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Foundation-structure interaction on high-rise buildings

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Article

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Abstract
This article addresses the importance of considering foundation-structure interaction in the design of high-rise buildings. Embedding the behavior of the foundation in the analysis of structures is fundamental to simulate the real deformability of these constructions. On the foundation design side, the addition of structure stiffness implies in the reduction of maximum settlement and angular distortions. On the structural dimensioning side, the consideration of the foundation settlement modifies the flexibility of the structure by changing internal efforts in several parts, which is against safety in many cases. The study of a building with 50 floors is presented, as well as the report of 13 cases of construction where the settlements measurements reached more than 10 times the results of the load test of the isolated element, illustrating the effect of the interaction between different foundation elements. There was a considerable increase in the loads of corner columns and an increase in the overall building's stability. The γ_z that is a parameter associated to second-order effects increased exponentially with the increase in the building's non-verticality.

1. Introduction

The structure and the foundation design of buildings are generally calculated separately, where the structure designer calculates the loads that reach the foundations of these structures without considering the soil behavior, and the foundation designer receives these loads and calculates the settlements without considering the building's stiffness.

The concern with this subject is not recent. Meyerhof (1953) evaluated the effects of absolute and differential settlement on the stresses that occur in the structural elements and in the foundation. Meyerhof's theory considered different foundation-structure relative stiffness and observed that the incorporation of differential settlement in the building design increased the stresses generated in the superstructure, mainly in the beams and columns of the first floors.

Rocha (1954), in the first Brazilian Congress of Soil Mechanics, presented a suggestion on how to calculate hyperstatic structures considering the settlement of foundations, using the traditional methods of displacement and force method. The foundation load-settlement ratio was considered linear, allowing the incorporation of proportionality coefficients (spring constants) in the equations. The calculation would be made in an iterative way, and the convergence would be evaluated in terms of the loads and the settlement of the columns.

Chamecki (1954) presented a methodology to calculate foundation settlement and superstructure support reactions, incorporating rigidity of both parts. It used load transfer coefficients between adjacent columns for the entire structure, noting the transfer of the loads from the most loaded elements to the least loaded. Consideration of the superstructure's stiffness in the foundation analysis caused the reduction of differential settlement.

Gusmão (1990, 1994) proposed a methodology to evaluate the effects of soil-structure interaction from the measurement and analysis of settlement. Using this concept, the redistribution of loads in the columns and the uniformity of the settlement deformation, by means of parameters defined by the author, was evaluated.

With the current trend of constructing higher and higher buildings in Brazil, in a similar tendency to the rest of the world, the theme of foundation-structure interaction is becoming more and more relevant in the study of the behavior and stability of these high and very flexible structures. This article presents and discusses the analysis of a

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hypothetical tall building evaluated with and without the foundation-structure interaction process, and it compares settlement measurements in real buildings, highlighting the importance of incorporating the settlement superposition effect between different foundation elements in the study of the interaction with the structure.

2. Problem analysis model

To try to solve the problem of the interaction between the superstructure behavior and that of the foundation, different methods of calculation have been employed. The main methods used are: calculation by iterative process (Rocha, 1954; Iwamoto, 2000; Araújo, 2009; Bahia *et al.*, 2016; Silva, 2018); the coupled method presented in Poulos (1975); and considering the superstructure and foundation as a unique problem in numerical models.

The iterative calculation starts from obtaining the loads on the columns considering the supports as fixed nodes. Using these loads, foundation settlements are obtained for each column, incorporating the interaction effect between different points of foundations. Considering "spring coefficient" as the ratio between load and settlement of each column, these coefficient values are assigned as the stiffness of the supports under each column, and the structure is recalculated, now supported by elastic supports, to obtain a new load set. The process is repeated iteratively, recalculating the settlement and the loads on the supports. This procedure is repeated until the result convergence of some variable, usually the settlement. The aforementioned authors (Iwamoto, 2000; Araújo, 2009; Bahia *et al.*, 2016; Silva, 2018), among others, highlight: the important load redistribution between columns located in areas of different settlements; that the lower floors are the most affected by changes of loads and moments in columns and beams; that a small number of iterations (3 to 5) are already sufficient to achieve the settlement convergence; and that the first iteration is responsible for a preponderant percentage of changes.

In the coupled calculation proposal, presented in Poulos (1975), the analysis of the soil-structure interaction (SSI) is performed by coupling the equations of the superstructure forces calculation with the foundation deformability equations. Eq. 1 describes the vector $\{V\}$ as the support reactions obtained considering the SSI; $\{V_0\}$ is the vector of the reactions calculated for the case of fixed supports; $\{\delta\}$ refers to the displacement vector of the supports considering the SSI; and $[SM]$ is the structure stiffness matrix, which relates the additional support reactions due to unitary displacements of other supports.

$$\{V\} = \{V_0\} + [SM]\{\delta\} \tag{1}$$

This formulation allows considering the structure as three-dimensional and with 6 degrees of freedom. The support reactions can be calculated considering external loads and the hypothesis of fixed supports. The stiffness matrix can be calculated by imposing unitary displacements to the supports. The displacement vectors and support reactions are unknown when considering the SSI process.

The soil-foundation interaction is governed by Eq. 2, in which $[FM]$ is the foundation flexibility matrix that relates the displacements of the foundation supports to unit loads.

$$\{\delta\} = [FM]\{V\} \tag{2}$$

Through these equations, the following relationships are obtained to reach the final loads.

$$\{V\} = \{V_0\} + [SM][FM]\{V\} \tag{3}$$

$$\{V_0\} = [[I] - [SM][FM]]\{V\} \tag{4}$$

Another way to consider soil-structure interaction is to adopt a unique 3-D model that gathers the superstructure and the foundation. To solve this problem, numerical tools are used, such as the finite element method (FEM) and the boundary element method (BEM), with a very high computational effort to try to incorporate specific models to the superstructure and soil materials (Poulos, 2013).

3. Evidence of soil-structure interaction

Gusmão (1990, 1994) developed a methodology to interpret settlement measurements in order to verify the effect of SSI on building performance. The settlement distribution was evaluated by means of the coefficient of variation (*CV*), which is the relationship between the standard deviation (σ) and the settlement average (w_m). The author highlights that the standard deviation is influenced by the average magnitude of the settlements, so the use of the coefficient of variation is indicated (*CV*).

Gusmão (1990, 1994) observed that the average settlements are related to the adopted stress-strain soil model, while the distribution of the settlement (evaluated by the *CV*) is related to the SSI model, where there is a tendency to smooth the settlement curves. It should be noted that increasing structure stiffness decreases the dispersion of settlement curves. The methodology developed and applied by Gusmão (1990) was also employed in 7 identical buildings in Recife-PE, with 18 slabs, based on precast concrete piles, in which it was observed that the measured *CVs* were smaller than the *CVs* theoretically estimated without the SSI effect, evidencing the tendency to the settlements uniformity (Gusmão & Gusmão Filho, 1994).

Gusmão *et al.* (2000) applied Gusmão (1990) methodology to a 15-story building built in Recife, with soil improvement using compaction piles. The settlement monitoring showed that the *CV* began to stabilize at the time of the construction of the first floors, in about 100 days, denoting that the building reached a limit of rigidity, after which the mechanism of SSI was less significant.

Mota (2009), through the settlement monitoring of a 28-story building located in Fortaleza-CE, observed that the average settlement increased and the *CV* decreased with the evolution of the construction, which reveals the tendency for settlement uniformity.

In Gusmão (1990), the absolute settlement factor (*AR*) was also defined to evaluate the effect of the SSI on load redistribution in the columns, calculated according to the following equation:

$$AR = \frac{w}{w_m} \tag{5}$$

in which w is the absolute settlement at a given support point and w_m is the average absolute settlement.

Differential settlements are responsible for the redistribution of loads among the columns when analyzing the SSI. When the estimated *AR* of a column is greater than one, it means that the estimated absolute settlement, without an SSI, is greater than the average absolute settlement. In these situations, there is a tendency for this column to suffer a load relief. Therefore, the measured *AR* value of this same column tends to decrease in relation to the estimated value. If the column has an estimated absolute settlement lower than the average (estimated *AR* less than one), there is a tendency to suffer an overload, and the measured *AR* value tends to increase and be higher than the estimated *AR*.

4. Interaction between foundation elements

4.1 Interaction between footings

The settlement of single footings can be calculated by several methods. The soil profile and the presence of the groundwater greatly interfere with the final result. In saturated clays, for example, the final settlement is composed of a short-term component (initial settlement) and another component caused by the consolidation process, which, as a rule, is the predominant one. In sandy soils or unsaturated soils in general, the settlement can be predicted using the theory of elasticity with good accuracy, since the elastic modulus of each soil layer could be estimated. Numerous elastic solutions for different loading sets and boundary conditions for the soil profile are presented in Poulos & Davis (1974). The Fadum's solution (1948) allows the easy composition of areas to represent the shape and stiffness of the footing, as shown in the equation below, already disregarding the small effect of the Poisson ratio:

$$w_i = I \sum_1^j \sum_1^n \frac{\Delta\sigma_i \Delta z_i}{E_i} \tag{6}$$

in which w_i is the settlement of an isolated footing; I refers to the stiffness factor (1 for central recalculation in flexible footing and 0.8 for rigid footing); j is the number of areas to fit the footing geometry; n is the number of layers in soil discretization; $\Delta\sigma_i$ is the vertical induced stress in the center of the layer i under the projection of the footing center; Δz_i is the thickness of each layer; and E_i is the elastic modulus of layer i.

To consider the interaction between two footings, Eq. 6 can be used when considering $\Delta\sigma_i$ as the vertical stress induced under the center of the neighboring footing and $I = 1$. Therefore, the compression on each footing will be the sum of all interactions to its isolated compression:

$$w_{if} = \sum_1^n w_{ij} \tag{7}$$

in which w_{if} is the total settlement of the foundation *I*, considering all the interactions of n neighbouring footings; w_{ij} is the stress induced by footing j in footing i.

4.2 Interaction on piling foundations

Aoki & Lopes (1975) used Mindlin's equations to estimate stresses and settlements in deep foundations. The loads are transmitted to the soil by the foundation elements considering the shaft and the base loads. When the foundation is a group of piles, the effects of interaction between the piles and between piles and soil can be estimated.

Poulos (1968) analyzed the effect of settlement increase due to the interaction between two identical piles admitting the behavior of the soil as an elastic medium, defining the percentage of the increase of the settlement as a pile-pile interaction factor. Through the superposition of interaction factors of all neighboring piles, it is possible to reach the total settlement of a pile, as seen in Eq. 8. The stiffness of the pile cap joining the piles is the boundary condition necessary to find the pile group settlement.

$$w_i = \sum_1^n \alpha_{ij} \frac{P_j I_j}{ES \cdot D} \tag{8}$$

in which w_i is the foundation total settlement (pile i); α_{ij} is the interaction factor between pile j and pile i; P_j is the load on pile j; I_j is an influence factor of the geometry and soil conditions in the calculation of the loaded pile j; Es is the average modulus of the soil profile along the pile; and D is the diameter of the piles.

