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REVIEW

Settlement monitoring and prediction towards an enhanced characterization of soil stratigraphy: a case study of a 23-story building

Monitoramento e previsão de recalques visando uma melhor caracterização da estratigrafia do solo: um estudo de caso de um edifício de 23 andares

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Abstract: The settlement is a critical factor in foundation selection. To ensure the safety and stability of a structure, engineers must accurately predict settlement and design the foundation accordingly. However, in cases where the soil stratigraphy is complex, a more detailed analysis is necessary to determine safe and cost-effective solutions for a building's foundation. This paper presents a case study in which settlement monitoring was performed on a 23-story building with spread footings laid on a stiff rock layer of limited thickness overlying more compressible layers of clay and sand. The building was monitored through four construction stages, during which data on vertical displacements and stress distribution on the soil were obtained. To improve the accuracy of the results, a methodology for calibrating the thicknesses of the soil strata was applied, based on the measured settlements, soil-structure interaction, and estimates from semi-empirical methods. Furthermore, a numerical analysis was performed using the finite element method, where more conservative displacements were observed. Finally, the safety of the building was evaluated by verifying angular distortions.

Keywords: settlement monitoring, settlement prediction, stress distribution, soil-structure interaction, numerical modeling.

Resumo: O recalque é um fator crucial na escolha de uma fundação. Para garantir a segurança e estabilidade de uma estrutura, os engenheiros precisam prever os recalques com um grau razoável de precisão e projetar a fundação de acordo. Entretanto, em casos em que a estratigrafia do solo é complexa, é necessária uma análise mais detalhada para determinar soluções seguras e econômicas para a fundação de uma edificação. Este artigo apresenta um estudo de caso no qual foi realizado o monitoramento de recalques de um prédio de 23 andares com sapatas assentadas em uma fina camada rochosa, sobrejacente a camadas mais compressíveis de argila e areia. O prédio foi monitorado através de quatro estágios de construção, para os quais foram obtidos valores de deslocamentos verticais e distribuição de tensões no solo. Buscando resultados mais precisos, uma metodologia para calibrar as espessuras das camadas de solo foi aplicada, com base nos recalques medidos, na interação solo-estrutura e nas estimativas de métodos semi-empíricos. Também foi realizada uma análise numérica pelo método dos elementos finitos, em que foram observados deslocamentos mais conservadores. Por fim, a segurança do prédio foi avaliada por meio da verificação de distorções angulares.

Palavras-chave: monitoramento de recalques, previsão de recalques, distribuição de tensões, interação solo-estrutura, modelagem numérica.

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1 INTRODUCTION

According to Darwin et al. [1], in order to restrict the settlement of structures, it is necessary to transmit their loads to a soil stratum of adequate strength and spread them over a sufficiently large area to minimize bearing pressure. To achieve this, shallow foundations are recommended as the primary solution due to their ease of construction and greater economy [2]. Additionally, historical cases demonstrate that the utilization of shallow foundations instead of deep foundations can lead to a reduction in structural cost of up to 20% [3].

Most of the time, a stiff top layer indicates that shallow foundations should be utilized. However, this layer has a significant impact on how vertical stresses are distributed, and estimates based on it may be wildly inaccurate [4]. Therefore, stresses and settlements should be meticulously predicted and observed.

In addition, it is essential to consider the uncertainty in the soil mass, as incorrect assumptions about the geological profile, such as the values used for strata thicknesses, can result in significant errors [5]. For design purposes, it would be impossible to drill enough boreholes to accurately determine the shape of the ground. In contrast, the measured settlements can provide a wealth of information about the subsoil conditions, which can be used to create a more accurate model of the soil across the entire site. This paper describes a calibration procedure for achieving this objective.

There are multiple methods for predicting settlement, but they all produce different results under identical conditions, making it difficult to determine which method to employ. To test prediction methods, it is crucial to observe and document how the structures of buildings function. In addition, if predictions are incorrect and problems arise, settlement observation during construction can alert the engineers so they can fix the issue before it becomes more severe.

Considering these aspects and the importance of efficient settlement prediction and monitoring, this paper aims to analyze a practical situation of a building with spread footings laid on a stiff rock layer overlying more compressible layers of clay and sand. By doing so, it contributes to a more reliable design of structures in similar cases.

2 FIELD CHARACTERIZATION

The analyzed building is a 23-story residential tower located in the city of Teresina/PI (Figure 1). With a height of 62.46 m, it features a basement, a ground floor, three garage levels, 16 typical residential floors, a mechanical floor, and a roof terrace.

Figure 1. Architectural model of the building.

The building is made of a reinforced concrete frame with a total of 69 columns, 24 of which are supported by spread footings, 44 by caisson foundations, and one is a floating column. The main focus of this investigation was the central core foundation, which bears the heaviest load and consists of 16 individual footings and one footing supporting four columns (see Figure 2).

The soil investigation employed both the Standard Penetration Test (SPT) and rotary drilling at three boreholes (BH-01, BH-02 and BH-03), as illustrated in Figure 2.

