

Numerical analysis of deep-anchored excavations in residual soil with reliquiaie layers

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Article

Keywords

Reliquiae layer
Residual soil
Numerical analysis
Anchored wall
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Abstract

The coastal region of Boa Viagem and Icaraí, located in the city of Niterói/RJ, has been the subject of several studies focusing on its geotechnical and geological characteristics, particularly due to the presence of a major fault extending into Guanabara Bay. Reliquiae weak layers are widespread in the local soil, with strength parameters significantly different from those of the surrounding residual soil, therefore commanding the stability conditions of excavations in the area. To support the construction of a new residential building complex near Boa Viagem Hill, several 30 to 40 m deep excavations are planned, with the stability ensured through anchored tieback walls and soil nailing systems. This paper aims to analyze the behavior of two key excavation sections considering the impact of the reliquiaie soil layers. A 2D numerical analysis was conducted using Plaxis 2D, applying the Hardening Soil Small Strain constitutive model for all soil layers. Displacement profiles and the evolution of average anchorage loads were assessed. The results showed that the highest lateral displacements were concentrated along the reliquiaie weak layers, particularly in the final three-quarters of the excavation, indicating a tendency of bottom heaving. At the same depth, the anchors experienced a 7% increase of the average axial loads by the final construction stage. The loads at the facing connections ranged from 60% to 80% of the maximum mobilized nail loads.

1. Introduction

In the day-to-day routine of project design, interpreting the results of the geotechnical investigation program is one of the main tasks related to the development of adequate earth-retaining structures projects. The Standard Penetration Test - SPT is the most utilized field test in Brazil due to its practicality and lower cost when compared to laboratory tests or even other field tests. In that case, the SPT test plays an important role in the design of geotechnical structures such as earth-retaining structures. The definition of strength parameters, utilized in slope analysis procedures, is often made by empirical expressions with SPT number - N_{spt} presented in the field charts. Nevertheless, the SPT test has its limits when it comes to analyzing the strength of soil with specific geotechnical and geological characteristics, such as soils with inherent anisotropic strength and stiffness. In those conditions, profound studies along with complementary

geotechnical investigation are extremely necessary in the design of earth-retaining structures.

Regarding the influence of anisotropic characteristics on soil behavior, the reference literature mainly discusses several case studies of excavations in sedimentary anisotropic soils whether associated with inherent cross anisotropy or soil stress-induced anisotropy (Li et al., 2022; Lisewska & Cudny, 2023; Zhang et al., 2023). In relation to natural residual soils, the effects of anisotropy regarding the strength parameters are also presented in the reference literature (Liu et al., 2021, 2023), but they usually are not related to excavation case studies accompanied by earth-retaining structures such as anchor walls and soil nailing systems.

In the city of Niterói/RJ, the region next to Boa Viagem and Caniço hills is marked by the presence of a geological fault that extends to Guanabara Bay. In this geological fault zone, historic processes of differential weathering took place, which resulted in the formation of reliquiaie layers with consistency and strength parameters

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significantly apart from the adjacent residual soil. The local stratigraphy is characterized by thick portions of residual soil with values of N_{spt} close to 40 at higher depths. Several deep excavations mark the region, with reliquiaie layers spotted during construction, serving as case studies described in the reference literature (Ehrlich, 2004; Saramago et al., 2010; Ehrlich & Silva, 2015; Moura et al., 2023). Regarding the study of the behavior of the excavations, the works reported in the reference literature (Ehrlich, 2004; Ehrlich & Silva, 2015; Moura et al., 2023) were associated to earth-retaining structures in previous failure conditions.

The construction of a new residential building complex is planned next to Boa Viagem Hill. Multiple deep excavations are set to be made in the area, with a maximum 40 m depth cut, the highest among the occurrences described in the reference literature. The stability conditions of the projected excavations will be guaranteed by tieback anchored walls and soil nailing systems, designed in project phase. The present paper aims to analyze the behavior of two excavations sections of the study area during construction by numerical modeling in finite elements utilizing Plaxis 2D. Displacement profiles and the evolution of average anchor loads were analyzed, considering the presence of the reliquiaie layers found in the region. The model was validated by the numerical analysis of one of the case studies held in the region, next to Caniço Hill (Ehrlich, 2004; Moura et al., 2023), that has good amount of instrumented data to support the validation.

2. Case study

2.1 General characteristics

The site of the present case study was originally characterized by natural slopes with average horizontal inclinations varying between 15° and 34°. Figure 1 shows the geological map of the fault zone situated in the coastal regions of Boa Viagem and Icaraí, and the respective occurrences described in the reference literature. All the presented occurrences are part of Ingá lithological unit. Centimetric reliquiaie structures characterized by layers of grey plastic clays with high activity index (>2.5) were identified during the execution of a 31 m excavation near Boa Viagem Hill (Occurrence 1), with dip of 58° (near the present case study) to 80° (near Occurrence 2) unfavorable to the excavation (Ehrlich & Silva, 2015). Similar reliquiaie layers were found next to Caniço Hill (Occurrence 3) during the execution of a 25 m high anchored wall (Ehrlich, 2004) and during complementary geotechnical investigation (Occurrence 4) developed for the execution of a 27 m excavation (Saramago et al., 2010). The present case study site is located next to Occurrence 1 (Ehrlich & Silva, 2015), where clay reliquiaie layers were identified during excavations procedures.

