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Lot 2: Lisbon Metro extension between Santos station and the terminal of Cais do Sodré station. Geological and geotechnical model and parameterisation

Lot 2: Extension du métro de Lisbonne entre la station Santos et la terminal de la station Cais do Sodré. Modèle géologique-géotechnique et paramétrage

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ABSTRACT: Lot 2 of the Extension of the Yellow and Green Lines of the Lisbon Metro starts at the Santos Station and runs southwards across Av. D. Carlos I and Av. 24 de Julho up to the Cais do Sodré Station, connecting the Yellow Line to the Green Line. In this lot, several stages of prospection were carried out, resulting in an extensive set of field and laboratory works that enabled the preparation of the geological model and the geotechnical parameterisation of the lithological units considered in that model. In a length of only 665 m, several materials are crossed. In the survey campaign of the construction phase, among a wide range of works, 73 boreholes were driven for the geological-geotechnical, hydrogeological and geo-environmental components. Based on all information available from previous surveys and from historical and geological mapping, it was possible to draw up the geological model adopted in the Construction Design. Given the large number of results, statistical processing was used to limit and estimate the characteristic values of the parameters required in safety checks.

RÉSUMÉ: Le lot 2 de l'extension des lignes jaune et verte du métro de Lisbonne commence à la station Santos et se dirige vers le sud en traversant l'Avenue D. Carlos I et Avenue 24 de Julho jusqu'à la station Cais do Sodré, reliant la ligne jaune à la ligne verte. Dans ce lot, plusieurs étapes de prospection ont été réalisées, aboutissant à un vaste ensemble de travaux sur site et de laboratoire qui ont permis d'élaborer le modèle géologique et la paramétrisation géotechnique des unités prises en compte dans ce modèle. Sur une longueur de seulement 665 m, plusieurs matériaux sont traversés. Lors de la campagne d'étude de la phase de construction, parmi un large éventail de travaux, 73 forages ont été réalisés pour les composantes géologiques-géotechniques, hydrogéologiques et géo-environnementales. Sur la base de toutes les informations disponibles, provenant des études et campagnes précédentes et de la cartographie historique et géologique, il y a été possible d'élaborer le modèle géologique adopté dans les études d'exécution. Compte tenu de la grande quantité de résultats, il a été possible de limiter et d'estimer les valeurs caractéristiques des paramètres nécessaires pour les contrôles de sécurité à effectuer, grâce à l'aide de traitements statistiques.

Keywords: Lisbon; geologic model; geotechnical survey; geotechnical parametrisation.

1 INTRODUCTION

Lot 2 of the Extension of the Yellow and Green Lines of the Lisbon Metro begins at the Santos Station and extends for about 665 m until the Cais do Sodré Station. From a geomorphological and topographical perspective, the alignment is divided into two parts. The first part is located on the slope of the Madragoa neighbourhood, where the Santos Station is located, and the alignment mostly runs underground. It then transitions to the lower area along the riverfront, where it continues in a trench.

This lower area riverside evolution is presented in Figure 1.