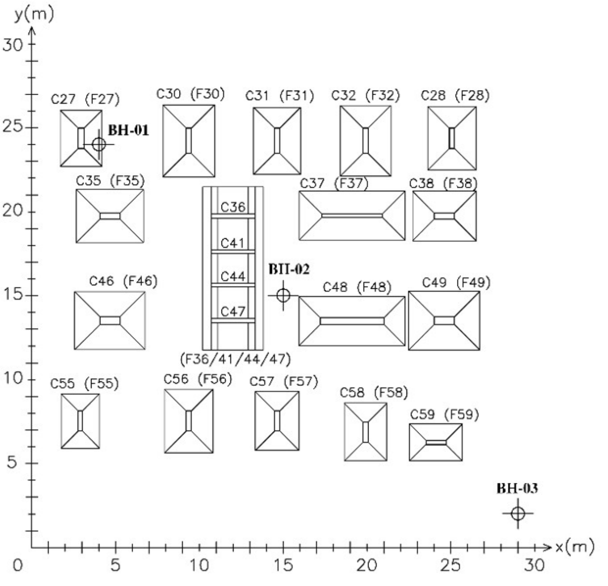

Figure 2. Locations of columns (C), footings (F), and boreholes (BH).

Figure 3 shows the soil profile of the boreholes, indicating the depths of the layers and their respective blow counts at each meter, along with the core recovery values (CR) of the rock layer. The stratigraphy reveals an upper layer composed of rocks fragments, silicified sandstone and quartz pebbles, underlain by a layer of soft sandy clay, and then a layer of very fine sand. Considering this arrangement, a spread footing solution for the central core of the building was established, taking advantage of the stiffness of the superficial rock layer.

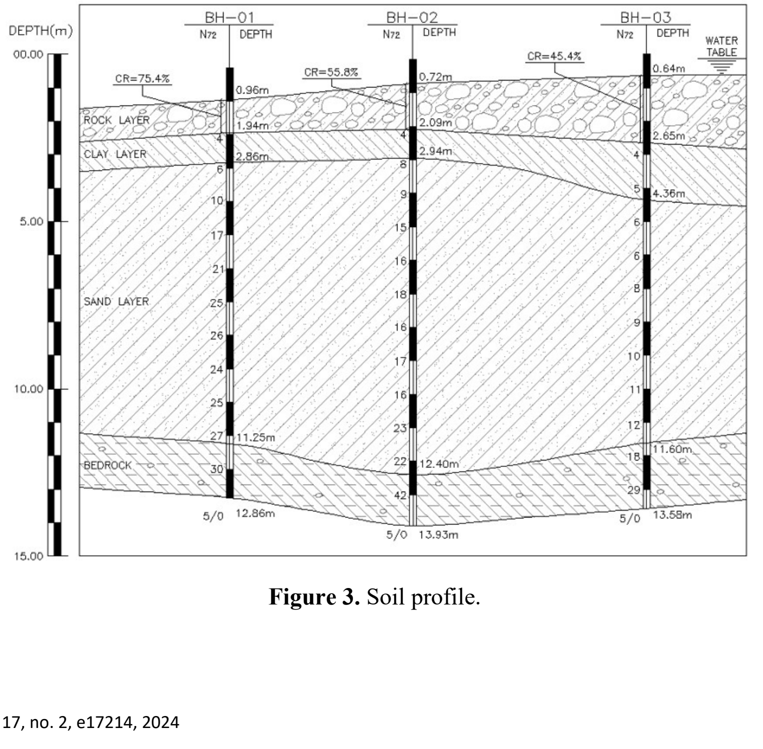

Figure 3. Soil profile.

3 SETTLEMENT MONITORING

The settlement monitoring was performed in the basement of the building through differential leveling, employing a digital level instrument with an accuracy of 1 mm/km and a bar-coded staff. This method involved determining the elevation disparities by measuring the height of the instrument and the reading of the staff, supported by steel rods fixed at 30 cm from ground level in each of the 20 monitored columns. The monitoring was carried out over four construction stages (Figure 4), allowing the settlements to be determined by comparing the elevation differences between consecutive stages. Table 1 presents the settlements, with the first stage serving as a reference point.

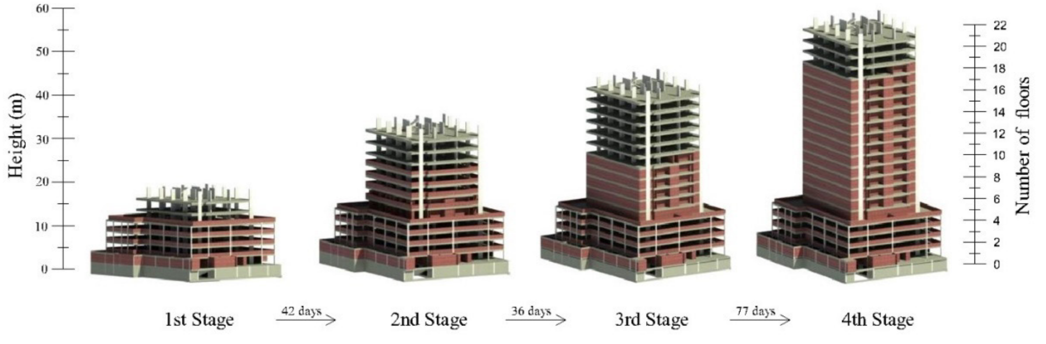

Figure 4. Monitoring stages.

Table 1. Measured settlements with reference to the 1st stage.