In a similar way to that described for footings, Eq. 8 also allows evaluation of the pile-group settlement considering the interaction of all the piles in the work.

On a piled raft foundation, the superposition of stress fields implies in the interaction between the surface plate (raft) and the various piles. Several papers (Hain & Lee, 1978; Clancy & Randolph, 1993; Poulos, 1994; Russo, 1995; El-Mossalamy & Franke, 1997; Bernardes *et al.*, 2019) presented numerical solutions incorporating the four interaction processes involved in a piled raft design: soil-soil, soil-pile, soil-pile, and pile-pile.

4.3 Interaction of nearby buildings

In extreme situations of nearby constructions, the stresses induced in the soil may induce additional stresses in neighbouring foundations. This effect is recurrent and noted when a large building is built close to old buildings, as the proximity can result in new cracks in the pre-existing building due to the suffered pressure increase. The more deformable the soil under those buildings, the more serious is the problem. The city of Santos, São Paulo, is a classic example of interaction between nearby buildings, resulting in tilt of several buildings in different directions, depending on the chronological order of construction of neighboring buildings.

5. Cases of buildings

Two real cases in Recife, Brazil, are here presented to show the effect of interactions on the overall behavior of buildings (Fig. 1). For both cases, the monitoring of settlements and the static load test (SLT) were performed.

5.1 Case 1 - Recife/PE - Brazil

The building has 30 floors and 17 columns in the main tower, with a total permanent load of 102.25 MN. The soil profile is composed of sand layers, as presented in Fig. 2. The foundation has 84 continuous flight auger piles

Figure 1. Location of Recife, Brazil.

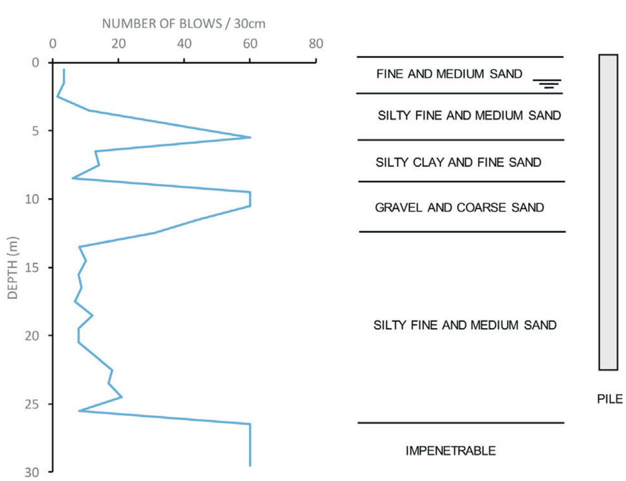

Figure 2. Soil Profile - Case 1.

with 50 and 60 cm in diameter, and length ranging from 22 m to 25 m (Fig. 3).

A static load test (SLT) was carried out in a pile with 500 mm in diameter and reached 2600 kN, which represented two times the project load. Figure 4 shows the pile test and the hyperbolic-method fit. For the value of 1300 kN (project load), the settlement was around 2 mm.

At a certain stage of settlement measurements, the total load acting on the building can be estimated, considering the construction stages completed up to that time. In this example, the last settlement measurement was carried out just after the completion but before receiving the residents. The load at this stage (only dead loads) was considered to be 85 % of the total load (dead and live loads). Figure 5 presented minimum, maximum, and average settlement with the values ranging from 9 to 19 mm, which are much higher than the 2 mm of the SLT.

The average absolute settlement of the building is independent of structure stiffness, while this last one influences only the settlements dispersion. Assuming the average load per pile as the division of the total permanent load by the number of piles, at each stage of measurements, it is possible to compare the average settlements and average pile loads. For the presented building, when 85 % of the construction was completed, the average load per pile would result in 1,034 kN. The average load per block is 5,794 kN.

Figure 6 compares the load curve of the SLT (without group effect) with the average settlement of the piles belonging to the foundations of P17 (corner column), P6 (central column); and the average of the whole building with the implicit group effect. The effect of the interaction between the foundations is clear, implying that the building settlement is much higher than that of the isolated pile.

The parameter (*Rs*), defined by Poulos & Davis (1980), is the relation of the pile settlement in a group compared with the pile settlement when isolated, and it represents the increase of the pile settlement as a function of the interaction of all the neighbouring piles. In this example, the value of *Rs* could be obtained by the ratio between the average pile settlement obtained during building monitoring and the settlement obtained in SLT, for the same load level.

Figure 7 compares calculated *Rs* values for piles in corner groups, central groups, and mean values across all piles. The monitoring showed a higher *Rs* value for the central groups of piles when compared with the corner piles. In average terms, the building showed settlements from 12 to 20 times higher than the measured SLT value for the same average load per pile. In this case, the effect of the interaction between the piles was clear. For the soil profile in question, the piles did not reach an impenetrable layer and therefore could be considered as floating piles, where the portion of lateral friction is predominant.

5.2 Case 2 - Recife/PE - Brazil

The building in Case 2 has 28 floors and 17 columns with a total permanent load of 79.58 MN. The foundation was designed using steel piles driven through soft layers and reached the impenetrable stratum, thus predominating the base resistance portion in the load capacity of these piles (Fig. 8).

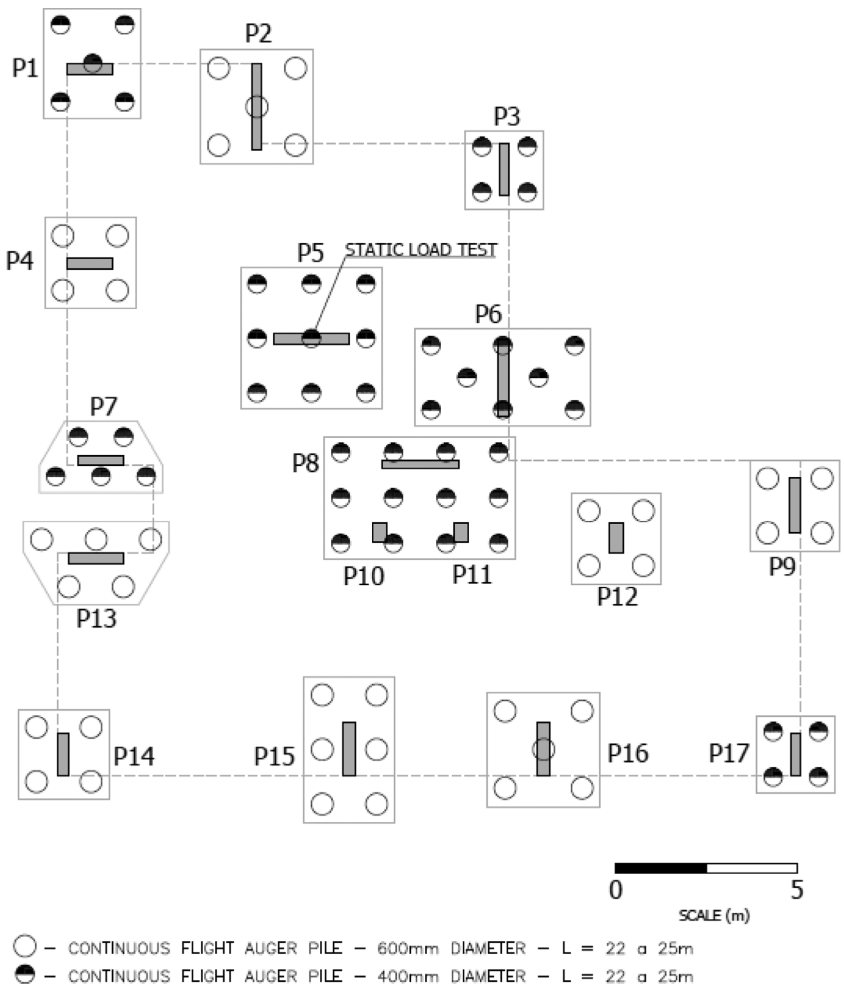

Figure 3. Foundation layout in plan - Case 1.

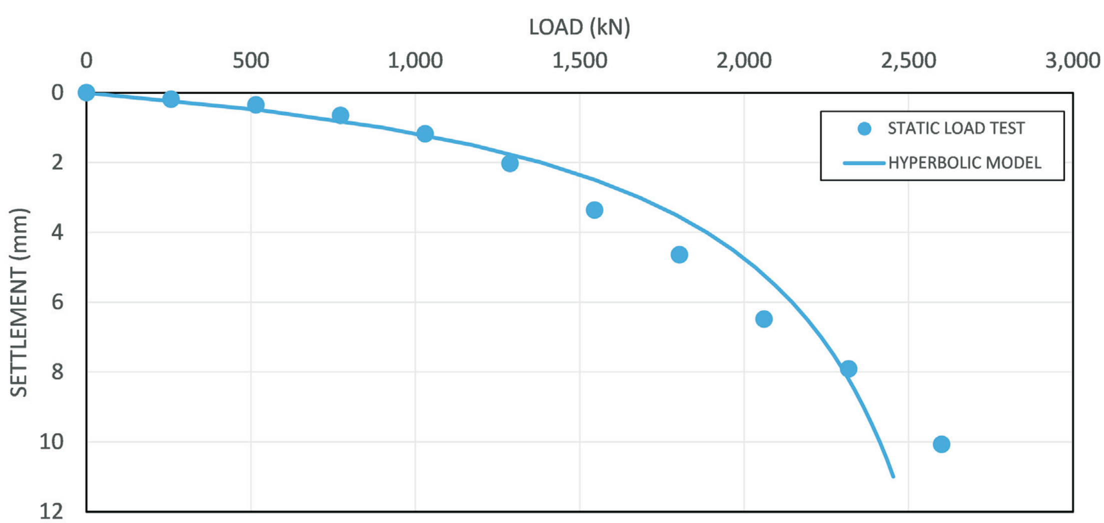

Figure 4. Pile load test and hyperbolic model fit for a pile with 500 mm diameter.

