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Analysis of the stress-strain behavior of compacted rock block layers in shallow foundation

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Article

Keywords
Compacted rock blocks
Shallow foundations
Three-dimensional analysis

Abstract

In geotechnical engineering, using layers of compacted rock blocks has become a practical alternative for increasing soil bearing capacity, especially for shallow foundations. This study aims to analyze the stress-strain behavior of isolated (rectangular) footings supported on this type of material. For this, the Mohr-Coulomb shear strength parameters obtained through back-analysis (under axisymmetric conditions) of a plate load test performed during a preliminary geotechnical investigation were used. These shear strength parameters were applied in both analytical methods (using the GEO5 software) and three-dimensional numerical modeling (considering the state of three-dimensional stresses and strains) with the RS3 software, employing the Finite Element Method (FEM). The results demonstrated consistency between the approaches and confirmed that the layer of compacted rock blocks significantly enhances bearing capacity, redistributes stress, and reduces settlement. Therefore, this study provides a solid technical basis for adopting this geotechnical solution in shallow foundation projects. The comparison between analytical and numerical methods reinforces the reliability of back-analysis procedures for determining shear strength parameters. Additionally, the results indicate that this type of material can serve as a viable and sustainable alternative to improve foundation performance on low-capacity soils. The study also emphasizes the need for further research under different loading conditions and layer thicknesses to optimize design and deepen understanding of this construction system's behavior in the field.

1. Introduction

According to Alkhamis & Al-Rashidi (2023), enhancing soil conditions is essential for ensuring the structural performance and safety of civil engineering projects. The authors emphasize that many construction failures are linked to inadequate foundation soil properties, highlighting the importance of thorough geotechnical investigations. Despite inherent challenges, conducting fieldwork remains vital for understanding soil behavior and guiding the selection of appropriate technical solutions. For soils with low shear strength, stabilization or replacement techniques become necessary to ensure proper performance and reduce deformations (Ismael, 2000). In this context, Lemos et al. (2025) proposed replacing the original material with a layer of compacted rock blocks, thereby providing greater bearing capacity and minimizing settlements, thereby facilitating the construction of a shallow foundation. The shear strength parameters of this layer were determined through back-analysis of a plate load test conducted under drained conditions, yielding satisfactory results even without direct laboratory or field test measurements.

In geotechnical engineering, traditional methods for estimating bearing capacity are still widely used (Akagwu & Aboshio, 2016; Acharyya & Dey, 2021), although recent technological advances have encouraged the use of software based on the Finite Element Method (FEM). The application of FEM to solve various geotechnical problems has proven to be highly effective (Sivakumar Babu & Srivastava, 2007; Millán et al., 2021). Several studies have also explored the bearing capacity of different soil types using analytical, numerical, and experimental approaches (Griffiths, 1982; Manoharan & Dasgupta, 1995; Hjiaj et al., 2005; Martin, 2005; Acharyya, 2019; Chihi & Saada, 2022).

However, there is a lack of study exploring the three-dimensional behavior of shallow foundations resting on compacted rock block layers, especially those that involve calibrating geotechnical parameters obtained from back-analysis of plate load tests. Gaining a better understanding of this behavior is crucial for designing safer, more stable, and cost-effective foundations. Accordingly, the present study uses the parameters from Lemos et al. (2025) to estimate the bearing capacity of a rectangular foundation on a compacted rock block layer.

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Analytical evaluations and three-dimensional numerical simulations were conducted, accounting for the foundation's actual dimensions. The analyses were performed using GEO5 (Fine) and RS3 (Rocscience). GEO5 was used for analytical assessments based on classical methods by Brinch Hansen (1961), Meyerhof (1963), and Vésic (1975), while RS3, based on the Finite Element Method (FEM), provided detailed three-dimensional insights into the foundation's behavior under different loading conditions. Finally, settlements under service loads were examined to better understand soil–foundation interaction and to support the design of safer, more efficient shallow foundations.