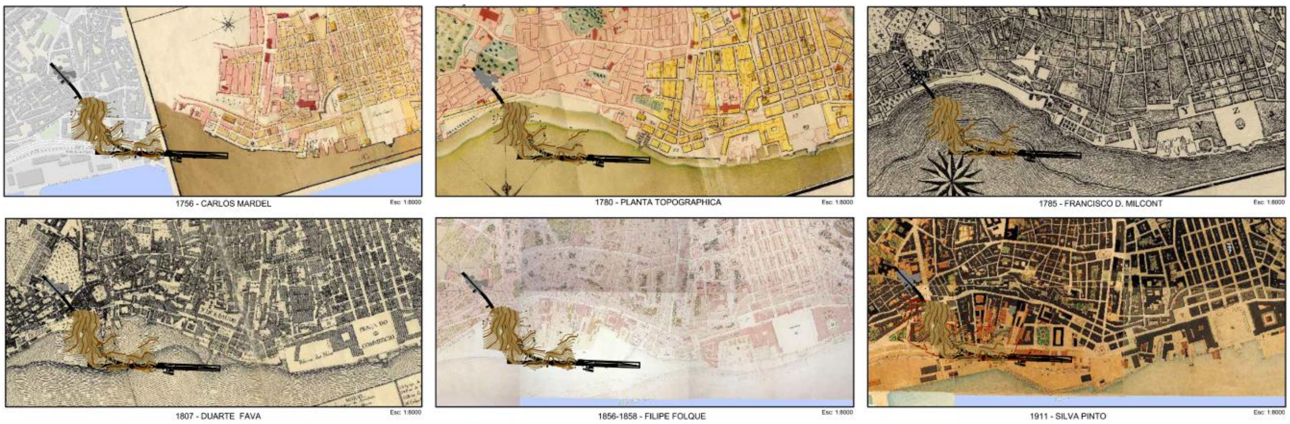


Figure 1. Maps dating from 1756 to 1911, showing the evolution of the riverside area of the Tagus River. The layout of the alignment shows that it lies partly on areas reclaimed from the river.

2 GEOLOGICAL MODEL

2.1 Geological and geotechnical zoning

In the initial part, the geological mapping is mainly composed of the Miocene unit of the Prazeres Clays (MPr), overlying materials from the Lisbon Volcanic Complex (CVL). In this area, in the upper part of the unit prevails basalt, often highly weathered and fractured and, in the lower part, tuffs and breccias. This complex overlies Cretaceous materials from the Bica Formation (mostly limestones). Approximately from Pk 1+575 onwards, the geology is dominated by the presence of alluvial soils underlying landfill deposits, as marked with lines in Figure 2. In this second part of the alignment, starting from the entrance into the Paleovalley (formed by a tributary of the Tagus and whose central zone broadly coincides with the designated Boavista landfill), it fits into an area with a flat surface morphology and the presence of alluvial deposits. In this zone, the geology is characterized, in the upper part, by the presence of landfill deposits – At, up to a depth of about 7.5 m (borehole SC22-Pz), beneath which prevail alluvial materials, predominantly sandy up to Pk 1+700, and predominantly clayey from there to the end of the trench section.

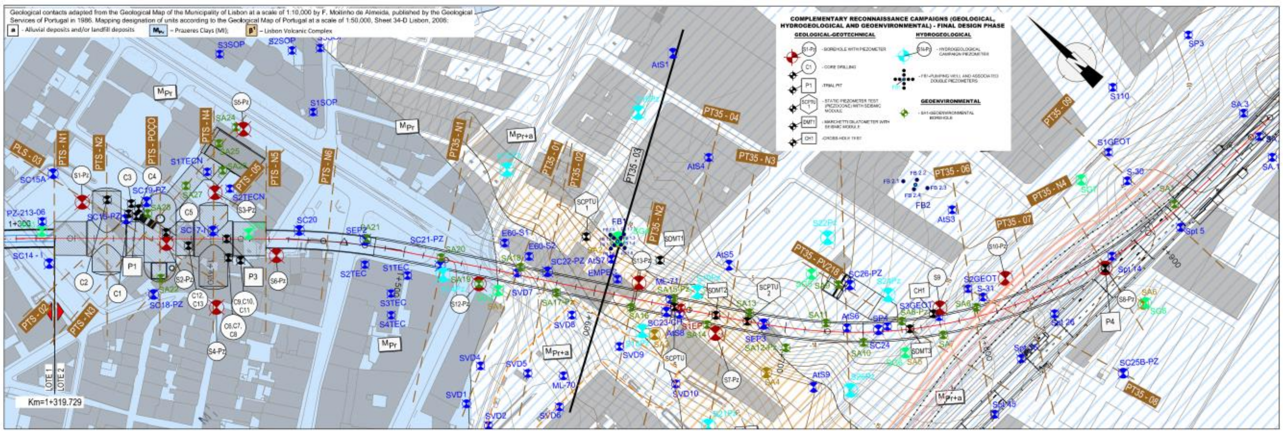


Figure 2. Geological map (adapted from geological mapping at a 1:10,000 scale). In blue, areas of the MPr-Prazeres Clays unit, and in brown, materials from the CVL-Lisbon Volcanic Complex ("line pattern" when under alluvial deposits). Contour lines indicate the topography of the substrate beneath the alluvial deposits in the paleovalley.

