging or structural instability.

It can be concluded that although the implementation of these inclusions can substantially reduce total and differential settlements, residual settlement may still be considerable, particularly in soft or highly compressible soils.

Furthermore, the vibratory installation methods which are typically used for stone columns can induce remolding effects in sensitive clays, potentially resulting in a pronounced reduction in the undrained shear strength.

Such strength reduction can compromise the stability and performance of the improved ground, thereby rendering stone columns unsuitable under such geotechnical conditions. The stone columns would exhibit bulging failure under excessive loading.

Ground improvement techniques analysis and verifications are complex and require an in-depth analysis to choose among the most suitable intervention. As such, 2D and 3D numerical analysis constitute an important part of the geotechnical analysis as it was in the case of this study. By the means of complex constitutive models for the soil it is possible to depict as realistic as possible the stress-strain behavior.

The numerical analysis was performed by generating 2D and 3D models in order to have a better overview of the results. Since, the behavior of Rigid Inclusions and Stone Columns is of a 3D nature due to the complex interaction between them and the surrounding soil when switching to a 2D model a calibration process is necessary. In this study, the calibration was done by using a reference axisymmetric model.

In similar situations, as recommended by the literature and emphasized in the Eurocodes, the use of static load tests is fundamental in capturing the behavior of the piles under the specific loading conditions, in the absence of such tests careful considerations should be taking during the modelling process.

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