2. Materials and methods

2.1 Back-analysis of the plate load test

The study by Lemos et al. (2025) was conducted on low-bearing-capacity soil at the Residencial Brisas do Parque development, located in Fortaleza, Brazil. To enhance the site's geotechnical conditions, the original soil was replaced with a compacted rock block layer, aiming to increase strength and reduce settlements. The plate load test was performed in accordance with NBR 6489 (ABNT, 2019) to evaluate the behavior of the approximately 0.89 m-thick layer. It is important to mention that the influence of suction was not considered in this study, as the analyses were based on saturated conditions consistent with the plate load test and the adopted geotechnical parameters.

The back-analysis of the test results, shown in Figure 1, was carried out using RS2 (Rocscience) software, applying the Finite Element Method (FEM) in an axisymmetric model. A perfectly elastic–plastic behavior was adopted, according to the Mohr–Coulomb criterion, to determine the shear strength and deformability parameters of the compacted rock block layer.

The validation of the parameters was performed by comparing the pressure–settlement curve measured in the plate load test with the numerical response generated using the calibrated parameters, as shown in Figure 2. The values obtained from the back-analysis are presented in Table 1, including the unit weight, Young's modulus, Poisson's ratio, the internal friction angle, and cohesion of the analyzed materials. The Poisson's ratio values adopted in this study were selected based on typical ranges reported by Thota et al. (2020), who state that the saturated Poisson's ratio varies approximately between 0.2 and 0.5. The values used in the present analysis fall within this interval, although they are closer to the upper bound.

The comparison shows strong agreement between the experimental data and the back-analysis results. The difference in maximum pressure was limited, and the predicted settlements closely matched the measured values, indicating that the adopted parameters adequately represent the mechanical behavior of the compacted rock block layer.

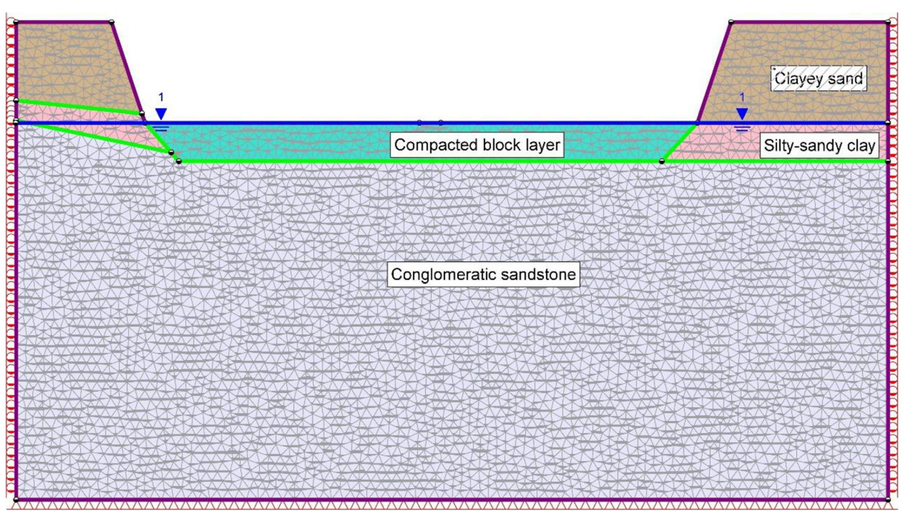

Figure 1. Back-analysis of the test results using RS2 (Rocscience) (adapted from Lemos et al., 2025).

Table 1. Shear strength and deformability parameters obtained from the back-analysis of the plate load test (Lemos et al., 2025).

Material	Unit weight (kN/m^3)	Young's modulus (kPa)	Poisson's ratio	Friction angle (º)	Cohesion (kPa)
Clayey sand	19	20000	0.35	12.4	79.7
Silty-sandy clay	21	20000	0.35	12.4	79.7
Compacted block layer	19	468000	0.49	23.0	110.0
Conglomeratic sandstone	22	468000	0.49	20.0	40.0