Figure 2a illustrates the location plan of the retaining walls and the defined cross sections analyzed in the present study, i.e., Section D and D.2. Figure 2b presents a 3D overview of the retaining structures designed in projected

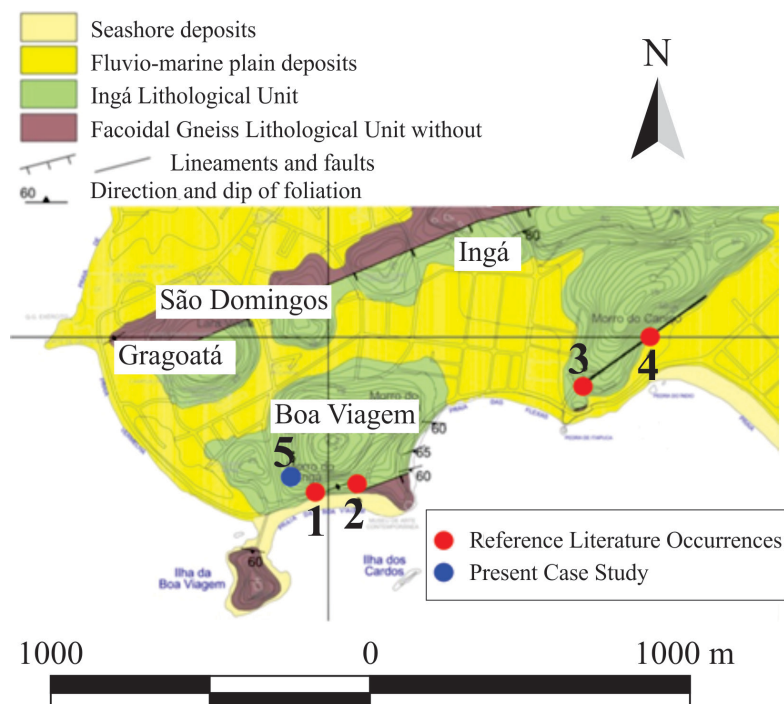


Figure 1. Ingá lithologic unit and its geological map (UFF, 2004) with the location of the case study site and the occurrences described in reference literature.

phase. As presented in Figure 2c, Section D is characterized by three 0.45 m thick anchored walls: Wall D, C, and G2. The lowest elevation is +10.74 m and the highest is +50.38 m, corresponding to a 40 m total depth excavation. The working anchor loads defined in project phase varied between 838 and 860 kN. Section D.2 (Figure 2d) is defined by two 0.45 m thick anchored walls and a soil nailing system with a 0.12 m shotcrete facing: Wall D, C, and Soil Nail 1.4. The lowest elevation is +16.00 m, corresponding to a 34 m depth excavation. The working anchor loads defined for Walls D and C were 838 kN. As for the nails, the value set was 400 kN.

2.2 Geotechnical and geological investigation

The geotechnical investigation of the case study started with extensive research of the geotechnical and geological characteristics of the local soil available in the reference literature, including field and laboratory tests data from previous occurrences (Figure 1). Regarding the present case study, the first stratigraphic analysis of the soil layers comprised three borehole campaigns: two mixed drilling campaigns (SPT + rotary drilling) and one SPT campaign. Figure 3 illustrates the positions of all borehole campaigns executed on the site and the location of the analysis sections D and D.2. The borehole campaigns indicated a stratigraphy defined by initial layers of red and brown sandy clay (mature residual soil) and yellow and white sandy silt (saprolitic

soil), the latter with values of N_{spt} higher than 40 blows. To further analyze the soil stratigraphy, undisturbed samples were continuously collected using Denison Sampler with diameters of 0.050 m and 0.075 m. The samples were collected at intervals of 1.5 m and 1.0 m, respectively, with the 0.050 m and 0.075 m aluminum tubes, reaching a maximum depth of 26 m and 24 m. These samples were collected next to borehole SP2 and analyzed at COPPE/UFRJ Geotechnical Laboratory. Figure 4a presents a sample collected at a depth of 17 m, showing the extension of the heterogeneity of the soil along its depth. Figure 4b illustrates a sample collected at a depth of 10.50 m with a high degree of heterogeneity along with the presence of grey plastic clay reliquia layers. Additionally, an investigation well with a depth of 15 m was also executed. The portion of the soil analyzed was situated between depths of 13 and 15 m, marked by multiple foliations with thicknesses below 0.01 m and dips varying from 50° to 60°. Figure 4c and Figure 4d illustrate the foliations observed in the inspected area of the well.

The geotechnical investigation was complemented by additional undisturbed sampling for laboratory tests. A box-shaped sample at a depth of 15 m (Sample A) was collected inside the investigation well. Figure 5a shows Sample A opened at COPPE/UFRJ Geotechnical Laboratory. The sample is again marked by its strong heterogeneity. Complete characterization, direct shear and drained triaxial tests were performed, with the direct shear tests conducted

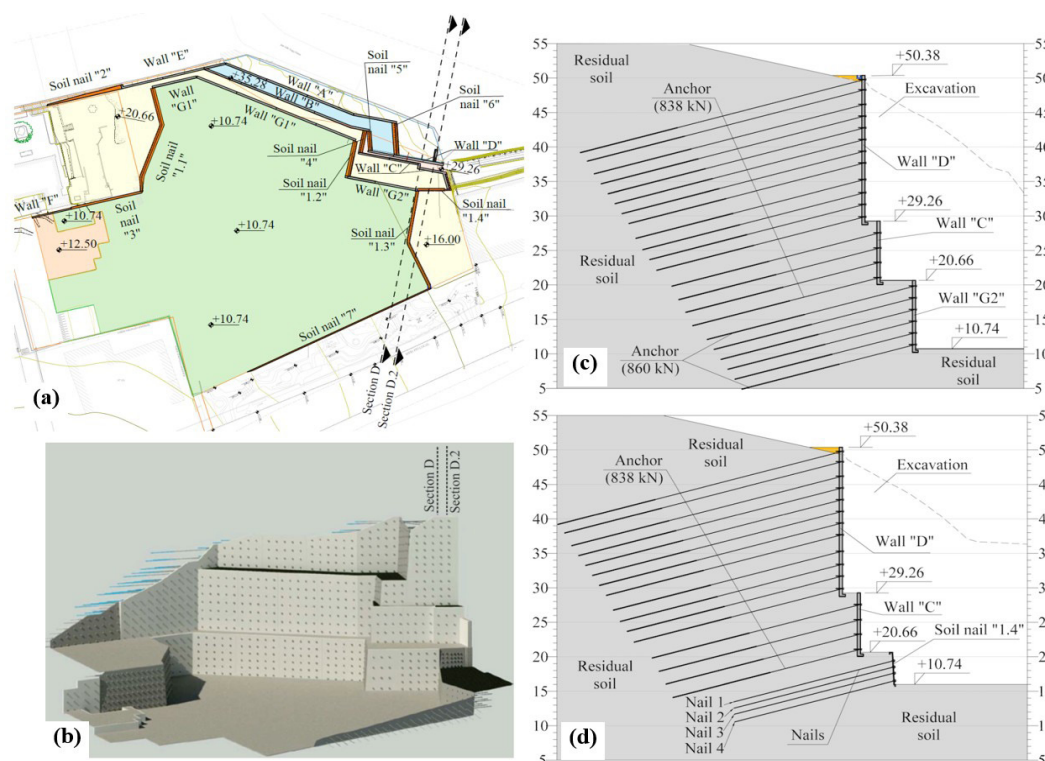


Figure 2. Location plan of the retaining structures (a); 3D overview of the retaining structures (b) and; main cross sections: D (c) and D.2 (d).