Column	Settlement (mm)		
	2nd Stage (Day 42)	3rd Stage (Day 78)	4th Stage (Day 155)
C27	-2.0	-2.3	-5.4
C28	-1.8	-4.1	-7.8
C30	-1.6	-3.7	-7.9
C31	-1.7	-5.0	-8.9
C32	-2.0	-4.2	-8.9
C35	-1.6	-2.9	-5.8
C36	-2.0	-4.0	-8.3
C37	-2.0	-4.2	-9.0
C38	-1.6	-4.5	-8.1
C41	-1.8	-2.6	-8.0
C44	-1.6	-4.0	-7.6
C46	-1.2	-1.5	-6.0
C47	-1.7	-3.4	-7.0
C48	-1.8	-3.4	-7.8
C49	-0.9	-2.6	-6.6
C55	-0.5	-0.9	-2.7
C56	-1.0	-2.1	-3.9
C57	-1.0	-2.2	-4.4
C58	-0.9	-2.1	-3.7
C59	-0.6	-1.6	-3.0

To ensure efficient leveling, it is recommended to establish at least two benchmarks that are properly protected and located at a considerable distance from each other. These criteria are essential to guaranteeing a backup benchmark in the event of the loss of the main benchmark. Additionally, it is important to plan a fixed path for all measurements, accounting for potential interference due to the construction process. Figure 5 illustrates the locations of the benchmarks and the adopted leveling path, where BM1 represents the primary benchmark, BM2 represents the secondary benchmark, and V0 to V5 represent the instrument positions.

The benchmarks consist of sufficiently long steel rods for attachment to the rock layer. In this case, BM1 was put close to the structure, which was not ideal because the structure's movement could cause the benchmark to move up or down, making it less reliable. To ensure this had not occurred, the levels were consistently compared using both benchmarks, considering that BM2 was located across the street, making it less susceptible to disturbances caused by the building's movements. The maximum difference observed was 0.3 mm, within the measuring instrument's accuracy limit, confirming that no significant displacement had occurred on the benchmarks and guaranteeing the safety of the determined settlement values.

This scenario demonstrates the importance of having at least two benchmarks, and it is important to note that while the results were unaffected in this instance, it is advisable to position them at a safe distance from the building and to continuously check for external interference.

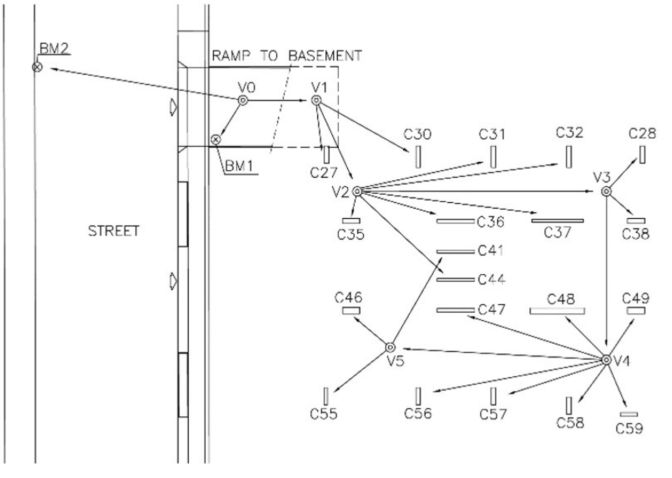

Figure 5. Leveling scheme.

4 STRUCTURAL LOADS AND STRESS DISTRIBUTION

The building's physical structure was modeled using Autodesk's Revit [6] software and then integrated with Autodesk's Robot [7] software to obtain the loads at the base of each column (Table 2). This procedure was carried out for every monitoring stage, making it crucial to record the building's condition at each phase to accurately model the structure later.

Initially, the footings were assumed to be fixed supports, and the structural loads consisted of dead loads from the reinforced concrete and masonry, whose values were obtained from the Brazilian standard ABNT NBR 6120/2019 [8]. Subsequently, the stresses on the building caused by the settlements will cause adjustments to these loads due to soil-structure interaction, which will be discussed later in this paper.

The loads obtained at the columns are then added to the weight of the footings and transmitted to the ground, causing modifications to the existing stresses on the soil. According to Burland et al. [4], the Boussinesq equations provide a reasonably accurate distribution of vertical stress changes in many practical cases. However, estimating these changes is more difficult when a stiff layer overlies more compressible layers. Hazzard et al. [9] demonstrated that Boussinesq overestimates the stresses when a rigid top layer is present, as it assumes a homogeneous material and does not account for the stiffness contrast between layers.

Another limitation of using Boussinesq's formula is related to the dimensions of the foundations. Taylor [5] has suggested that the loading area should have dimensions that are less than one-third of the depth to treat loads as point loads, which is not the case in the current scenario. Nonetheless, this problem can be solved by integrating the Boussinesq equations over a specified area. One of the most practical solutions to this is Newmark's method [10], which evaluates the case of a rectangular area that is uniformly loaded, even though it does not consider the stiffness contrast between layers.

Table 2. Loads at the base of each column per stage.

Column	Loads (kN)			
	1st Stage	2nd Stage	3rd Stage	4th Stage
C27	1235	1918	2468	3329
C28	1506	2160	2698	3513
C30	1357	2100	2713	3630
C31	1242	1962	2584	3543
C32	1207	1959	2574	3511
C35	1492	2165	2768	3736
C36	1278	1977	2630	3626
C37	1879	2960	4095	5614
C38	1112	1725	2303	3159
C41	734	1235	1770	2597
C44	752	1282	1806	2627
C46	1794	2657	3458	4585
C47	1456	2196	2897	3849
C48	2246	3548	4879	6495
C49	1585	2406	3192	4212
C55	1154	1580	1984	2579
C56	1271	1910	2456	3225
C57	1165	1709	2194	2847
C58	1130	1638	2091	2761
C59	1164	1578	1997	2523
Total	26757	40665	53557	71959

Therefore, to evaluate stress changes in this situation, a uniform distribution over an area was considered, increasing with depth through a load spread angle (φ_o). According to Barata [11], despite being simple, this approach approximates reality in the case of overloads from rigid structures, where the value of φ_o varies according to the soil's strength (Table 3).