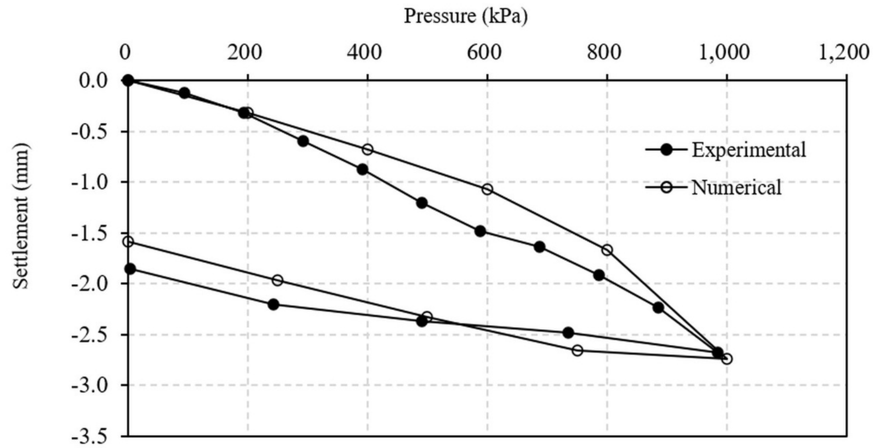

Figure 2. Comparison between pressure–settlement curves obtained from the plate load test and the back-analysis (adapted from Lemos et al., 2025)

Based on the results, a three-dimensional model of the rectangular footing was developed, enabling a more comprehensive evaluation of the soil–foundation system behavior under different loading conditions.

2.2 Bearing capacity estimates

In this study, the software "Footing" from the GEO5 package was used, which employs analytical methods to estimate the bearing capacity of the soil–foundation system. The program requires defining the soil stratigraphy and the geotechnical properties of the involved layers, taking into account the loads transmitted by the structure, the footing's plan dimensions, and the foundation level.

The bearing capacity is determined using well-known theoretical models, such as those proposed by Brinch Hansen (1961), Meyerhof (1963), and Vésic (1975), along with international standards including DIN 4017 (DIN, 2006), CTE DB SE-C (España, 2019), B1/VM4 (2014), SP 22.13330.2016 (2016), and CSN 73 1004 (ČSN, 2020), ensuring alignment with globally accepted engineering practices.

The allowable bearing stress is determined by applying a safety factor to the calculated bearing capacity, as recommended by NBR 6122 (ABNT, 2022), to ensure the foundation operates within acceptable safety limits. The loads transmitted by the columns to the isolated footings must account for both permanent and variable actions, weighted by the partial safety factors established in NBR 8681 (ABNT, 2025). The resulting design loads are then used as input data for the foundation analysis, enabling the software to provide a comprehensive assessment of bearing capacity.

2.3 Stress–strain analysis of the footing

For the three-dimensional numerical analysis, the RS3 software from Rocscience was used to evaluate stresses and strains. It is important to note that the three-dimensional analysis was necessary because an axisymmetric or plane-strain model cannot accurately represent the behavior of a rectangular footing. To simulate the behavior of foundation soil, the perfectly plastic, elastic Mohr-Coulomb constitutive model was adopted.

The fundamental assumption of this study is the treatment of the compacted rock block layer as a highly fractured rock mass, whose behavior can be reasonably represented by a generalized Hoek-Brown model with perfectly elastic–plastic response. The derivation of Mohr–Coulomb parameters based on the approach proposed by Hoek et al. (2002) aimed to enable their application in analytical formulations for bearing capacity estimation, as detailed in this study. To ensure consistency between analytical and numerical approaches, the same Mohr–Coulomb constitutive model was adopted in the numerical simulations. It is acknowledged that the use of more advanced constitutive models (e.g., strain hardening/softening or nonlinear elasticity) may provide additional insights; however, their application within the context of rock mechanics for this type of material still requires further investigation.

The initial geometry was modeled in RS2 by Lemos et al. (2025) and later converted to RS3 for this study. To better address the problem, a larger model area than the footing itself was selected. This approach enhances understanding of how applied loads propagate into depth and provides detailed insights into the resulting displacements. The numerical ground model was defined with the following main dimensions: a width of 8.00 m, a length of 18.28 m, and a depth of 7.898 m, measured from the top of the compacted rock layer to the bottom of the soil layer. The footing, measuring 1.90 m × 3.00 m (width × length), is centered on the surface of the modeled area.