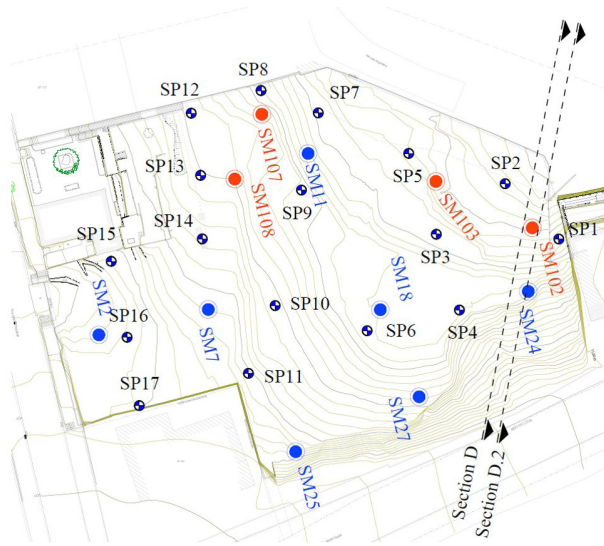


Figure 3. Location of the borehole campaigns and sections D/D.2.

in submerged conditions, where the shear planes followed the direction of the failure planes observed in the results of the triaxial tests (Figure 5b and Figure 5c). Figure 6 presents the triaxial test curves of Sample A. The overall results of the laboratory tests are presented in Table 1.

Figure 7a and Figure 7b illustrate photos of the site during excavation procedures of Wall D. Figure 7a shows yellow and white sandy silt located side by side with a portion of red sandy clay, i.e., two portions of completely different types of soil materials. Figure 7b presents red sandy clay marked by foliations of yellow and white sandy silts. In addition, reliquiaie layers were spotted during the execution of Wall D, closely resembling the clay layers found in Occurrence 1 (Ehrlich & Silva, 2015), as shown in Figure 7c.

Sample A was collected at a depth classified as impenetrable to percussion according to the results from SP-02. The laboratory test results of Sample A assessed the impact of the heterogeneity of the soil on its shear strength properties, which were not identified in the results of the nearest borehole (SP-02). Figure 8 presents the geotechnical profile of Sections D and D.2, according to the results from the borehole campaigns. Despite the heterogeneity of the local soil, a simplified stratigraphy was defined based on the geotechnical investigation (borehole campaigns) for both sections. The excavation behavior and its plastic deformations are controlled by the weak layers present in the local soil. Therefore, the use of a simplified residual soil stratigraphy is adequate for the numerical analysis performed.

3. Model validation

The validation of the case study's numerical analysis was performed by modeling a 30-meter deep excavation next to Icaraí Beach, referred to as Occurrence 3 (see

Table 1. Laboratory test results of Sample A.

Atterberg's limits		Percentage finer than			G_s
LL (%)	PI (%)	2	20	2	
		(mm)	(μ m)	(μ m)	(g/cm ³)
58	31	15	31	54	2,57
Triaxial and Direct Shear tests					
Identification	Strength Parameters				
			ϕ (°)	c (kPa)	
Triaxial Test			18	33	
Direct shear test 1			19	32	
Direct shear test 2			26	15	

Figure 1), using Plaxis 2D. This excavation is well-documented in the literature (Ehrlich, 2004; Moura et al., 2023) and is supported by substantial amount of monitoring data. Occurrence 3 is 1 km apart from the present case study and was originally characterized by an anchored wall (Itapuca Wall) with working loads of 280 kN. The referred excavation was chosen as the validation model not only due to its well-documented data, but also due to its similarity with the present case study. As reported by (Ehrlich, 2004; Moura et al., 2023), the overall residual soil characteristics, the nature of the weak layers and the dimensions of both excavations exhibit notable similarities with the current case study, further supporting the use of Occurrence 3 as a validation model.

For the development of a residential building complex, complementary downstream excavations accompanied by a new anchor wall (Wall 2) with working loads of 280 kN were planned in the area. Before the execution of Wall 2, the existing anchored wall (Itapuca Wall) was reinforced with anchor grids with working loads of 350 kN. During the construction of Wall 2, at level +5.30 m, continuous cracks were identified upstream of the entire excavation. The construction works were interrupted for further investigation, which led to the identification of reliquiaie layers with a dip of 60°, unfavorable to the excavations. To monitor the remaining construction stages, a set of inclinometers was installed upstream of the excavation, with load cells added to some anchors of the reinforcement grid and Wall 2. In addition, new tieback anchors were installed in Wall 2 with working loads of 350 kN. The groundwater level was spotted at +4.3 m. Figure 9 (Moura et al., 2023) illustrates the main excavation section of Occurrence 3.

Following the successful approach studied by Moura et al. (2023), the reliquiaie layers were modeled as interface elements spaced every 0.25 m, with length extending 12 m below the final excavation level, where the adjacent residual soil was taken as a single soil matrix. The spacing adopted for the interface elements followed the characteristics observed in the field (Ehrlich, 2004; Moura et al., 2023) and parametric sensitivity analysis performed by Moura et al. (2023). The referred authors obtained consistent results utilizing the Hardening