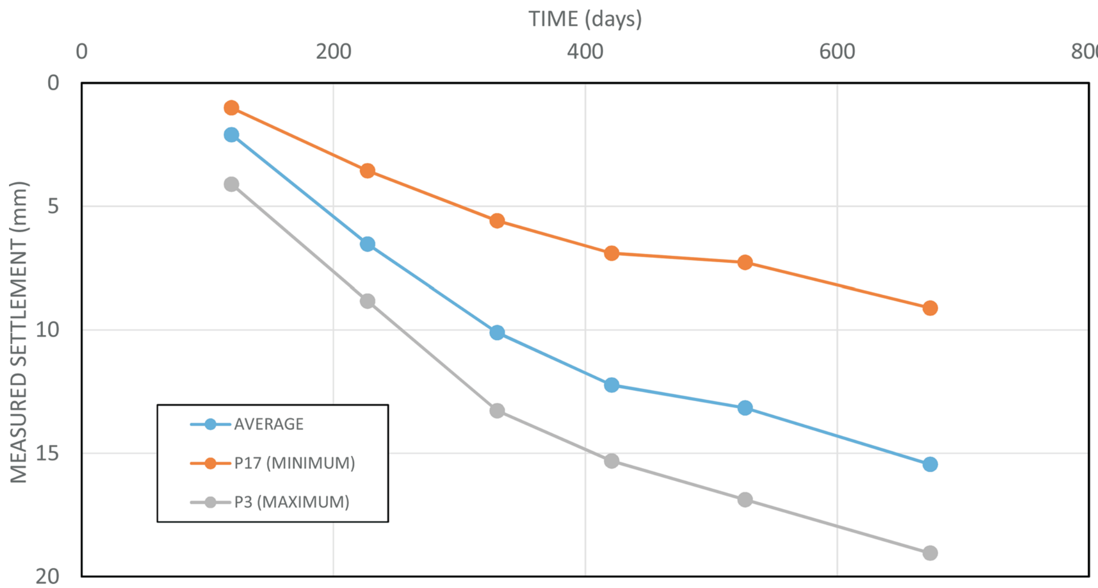

Figure 5. Settlement measurements during construction - Case 1.

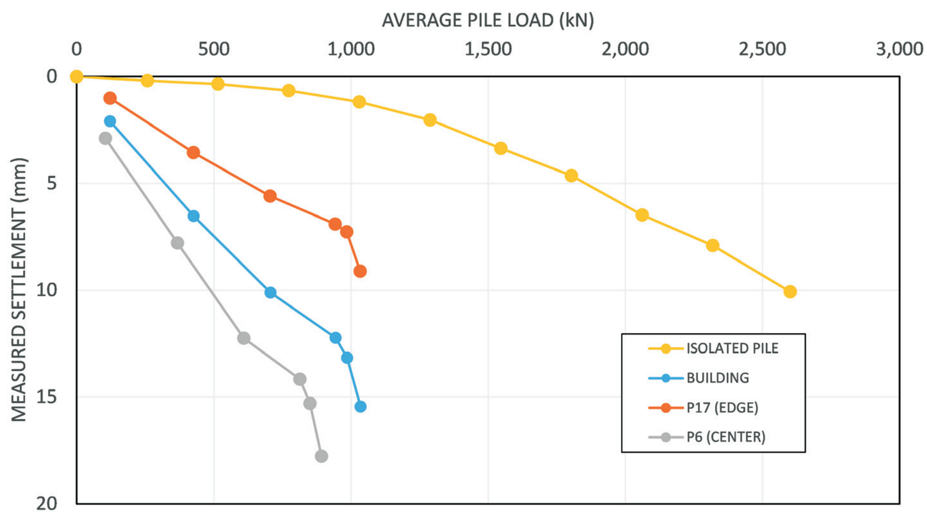

Figure 6. Comparison of SLT curve (isolated pile) and monitoring settlements of the piles - Case 1.

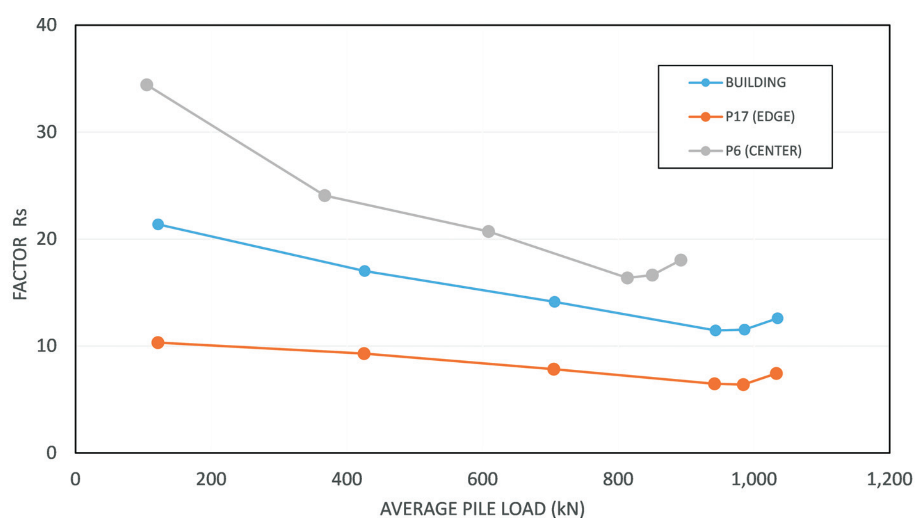

Figure 7. *Rs vs.* load, comparing the SLT and settlement monitoring - Case 1.

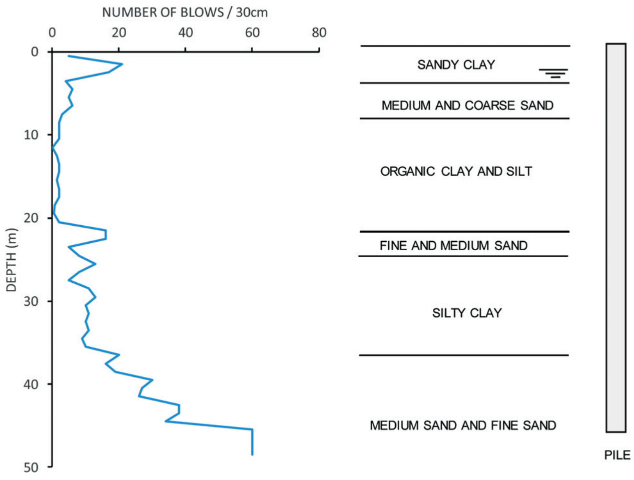

Figure 8. Soil Profile - Case 2.

The steel piles had 45 m in length and H-section of type HP310x93 and HP-310x110 (Fig. 9). Sharing the total project load for all piles would result in an average design load of 1,206 kN. A static load test was carried out at the beginning of the construction work (Fig. 10), and the piles were monitored during the construction period.

Figure 11 presents the maximum, minimum, and average setlements until building completion (around 85 % of total permanent load). In the last stage of measurements, the settlements ranged from 2 to 8 mm, close to the measured values of SLT.

In a similar way, described in Case 1, Fig. 12 compares the average pile load-average settlement behavior for one edge column (P13); one center column (P8) and for the building mean, and these curves were also compared with the static load test result for a single pile. It is observed that

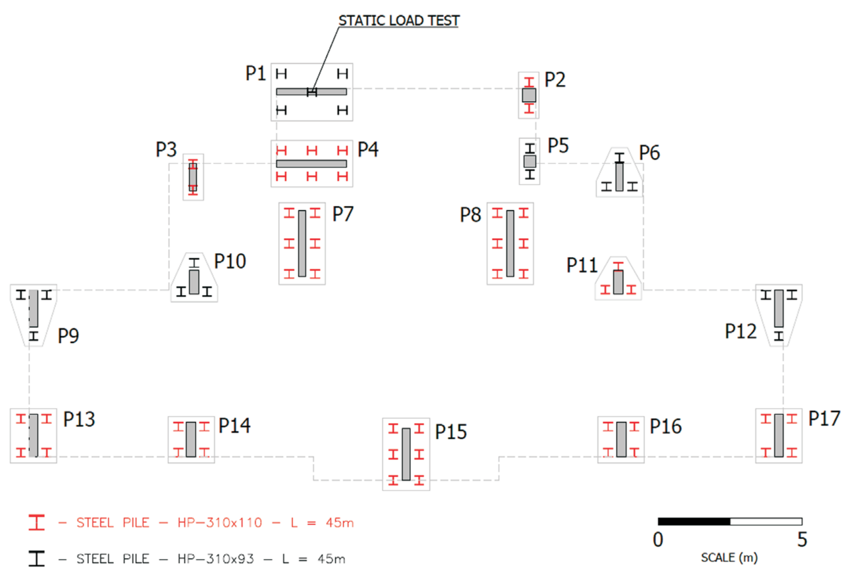

Figure 9. Foundation plan - Case 2.

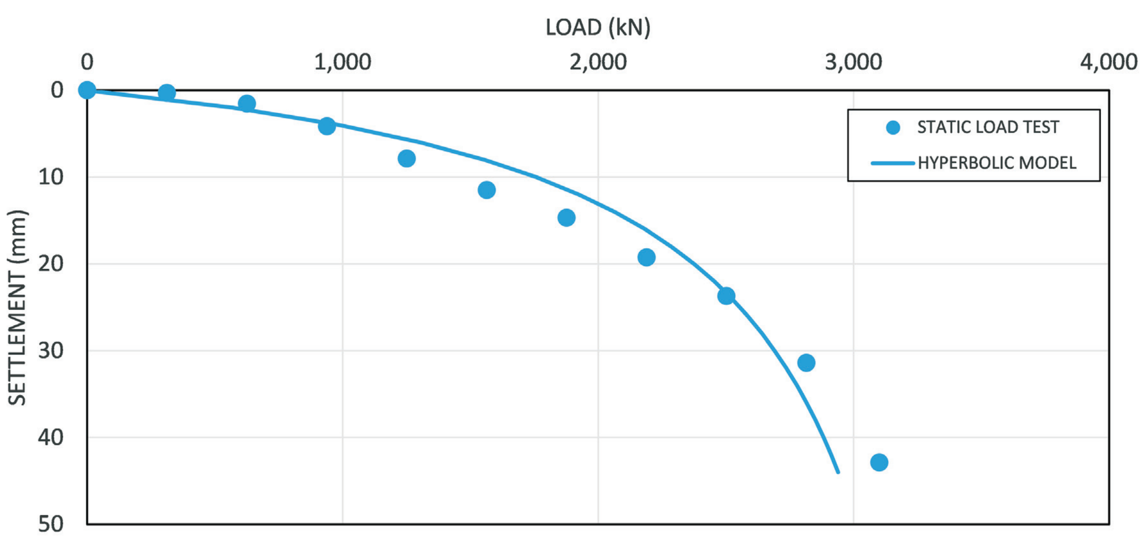

Figure 10. Static load test on a HP-310x110 pile - Case 2.

