Performance Evaluation of Rigid Inclusion Foundations in the Reduction of Settlements

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Abstract. In this study, numerical modeling is used to evaluate the performance of rigid inclusion foundations for settlement control considering the characteristic soils of the city of Brasília, Federal District, Brazil. Two- and three-dimensional (2D and 3D) PLAXIS software models were used considering the Hardening Soil constitutive model parameters previously obtained, calibrated and validated by the authors. First, the general concepts regarding systems with rigid inclusions are presented. Then, parametric 2D axisymmetric numerical modeling is shown, where the spacing between inclusions, the height of the distribution layer and the soil conditions were varied. The load transfer mechanisms were analyzed, including the performance of rigid inclusions for settlement control. Finally, 3D modeling was performed with the information from a real project located in the Federal District. In the 3D modeling, the performance of the rigid inclusion foundation was compared with that of a slab foundation solution; then, the obtained settlements and angular distortions were compared with the serviceability limit states indicated in the literature. The results show that for the analyzed conditions, rigid inclusion foundations can be considered to be reliable foundation solutions. However, feedback from instrumentation cases in the city of Brasília is required to further validate the design considerations.

Keywords: 3D model, angular distortion, numerical modeling, rigid inclusions, settlement control performance.

1. Introduction

The region of the Federal District of Brazil is covered by a detritus-laterite soil mantle from the Tertiary-Quaternary age called “porous clay”. This superficial clay layer presents a porous and highly unstable structure, with high void ratio and low shear strength resistance; therefore, deep foundations are widely used. In this study, numerical modeling is used to evaluate the performance of rigid inclusion foundations for settlement control.

Inclusions have been commonly used all over the world as foundation solutions, primarily for road and railway embankments (Zanziger & Gartung 2002; Quigley et al., 2003; Wood 2003; Almeida et al., 2011; Okuy et al., 2014; Fonseca & Palmeira, 2018). Since the late twentieth century, in North America (López et al., 1999; Santoyo & Ovando, 2006; Rodríguez, 2001, 2010; Rodríguez & Auvinet, 2006; Auvinet & Rodríguez, 2006) and in Europe (Combarieu, 1990; Pecker, 2004; Simon & Schlosser, 2006; ASIRI, 2011; Briançon et al., 2015), this solution has been studied and employed for settlement control and to lower the costs of deep foundations for buildings on difficult soil conditions. Currently, the use of inclusions is one of the most employed deep foundation techniques under these conditions due to good performance (Briançon et al., 2015) and low cost compared to other solutions (Rodríguez & Auvinet, 2006). Therefore, the objective of this paper is to evaluate the use of this type of foundation for situations involving superficial layers of collapsible porous soils, such as those present in the stratigraphy of the city of Brasilia in the Federal District of Brazil.

The settlement reduction obtained with the use of rigid inclusions is due to the transfer of a significant load supported by the soil to these elements.

In this study, the performance of rigid inclusion foundations in settlement control is evaluated through numerical modeling. Two- and three-dimensional (2D and 3D) PLAXIS software models were used considering the mechanical parameters of the characteristic soils of the city of Brasilia, obtained, calibrated and validated (Rebolledo et al., 2019) for the Hardening Soil (HS) model based on laboratory and field test results obtained in previous studies conducted in the Experimental Field of the University of Brasília (CEGUnB).

To evaluate the influence of the main geometric variables of the foundation on settlement control, parametric 2D axisymmetric modeling was performed. The analyses were performed using the soil stratigraphy and properties of the CEGUnB for natural moisture conditions and with the first 3.5 m of the saturated soil. The latter condition was investigated to consider the significant increase in the soil moisture in this active zone during the rainy season.

Additionally, 3D modeling was done with information from a real project located in the Federal District. The
performance of the rigid inclusion foundation was compared with that of a single-slab foundation solution; then, the settlements and angular distortions obtained were compared with the serviceability limit states indicated in the literature.

2. Rigid Inclusions

2.1. Main characteristics

A foundation with rigid inclusions has five components that interact with each other, as shown in Fig. 1: the foundation (slab or footing), the distribution layer or load transfer platform, the rigid inclusions, the column caps (optional), and the surrounding soil. A rigid inclusion foundation solution should incorporate all of these components.

Commonly, in embankment projects, the distribution layer is composed of granular soils reinforced with geo-synthetics, but for building foundation projects, the distribution layer is commonly composed of compacted soils. To increase the shear strength and stiffness of the distribution layer, the material of this layer can be mixed with cement, lime or another chemical or physical additives.