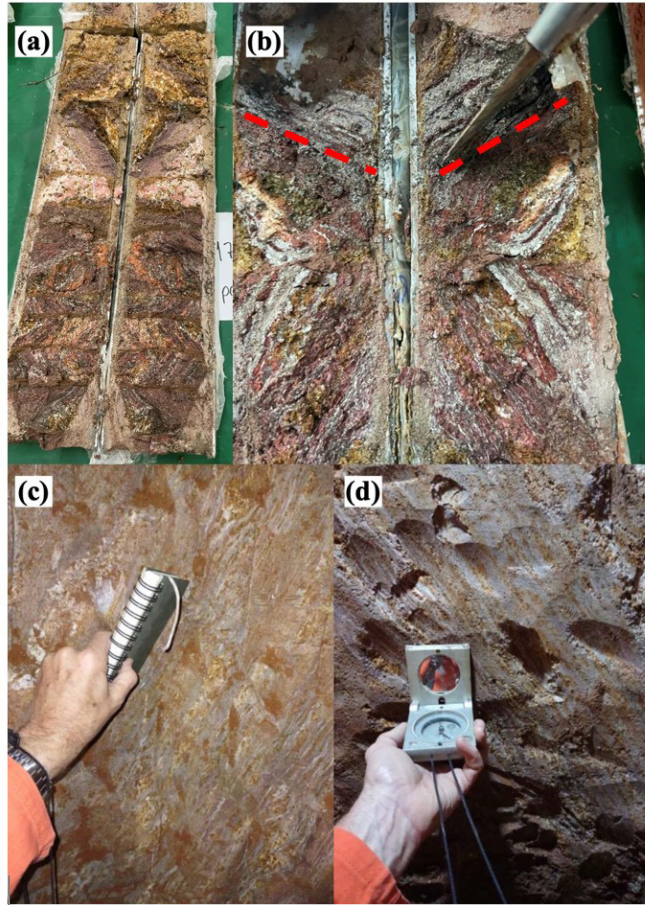


Figure 4. Undisturbed Denison samples: (a) evident heterogeneity observed at a depth of 17 m; (b) grey plastic clay layers identified at a depth of 10.50 m. Multiple foliations, shown in (c) and (d), were observed in the investigation well.

Soil Small Strain - HSS model (residual soil matrix) and Mohr Coulomb with stiffness parameters set as depth-dependent for the reliquiaie layers. In the present validation model, HSS was the constitutive model utilized in both the residual soil matrix and reliquiaie layers. The use of HSS for deep excavations, where depth-dependent stiffness is an important aspect, is well supported by the reference literature (Benson Hsiung & Dao, 2014; Moura et al., 2023; Nejjar et al., 2024).

Table 2 summarizes the soil parameters utilized in the calibration model. To adequately validate the calibration model with instrumental data of Occurrence 3, the soil strength and stiffness parameters followed the values considered by Moura et al. (2023). In the case of the reliquiaie layers, the referred authors utilized the Mohr-Coulomb model with depth-dependent incremental values of the Young's Modulus E throughout the depth of the model. Therefore, the E_{50} vs. σ_c curves of the calibration model were adjusted to match those reported by Moura et al. (2023), by varying the values of E_{50}^{ref} and the exponent m though a trial-and-error process.

Regarding the support properties, the tieback anchors were modeled as node-to-node anchors (free length) and

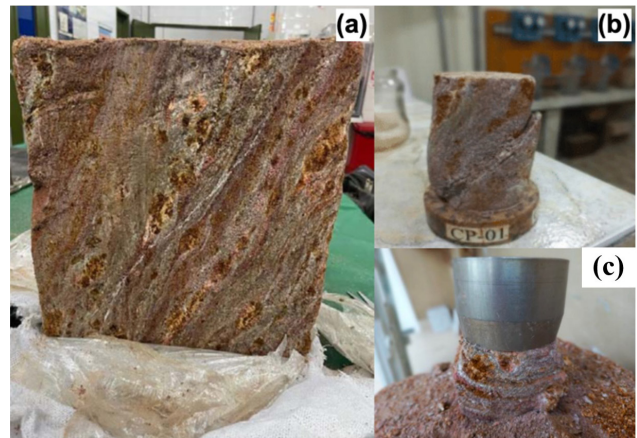


Figure 5. Undisturbed box-shaped sample (Sample A) opened at COPPE/UFRJ Geotechnical Lab. (a), (b) failure planes observed in one of the triaxial tests samples, and (c) sampling of one of the direct shear rings.

embedded beam rows (fixed length). To properly account for the change in stiffness of Itapuca Wall in the execution of the

Table 2. Soil strength and stiffness parameters utilized in the validation model (Ehrlich, 2004; Moura et al., 2023).

Identification	Residual Soil	Reliquiae
	Matrix	Layer
Model	Soil Element	Interface Element
Element		
ϕ (°)	44	30
c (kPa)	20	20
γ_{nat} (kN/m ³)	20	20
γ_{sat} (kN/m ³)	21	21
ψ (°)	14	0
E_{50}^{ref} (MPa)	38	44
E_{oed}^{ref} (MPa)	38	44
E_{ur}^{ref} (MPa)	1.1E2	1.3E2
m	0.70	0.80
ν_{ur}	0.20	0.30
G_0^{ref} (MPa)	1.2E2	1.2E2
$\gamma_{0.7}$	1.0E-4	1.0E-4
K_0	0.31	0.50

anchor grid, the wall facings were modeled as linear elastic layers. Nevertheless, the shotcrete facing of the upstream slope was modeled as a linear elastic plate element. Table 3 summarizes the support and facing parameters utilized in the validation model.

The validation model was built considering the plane strain state, with boundary conditions set as totally fixed at the base of the model, with the left and right sides fixed horizontally while allowing free vertical movement. Figure 10 presents the validation model geometry. The downstream surcharge load of 170 kPa represents the load imposed by the 17-storey building projected on the site.

Lateral displacement profiles and development of the average anchor loads throughout the construction stages of the validation model were compared with the results presented by the monitoring data of Occurrence 3 (Ehrlich, 2004) and the results obtained by Moura et al. (2023). The lateral displacement profiles were analyzed at three excavations levels (+3.60 m, +1.80 m, and +0.80 m) for each inclinometer's position (see Figure 9). The results, presented in Figure 11, showed good agreement with both the results of the monitoring data and the presented by Moura et al. (2023). The values of average anchor loads (anchor grid and Wall 2) throughout the construction stages are shown in Figure 12. The results obtained were also compatible with Occurrence 3's measured data and with the results from Moura et al. (2023), the latter plotted in Figure 12 as simplified data.

Table 3. Support and facing parameters utilized in the validation model (Ehrlich, 2004; Moura et al., 2023).