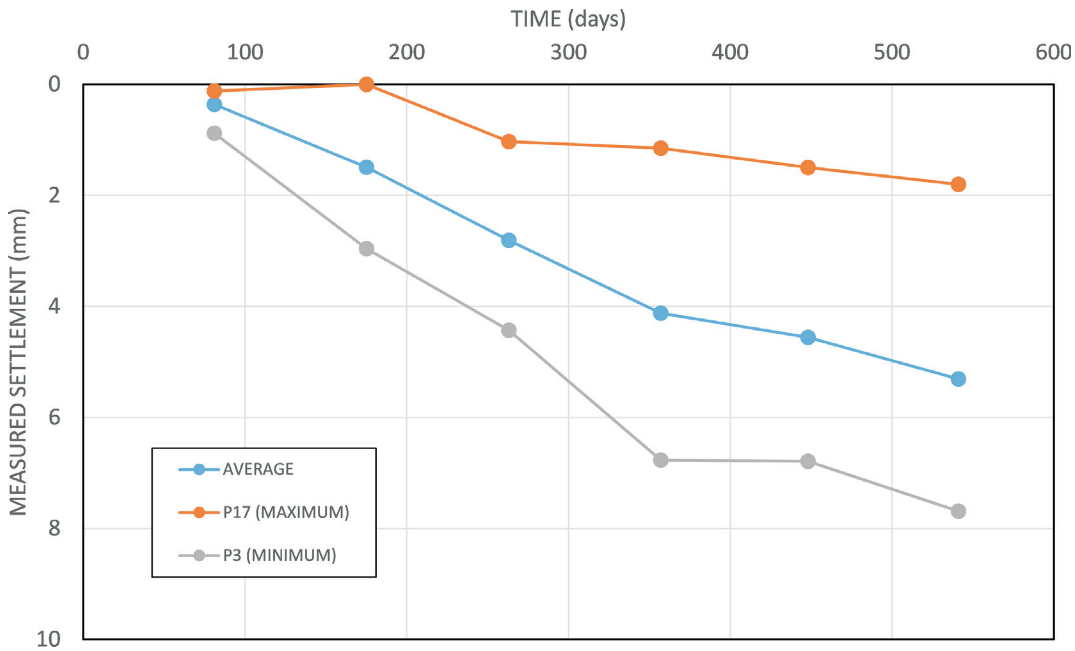

Figure 11. Evolution of measured settlements - Case 2.

the curves behaved in a very similar way, indicating that the interaction between piles was not relevant. The explanation lies in the fact that the support layer of the tip was quite rigid and did not induce relevant settlements in the neighboring piles. Adding to this fact, Sales *et al.* (2017) point out that the process of installation of pre-molded piles creates a thin layer of soil (shearband) along the pile, where the soil structure is destroyed, and cannot induce important settlements in the vicinity.

The settlement ratio (*Rs*) was also calculated for columns in different positions in Case 2. Figure 13 presents the values obtained for *Rs*, noting that all cases are close to 1. Unlike the previous case, in this building, the behavior of tip piles indicated that the interaction effect between piles was practically negligible.

5.3 Other cases

Figure 14 presents a database of 14 buildings (represented by different letters) in the Metropolitan Region of Recife, where it can be observed that the settlement ratio (*Rs*) was between 1 and 22, considering different percentages of building loads (Almeida, 2018).

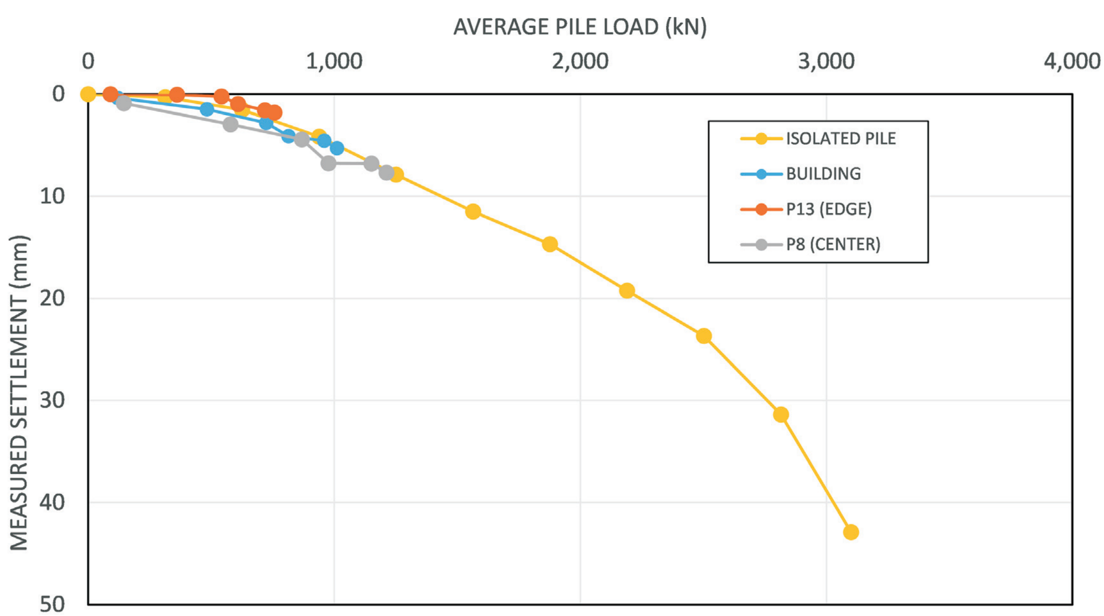

Figure 12. SLT curve (isolated pile) and monitoring pile settlements - Case 1.

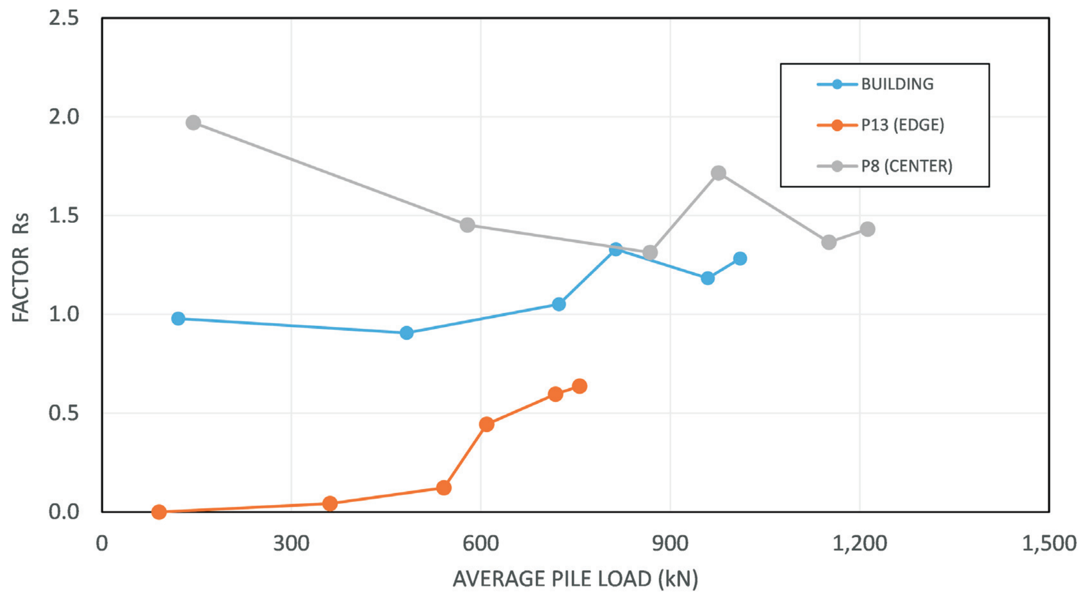

Figure 13. *Rs vs* average load, comparing SLT and settlement monitoring in building 2.

The value of *Rs* did not vary much with the stage of the work, but it can be quite different from one construction to another depending on the number of piles, proximity of columns, and soil profile.

6. High-Rise building (A theoretical study)

Silva (2018) evaluated the behavior of hypothetical high-rise buildings, considering and not considering the foundation-structure interaction by iterative process described in Section 2 of this paper. The case presented in this article simulates the behavior of a 50-story building, with rectangular geometry in plan projection, as illustrated in Fig. 15. The structure was analysed using the software TQS (2016) and the foundation behavior was evaluated with the GARP software presented in Small & Poulos (2007).

Figure 16 presents the soil profile considered (sandy non-saturated clay from 0-5 m and sandy residual silt below this elevation). Two possibilities of foundations were taken into account: the first representing the case where the foundation is little embedded (inferior raft surface at -3 m elevation) and the second representing a 7 m excavation and thus the raft is laid at -10 m elevation. The first alternative foundation, called RFA140, resulted in the use of a 3 m thick raft resting on 220 piles with 1.4 m diameter and 21 m length under the raft. The second foundation option, case RFO140,

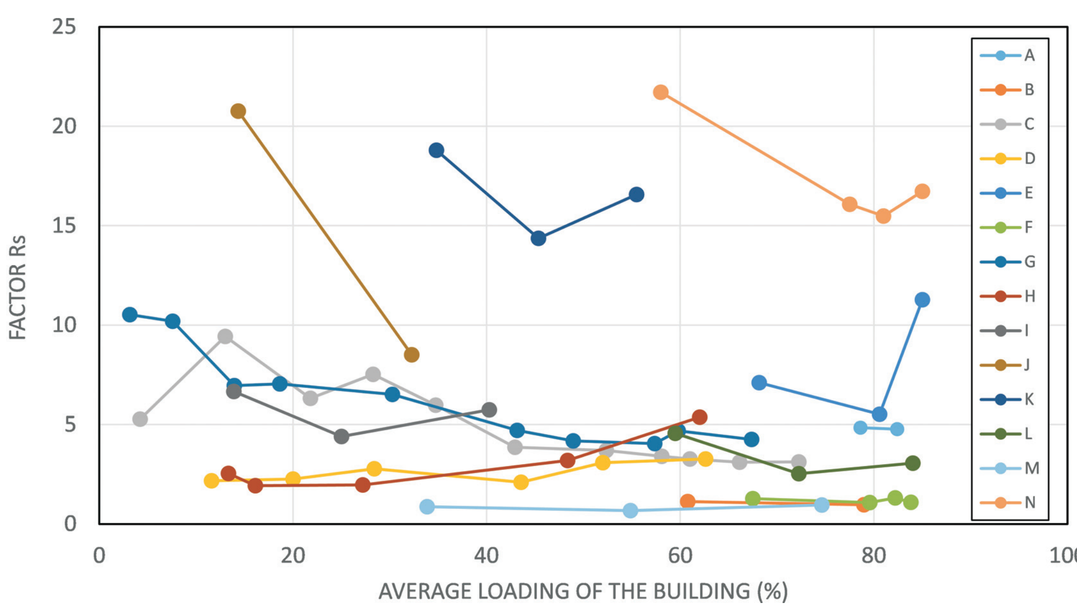

Figure 14. *Rs*-values *vs.* percentage of loading of 14 buildings (Almeida, 2018).