Considering safety standards, the original foundation design of the presented building assumed that $\varphi o = 45°$ for the whole soil mass. However, in this research, it was adopted $\varphi_o = 70°$ for the rock layer, $\varphi_o = 40°$ for the soft clay layer and $\varphi_o = 45°$ for the sand layer.

Table 3. Values for the load spread angle (Kögler and Scheidig [12] apud Barata [11]).

Soil Type	Load spread angle
Very soft soils	$\varphi 0 < 40°$
Sands	$\varphi 0 \approx 40°$ to $45°$
Hard clays	$\varphi 0 \approx 70°$
Rocks	$\varphi 0 > 70°$

To emphasize the importance of considering the variance of stiffness among the soil strata, the vertical stresses obtained along the depth below the center of each footing were compared with the results obtained through Newmark's method. Figure 6 illustrates this comparison for F49, which provides a good example of the general behavior in this case, showing that the stresses would be far overestimated by Newmark's solution, especially at the interface of rock and clay (E_{rock} / $E_{clay} \approx 2300$), where it presents a result six times higher.

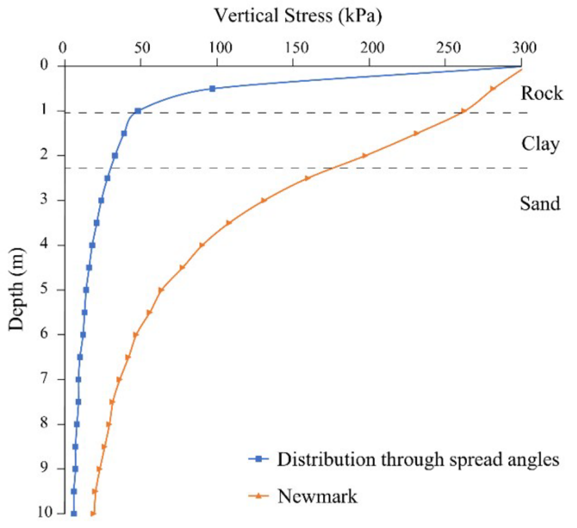

Figure 6. Vertical stresses below the center of F49

5 SEMI-EMPIRICAL METHODS

Semi-empirical methods are those in which a theoretically correct model, or an adaptation of it, is used with parameters obtained by correlation with in-situ tests, most commonly static (CPT) and dynamic (SPT) penetration tests [13]. Since the sand stratum is the thickest and most affected by immediate settlements, the following well-known semi-empirical methods were chosen to compare settlements on sand in this study:

- Barata [14]: The author investigated the settlement of shallow foundations in highly compressible soils and presented his modifications to the theory of elasticity in order to predict settlement under these conditions;
- Schultze and Sherif [15]: It was based on the observation of settlement at 48 sites with shallow foundations on sand. It was reported that the proposed method had an accuracy of ± 40% in predicting settlements;
- Berberian [16]: It was based on the theory of elasticity and data collected from a retrospective analysis of settlement monitoring and load tests at several sites with spread footings laid on sand;
- Schmertmann [17]: The author evaluated changes in the vertical strain distribution on sands and proposed a strain distribution diagram to calculate settlements over a zone of influence, as well as correction factors due to embedment and creep. This method was designed primarily for use with CPT data, but it can also be applied to SPT data using conversion ratios between blow count values and cone tip resistance.
- Schmertmann et al. [18]: Modification of Schmertmann [17] method, in which new diagrams are proposed for the strain distribution, mainly aiming to account for differences in footing shapes;
- Terzaghi and Peck [19], [20]: Based on relationships between the SPT blow count, footing width, and bearing capacity charts. In addition, the authors provided correction factors for embedment, water table, and blow count data;
- Meyerhof [21], [22]: The author revised Terzaghi and Peck [19], [20] methodology, finding it to be quite conservative. So, to obtain more accurate results, it was suggested an increase in the bearing capacity values;
- Anagnostopoulos et al. [23]: The authors suggested grouping settlement estimates by SPT blow counts and footing widths. This was based on a statistical evaluation of settlement measurements and back analyses.

The settlements on clay were estimated using the solution presented by Janbu et al. [24], which adapts the theory of elasticity to account for constant deformations within a finite layer of homogeneous elastic soil, representative of saturated clays. The total displacement at the base of each footing was calculated as the sum of the settlements of the clay and sand layers beneath the geometric center of the footing.

Formulations and calculation examples for the presented methods are available in Carvalho [25].