Inclusions are cylindrical or prismatic elements with no direct contact with the foundation (slab or footing) that can be placed in the ground using different techniques, such as bored piles, jacked piles, precast-concrete pile driving, steel pipe pile driving, micropiles, continuous flight augers, low-pressure grouting, jet grouting, and stone columns; i.e., any type of deep foundation that has a rigidity considerably greater than that of the ground the foundation reinforces. According to the Soil Improvements through the use of Rigid Inclusions (ASIRI) National Project (2011), the concept of rigid inclusions is based on the hypothesis that the structural stability of an element is guaranteed without the lateral confinement of the soil.

In this study, we assumed that the rigid inclusions were constructed using the continuous flight auger technique because, compared with other techniques, the flight auger technique produces small disturbances in the excavated soil, has relatively high performance and is widely used in Brazil; however, any of the methods mentioned above can be used.

2.2. Load transfer mechanism

The settlement reduction obtained with the use of rigid inclusions is due to the transfer of a significant load supported by the soil to these elements. A complex interaction develops between the reinforced soil, the inclusions and the distribution layer, as shown in Fig. 2a. To understand the inclusion-soil and soil-inclusion load transfer, we can use concepts similar to those accepted for piles, as shown in Fig. 2b (Vesic, 1970; Rodríguez, 2010; Combrieu, 1990; Rodríguez et al., 2015; Briançon et al., 2015). Initially (Fig. 2a), the distribution layer transfers the load of the structure to the inclusion cap (q) and to the reinforced soil (qs); then, the reinforced soil transfers the load to the upper inclusion shaft as negative skin friction (f'); finally, the inclusion transfers the load through the inclusion tip (qp) and through the lower inclusion shaft as positive skin friction (f'’). Both negative and positive friction are separated by a neutral point (z0), where no relative displacement occurs between the inclusion and soil.

The distribution layer or load transfer platform is intended to transfer most of the load from the structure to the rigid inclusions, q. The geotechnical and geometric characteristics of this layer influence the efficiency of the transfer because these characteristics can increase the stresses at the heads of the inclusions (qs, Fig. 2a) and reduce the stresses in the soil to be reinforced (qs). Additionally, the distribution layer absorbs the loads transferred by the inclusion heads (column caps or top of the piles, according to the case), which prevents the inclusion heads from punching the foundation slab, and homogenizes the settlements, ensuring efficient foundation performance. The parameters that have the most influence on this efficiency are the friction angle of the compacted material, the spacing and head diameter of the inclusions and the thickness of the distribution layer (BSI, 2010). According to Fonseca & Palmeira (2018), to obtain the load transfer efficiency of geosynthetic reinforced piled embankments, analytical methods are commonly used, but the results obtained are significantly different. Generally, methods based on arching stresses such as the British Standard BS 8006 (BSI, 2010) showed satisfactory results.

According to Rodríguez (2010), the use of rigid inclusion systems is more economical than the use of other deep foundation solutions, mainly because:

1) inclusions do not require steel reinforcement (Fig. 2) because only small compressive loads develop in the element,
2) the foundation slab or footing does not require steel reinforcement (Fig. 2) to transfer load to the inclusions, since the top of the inclusions are located at a depth sufficient to prevent the inclusions from reacting as point load to the foundation slab or footing, and
3) the magnitudes of the loads transmitted by the inclusions are low, so a concrete with low resistance can com-
monly be used (compressive strength of the order of 10 MPa).

3. Aspects of the Numerical Modeling

3.1. Stratigraphy used in the modeling

For this study, the stratigraphy of CEGUnB, shown in Fig. 3, was considered. This program provides valuable geotechnical information obtained from surveys, in situ tests, laboratory tests and loading tests on superficial and deep foundations (Pérez, 2017; Jardim, 1998; Sales, 2000; Guimarães, 2002; Mota, 2003; Coelho, 2013; Sales et al., 2015). According to this information and the tropical soil profiles proposed by Cruz (1987) and Cardoso (2002), Rebolledo et al. (2019) defined the typical stratigraphic profile of the CEGUnB, as shown in Fig. 3.

For the numerical simulation of soil behavior based on the information from the CEGUnB, the HS model (Schanz et al., 1999; Brinkgreve et al., 2014, 2015) of the software PLAXIS was used. The HS model is one of the most complete constitutive models of PLAXIS and is capable of:

1) calculating the total strains using a stress-dependent stiffness that is different for loading and unloading/reloading conditions, and

2) modeling irreversible strains due to primary deviatoric loading (shear hardening) and modeling irreversible plastic strains due to primary compression under oedometric and isotropic loading (compression hardening).