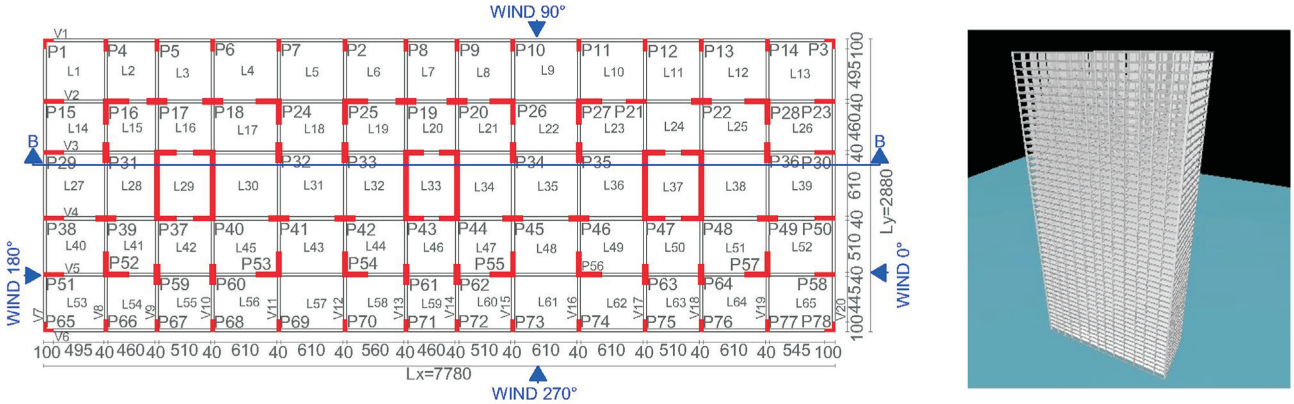

Figure 15. Modeled building - Location of the columns.

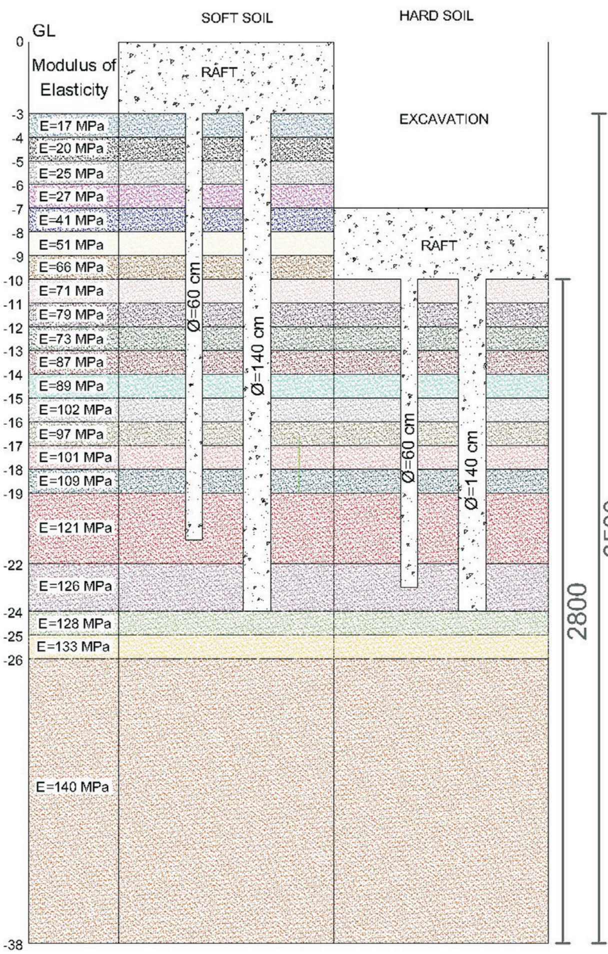

Figure 16. Soil profile and analysed foundation.

would be a raft with the same thickness supported on 27 piles (1.4 m in diameter and 14 m in length). The quantity of piles was determined by considering the contribution of the soil under the raft and satisfying a minimum Safety Factor of 2.5 for the load capacity when both raft and piles are considered, in the form of Eurocode7 (2004). Figure 17 illustrates the distribution of the piles under the raft.

The results presented by Silva (2018) revealed that after the third iteration of the calculation of the structure and foundations, the settlement values in successive interactions were very close. The following sections discuss the changes in settlement results, angular distortion, loads on the columns, bending moments in the raft when the building is calculated with or without the SSI, and spring coefficients to make the supports more flexible.

6.1 Settlements

Figures 18a and 18b show the settlement curves for the columns near the BB cross section, shown in Fig. 15, for the two alternatives of foundations: RFA140 and RFO140.

In both cases, the incorporation of the process of interaction between the foundation and the structure led to a load increase on the corner columns. The internal columns P31 and P36 did not present a defined behavior of increase or reduction of the settlements, but the other internal columns presented a reduction of the settlements with the interaction.

The corner (P1), lateral (P7), and inner (P43) columns were selected for the calculation of the relative percentage change of settlements (ΔV_j), which was obtained by means of Eq. 9, in which Δw_j is the difference between the settlement of the current iteration and the previous one, of the column j; and $w_{0,j}$ is the settlement obtained without iteration for this column.

$$\Delta V_j = \frac{\Delta w_j}{\Delta w_{0,j}} \tag{9}$$

Figures 19a and 19b show the relationship between the percentage variation of settlements and the number of iterations for the cases RFA140 and RFO140, respectively. In both analyses, the corner column (P1) showed a settlement increase in relation to the previous iteration (positive variation) of about 10 % in the first iteration and smaller increments, but still positive variations of its value in the 2nd and 3rd iterations. The internal column (P43) presented in

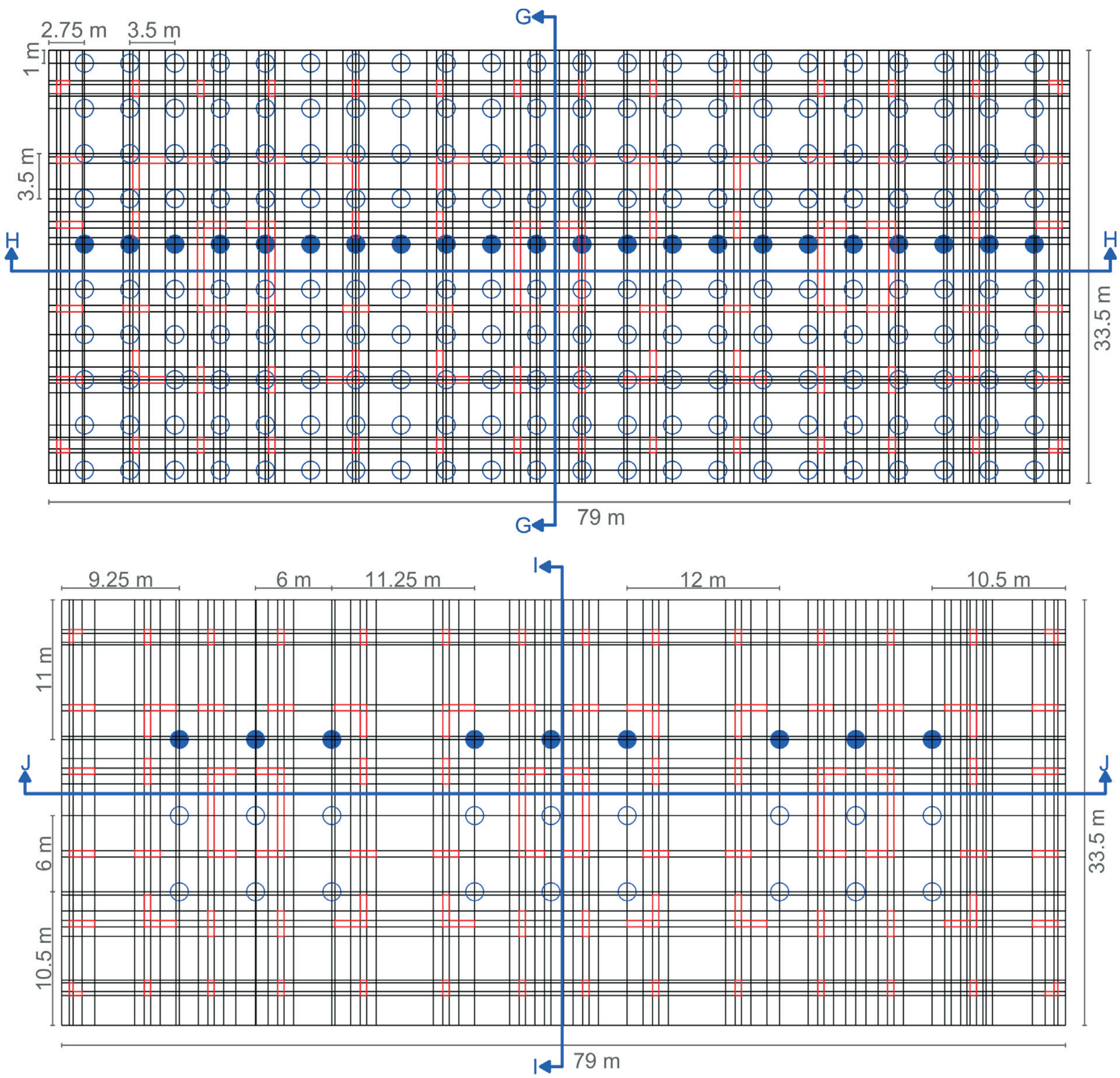

Figure 17. Modeled building - Pile location.

the first iteration a settlement reduction, around 5 %, and smaller reductions of the settlement w ith the following iterations. In both cases, the lateral column (P7) presented an intermediate behaviour. Increasing the number of iterations, the settlement variation curves converge to values close to zero in all columns.

Table 1 shows the average settlements (w_m), standard deviation (σ) and the coefficient of variation (CV), with (SSI-soil structure interaction) and without (FS-Fixed supports) interaction. The case RFO140 showed higher average settlements than RFA140. For both cases, the difference between the mean settlement with and without the interaction was less than 1 %, *i.e.*, the foundation-structure interaction process hardly affects the mean settlements prediction. However, with the interaction, there was a 26 % relative reduction in the coefficient of variation in the case RFA140, and a 33 % reduction in the case RFO140. The reduction in CV indicates greater uniformity (less differential settlements) when performing the foundation-structure interaction. Thus, the soil-structure interaction had a greater influence on smoothing the settlement curve than on the reduction of the mean settlements magnitude.

6.2 Angular distortions

Disregarding the perfect building verticality or not, highest rotations or angular distortions (ratio of the settlement difference to the columns distance) of five pairs of columns were calculated after 3 iterations steps, and these results are shown in Figs. 20a and 20b.