6 SOIL CALIBRATION

A soil calibration method was followed in this research to get a more realistic stratigraphy model based on Bowles' [26] finite element solution, which was used to obtain stresses and displacements in layered and anisotropic soils. The proposed calibration in this paper, however, used results from semi-empirical methods along with data from the monitored building and proceeded as follows:

- A soil investigation was carried out to identify the soil strata and thicknesses at three boreholes (Figure 3);
- The structure was monitored in stages to obtain the settlements (Table 1) and loads (Table 2) at each column;
- Two soil limit models were defined, based on the smallest and largest thicknesses found for the rock stratum;
- The settlements were estimated through semi-empirical methods for each of the two limit models defined;
- The limit model and semi-empirical method with the best results were selected;
- Initial consideration was limited to the ground stratum (sand) and the surface stratum (rock). In this step, the thickness of the rock layer was varied until the settlements of the selected semi-empirical method converged with the measured settlements at each column;
- The clay stratum was then added, and its thickness was varied until the estimated settlements converged again with the measured settlements at each column;
- For more accurate results, the soil-structure interaction was considered.

The soil investigation and building monitoring processes were previously detailed. To define the limit models, the boreholes that indicated the smallest (BH-01) and largest (BH-03) thicknesses of the rock stratum were used, since it is the most rigid layer. These values were then assumed for the entire soil mass, and the settlements were calculated using the selected semi-empirical methods. Figure 7 shows the settlement values obtained in each column for both boreholes. From there, it was possible to determine the thickness that best described the general reality of the soil and verify which semi-empirical method most closely matched the measured settlements.

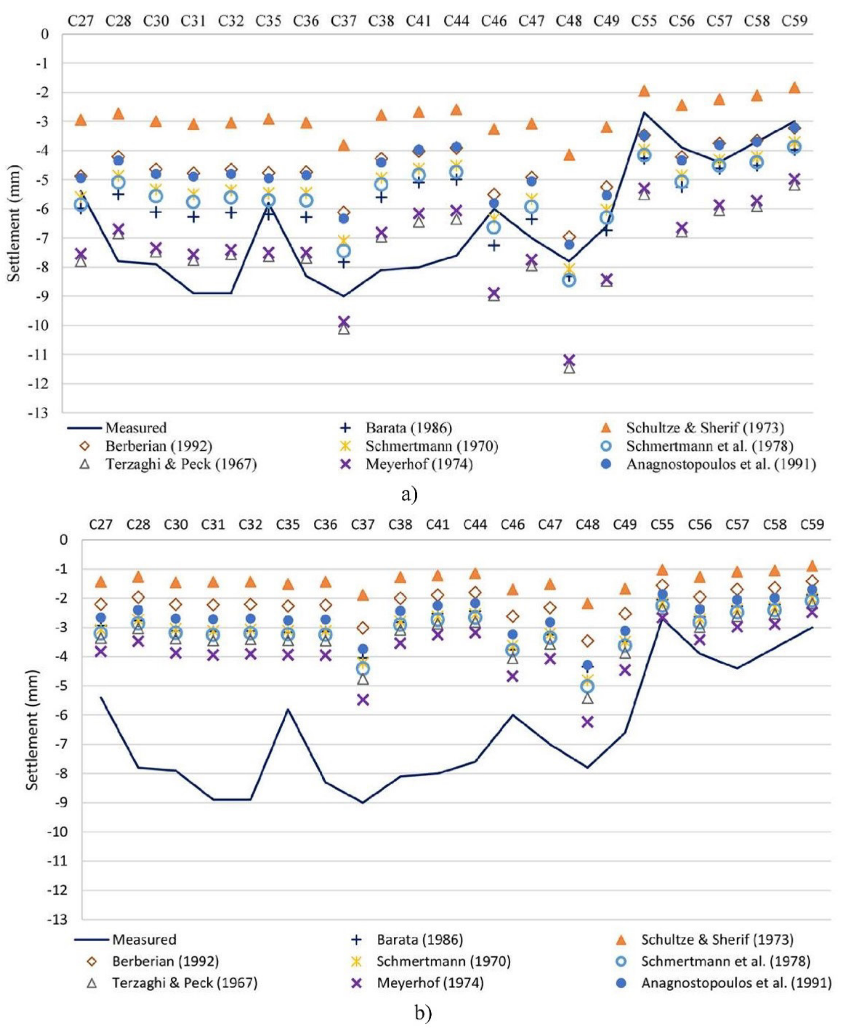

Figure 7. Settlements obtained with the values of: (a) BH-01; (b) BH-03.

The calculations based on the model with the greatest thickness of the rock layer (BH-03) resulted in inaccurate estimates. On the other hand, the model with the smallest thickness (BH-01) came closer to the measured values. Consequently, this model was chosen as the basis for calibration. To provide a more comprehensive analysis of the semi-empirical methods' results, the ratio of estimated settlements to measured settlements was evaluated for each column. The mean values and their respective coefficients of variation were calculated and are presented in Table 4.

Table 4. Mean values of the ratio estimated/measured settlements.

Method	BH-01		BH-03	
	Mean	**CV**	**Mean**	**CV**
Barata	0.97	28.1%	0.49	27.1%
Schultze & Sherif	0.46	24.8%	0.23	29.0%
Berberian	0.76	30.3%	0.36	28.7%
Schmertmann	0.88	30.1%	0.50	28.7%
Schmertmann et al.	0.92	30.0%	0.52	28.7%
Terzaghi & Peck	1.24	29.7%	0.55	28.4%
Meyerhof	1.20	29.5%	0.63	28.3%
Anagnostopoulos et al.	0.78	29.9%	0.43	28.5%

The coefficient of variation (CV) represents the ratio of the standard deviation to the mean and is a useful metric to evaluate the variability of results in relation to the mean [27]. Ideally, the mean should be 1.00 and the CV 0.0%, indicating minimal variability in the results. In this study, the calibration method proposed by Barata [14] was chosen as the reference due to its close approximation to the ideal results for the BH-01 model. An iterative process was then performed to adjust the thicknesses of the layers until the estimated settlements converged with the measured settlements, as previously described.