Rebolledo et al. (2019) developed a methodology to obtain, adjust and validate the mechanical parameters of characteristic soils of the city of Brasília for the HS model, making use of laboratory and field test results obtained in previous studies conducted in the CEGUnB. The methodol-
ogy presented began with the evaluation of the strength and compressibility parameters of triaxial CU tests (with isotropic and anisotropic consolidation) and one-dimensional consolidation tests, respectively (Guimarães, 2002). Then, the parameters obtained for the HS model were calibrated through the explicit numerical modeling of the tests using the finite element method (FEM) and the SoilTest module of the PLAXIS software. Based on the evaluation and calibration of these parameters and the proposed soil profile (Fig. 3), a geotechnical model based on the natural moisture state of the CEGUnB was proposed for the HS model, shown in Table 1. This geotechnical model was validated through numerical modeling of the load testing of footings and piles conducted in the CEGUnB (Sales, 2000; Guimarães, 2002).

Using the same methodology and with the triaxial and consolidation tests performed by Guimarães (2002), Pérez (2017) determined the HS model parameters for the first 3.5 m in depth of porous clay from Brasília in the saturated moisture state, as shown in Table 2.

3.2. Definition of the problem geometry

To define the diameter of the inclusions, the studies by ASIRI (2011) and Guimarães (2002) were considered. According to ASIRI (2011), for nonreinforced concrete inclusions constructed on site that do not rely on a micropile-type technique, the typical minimum diameter is 25 cm. According to Guimarães (2002), mechanically excavated piles, which are of great use and versatility in the Federal District of Brazil, can reach 25 m in depth, with diameters varying from 30 to 110 cm.

In this study, the inclusions were modeled with a diameter of 30 cm and placed at a depth of 9.5 m, thus penetrating 1 m into the noncollapsible soil layer to ensure the load transfer from the inclusions to a more competent stratum. For the load distribution layer, thicknesses ranging from 0.5 to 2.5 m were considered, according to the recommendations of ASIRI (2011). The slab was considered flexible with a thickness of 0.20 m with the properties described in 3.3.

According to ASIRI (2011), the minimum center-to-center spacing between inclusions is three times the diameter of the element (3D) if the inclusions are constructed on a site with minimum soil disturbance. Hence, the minimum inclusion spacing considered in the parametric analysis was 1 m (≈ 3D).

Table 1 - Geotechnical model proposed by the CEGUnB for the natural moisture state for the HS model (Rebolledo et al., 2019).

| Parameters | Layer number | Depth (m) | 1 | 2 | 3 | 4 | 5 | 6 |
|------------|--------------|-----------|---|---|---|---|---|---|
| Porous sandy clay | | 0 - 1.5 | 13.1 | 12.8 | 13.9 | 14.3 | 16.0 | 18.2 |
| Lateritic residual soil | | 1.5 - 3.5 | 5 | 5 | 5 | 20 | 75 | 20 |
| Saprolitic soil | | 3.5 - 5.0 | 25 | 25 | 26 | 32 | 20 | 22 |
| | | 5.0 - 7.0 | 0 | 0 | 0 | 0 | 0 | 0 |
| | | 7.0 - 8.5 | 3.2 | 2.5 | 4.0 | 12.0 | 13.2 | 12.2 |
| | | 8.5 - 20.0 | 4.9 | 1.45 | 2.2 | 6.9 | 7.0 | 5.7 |
| | | | 14.0 | 14.0 | 36.9 | 37.5 | 54.0 | 54.0 |
| | | | 0.5 | 0.5 | 0.5 | 0.5 | 0.5 | 0.7 |
| | | | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 |
| | | | 100 | 100 | 100 | 100 | 100 | 100 |
| | | | 0.8 | 0.8 | 0.9 | 0.9 | 0.9 | 0.8 |
| | | | 65.7 | 31.8 | 0 | 31.4 | 0 | 0 |
| | | | 0.58 | 0.58 | 0.56 | 0.47 | 0.66 | 0.63 |
| | | | 1.37 | 0.77 | 0.56 | 0.56 | 0.66 | 0.63 |

\(\gamma\): unit weight of moist soil, \(c’\) and \(\phi’\): the effective shear strength parameters, \(\psi\): dilatancy angle, \(E_{50}^{ref}\): the reference secant stiffness modulus for the drained triaxial test, \(E_{oed}^{ref}\): the reference tangent stiffness modulus for oedometric loading, \(E_{ur}^{ref}\): the reference stiffness modulus for unloading and reloading conditions, \(m\): the exponent that defines the strain dependence of the stress state, \(\nu_{ur}\): unloading/reloading Poisson’s ratio, \(p_{0}^{ref}\): the reference isotropic stress, \(R_f\): the failure ratio, POP: the preoverburden pressure, \(K_0^{ur}\): the coefficient of the earth pressure at rest for normal consolidation, and \(K_0\): coefficient of earth pressure at rest.
3.3. Properties considered for concrete elements

For the modeling of slab and inclusions, both in concrete, the linear elastic constitutive model was assumed because the stiffness and the strength of this material are considerably higher than those of the reinforced soil. Table 3 presents the parameters of the constitutive model adopted for each concrete element.