The boundary conditions ensure a unique solution to the problem. The lateral surfaces (vertical faces) were constrained to restrict displacements in the x and y directions, preventing horizontal movements along these faces.

At the bottom interface, corresponding to the base of the soil layer, displacements were entirely restricted in the x, y, and z directions, simulating a rigid support. To generate the pressure-displacement graph, nine loading stages were applied, with the vertical load on the footing increasing from 0 to 3000 kPa. As shown in Figure 3, the 3000 kPa values at the footing's edges indicate that this load is evenly distributed over the entire foundation area, not a point load.

The numerical analysis was performed by discretizing the model with a finite element mesh, dividing the domain into small regions, thereby enabling the evaluation of stress–strain behavior. The mesh was created using four-node tetrahedral elements, with uniform grading and user-controlled distribution. The element size was set to 0.3 m, resulting in approximately 394,757 elements in the model, as shown in Figure 4.

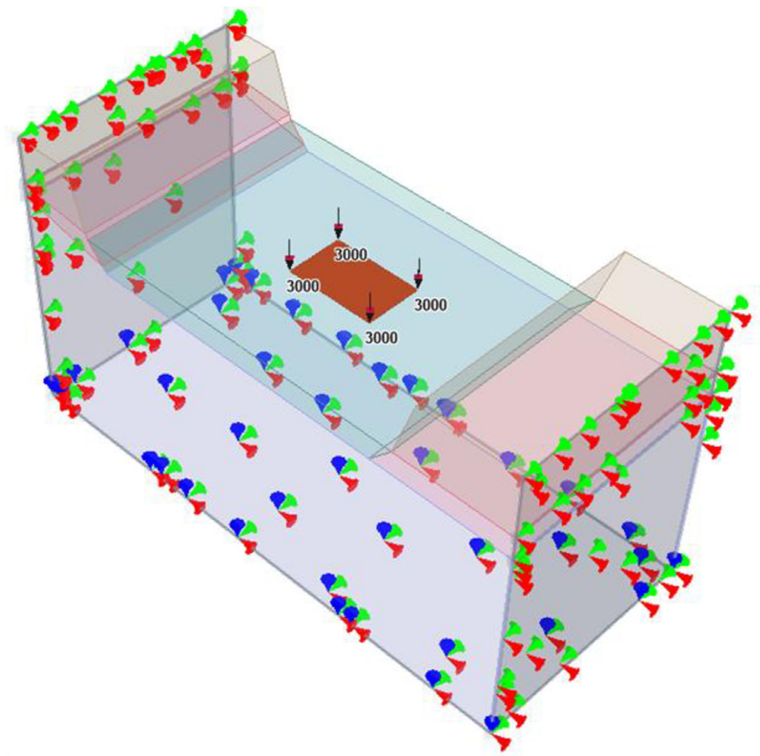

Figure 3. Boundary conditions applied in the numerical model, showing the compacted rock block layer, clayey sand, silty-sandy clay, and conglomeratic sandstone.

Figure 4. Schematic representation of the finite element mesh.

Although the strength and deformability parameters were obtained from an axisymmetric back-analysis of the plate load test, their use in the three-dimensional model is justified by the understanding that the axisymmetric representation of a circular plate provides a simplified but consistent approximation of the three-dimensional stress-strain material behavior for the investigated material. Therefore, both the strength parameters and boundary conditions derived from the axisymmetric analysis were adopted in the 3D numerical model. The reliability of this extrapolation may be further assessed through ongoing field instrumentation of the foundation system, which will allow future validation of the numerical predictions.

3. Analysis and results

3.1 Bearing capacity estimation using GEO5 software

Figure 5 compares soil bearing capacity estimates calculated with GEO5. The results demonstrate that the Meyerhof (1963) method provides the highest bearing capacity, followed by the Brinch Hansen (1961) method, and then the Vésic (1975) model.