6.1 Soil-structure interaction

Iwamoto [28] demonstrated that considering soil-structure interaction provides numerous advantages, one of which is the possibility to estimate the effects of stress redistribution on structural elements as well as the form and intensity of differential settlements. By incorporating soil-structure interaction into designs, they can become more efficient and reliable.

In this instance, soil-structure interaction was incorporated into the calibration procedure to ensure the accuracy of the soil stratigraphy. The Winkler hypothesis was used to accomplish this. According to this hypothesis, settlements are directly related to the contact pressures between the foundation and the soil. Thus, the spring coefficient (Kv) can be determined by Equation 1, calculating the quotient between the load of each column (q) and its corresponding vertical displacement (w):

$$K_v = \frac{q}{w} \quad (1)$$

After obtaining the monitored settlements of each column and their loads, the initial spring coefficients were calculated and then imposed on each support within the structural model of the building, which initially considered only fixed supports. This procedure enabled the acquisition of new loads, which were substituted for the existing ones in Barata's [14] method. This required varying the thicknesses of the layers until the estimated and measured settlements converged. Consequently, using the same measured settlements and previously obtained loads, new values for the spring coefficient were generated. On the recommendation of Santos and Corrêa [29], the procedure was repeated iteratively until a stopping criterion (Equation 2):

$$\sum_{i=1}^{n^\circ\ of\ footings} \frac{\|P_i - P_i^*\|^2}{\|P_i\|^2} \leq \xi \quad (2)$$

Where P_i is the vertical load on the footing i in the current iteration; P_i^* is the vertical load on the footing i in the previous iteration and ξ is the tolerance, adopted $\xi = 10^{-3}$.

7 NUMERICAL MODELLING

Several of the limitations of semi-empirical methods for analyzing settlements can be overcome by employing numerical models. According to Burd and Frydman [30], the algorithms utilized by these models ensure that equilibrium and compatibility requirements are satisfied, or at least closely approximated, by transforming the soil mass into discrete elements.

This study utilized the finite element analysis software Plaxis 3D [31] to perform numerical modeling, which allowed for considering stress overlap among footings and evaluating its effect on settlements. This is a complicated phenomenon that is difficult to evaluate using semi-empirical methods. The behaviors of sand and clay were analyzed using the Mohr-Coulomb constitutive model, which considers the soil as a linear elastic-perfectly plastic material and requires the parameters: γ_{sat} (soil's unit weight), E (Young's modulus), ν (Poisson's ratio), c (cohesion), ϕ (friction angle) and ψ (dilatancy angle). On the other hand, the Hoek-Brown model was applied to the rock mass as it provides a better non-linear approximation of the rock behavior, requiring the parameters m_i (intact rock parameter) and *GSI* (Geological Strength Index).

Due to the absence of field and laboratory tests, the required parameters (Table 5) had to be estimated through back analyses from the monitored settlement values in conjunction with empirical correlations, following tabulated values recommended by Cintra et al. [32] for soils and Sjöberg [33] for rocks. Consequently, an ideal numerical model for this situation was unfeasible. Nevertheless, the model presented in this study still holds significance as a complementary analysis, which could be validated through the other presented methods and site instrumentation.

Table 5. Adopted values at each stratum.

Stratum	γsat (kN/m^3)	E (MPa)	ν	c (kN/m^2)	ϕ (°)	ψ (°)	*mi*	*GSI*
Rock (Sandstone)	24.5	29000.0	0.25	-	-	0	19	65
Clay	18	12.6	0.40	0.2	25	0	-	-
Sand	20	28.0	0.20	0.2	34	4	-	-
Bedrock	24.5	79000.0	0.25	-	-	0	22	100

To determine the settlements (Figure 8), the soil stratigraphy was modeled based on the calibration results that accounted for soil-structure interaction. The analysis was limited to the construction site because the primary focus of the modeling was on the vertical displacements of the footings and their interactions, ignoring any off-site effects. Nonetheless, it is evident that the area of influence of settlements extends significantly beyond the loaded region, and this expansion must be accounted for in settlement analyses conducted off-site, which could potentially affect neighboring structures. In the modeling, only the loads from the first and last construction stages were considered, ignoring the impact of the entire construction process.

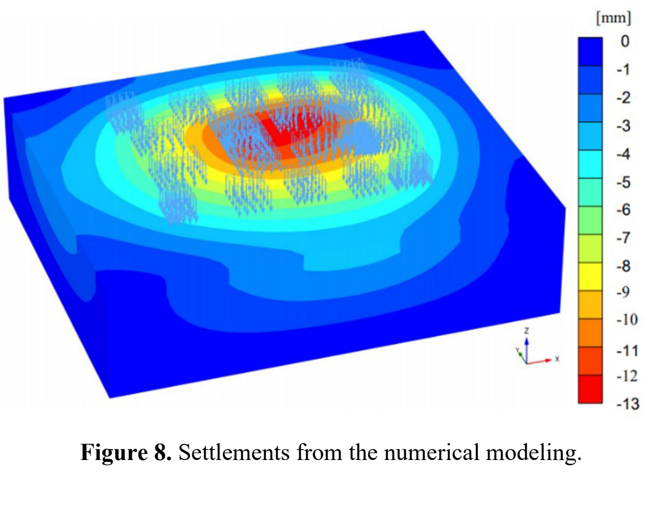

Figure 8. Settlements from the numerical modeling.