The concrete Young's modulus was calculated according to the equation proposed in Brazilian standard NBR 6118 (ABNT, 2014) as a function of the strength characteristics of the concrete subjected to simple compression. Therefore, a compressive strength of 20 MPa was assumed for the slab and 10 MPa for the inclusions.

As proposed in NBR 6118 (ABNT, 2014) and by ASIRI (2011), a Poisson's ratio for concrete equal to 0.2 was adopted for both elements.

3.4. Properties considered in the distribution layer

For the distribution layer (improved soil), the Mohr-Coulomb model was adopted. Research performed by Otálvaro (2013) at the Geotechnics Laboratory of the University of Brasília (UnB) provided the estimates of the parameters for tropical soil improved by compaction that were used in this study, and these parameters are presented in Table 4. The compacted tropical soil, of the laterite type and highly weathered, was collected in the Brazilian Cerrado region in the city of Brasília. The material was classified as ML (low plasticity silt) according to the Unified Soil Classification System (USCS). The γ value was obtained from the results of Proctor Standard testing (compaction energy of 600 kN-m/m3), from values of \( w_{opt} = 24 \% \) and \( \gamma_{max} = 15 \text{ kN/m}^3 \). Parameters \( E, \phi' \) and \( c' \) were obtained from CD (consolidated-drained) triaxial tests performed on the same compacted soil. Echevarría (2006) obtained similar parameters for numerical simulation of tropical porous compacted soil.

3.5. Analysis steps

The analysis steps of the numerical models were established according to the construction process of foundations with rigid inclusions. Initially, five steps were defined: Step I consists of the excavation of the natural soil, upon which the distribution layer will be built; in Step II, the rigid inclusions are installed; in Step III, the distribution layer is built; in Step IV, the slab is built; in Step V, the load is applied to the foundation.

4. Parametric Modeling of Rigid Inclusions

4.1. General considerations

An infinite group of rigid inclusions over an infinitely large distribution layer and slab was considered (Fig. 4a). The area of influence, or influence cell, of each internal inclusion (Schlosser et al., 1984) is hexagonal but can be idealized as a circular unit cell (Rodríguez et al., 2015); the problem then becomes axisymmetric (Fig. 4b). The radius \( R \) of this area, corresponding to the radius of a finite element axisymmetric mesh, is approximately equal to half the spacing between inclusions \( \left( \frac{S}{2} \right) \). For the inclusions located in the periphery, the conditions are no longer axisymmetric, and the results obtained by such a model are less accurate.

### Table 2 - Geotechnical model proposed for the CEGUnB for the first 3.5 m in depth in the saturated moisture state for the HS model (modified from Pérez 2017).

| Parameters | Layer number |
|------------|-------------|
|            | 1           |
|            | 2           |
| Depth (m)  | 0-1.5       |
|            | 1.5-3.5     |
| \( \gamma \) (kN/m³) | 16.5       |
|            | 16.4       |
| \( c \) (kPa) | 26         |
|            | 26         |
| \( \psi \) (°) | 0          |
|            | 0          |
| \( E_{w}'' \) (MPa) | 2.2         |
|            | 2.1         |
| \( E_{m}'' \) (MPa) | 0.96       |
|            | 0.83       |
| \( E_{m}'' \) (MPa) | 13.0       |
|            | 13.0       |
| \( m \) | 0.65        |
|            | 0.80       |
| \( v_{w}'' \) | 0.2         |
|            | 0.2         |
| \( p'' \) (kPa) | 50         |
|            | 50         |
| \( R_{f} \) | 0.75       |
|            | 0.75       |
| POP (kPa) | 16.1       |
|            | 6.59       |
| \( K_{nf} '' \) | 0.56       |
|            | 0.56       |
| \( K_{n} '' \) | 0.75       |
|            | 0.75       |