Retaining Structures	Itapuca Wall/ Grid/Wall 2	Uphill facing
	Anchors	
Support Element	Free	Fixed
	Length	Length
ϕ_s (cm)	3.2	3.2
ϕ_d (cm)	-	10
EA (MN)	1.7E2	1.7E2
EI (MNm ²)	-	0.11
E (GPa)	205	21.5
L_{spa} (m)	1.75/ 2.24/ 2.24	1.75/ 2.24/ 2.24
ν	0.20	0.20
Facing Element	Reinforced Concrete facing	Shotcrete facing
d (cm)	20/19/30	15
γ (kN/m ³)	25	25
EA (MN)	-	2.7E3
EI (MNm ²)	-	5.1
E (GPa)	18	18
w (kN/m/m)	-	1.1
ν	0.2	0.2

4. Numerical analysis of the case study

4.1 Definition of model parameters

The geotechnical investigation and the observations made during excavation procedures indicated that the soil stratigraphy was characterized by layers of residual soil

Figure 10 consists of two subplots, (a) and (b), showing the mechanical behavior of four samples (CP1, CP2, CP3, and CP4) under axial strain.

Subplot (a) shows Deviatoric stress (kPa) on the y-axis (0 to 700) versus Axial strain (%) on the x-axis (0 to 20.0). The curves show that CP4 (purple) has the highest deviatoric stress, followed by CP3 (yellow), CP2 (brown), and CP1 (blue). CP1 and CP2 show a slight decrease in stress after a peak around 12% strain, while CP3 and CP4 continue to increase or plateau.

Subplot (b) shows Volumetric strain (%) on the y-axis (0 to 7.0) versus Axial strain (%) on the x-axis (0 to 20.0). The curves show that CP4 (purple) has the highest volumetric strain, followed by CP3 (yellow), CP1 (blue), and CP2 (brown). All samples show an increase in volumetric strain with increasing axial strain.

elements spaced every 0.25 m with length extending 15 m below the final excavation level of both sections. Both the length and the positions of the interface elements were defined after a sensitivity analysis of their influence on the excavation behavior. The 2D plane strain conditions were adopted in both sections, following the same boundary conditions as those of the calibration model. Figure 13a and Figure 13b respectively present the model geometry of Section D and D.2.

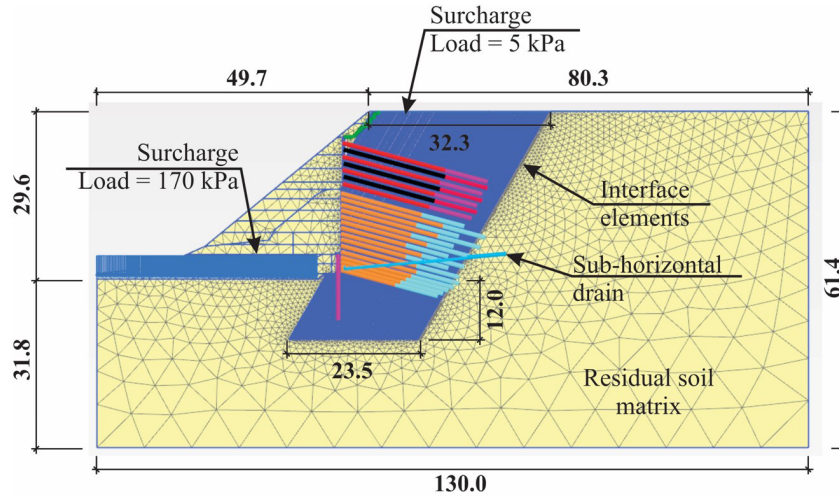


Figure 10. Validation model geometry developed in Plaxis 2D.

Table 4 summarizes the strength and stiffness parameters defined in both model sections. For the residual soil and reliquiaie layers, the HSS constitutive model was utilized. As previously noted, the heterogeneity of the local soil can significantly influence the strength parameters of the laboratory tests, as shown in Table 1. To better represent the strength and stiffness parameters of the overall residual soil, the strength parameters defined for the residual soil layers were based on extensive laboratory tests conducted at Occurrence 1 (Ehrlich & Silva, 2015), adjacent to the present study site, and Occurrence 2 (Proto, 2005; Lima, 2007; Saré, 2007), which included triaxial and direct shear tests on several undisturbed samples collected on these areas. Additionally, the strength parameters of the reliquiaie layers were adopted based on the existing results from laboratory tests performed in plastic grey layers identified in Occurrence 1, supported by a back-analysis of the observed failure conditions during the execution of an anchored tieback wall (Ehrlich & Silva, 2015).

The stiffness parameters of all soil layers were defined according to the study performed by Silva (2017), where the results were based on the numerical analysis of Occurrence 1, supported by instrumental and laboratory test data. For the saprolitic soil layer, the dilatancy angle ψ was defined as a third of the friction angle ϕ (Obrzud & Truty, 2020). The reference initial shear modulus $G_{0,ref}$ followed the charts presented by Alpan (1970). To account for the depth-dependency of the reference strain threshold parameter $\gamma_{0.7}$, the saprolitic layer was divided into sublayers with a depth approximately of 10 m, with values of $\gamma_{0.7}$ determined by the research of Darendeli (2001). As for the reliquiaie and sandy clay layers, the strain threshold parameter was defined following the results presented by Vucetic & Dobry (1991), considering the material plasticity observed in Occurrence 1 (Ehrlich & Silva, 2015).

The tieback anchors were modeled as elastic node-to-node anchors (free length) and embedded beam rows (fixed length). The nails and the facing of the retaining structures

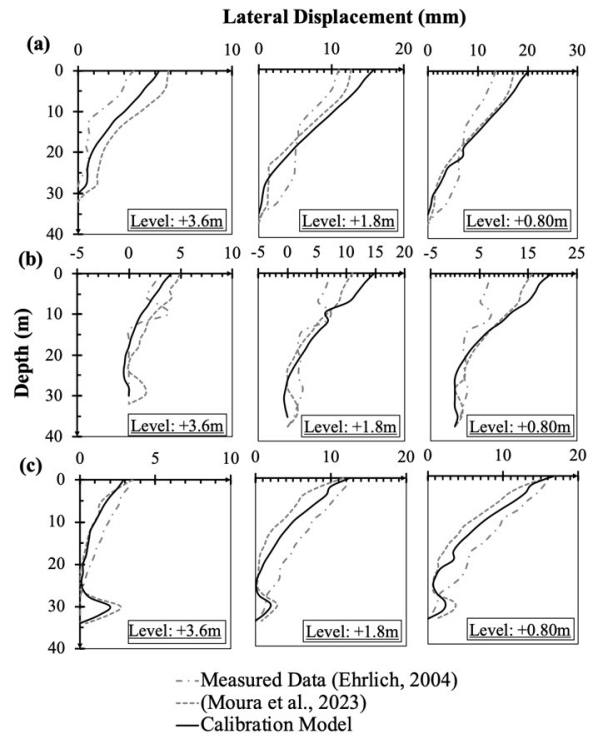


Figure 11. Lateral displacement profiles of (a) I-101, (b) I-102, and (c) I-103 (Ehrlich, 2004; Moura et al., 2023).