8 RESULTS AND DISCUSSIONS

8.1 Calibration results

Figure 9 displays the variation of strata thicknesses resulting from the calibrations. Among the strata presented, the rock stratum exhibited the least change, with a graphical pattern closely resembling the measured settlements, indicating that this stratum has the most significant influence over the structure's displacements. Since the clay and sand strata were considered complementary, their graphics are symmetrical, and due to the clay stratum's significant variation in proportion to its thickness, it can be regarded as the least influential stratum over the settlements.

Four iterations were needed to get the results for soil-structure interaction, and the most significant changes were seen between the first two. In the rock layer, the most substantial thickness reductions due to SSI were noticeable below the two most central columns, C41 and C44, while C35 and C55 exhibited the most considerable increases. It was also evident that there were only three increases in thickness along the clay layer due to SSI, resulting in three decreases in the sand layer below columns C27, C41, and C47, with a general tendency towards uniformization.

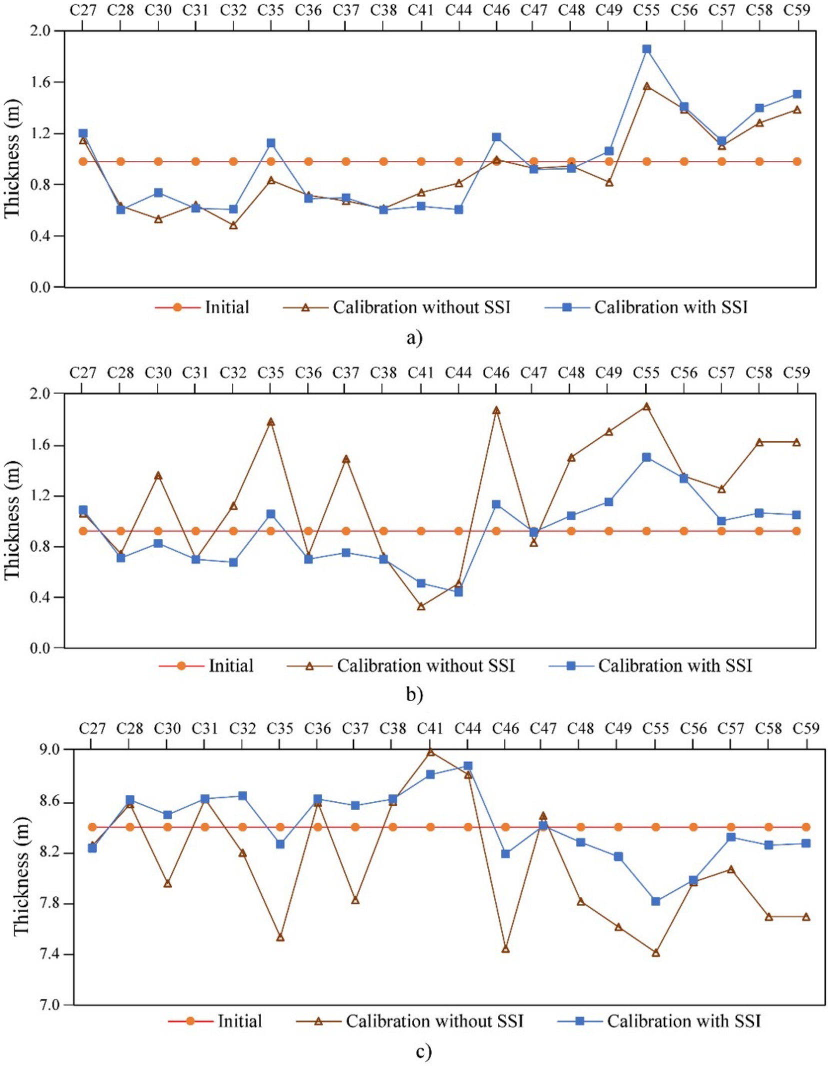

Figure 9. Variation of thicknesses on: a) Rock layer; b) Clay layer; c) Sand layer.

8.2 Analysis of the settlement estimate methods

The final settlement results for each method after the soil calibration, considering the effect of the soil-structure interaction, are displayed in Figure 10.

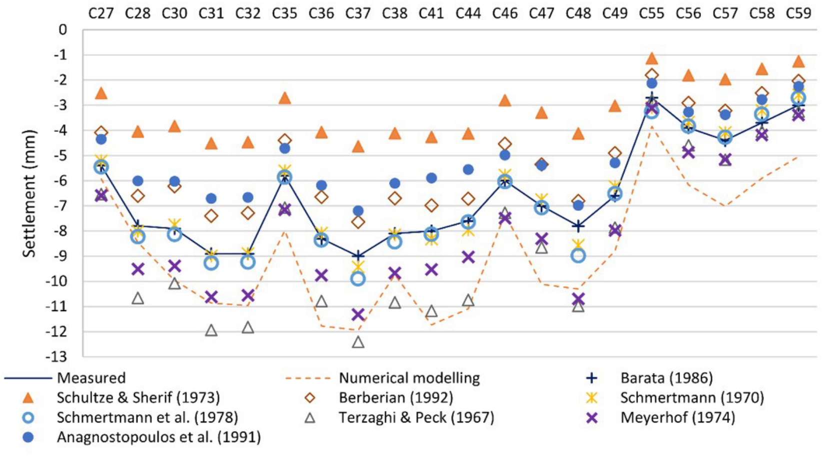

Figure 10. Final settlements after calibration.