### Table 3 - Parameters of the slab and the rigid inclusions.

| Parameters              | Slab | Inclusions |
|-------------------------|------|------------|
| Unit weight of concrete, \( \gamma \) (kN/m³) | 24.0  | 23.0       |
| Young’s modulus of concrete, \( E \) (GPa) | 25.0  | 17.7       |
| Normal stiffness, \( EA \) (kN/m) | \( 5.0 \times 10^5 \) | - |
| Bending stiffness, \( EI \) (kN/m²/m) | \( 1.67 \times 10^7 \) | - |
| Poisson’s ratio, \( \nu \) | 0.20  | 0.20       |

### Table 4 - Estimated parameters for soil improved by compaction.

| Parameter               | Value |
|-------------------------|-------|
| Unit weight, \( \gamma \) (kN/m³) | 18.6  |
| Young’s modulus, \( E \) (MPa) | 60    |
| Cohesion, \( c' \) (kPa) | 80    |
| Friction angle, \( \phi' \) (°) | 38    |
| Poisson’s ratio, \( \nu \) | 0.25  |
representative. However, according to Schlesser et al. (1984), for large groups of inclusions where the boundary conditions become less important, the influence cell model can capture the essence of the physical phenomena.

The centerline of the axisymmetric model coincides with the axis of the rigid inclusion. The right boundary was placed halfway between the inclusions. The lower boundary was established at a depth of 20 m, beyond which the $N_{ip}$ was larger than 40 blows, and the soil was classified as very compact, according to the Brazilian standard NBR 6484 (ABNT, 2001). Therefore, the lower boundary was considered 10.5 m below the tip of the inclusion.

The parametric modeling of the rigid inclusions was carried out using PLAXIS 2D software (Brinkgreve et al., 2014). The problem was discretized using a finite element mesh with more than 6,700 15-node triangular elements. A mesh densification was considered around the inclusions. The slab was modeled using 5-node beam elements. The lateral boundaries were fixed in the horizontal direction, and the bottom boundary was fixed in both directions. The sensitivity studies showed that the mesh was dense enough to provide accurate results. To adequately consider the interaction between the inclusion surface and the soil, five pairs of node interface elements were added.

### 4.2. Cases analyzed

To evaluate the performance of the rigid inclusion foundation for settlement control, the case of a single-slab foundation (without inclusions) was analyzed, and the results obtained from both cases were compared. Parametric analyses were performed considering the stratigraphy previously presented in two situations: with the soil at natural moisture conditions (Case 1) and with the first 3.5 m of the soil saturated (Case 2). The influence of two parameters was considered: the spacing between inclusions (1, 1.5, 2, 2.5, and 3 m) and the distribution layer thickness (0.5, 1, 1.5, 2, and 2.5 m). In each model, the results for loads on the slab ($q$) equal to 10, 20, 40, 60, 80, 100 and 120 kPa were obtained. The base of inclusion was placed at a depth of 9.5 m, and the slab thickness was 0.2 m.

### 4.3. Loads developed on the inclusions

As part of the results, Fig. 5a shows the axial load developed in the inclusions for spacing between elements ($S$) of 1 to 3 m, distribution layer thickness ($H_{dL}$) equals to 1.5 m, and load ($q_0$) of 60 kPa. As explained previously (Fig. 2a), the model highlights the development of the cap and tip forces and those due to negative and positive skin friction. Similar behavior was observed by Briançon et al. (2015) in instrumented rigid inclusions that were part of the foundation of an industrial structure. The author notes that the shape of the strain curve inside both instrumented rigid inclusions indicates an evolution of the strain during the construction of the building, indicating the interval where the neutral point between the negative and positive friction was located.

Figure 5a shows that the magnitudes of the cap, negative friction and positive friction loads significantly increase when $S$ increases. The load on tip does not increase with increasing $S$, probably because the tip bearing capacity has been reached at this point; therefore, the inclusions respond mainly by lateral friction, as in the case of friction piles.

Figure 5b shows the axial load in the inclusion for different $H_{dL}$ values; when $H_{dL}$ increases, the maximum axial load increases because the unit weight of the distribution layer is 18.6 kN/m$^2$ and that of the substituted superficial soil is approximately 13 kN/m$^2$, which gives an overload of 5.6 kN/m$^2$ for each meter of thickness. Then, when $H_{dL}$ increases, the overload is transferred to the inclusions mainly by the caps, with the negative skin friction almost constant; hence, the positive skin friction increases. The increase in positive skin friction means that additional settlements can occur. Due to the above and for economic reasons, the thickness of the distribution layer cannot be large; this layer has to enable partial load transfer to the inclusion cap, surface settlement reduction and homogenization, thereby guaranteeing the durability and functionality of the surface structure.