The differences between the Brinch Hansen (1961) and Vésic (1975) models are slight, indicating that both use similar approaches to shape and depth factors. It is important to note that each technique may produce slightly different results due to its specific theoretical basis. Furthermore, with a safety factor of 3.0 and given the method's theoretical nature, all bearing capacity estimates met the design allowable stress of 500 kPa. The Meyerhof (1963) method achieved a safety factor of 4.04, followed by Brinch Hansen (1961) with 3.19, and Vésic (1975) with 3.11. Nonetheless, all three methods show overall consistency, confirming that these approaches are reliable for estimating bearing capacity in geotechnical studies. The superior performance of the Meyerhof (1963) model may be due to its broader inclusion of correction factors related to load inclination and foundation shape, which could have influenced the bearing capacity results. This underscores the importance of choosing an analytical method suitable for the soil and loading conditions. Combining analytical and numerical methods improves the analysis of soil–foundation interaction.

3.2 Three-dimensional analysis using RS3 software

The three-dimensional analysis was performed to carefully evaluate the system's behavior, incorporating the properties of the geological-geotechnical profile obtained through back-analysis of the plate load test. The actual dimensions of the footing, as specified in the structural design, were used. Therefore, in Figure 6, it is shown that for pressures from 0 to 1000 kPa, displacement gradually increases to 1.54 mm; however, it rises more rapidly between 1250 and 2000 kPa, reaching 12.58 mm. Beyond 2000 kPa, the displacement increases sharply, reaching 36.35 mm at 3000 kPa.

It is observed that the footing remains within operational limits up to a pressure of 2500 kPa, as the vertical displacement remains below the recommended threshold of 25 mm, as reported by Terzaghi et al. (1996). In other words, beyond this pressure, as shown by the three-dimensional numerical simulation, the foundation element would no longer be sufficient.

The stress–strain analysis of the foundation soil, based on the curve, indicates elastic–perfectly plastic behavior. Up to approximately 500 kPa, the curve remains in the elastic regime, where deformations are still recoverable. Beyond this point, plastic deformations occur.

The results shown in Figure 5 were overlaid in Figure 6 to allow a direct comparison of the methods analyzed. The method proposed by Meyerhof (1963) produced the highest bearing capacity, corresponding to an approximate settlement of 13 mm – a value below the recommended limit.

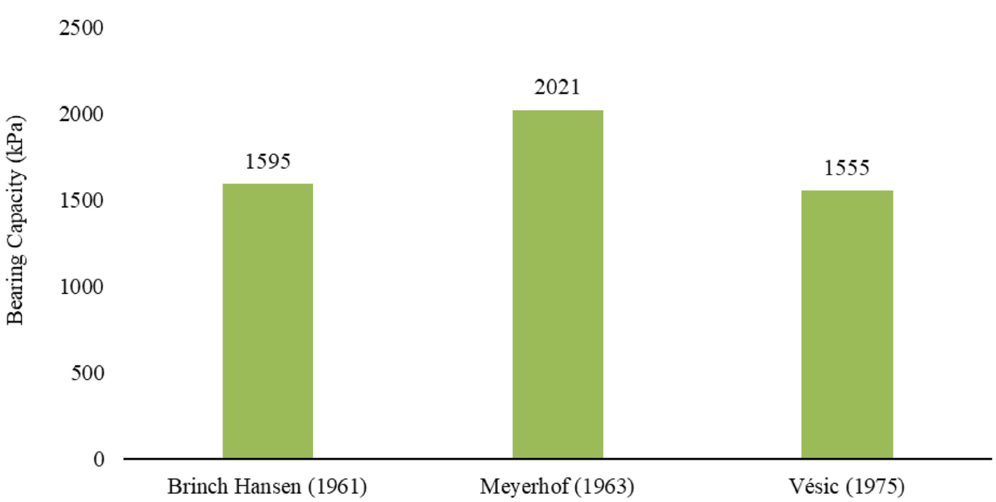

Figure 5. Comparison of the actual footing bearing capacity using analytical methods.

Figure 6 also shows a behavior similar to that obtained through the two-dimensional numerical modeling (back-analysis of the plate load test) using the axisymmetric model studied by Lemos et al. (2025). In both cases, the displacement increases progressively with increasing pressure, reinforcing the consistency of the analyses.