were modelled as linear elastic plate elements. The support and facing parameters are presented in Table 5. To simulate the descending execution procedure of the tieback walls, the following steps were adopted for each anchor level: (a) initial excavation with a 1.5 m offset from the wall face and a downward slope of 34° , and (b) vertical excavation with activation of the wall facing and anchor at its respective prestressing load. During the execution of Wall D, the

Table 4. Strength and stiffness parameters utilized in the numerical analysis.

Identification	Sandy Clay	Sandy Silt	Plastic Clay
	Mature Residual Soil	Saprolitic Soil	Reliquiae Layer
Model	Soil	Soil	Interface
Element	Element	Element	Element
ϕ (°)	29	36	28
c (kPa)	10	27	12
γ_{nat} (kN/m ³)	18	20	18
ψ (°)	0	12	0
E_{50}^{ref} (MPa)	8.6	23	16
E_{oed}^{ref} (MPa)	8.6	23	16
E_{ur}^{ref} (MPa)	26	68	48
m	1	1	1
ν_{ur}	0.2	0.2	0.2
G_0^{ref} (MPa)	56	101	80
		1.5E-4 ^a / 1.9E-4 ^b / 2.1E-4 ^c / 2.3E-4 ^d	7.0E-4
$\gamma_{0.7}$	4.0E-4		
K_0	0.51	0.41	0.53

^a10-20 m; ^b20-30 m; ^c30- 40m; ^d>40 m.

anchors were initially prestressed to 390 kN (about 45% of the design working load). By the completion of Wall D, all the anchors executed were prestressed to 750 kN (about 90% of the design working load), with the supports of Wall C and Wall G2 also prestressed to 750 kN.

4.2 Lateral displacement profiles

The lateral displacement profiles throughout the construction stages of both model sections were analyzed in this study. Figure 14a and Figure 14b shows respectively the lateral displacement profiles of Section D and D.2 (at a 1 m offset distance from the top of Wall D) after the final stage of each earth-retaining structure. Figure 14c illustrates the final stage trends of lateral displacements δ_x along the excavation depth z , both relative to the respective total excavation height H . In order to better assess the impact of the reliquiae layers, Figure 14c presents data of the numerical analysis with and without weak layers (WL). The results show that the highest values of lateral displacements for both sections (with weak layers) occurred in the last three quarters of the total excavation height H , with maximum values of 0.30% and 0.25% of H , respectively, for Section D and D.2. The

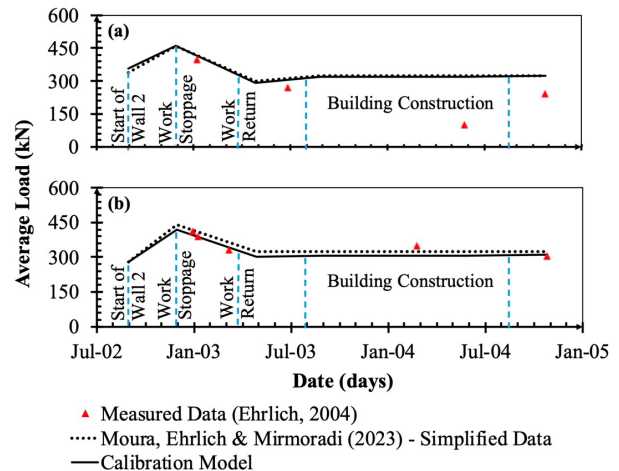


Figure 12. Development of average anchor loads of (a) the reinforcement anchor grid, and (b) of Wall 2 (Ehrlich, 2004; Moura et al., 2023).

results also point out the impact of the reliquiae layers on the lateral displacement profiles. The maximum displacement values, when in the absence of the weak layers, corresponded

Table 5. Support and facing parameters adopted in the numerical analysis.

Retaining Structures	Wall D/Wall C/ Wall G2		Soil
Support Element	Anchors		Nails
	Free Length	Fixed Length	
ϕ_s (cm)	6.1	6.1	4.1
ϕ_d (cm)	-	15	-
EA (MN)	6.0E2	6.0E2	2.3E2
EI (MNm ²)	-	8.4E-1	2.4E-2
E (GPa)	205	34	205
L_{spa} (m)	2.2/2.0/1.9	2.2/2.0/1.9	1.2
ν	0.20	0.20	0.2
Facing Element	Reinforced Concrete facing		Shotcrete facing
d (cm)	45		12
γ (kN/m ³)	25		25
EA (MN)	1.3E4		3.5E3
EI (MNm ²)	2.2E2		4.0
E (GPa)	29		29
w (kN/m/m)	2.3		0.60
ν	0.2		0.2

about 20% of the values gathered in the analysis with WL for both sections.

The highest values of total lateral displacement for Sections D and D.2 were 120 mm and 85 mm, respectively. The lower values presented by Section D.2 are attributed to the smaller total height excavation of 34 m in comparison to Section D's total height of 40 m. In the early construction stages of both sections, the lateral displacement profiles exhibited negative values of δ_x (approximately 50 mm). This can be associated with the presence of anchors with high prestressing load values while the excavation depths were still small. Nevertheless, in both sections, the highest lateral displacement values were observed during the final

construction stages of the Wall C. Additionally, significant lateral displacements occurred at depths below the final excavation surface level, reaching about $1.25H$ for both sections. For Section D, the lateral displacement values varied between 0.1% and 0.3% of H , while for Section D.2, the values fluctuated between 0.1% and 0.25% of H , indicating a tendency of bottom heaving.