Figure 6 shows that when the first 3.5 m of the soil saturate, the load on the inclusion cap increases, the negative friction load decreases, and both positive skin friction and tip load remain constant. These results indicate that saturated soil is less resistant and more compressible than the soil at natural moisture condition and thus is not able to transmit the same load to the shaft of the inclusion (negative friction); therefore, the load transferred by the distribution layer to the inclusion tip increases, and no additional
load is transferred to the reinforced soil, which means that inclusions can work properly for both natural and saturated conditions.

4.4. Settlement control

The performance of the rigid inclusion foundation in controlling surface settlement was determined using the proposed settlement reduction factor (SRF):

\[
SRF = 1 - \frac{\delta'_s}{\delta}
\]  

(1)

where \(\delta'_s\) is the settlement of the soil reinforced by rigid inclusions and \(\delta\) is the settlement of the soil without reinforcement, both obtained at the center of the slab. When \(SRF = 1\), the settlement is fully reduced, and the performance of the inclusion system is at the maximum; when \(SRF = 0\), the settlement reduction is null, and the performance of the system is at the minimum.

The analyses were performed for several values of spacing between inclusions (S), distribution layer thicknesses (HDL), and load levels (q0), considering cases of stratigraphy with natural moisture (Case 1) and with the first 3.5 m of the soil saturated (Case 2).

Figure 7 shows graphs of settlement \(\delta'_s\) versus \(q_0\) at the center of the slab, for \(HDL = 1.5\) m, for different S values and for Cases 1 and 2 (Figs. 7a and 7b, respectively). The results for \(\delta'_s\) are presented by continuous lines, and those for \(\delta\) are presented by dashed lines.
For Case 1, \( \delta' \) varies from 2.7 to 40.2 cm and for Case 2, from 4.2 to 57.4 cm; an increment of 43 to 56% of the total settlement is obtained from one case to the other. When inclusions are added for both cases (Figs. 7a and 7b), very similar values of \( \delta' \) are obtained for the different \( S \) values; a maximum difference of 16% is calculated for \( q_0 = 120 \) kPa and \( S = 3 \) m. This result means that, as shown in Fig. 8, the performance of rigid inclusions for settlement control (SRF values) significantly increase from Case 1 (Fig. 8a) to Case 2 (Fig. 8b). As mentioned before, rigid inclusion foundations are more efficient when the soil to be reinforced is more compressible and less resistant because the distribution layer transfers more load to the head of the element and less load to the reinforced soil.

Figures 7 and 8 show that very similar results are obtained for \( S = 1.0 \) m and 1.5 m. For \( S \) values greater than 1.5 m, the SRF passes from a maximum value of 0.6 to 0.35 for Case 1 and from 0.75 to 0.48 for Case 2. On the other hand, as demonstrated in Fig. 9, when \( S \) is held constant (\( S = 1.5 \) m) the settlement reduction achieved with the inclusions for different \( H_{al} \) values is practically constant for \( q_0 \) values greater than 40 kPa. For \( q_0 < 40 \) kPa, the overload generated by the distribution layer influences the inclusion performance for \( H_{al} > 1.5 \) m; inclusive negative SRF values can be obtained.

The axisymmetric analyses show high performance for the rigid inclusions regarding settlement control for the two cases analyzed. According to the research developed by Briançon et al. (2015) related to the monitoring and numerical investigation of rigid inclusion-reinforced industrial buildings, the settlement measured both at the pile head and the soil surface showed that the supporting system can significantly reduce settlements.

According to the results obtained, a thickness of 1.5 m for the distribution layer and an \( S \) value close to 1.5 m were proposed for the following 3D analyses.

5. 3D Modeling

5.1. General considerations

The 3D modeling was based on a real project located in the Meireles Sector in the city of Santa Maria, DF, as presented by Castillo (2013). Lot 401, chosen for the study, encompasses three types of residential blocks with pile group and single-pile foundations; the Type II block was chosen for these analyses. Type II block consists of four floors (ground floor plus three decks), with two two-bedroom...
The original project foundation was formed by a total of 32 bored concrete piles with diameters from 30 to 50 cm and depths of 12 to 17 m, arranged as shown in Fig. 10.

For practical purposes and as a demonstrative example only, the stratigraphy and properties considered for this analysis were those described in section 3.1.

Figure 10 shows the floor plan for the locations of the columns and the load acting on each (serviceability state load combination). To simplify the simulation, the total load imposed by the superstructure (9,740 kN) was obtained through the sum of loads of all columns (Fig. 10). This total load was divided by the total area of the slab (202.4 m$^2$) and by the number of floors (five), and then, a distributed load of the same value (9.7 kN/m$^2$) was applied directly to each slab.