The soil's stress distribution is shown by a color gradient, known as the "pressure bulb", which highlights areas experiencing higher stress due to a maximum pressure of 3000 kPa (Figure 7).

Figure 7 shows the pressure bulb center in deep blue as the area of highest stress, located directly beneath the footing. As the distance from this center increases, the colors lighten, illustrating the vertical distribution of stress and how it decreases with increasing distance from the point of application.

The pressure bulb is independent of the material type, as it is a geometric phenomenon. Even in the presence of a soft clay layer–or when the bulb is entirely within rock–the material's capacity may lead to footing failure or excessive settlements. When this clay is replaced with high-quality material, the pressure bulb remains in the rock but now exhibits satisfactory behavior. This was confirmed in the direct load test. The pressure bulb remains unchanged with the material replaced, restoring the desired conditions. If the rock layer can support this pressure, the underlying material–conglomeratic sandstone–can also sustain it. As the stiffest layer in the system, the sandstone acts as the main supporting base. In Figure 7, it was identified that the pressure bulb reached approximately 6.23 m. Although there is a slight difference compared to the estimated 5.70 m for rectangular footings, according to Cintra et al. (2011), the values are close, indicating the results' consistency.

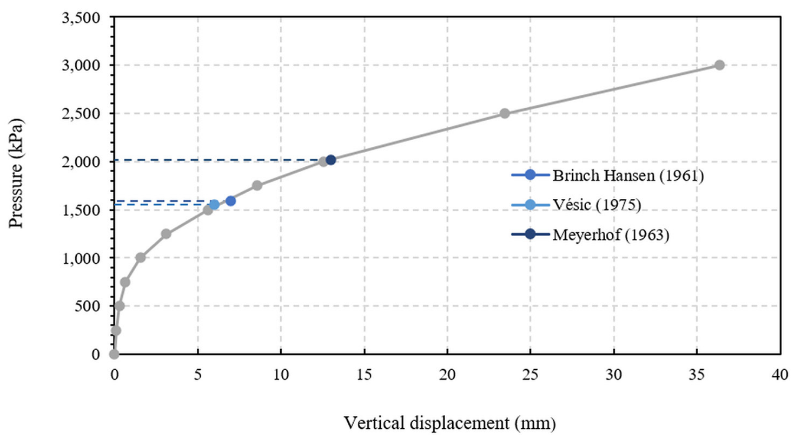

Figure 6. Pressure vs. vertical displacement graph obtained from numerical extrapolation.

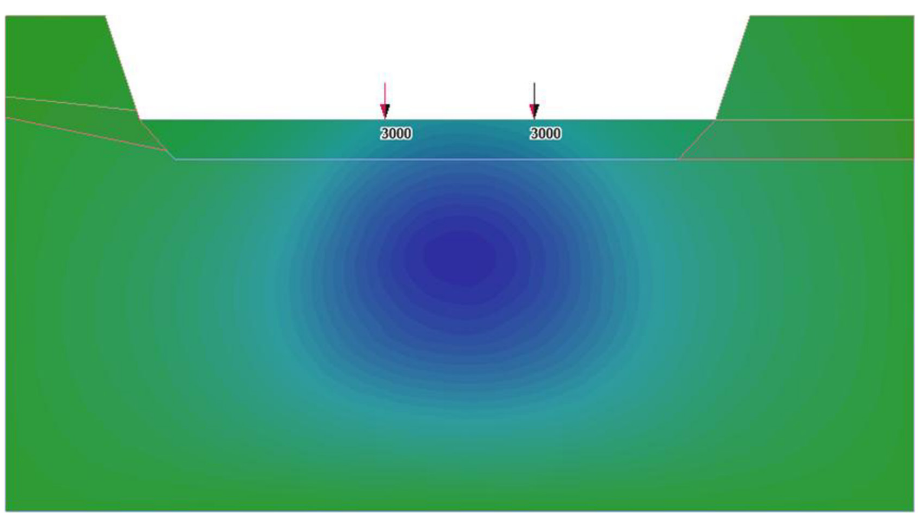

Figure 7. Pressure bulb beneath the footing obtained from the RS3 numerical model under a pressure of 3000 kPa.