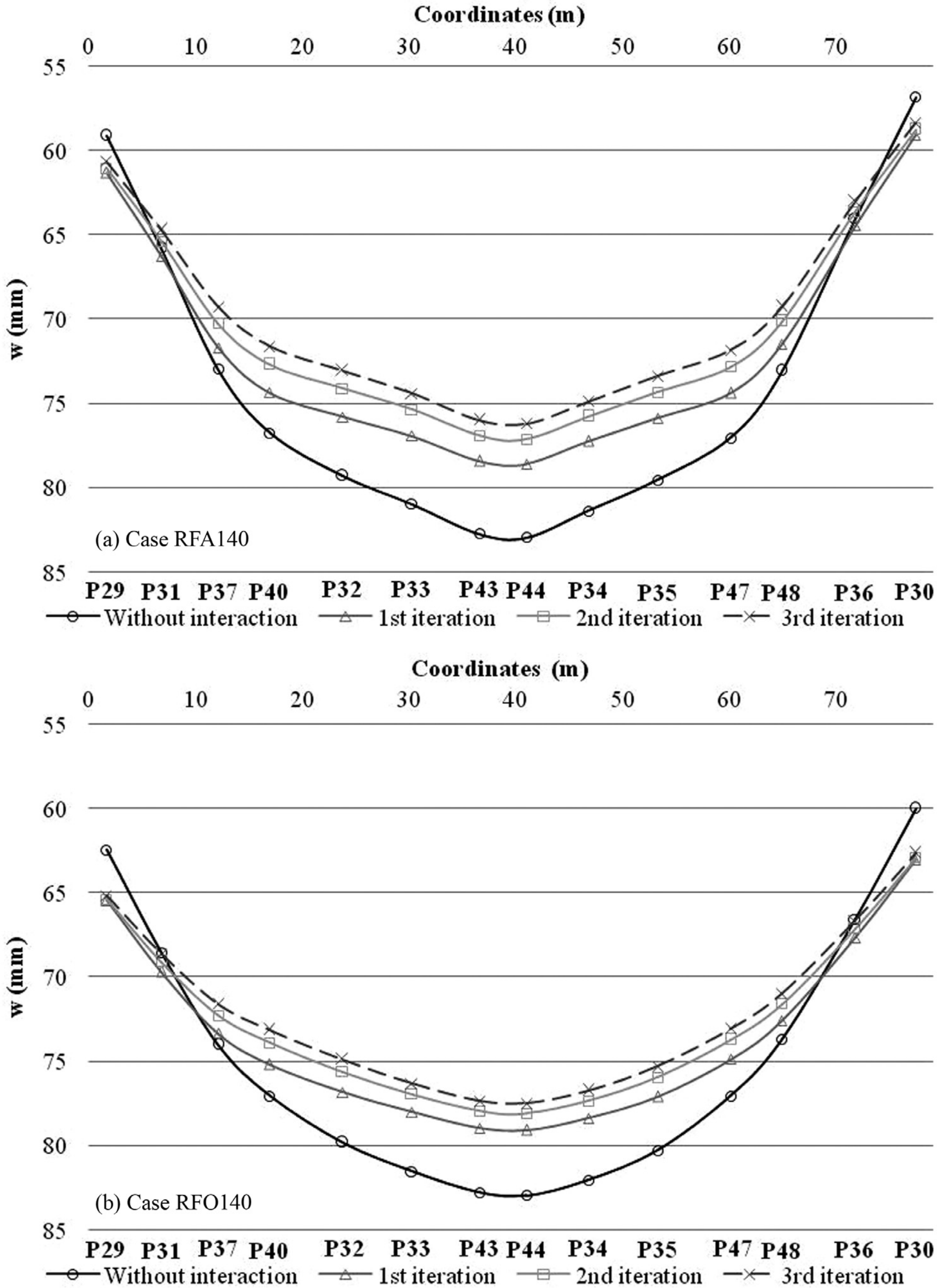

Figure 18. Settlement prediction for the BB-cross section.

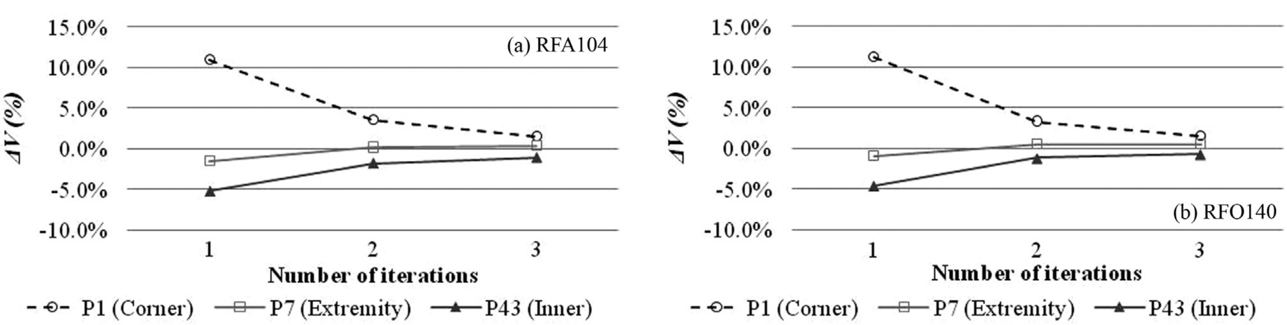

Figure 19. Relative percentage change of settlements with the number of iterations.

Table 1. Average setllements and coefficient of variation for the analysed cases.

Settlement	RFA140		RFO140	
	FS	SSI	FS	SSI
w_m (mm)	62.71	61.50	67.13	66.84
σ	11.76	8.50	9.65	6.40
CV	0.19	0.14	0.14	0.10

Comparing with the ratio of "1:500", which would correspond to situations of unlikely appearance of cracks in the building, as suggested in Skempton & MacDonald (1956), and Poulos (2017), Figs. 20a and 20b show that, for both analysed foundations, the interaction process resulted in less angular distortions than the conventional project without interaction. In the case of RFA140, the results obtained without the SSI could have been discarded or altered by excessive distortion between nearby columns, but the evaluation considering the interaction pointed out that it could be accepted by the criterion mentioned above.

6.3 Column loads

Figure 21 shows the load evolution in different columns with the increase in the number of iterations, for the cases RFA140 and RFO140. The loads varied more in the first two iterations and showed the tendency of convergence for the following steps. It should be noted that the P1 (corner) column now has an increased load of approximately 60 % in relation to the predicted value without foundation-structure interaction (P0 = initial loads without interaction). Column 33, in a more central position, had its load reduced between 20-30 % for the two foundation alternatives studied. In turn, the P29, located in an intermediate position, had little change in its load over the several iteration steps. The comparison between the RFA140 and RFO140 foundations shows that the stiffness of the foundation also interferes with the load redistribution process.

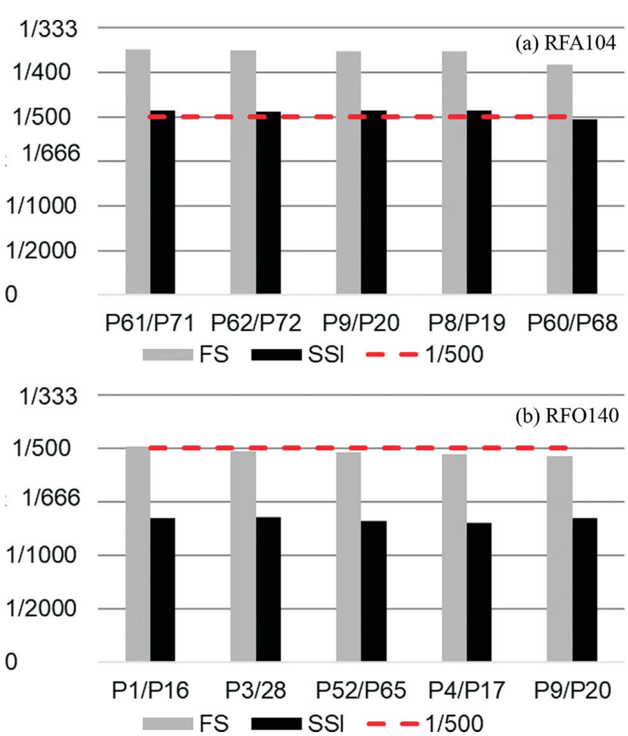

Figure 20. Angular distortions obtained with and without interaction for both foundations options.

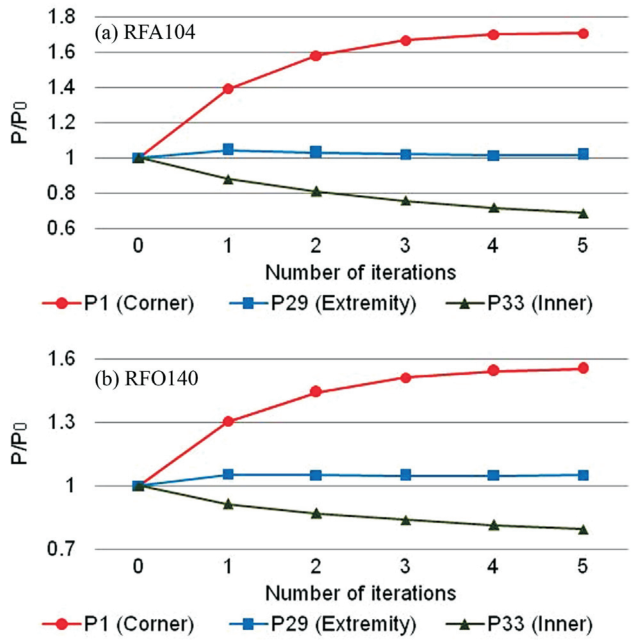

Figure 21. Load changes in three different columns considering SSI for both foundations alternatives.

Load redistribution can also be evidenced by means of the *AR* parameter, which is calculated using foundation settlements according to Eq. 5. In regions where *AR* is less than 1, the interaction generates overload, and if *AR* is greater than 1, the tendency will be to relieve the load on the columns. Figures 22a and 22b show the calculated *AR* values for the same columns shown in section BB in Fig. 15, for cases RFA140 and RFO140. It is observed in the central region that the *AR* values are higher than 1, *i.e.*, their settlement exceeds the mean settlement, implying a tendency of load reduction when considering SSI effect. Reverse behavior occurs in the extremities.

Figure 23 presents a relationship between AR obtained with SSI (AR_{SSI}) and without interaction (AR_{CONV}) of all the columns of the building evaluated for the two alternative foundations. It can be noted that when AR_{CONV} was greater than unity the SSI process resulted in a settlement decrease in relation to the first forecast (conventional), indicating that the region of settlement reduction will also be the region of columns that will have their loads reduced at the end of the interaction process. Conversely, the region

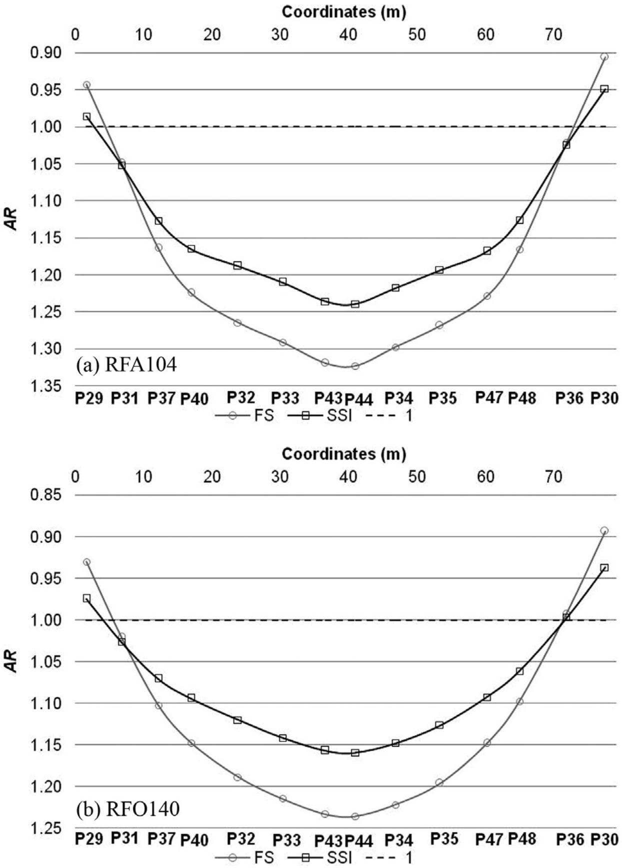

Figure 22. Values of *AR*'*s* obtained with and without SSI.