4.3 Support loads

The development of the support loads throughout the construction stages of the retaining structures was analyzed in this study. Figure 15 shows the mobilized average anchor

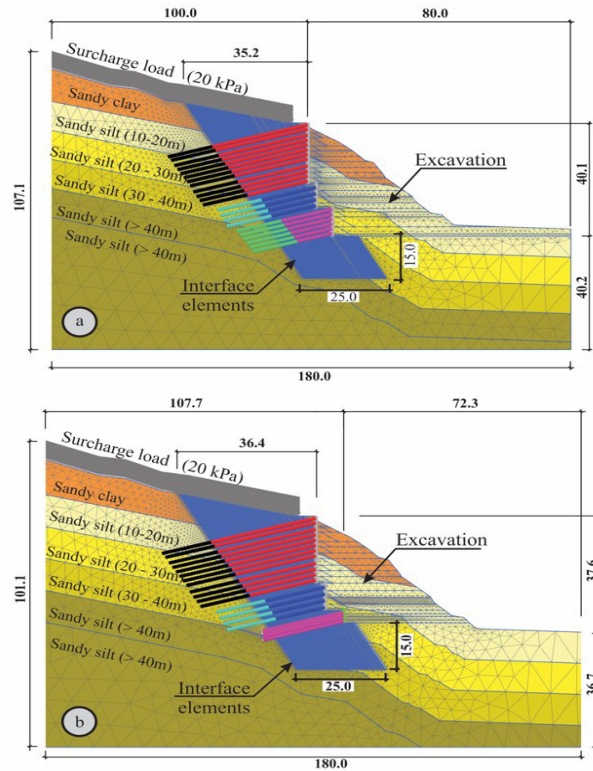


Figure 13. Model geometry and mesh of (a) Section D, and (b) Section D.2.

loads in Wall D, Wall C and Wall G2 at excavation depth z relative to the final depth H . The results indicated a reduction in average anchor loads (about 25%) at excavation depths less than $0.40 H$ compared to the initial prestressing loads. This decrease can be attributed to the high prestressing values of the anchors, which caused displacements in the opposite direction of the excavation during the early stages, leading to a reduction in the average anchor loads. With the advancement of the excavation stages, the support loads increased, and at the final excavation stage, the average anchor loads of Wall D and Wall G2 were, respectively, 8% and 10% smaller than the design working loads. Nevertheless, due to the highest lateral displacements in the final three quarters of the excavation, the mobilized anchors' loads in the Wall C were 7% higher than the design working loads. This indicates the need to monitor the increase of the respective anchors loads of Wall C during its execution and, if necessary, adjusting the values back to the design working loads.

The mobilized axial loads in the support nails of Section D.2 were also analyzed in this study. Figure 16 illustrates the charts of mobilized axial loads for Nails 1, 2, 3 and 4 along their respective lengths at different construction stages. In the final construction phase, Nail 1 showed a maximum mobilized axial load of 150 kN/m at a 9 m distance from the soil nail facing. Nail 2 exhibited a maximum axial load of 165 kN/m at a 6 m distance from the shotcrete facing.

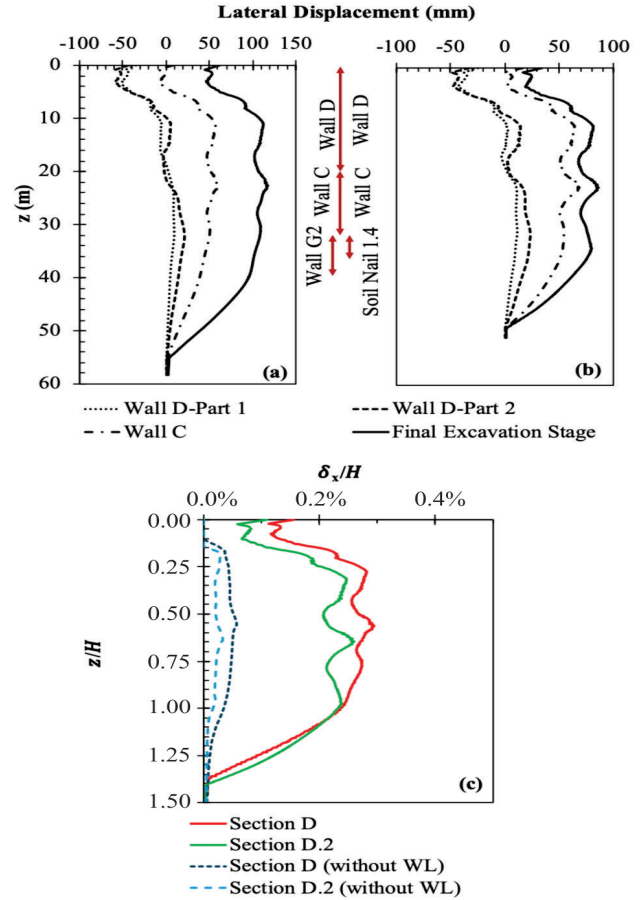


Figure 14. Lateral displacement profiles: (a) Section D and; (b) Section D.2 after the final stages of each retaining structure and; (c) trends of lateral displacement δ_x throughout depth z in relation to the final excavation depth of each section with and without weak layers.

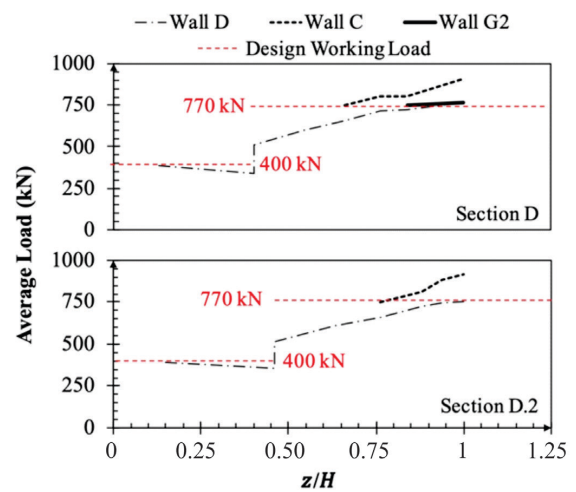


Figure 15. Average anchor loads of Wall D, Wall C and Wall G2 at the excavation depth z relative to the final depth H in Section D and Section D.2.