The presence of the compacted rock block layer is crucial in limiting stress propagation to deeper regions. This layer provides additional strength, acting as a structural "barrier" that helps dissipate and distribute the stresses.

Figure 8 shows the pressure field from numerical extrapolation, illustrating the relationship between applied pressure and the vertical displacement of the footing. It is observed that the plasticization area is mainly located beneath the center of the footing, gradually extending toward the edges. This suggests that the applied pressure has surpassed the elastic limit of the soil.

The color transition — from red in the most stressed areas to blue in the less affected regions — indicates a decrease in stress as the distance from the point of maximum pressure increases. This distribution reflects how stresses dissipate into the lower and lateral layers, where the ground stays within the elastic range. The visualization suggests that the load is generally well distributed, but the center of the footing bears the highest stresses. The soil has been compressed to the point where it can no longer withstand additional stress without permanent deformation; further pressure increases will lead to greater plastic deformation.

Overall, the observed behavior aligns with expectations for shallow foundations, where the load is generally well distributed, but stresses are more concentrated at the center of the footing. The study had several limitations, mainly due to variability in geotechnical properties caused by the natural heterogeneity of soils and rocks. The complexity of geotechnical behavior required simplifications in the numerical simulations, which, although effective, might not fully capture real field conditions.

The numerical modeling highlighted important aspects related to the stiffness contrast between the compacted rock block layer and the underlying sandstone. The high stiffness of the compacted layer facilitated a more uniform transfer of stresses at the interface, reducing the sharp gradients often seen in heterogeneous geotechnical profiles. The confinement effect provided by the surrounding soil was also evident. The lateral restriction acted as a natural retaining boundary, leading to a slight increase in the apparent bearing capacity due to the development of passive resistance along the edges of the mass. The three-dimensional analysis revealed that the displacement vectors beneath the footing follow a downward and outward path, creating a distinctive deformation bulb consistent with evidence from full-scale plate load tests. Overall, the results enhance the understanding of the combined behavior between the compacted layer and the natural subgrade. It was confirmed that using granular or block-reinforced materials increases the overall stiffness of the foundation system, promotes even more stress distribution, and reduces the risk of progressive failure under repeated or long-term loading conditions. In this context, Table 2 presents a comparative summary of the results obtained through analytical methods and numerical modeling, allowing an integrated interpretation with the stress–strain behavior observed in Figure 6.

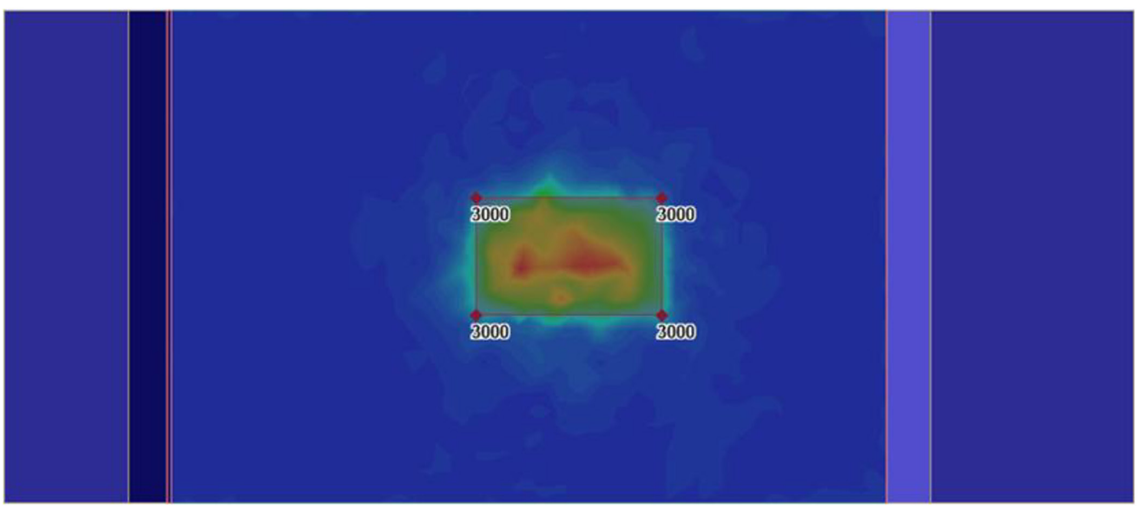

Figure 8. Plastic strain distribution beneath the footing obtained from the three-dimensional numerical analysis.