To obtain the magnitudes of the settlements that needed to be minimized and to evaluate the performance of the rigid inclusion system, the foundation was initially modeled considering only a single-slab foundation. Subsequently, analysis was performed considering the rigid inclusion system.

The numerical simulation presented below was performed using PLAXIS 3D (Brinkgreve et al., 2015).

### 5.2. Modeling of the foundation with a single-slab foundation

As shown in Fig. 11, for modeling a single-slab foundation, the symmetry conditions of the problem were considered. The medium was discretized by a finite element mesh with more than 112,200 10-node tetrahedral elements, the foundation and floor slabs were discretized by

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**Figure 9** - Settlement reduction factor (SRF) as a function of $q_0$, for $S = 1.5$ m, different $H_{dl}$ values, and a) Case 1: stratigraphy with natural moisture conditions and b) Case 2: stratigraphy with the first 3.5 m of the soil saturated.

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**Figure 10** - Floor plan for locations of piles and columns in the type II block (Castillo, 2013).
six-node triangular plate elements (0.2 m thick) and the columns with three-node beam elements. The densification of the mesh around (14 m along the x-axis and 10 m along the y-axis) and under (12 m in z-axis) the footing was considered. The lateral boundary conditions were fixed in the horizontal direction, and the bottom boundary conditions were fixed in both directions. The sensitivity analyses showed that the mesh density was sufficient for accurate results.

5.4. Analysis of the results

According to the Brazilian standard NBR 6122 (ABNT, 2010), the maximum allowable displacements supported by the structure, without prejudice to the serviceability limit states, shall comply with the requirements of NBR 8681 (ABNT, 2003). These displacements, both in absolute terms (total settlements) and in relative terms (differential settlements), must be established by the designer according to the importance of the construction work. As shown in Table 5, several publications propose reference values; in this study, the admissible settlement value adopted was 50 mm.

Figures 14 and 15 show the settlements obtained in the slab for Cases 1 and 2, respectively, and Table 6 shows the maximum values. For Case 1, the maximum total settlement obtained was 127.8 mm, and that for Case 2 was 318 mm. The foundation does not meet the maximum admissible settlement criterion, requiring the use of rigid inclusions to minimize the total surface settlements. On the other hand, Fig. 16 shows the settlements obtained for the foundation with rigid inclusions, considering: a) Case 1 and inclusions at a depth of 9.5 m, b) Case 2 and inclusions at a depth of 9.5 m, c) Case 1 and inclusions at a depth of 12 m, and d) Case 2 and inclusions at a depth of 12 m. Table 6 in-
includes the maximum values obtained for all the cases analyzed.

For Case 1 (Fig. 16a), the use of rigid inclusions placed at a depth of 9.5 m yields a settlement reduction of 62.5% compared with the foundation with the single-slab, for inclusions placed at 12 m (Fig. 16c), the reduction was 71.8%, and for Case 2 (9.5 m in depth, Fig. 16b, and 12 m in depth, Fig. 16d), the settlement reductions were 83.0 and 87.1%, respectively.

To control the angular distortion, a limit of 1/500 was adopted, according to those proposed by Bjerrum (1963), as shown in Table 7. The angular distortion value was calculated from Figs. 14, 15 and 16 as the ratio of the differential settlement between two neighboring columns (Fig. 12) and the distance between axes.

Table 8 summarizes the maximum angular distortion for all the cases analyzed. For the single-slab foundation in Case 1 (Fig. 14), the maximum angular distortion was 1/396, and that for Case 2 (Fig. 15) was 1/1000. According to these results, the single-slab foundation does not meet the angular distortion limit in Case 1, and therefore, the use of rigid inclusions is required. With rigid inclusions, as

Figure 12 - a) Floor plan of the original project highlighting the area considered in the numerical modeling and b) details of the distribution of the rigid inclusions.
shown in Table 8 and Fig. 16, all the cases analyzed meet the maximum angular distortion limit (1/500).

In general, for the cases analyzed, the rigid inclusions perform well, reducing the settlement by more than 80% and homogenizing the vertical displacements at tolerable values meeting with the serviceability limit states.