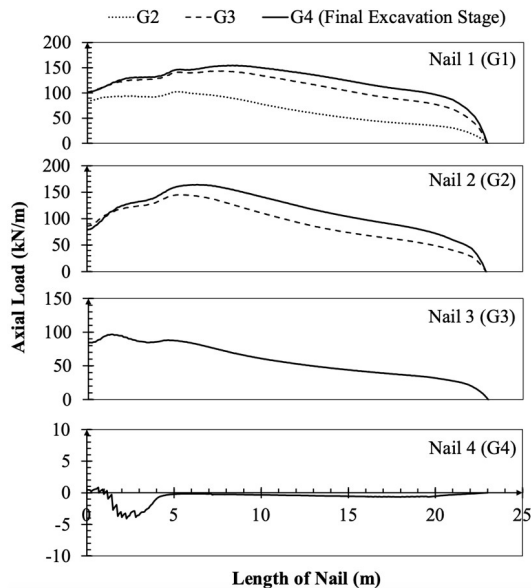


Figure 16. Mobilized axial loads of Nails 1, 2, 3 and 4 along their respective length.

Meanwhile, Nail 3 recorded a highest value of 100 kN/m at a 2 m distance from the facing. In addition, the developed axial loads at the connections of the shotcrete facing, during the final construction stage, varied between 60% and 80% of the maximum mobilized axial load in the nails. The mobilized values in the facing for Nails 1, 2, and 3 were respectively 100 kN/m, 90 kN/m, and 80 kN/m, in good agreement with those established in the design phase (160 kN/m load, 200 kN per nail). Nail 4 did not exhibit significant values of mobilized axial loads, likely due to its minimal contribution relative to the overall influence of the 34 m deep excavation.

5. Conclusion

The study area is characterized by the presence of reliquiaie layers at multiple sites along the coastal region. Following extensive investigation and careful observations during excavation, the case study reveals a highly heterogeneous geological profile, featuring numerous steep foliations and thin weak soil layers that differ significantly from the surrounding residual soil.

2D numerical analyses of two sections of the earth-retaining structures project of the area were developed with Plaxis 2D. Unlike the reported on the reference literature, the excavations analyzed were not related to any previous failure conditions, with the numerical analysis focused on the working conditions of the respective retaining structures. The HSS constitutive model was adopted for all soil layers, including the reliquiaie weak layers. The numerical modelling was validated by the results presented by the calibration model of Occurrence 3, being in good agreement with its instrumental data and previous results presented in reference literature (Ehrlich, 2004; Moura et al., 2023).

The lateral displacement profiles showed the effects of the reliquiaie layers in the excavation behavior, especially when compared with the results without considering the weak interfaces. Significant displacement values were induced through the weak layers and extended below the final excavation levels (about $1.25 H$), indicating a tendency for bottom heaving. The persistence of these weak layers—similar to those identified in the coastal regions of Boa Viagem and Icarai—has important impact on the stability conditions of the proposed excavations. Moreover, when designing earth-retaining structures in similar soil conditions, it is essential to consider the depth and continuity of the weak layers beneath the final excavation level, as this may lead to more critical conditions in slope stability analysis.

Numerical modeling suggests that the highest lateral displacement values are concentrated in the final three quarters of the total excavation height, and that anchors in the Wall C experiment a 7% increase in loads compared to the design working loads. This suggests the potential need for a slight reduction in the pretension load applied to the anchors, in order to maintain the anchorage loads within the defined working loads. Additionally, the maximum mobilized axial loads of Nails 1, 2, and 3 were lower than the design working loads, with the loads at the connections to the shotcrete facing ranging from 60% to 80% of the maximum mobilized axial nail loads.

This case study underscores the impact of reliquiaie layers in the behavior of deep-anchored excavations in residual soil derived from gneiss with high N_{spt} values. It highlights the need to consider these layers in the analysis and design of such structures, supported by comprehensive geotechnical investigations, field observations, and monitoring programs.

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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

Antonio Alves do Couto Júnior: conceptualization, formal analysis, investigation, methodology, project administration, visualization, writing - original draft. Maurício Ehrlich: conceptualization, methodology, project administration, funding acquisition, resources, supervision, validation, writing - reviewing and editing. Robson Palhas Saramago: methodology, resources, supervision, validation, writing - reviewing and editing.

Data availability

The datasets generated analyzed in the course of the current study are available from the corresponding author upon request.

Declaration of Use of Generative Artificial Intelligence

This work was not prepared with the assistance of generative artificial intelligence (GenAI).

List of symbols and abbreviations

c	Cohesion intercept of soil
d	Facing thickness of retaining structures
m	Power parameter for stress-level dependency of stiffness
w	Specific weight
z	Excavation depth
CAPES	Coordenação de Aperfeiçoamento de Pessoal de Nível Superior
COPPE/UFRJ	Instituto Alberto Luiz Coimbra de Pós-Graduação e Pesquisa de Engenharia da Universidade Federal do Rio de Janeiro
E	Young's Modulus for elastic materials
EA	Axial Stiffness
EI	Bending Stiffness
E_{50}^{ref}	Reference stiffness modulus for primary loading of the Hardening Soil Model
E_{ur}^{ref}	Reference stiffness modulus for unloading and reloading of the Hardening Soil Model
E_{oed}^{ref}	Reference stiffness modulus for primary oedometer loading of the Hardening Soil Model
G_s	Specific gravity
G_0^{ref}	Reference initial shear modulus for very small strains
GWT	Ground Water Table
H	Total excavation height
HSS	Hardening Small Strain constitutive model
K_0	At-rest earth pressure coefficient
LL	Liquid Limit of soil
L_{spa}	Support's out-of-plane spacing
N_{spt}	Standard Penetration Test number
PI	Plasticity Index of soil
R_f	Failure ratio
$SPTS$	tandard Penetration Test
WL	Weak layers
δ	Lateral displacement
ϕ	Friction angle of soil
ϕ_s	Support's steel diameter
ϕ_d	Support's drilling diameter

γ	Unit weight of facing materials in retaining structures
γ_{nat}	Natural unit weight of soil
γ_{sat}	Saturated unit weight of soil
$\gamma_{0.7}$	Threshold shear strain parameter of the stiffness degradation curve
ν	Poisson's Coefficient
ν_{ur}	Poisson's Coefficient for unloading and reloading of the Hardening Soil Model
σ_c	Confining Soil Pressure
ψ	Dilatancy angle of soil

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