Table 2. Summary of analytical and numerical results for bearing capacity.

Method	Bearing capacity (kPa)	Estimated settlement (mm)
Meyerhof (1963)	2021	~13
Brinch Hansen (1961)	1595	~ 7
Vésic (1975)	1555	~ 6
RS3 numerical model	2500	25

The Meyerhof (1963) method yields a bearing capacity of 2021 kPa with a settlement of approximately 13 mm, whereas Brinch Hansen (1961) and Vésic (1975) indicate lower bearing capacities (1595 kPa and 1555 kPa) associated with smaller settlements (~7 mm and ~6 mm, respectively). In contrast, the numerical model (RS3) provides the highest bearing capacity (2500 kPa), accompanied by the largest settlement (25 mm).

4. Conclusion

The article outlined the integration of plate load test back-analysis, analytical methods, and three-dimensional modeling as a practical approach for assessing the stress–strain behavior of shallow foundations on compacted rock block layers. Calibrating geotechnical parameters via back-analysis was crucial for validating the numerical model, as the results demonstrated strong agreement with the experimental data. These calibrated parameters provided a reliable foundation for three-dimensional modeling, enabling a realistic simulation of soil–foundation interaction.

The three-dimensional analysis stood out for its ability to more accurately simulate the stress and strain states within the mass, highlighting the performance of the compacted rock block layer under applied loads. The three-dimensional model also enabled the assessment of plasticization mechanisms based on the Mohr-Coulomb failure criterion. The results showed that the foundation could withstand the maximum design pressures, meet the established safety factor, and keep settlements within permissible limits. Therefore, the three-dimensional modeling proved to be a valuable tool for designing shallow foundations.

Even considering the simplifications inherent in numerical simulations, the results aligned with both theoretical and experimental data, confirming the reliability of three-dimensional modeling in the assessment and design of shallow foundations on improved geotechnical materials. Additionally, it is recommended that future studies conduct a sensitivity analysis of the boundary conditions used in the numerical model. This approach will help evaluate how boundary constraints influence the distribution of stresses and strains and the overall response of the foundation system. Future research should also include a sensitivity analysis of the adopted geotechnical parameters and, if possible, their validation through laboratory testing, especially for the foundation soil. Such analyses will also help in understanding the behavior of the construction system through numerical simulations. Finally, further research may support the development of specific design guidelines for shallow foundations on compacted rock block layers, advancing geotechnical engineering applied to reinforced soils and unconventional materials.

Building on the results obtained, it is recommended that future research expand the investigation of the behavior of compacted rock block layers under various loading conditions and geotechnical variations. Field monitoring of full-scale foundations is also advised to validate numerical predictions and improve model performance, thereby enhancing the reliability of the results. This integration of simulation and field observation is crucial for advancing the design methods of shallow foundations resting on compacted rock block layers. Progress in these studies may lead to the development of technical guidelines and practical design recommendations, thereby contributing to safer, more efficient solutions for improved soils.

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Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Bruna Leticia da Silva Lemos: conceptualization, data curation, visualization, writing – original draft. Silvrano Adonias Dantas Neto: methodology, supervision, validation, resources, software, writing – review & editing.

Data availability

Data generated and analyzed in the course of the current study are available in the institutional repository of the Federal University of Ceará repository, https://repositorio.ufc.br/handle/riufc/81196.

Declaration of use of generative artificial intelligence

This work was prepared without the assistance of any generative artificial intelligence (GenAI) tools or services. All aspects of the manuscript were developed solely by the authors, who take full responsibility for the content of this publication.

List of symbols and abbreviations

ABNT	Brazilian Association of Technical Standards
FS	Safety Factors
GSI	Geological Strength Index
FEM	Finite Element Method

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