Based on measurements performed one year after construction at superficial benchmarks located at eight points around the perimeter of three five-floor buildings with rigid inclusion foundation systems in soft soils, Rodri-

**Table 5 - Maximum allowable settlements (Castillo, 2013).**

| Publication | \( \delta_{\text{max}} \) (mm) |
|-------------|-------------------------------|
| Eurocode 7  | < 50                          |
| Eurocode 1 (1993) | 50                          |
| Teixeira & Godoy (1998) | 90                          |
| Burland et al. (1977) | 65-100                      |
| Bowles (1977) | 64                           |
| Terzaghi & Peck (1967) | 50                          |
| Skempton & MacDonald (1956) | 90                          |

**Table 6 - Maximum total settlements (mm).**

|                | Single-slab | Inclusions up to 9.5 m | Inclusions up to 12 m |
|----------------|------------|------------------------|------------------------|
| Case 1         | 128        | 48                     | 36                     |
| Case 2         | 318        | 54                     | 41                     |

**Table 7 - Values of angular distortion limits proposed by Bjerrum (1963).**

| Damage category                                           | \( \eta \) |
|-----------------------------------------------------------|------------|
| Danger to settlement-sensitive machines                   | 1/750      |
| Danger to landmarks with diagonal cracks                  | 1/600      |
| Safe limit for cracks not occurring in buildings*         | 1/500      |
| First cracks on walls                                     | 1/300      |
| Problems with overhead crane                              | 1/300      |
| Slant of tall buildings becomes visible                    | 1/250      |
| Considerable cracking of brick walls and panels           | 1/150      |
| Risk of structural damage to general buildings            | 1/150      |
| Safe boundary for flexible brick walls, L/H > 4 *          | 1/150      |

*The safe limits include a safety factor.

**Table 8 - Maximum angular distortion values.**

|                | Slab-only | Inclusions up to 9.5 m | Inclusions up to 12 m |
|----------------|-----------|------------------------|------------------------|
| Case 1         | 1/396     | 1/616                  | 1/821                  |
| Case 2         | 1/1000    | 1/616                  | 1/593                  |
guez & Auvinet (2006) concluded that the mean differential settlement was approximately 0.5 cm, and hence, the behavior of the buildings was adequate during that period.

6. Conclusions

In this study, numerical modeling was used to evaluate the performance of rigid inclusion foundations for settlement control considering the characteristic soils of the city of Brasília in the Federal District of Brazil. PLAXIS 2D and 3D software was used considering the Hardening Soil constitutive model parameters previously obtained, calibrated and validated by the authors.

The settlement reduction obtained with the use of rigid inclusions is due to the transfer of a significant load from the soil to these elements. Complex interactions develop between the reinforced soil, the inclusions and the distribution layer.

To evaluate the performance of a rigid inclusion foundation for settlement control, the case of a single-slab foundation (without inclusions) was analyzed, and the results obtained for both cases were compared.

The numerical modeling highlights the development of forces in the cap and inclusion tip, and those due to negative and positive skin friction. Similar behavior was observed in instrumented rigid inclusions that were part of the foundation of an industrial structure.

The magnitudes of the cap, negative friction and positive friction loads increased significantly when the inclusion spacing increased. However, the tip load does not increase in this case, probably because the tip bearing capacity had been reached at this point; therefore, for the cases analyzed, the inclusions mainly responded by lateral friction, as in the case of friction piles.

When the thickness of the distribution layer increased, an overload was generated and was transferred to the inclusion mainly by the cap, while the negative skin friction was almost constant, and hence, the positive skin friction increased. Thus, additional settlements could occur. Due to the above results and economic reasons, the distribution layer cannot be large in thickness; this layer has to enable: i) partial load transfer to the cap, ii) surface settlement reduction, and iii) homogenization; thereby guaranteeing the durability and functionality of the surface structure.

When the first 3.5 m of the soil was considered to be saturated, the load on the inclusion cap increased, the negative skin friction load decreased, and both positive skin friction and tip load remained almost constant. These results indicated that saturated soil is less resistant and more compressible than soil at natural moisture conditions and thus is not able to bear the same load to the shaft of the inclusion (negative friction); therefore, the load transferred by the distribution layer to the inclusion tip increases, and no additional load is transmitted to the reinforced soil. This effect means that inclusions can work properly for both natural and saturated conditions without a significant increase in settlement.

Figure 16 - Vertical displacements for all the cases analyzed with rigid inclusion foundations.
The performance of the rigid inclusion foundation for controlling surface settlement was determined using the proposed Settlement Reduction Factor. Rigid inclusion foundations are more efficient when the soil to be reinforced is more compressible and less resistant because the distribution layer transfers more load to the head of the element and less load to the reinforced soil.

In general, for the 3D cases analyzed, the rigid inclusions perform well, reducing the settlement by more than 80% and homogenizing the vertical displacements at tolerable values that meet the serviceability limit states. However, feedback from instrumentation cases in the city of Brasília is required to further validate the design considerations.

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