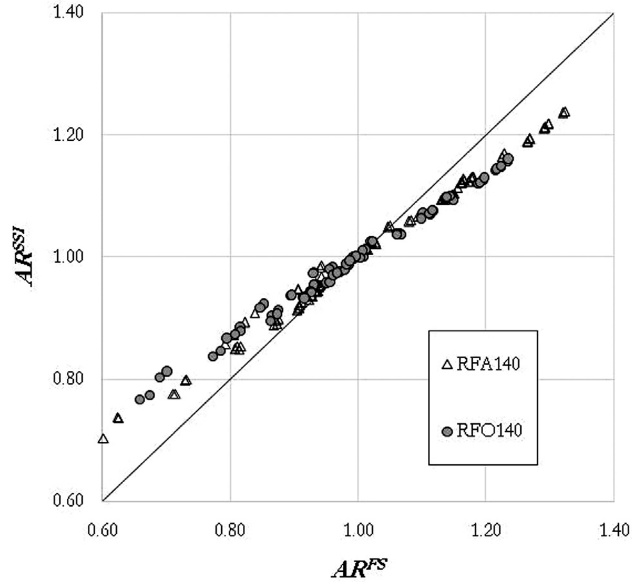

Figure 23. *AR* parameter relationship with and without interaction.

with settlements below the average (AR_{CONV} < 1) tends to have its settlements and loads increased by the interaction process. Figure 24 illustrates the regions where relief (shaded zone) and overload occurred after the SSI process.

6.4 Bending moment in the raft

The effect of the SSI, previously described, implies lower differential settlements and a tendency to smooth out the "settlement basin". When the foundation is designed as a piled raft, as in the analysed case, considering the SSI implied in a lower raft bending, thus reduced the internal moments.

Figure 25 shows that for a cross-section in the smallest dimension (Y-direction) of the building, close to the P8 column (see Fig. 15) the results were similar near the edges, but there was a clear reduction in the central part of the foundation when considering the SSI. In the greater building direction (X-direction) of the raft, the internal moments are presented in Fig. 26. Reductions between 25-50 % were found in the full extent of the raft. As the concrete reinforcement is directly proportional to the value of the bending moment, the reductions would result in considerable cost reduction when considering the SSI effect.

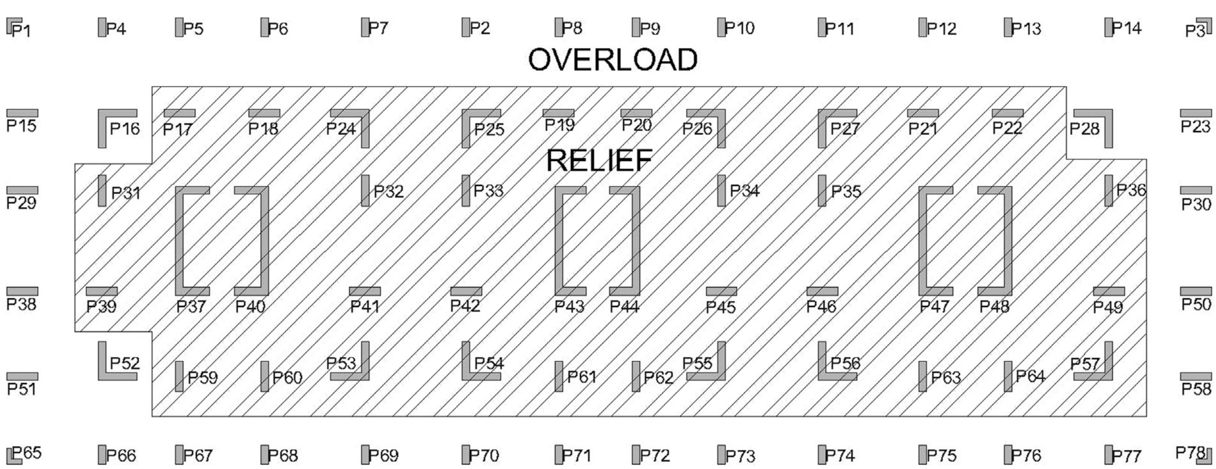

Figure 24. Relieved and overload regions due to the SSI process for the case RFA140.

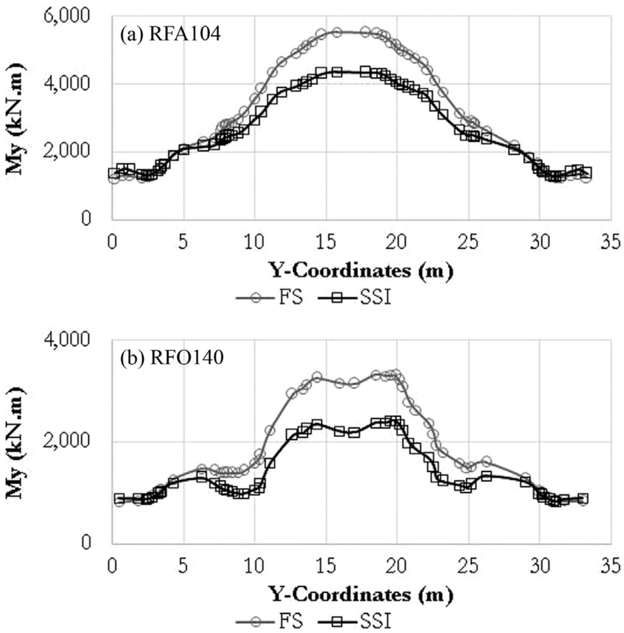

Figure 25. Bending moment on Y-direction with and without interaction for rectangular building.

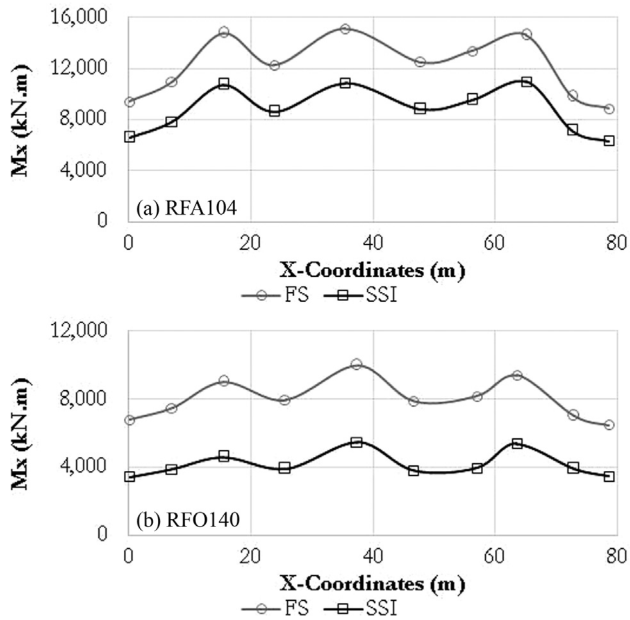

Figure 26. Bending moment on the X-direction with and without interaction for rectangular building.

6.5 Spring effect

In the search for a structural model closer to reality, structural designers replace the fixed supports under the columns by elastic ones. The parameter describing the stiffness of these supports is, by similarity, called "spring coefficient". An attempt is made to relate the spring coefficient to the deformability of the foundations, and some geotechnical engineers have been consulted frequently about what spring coefficient they would indicate to that specific project. The most frequent choice is to try to find a spring coefficient based on the vertical subgrade modulus (k_v), defined by the ratio of the vertical stress acting on soil surface (q) and the resulting settlement (w):

$$k_v = \frac{q}{w} \tag{10}$$

The technical incoherence lies in the fact that the vertical reaction modulus is not a soil property. It depends on the scale factor, the geometry of the foundation, and the soil heterogeneity, and mainly, it does not incorporate the interaction effect between all the elements of the foundation. As in the theory of elastic beam, the spring coefficients do not reflect the continuity of the soil surrounding the foundations.

Antoniazzi (2011) used the Soil Structure Interaction System (SSIS) developed by TQS (2016), which uses a model that connects the superstructure and foundation by using vertical and horizontal reaction coefficients. In this methodology, the interaction between the foundation elements is not considered. However, the results obtained by that author were closer to reality when compared to the hypothesis of fixed supports.

Poulos (2018), after calling the attention to the importance of considering the process of interaction between foundations, suggests that estimating the spring coefficients as the load-settlement ratio, obtained in load tests, is more accurate than using the vertical reaction modulus.

The cases nominated as RFA140 and RFO140 are piled raft foundations, where the piles function fundamentally as settlement reducers. On the other hand, there are interactions between the piles that increase the settlements. The soil profile in question is stratified, which makes the use of spring coefficients difficult to succeed if the coeficients are estimated by means of vertical reaction moduli.

Comparisons were made between the predicted settlements for the studied building, with and without the SSI, and with the described use of spring coefficients. The values of these coefficients were estimated from vertical reaction moduli of 1 and 5 MN/m^3, for the soils RFA and RFO, respectively, chosen based on tabulated values in the literature (Bowles, 1996) for the shown soil profile. Figure 27 presents only the result for 1 MN/m^3, since the results for 5 MN/m^3 were very similar. It can be observed that the use of the spring coefficients calculated from k_v did not practically change the results of the settlements of the initial assessment without the use of the SSI.

Figure 28 includes the results after the first interaction applying the SSI. These results were much more effective to approximate the final convergence values than the case using spring coefficients. As the format of the settlement curves is directly linked to load redistribution, it can

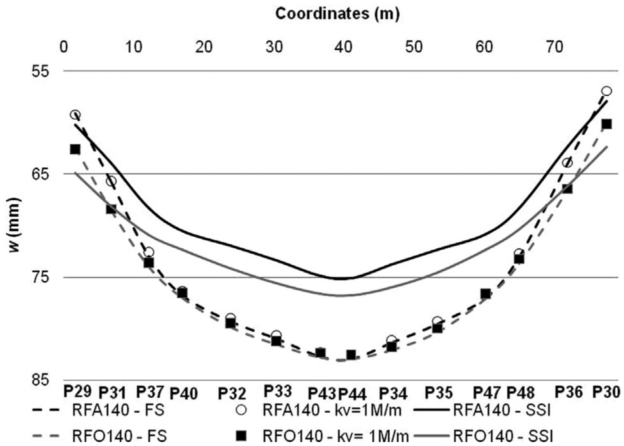

Figure 27. Comparison of settlements predictions considering (SSI) or not (FS) the SSI with the results based in the use of spring coefficients to substitute the foundatios.

be concluded that the use of spring coefficients does not simulate SSI for the presented cases.