Estimated-to-measured settlement ratios and coefficients of variation can be derived for each analyzed method (Table 6). Since Barata's [14] method was utilized during the calibration phase, it is evident that its results correspond to the measured settlements. The decrease in the coefficients of variation is quite significant when compared to the BH-01 model initially considered (refer to Table 4), which confirms the impact of calibration in achieving improved settlement prediction outcomes.

Table 6. Estimated/measured settlement ratio after the calibration.

Method	Mean	CV
Barata	1.00	0.0%
Schultze & Sherif	0.48	7.9%
Berberian	0.78	8.5%
Schmertmann	0.99	7.0%
Schmertmann et al.	1.02	6.8%
Terzaghi & Peck	1.26	9.0%
Meyerhof	1.20	4.4%
Anagnostopoulos et al.	0.78	5.3%
Numerical Modelling	1.37	12.3%

Regarding the semi-empirical methods, it is evident from Table 6 that the Schmertmann [17] and Schmertmann et al. [18] methods provided excellent approximations, while the Meyerhof [21], [22] and Terzaghi and Peck [19], [20] methods yielded more conservative results. On average, Berberian [16] and Anagnostopoulos et al. [23] underestimated the settlements by 22%, whereas Schultze and Sherif [15] presented the least reliable method, underestimating the settlements by 52%.

As anticipated, the numerical model results were the most conservative because the semi-empirical methods did not account for the stress overlap arising from the footings' proximity. Nevertheless, it is evident that the numerical model's conservativeness is not very far from the measured settlements, being only 37% higher on average. Similar conclusions were reached by Barata [14], who argued that the presence of a stiff rocky layer at low depths prevents significant stress overlap among the footings despite their proximity. Consequently, each footing's settlement is mostly due to the action of its own pressure bulb.

8.3 Analysis of angular distortions

The Brazilian Foundations Standard ABNT NBR 6122 [34] does not specify settlement limits for spread footings. However, Skempton and MacDonald [35] established that the allowable limit for angular distortions (β) should not exceed 1/300 to prevent cracking in buildings, recommending a more conservative limit of 1/500 for safety. Based on this guideline, the maximum angular distortion values for each axis of the building's footings were verified using the measured settlements (Figure 11). The results indicate that the vertical axes experienced greater angular distortions, with a maximum value of 1/1092 observed at the C32-C58 axis, thereby confirming the structural integrity of the building with respect to angular distortions.

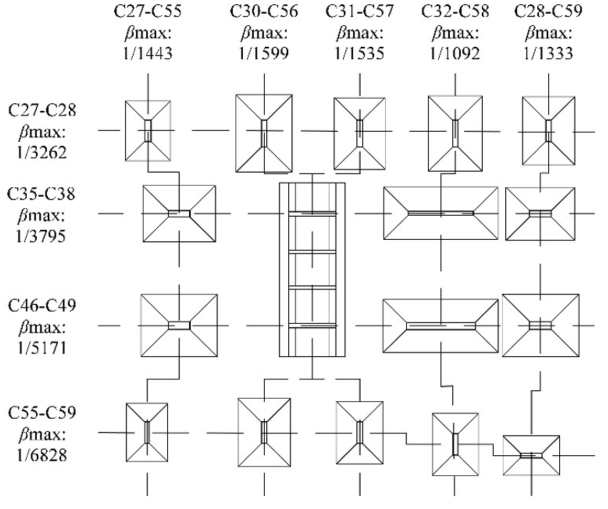

Figure 11. Maximum angular distortion per axis.

9 CONCLUSIONS

The present study focuses on a 23-story residential building supported by spread footings resting on a stiff rock layer, which overlays more compressible layers of clay and sand. To evaluate the vertical stress distribution, a uniform load spreading method was used, with different angles determined by the soil type. This simple yet effective approach provided more reliable values compared to Newmark's [10] method, which tended to overestimate the stresses.

Settlement monitoring was conducted over four stages of construction using the differential leveling method. To ensure reliable results, several measures must be taken, such as establishing at least two well-protected benchmarks, planning the measurement stages, and thoroughly documenting the building's condition at each stage.

To enhance the precision of settlement monitoring, a methodology was implemented to calibrate the thickness of the soil strata, considering soil-structure interaction. This process led to adjustments in the strata, including an increase in thickness of the rock layer below the central columns and a tendency toward uniformization of the clay and sand layers.

The calibration process also enabled a better evaluation of the semi-empirical methods for settlement prediction, leading to the following conclusions: Barata's [14], Schmertmann's [17], and Schmertmann et al.'s [18] methods provided excellent settlement estimates, while Anagnostopoulos et al. [23], Berberian [16], and Schultze and Sherif [15] underestimated the settlements, with the latter presenting the most distant results. Terzaghi and Peck [19], [20] and Meyerhof [21], [22] presented somewhat conservative results, but still good approximations.

Given that the semi-empirical calculations did not account for stress overlap between footings, a finite element analysis was conducted to evaluate this effect on settlements. The analysis revealed that settlements were, on average, only 37% higher than the measured values. This suggests that the presence of a stiff rock layer at shallow depths prevented sensitive stress overlap among the footings, causing the system to behave like a natural raft. Therefore, it was determined that the stratigraphy presented was favorable for the design of spread foundations, allowing for a more cost-effective solution.

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