7. Effect of SSI on γ_z

Franco & Vasconcelos (1991) developed one method to evaluate the global stability of buildings, denoted by "parameter γ_z", mentioned by NBR 6118 (2014), which is valid for structures with 4 or more floors. This parameter is obtained from a linear analysis of the loaded structure, considering the physical nonlinearity by reducing the structural elements stiffness. The parameter γ_z is calculated by the equation:

$$\gamma_z = \frac{1}{1 - \dfrac{\Delta M_{1,tot,d}}{M_{1,tot,d}}} \tag{11}$$

in which $M_{1,tot,d}$ is the tipping moment, that is, the sum of the moments produced by the horizontal forces in relation to

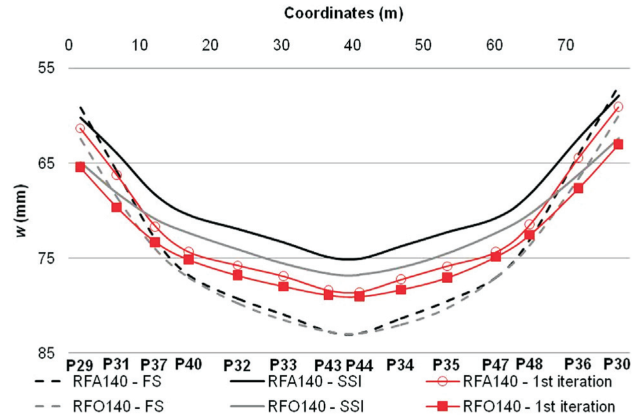

Figure 28. Comparison of the first interaction results with the simulations with (CI) and without (SI) the SSI.

the base of the structure; and $\Delta M_{tot,d}$ is the sum of the products of all vertical forces acting on the structure by the horizontal displacements of their respective points of application, obtained from the 1st order analysis, in the considered combination.

When the parameter γ_z is less than 1.1, according to NBR 6118 (2014), second order effects are not considered, and the structure is classified as having fixed nodes; otherwise, second order effects must be considered, and the structure is classified as having mobile nodes. If the value is between 1.1 and 1.3, this effect is considered approximately by multiplying the horizontal forces by a factor of $0.95.\gamma_z$. Finally, when this parameter is greater than 1.3 the second order effects need to be calculated more precisely.

The value of the parameter γ_z is influenced by the tilt and geometrical imperfections of the building columns. This rotational displacement is calculated considering the perfectly fixed supports, as shown in Fig. 29. However, as already discussed, the foundation will suffer differential

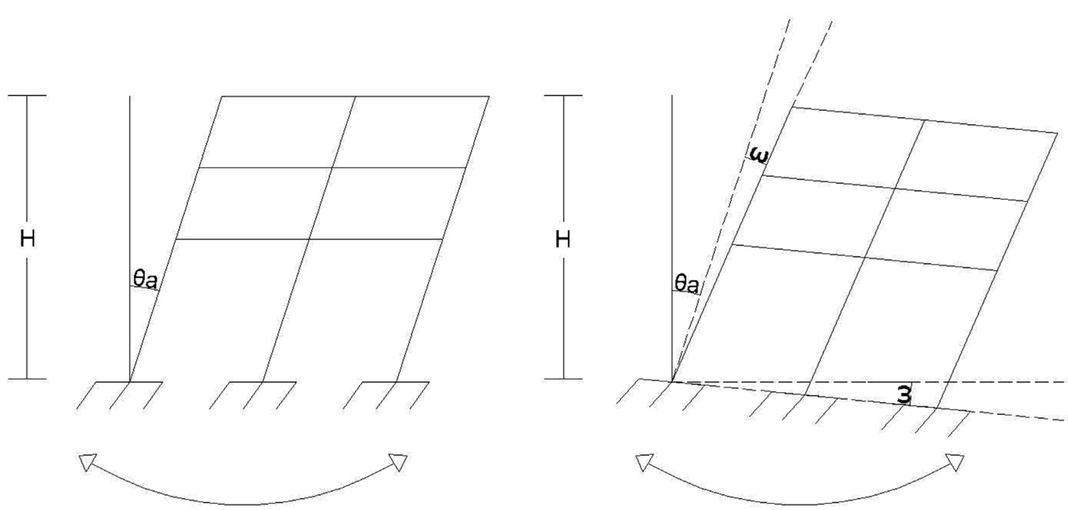

Figure 29. Superposition of the lack of verticality of the structure (θ_a) and diferential foundation settlements (ω).

settlements that will affect the final verticality of the superstructure.

Borges (2009) evaluated the effect of the SSI at the value of γ_z and found that considering the interaction effect between foundation-structure as well as possible rotations of the foundations resulted in an increase of up 37 % in the values of γ_z. Silva (2018) analyzed two high-rise buildings with seven different alternative foundations and also found the magnification of the γ_z factor when incorporating the effect of foundation deformability (SSI).

Based on these results, Gusmão (2018) defined the parameter β_z, expressed by Eq. 12, as the amplification factor of the parameter y_z due to the interaction between the foundations and the superstructure of a building.

$$\beta_z = \frac{\gamma_z^{SSI}}{\gamma_z^{FN}} \tag{12}$$

in which γ_z^{SSI} is the coefficient γ_z calculated with the SSI and γ_z^{FN} is the initial coefficient γ_z calculated without interaction and using fixed supports.

Tables 2 and 3 summarize the values calculated for β_z from the off plumb buildings (ω) obtained by Silva (2018) and Borges (2009), respectively, when considering the SSI. In all cases presented in Tables 2 and 3, the consideration of interaction increased the parameter γ_z. The greater the lack of verticality of the building, the greater is the amplification factor (β_z) due to the SSI, *i.e.*, when the SSI is considered the global stability coefficient will always be higher than the initial value of γ_z, calculated for a building over fixed supports. The data compiled from both works are plotted in Fig. 30 and indicate an exponential growth of β_z when the tilt ratio is close to or higher than 1:1000.

8. Conclusions

This article presented the importance of considering the interaction between different elements of a foundation in predicting the building settlement, as well as the effect of including the settlements of the foundation in the performance of the structure of a high-rise building. Many examples of settlement monitoring were presented to explain the SSI process. Results of iterative process to estimate loads and settlements of buildings supports were discussed. The process concerns in repeating the structure analysis considering the stiffness of each support based in previous settlement prediction. This procedure is repeated until the convergence of the support settlement results.

The highlights are listed below:

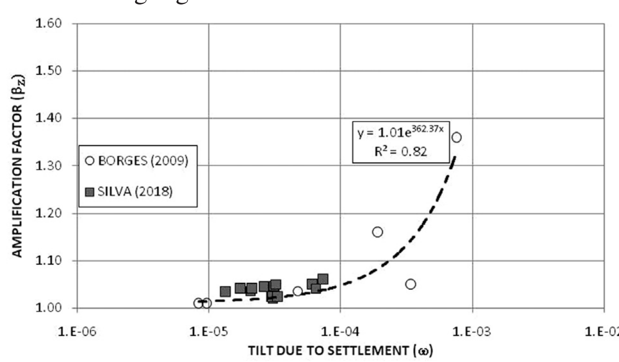

Figure 30. Tilt *vs.* amplification factor of parameter $\boldsymbol{y_z}$ (β_z).

Table 2. Amplification factor (β_z) due to the SSI (Silva, 2018).

Case	X-direction				Y-direction			
	ω	γ_z^{FS}	γ_z^{SSI}	β_z	ω	γ_z^{FS}	γ_z^{SSI}	β_z
QFA60	1/16386	1.095	1.15	1.05	1/74746	1.091	1.128	1.03
QFA140	1/15357	1.095	1.161	1.06	1/57276	1.091	1.136	1.04
QFO	1/13613	1.095	1.167	1.07	1/38011	1.091	1.139	1.04
RFA60	1/32558	1.084	1.106	1.02	1/48110	1.086	1.124	1.03
RFA140	1/33476	1.084	1.109	1.02	1/47059	1.086	1.131	1.04
RFO60	1/32806	1.084	1.109	1.02	1/31724	1.086	1.135	1.05
RFO140	1/29936	1.084	1.11	1.02	1/30916	1.086	1.139	1.05

Table 3. Amplification factor (β_z) due to the SSI (Borges. 2009).

Case	X-direction				Y-direction			
	ω	γ_z^{FS}	γ_z^{SSI}	β_z	ω	γ_z^{FS}	γ_z^{SSI}	β_z
1	1/5192	1.25	1.45	1.16	1/1312	1.18	1.61	1.36
2	1/21100	1.14	1.18	1.04	1/2936	1.2	1.26	1.05
3	1/119286	1.12	1.13	1.01	1/103214	1.1	1.11	1.01

- In order to predict the building settlements, it is essential to consider the interaction effect between the different elements of its foundation;
- The result of a load test on a single element cannot be directly compared with the measured settlements during building construction. While the first represents the behaviour of an isolated foundation, the latter is the result of the entire interaction process;
- Two actual cases of tall buildings on piles were presented, and they behaved very differently. The case with floating piles resulted in a relevant process of interaction between the piles, while the case employing fixed-tip showed nearly no interaction between the piles;
- In a set of 13 monitored constructions, values of *Rs* factor between 1 and 22 were observed, which made clear the increase of the measured settlement in relation to that of an isolated pile. This number depends on the number of piles, their proximity, and the soil profile;
- Calculating the building superstructure, considering the supports as rigid (conventional calculation) and the subsequent isolated calculation of the foundations, does not well represent the performance of the building in terms of load distribution and loads on the columns;
- The foundation-structure interaction process, or more simply denoted by soil-structure interaction (SSI), represents the coupling of the stiffness of the building parts and is closer to the construction behavior;
- A few steps of iteration (suggestion of 3) within the process of interaction between the foundation and the structure are sufficient for commercial designs. The first iteration already points out more than 2/3 of the changes coming from SSI;
- Columns that in the conventional calculation have settlements below the average of the building, which in general are the columns of the periphery, will have the tendency to increase the load and the settlements. On the other hand, the columns with initially predicted above average settlement will lose load and settle less in a load redistribution process;
- Using SSI, the predicted settlements result in lower values of angular distortions in the foundations, which may make possible the use of an alternative foundation that would not satisfy the criterion of maximum distortions for the conventional calculation;
- In the presented examples, the foundations in rafts or piled rafts presented reductions in the maximum bending moments in the order of 20 % to 50 %, when the SSI was employed. This points to the possibility of a more economical foundation design;
- The global stability parameter, γ_z, is also affected by the calculation with or without the SSI. When considering the interaction process between the foundation and the structure, the values obtained for γ_z were higher;
- The article presents a factor β_z to represent the ratio of the increase in the value of γ_z when it is calculated with and without SSI. The increase observed depends on the building's non-verticality, with a non-linear and accelerated growth as it approaches a ratio of 1:1000.

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