In the second section there is a noticeable rise of the Miocene substrate, reaching a maximum roughly between Pk 1+725 and 1+750, at the final of the paleovalley to the southwest. It is noteworthy that there is a prominence of the substrate in relation to the general base level trend toward the east throughout the alluvial/landfill area, with a positive differential of about 7 m. This substrate rise is attributed not only to the prominence of the former stream bank on the left side, but also because the alignment turns eastward here, running along the substrate. On the other hand, this local prominence of the substrate also marks, to some extent, the separation of alluvial deposits from the former stream (paleovalley) from those deposited by the Tagus River. The overall structure of the Miocene monoclinal unit exhibits a general dip toward S-SE with strata dipping generally less than 15º, ranging from 5 to 10º, similar to the levels of the CVL. Despite the eventual faults in the second part of the alignment, it was decided not to represent them, as the model adopted allowed for the interpretation of the contacts between units without materializing disruptions caused by structural discontinuities. Another aspect worth noting in the model is its transversal variability, particularly around the paleovalley, as can be seen, for example, in the section at Pk 1+605, presented below in Figure 4

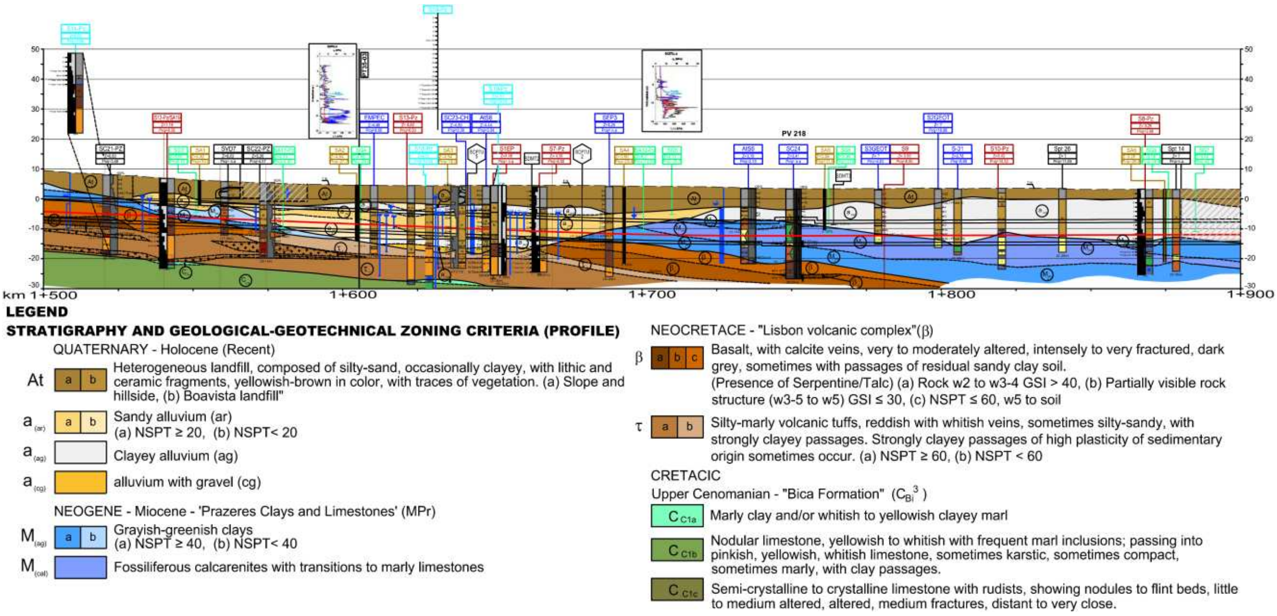


Figure 3. Geological model in the lower area, from pk 1+500 to pk 1+900. In the legend, stratigraphy and zoning criteria adopted in the geological and geotechnical model, in Bento et al. (2022).

The construction of the trench in the lower area, in an alignment that positions it as a barrier to the flow of groundwater along the paleovalley, also led to the need to carry out detailed hydrogeological studies. Taking into account the constraints arising not only by the trench to be built, but also by the basements and pumping systems of pre-existing buildings, these studies addressed the modelling of the current and post-construction conditions. These studies made it possible to design a solution for water to pass through the walls of the trench above the rail tunnel, achieved by drilling holes in the structure.

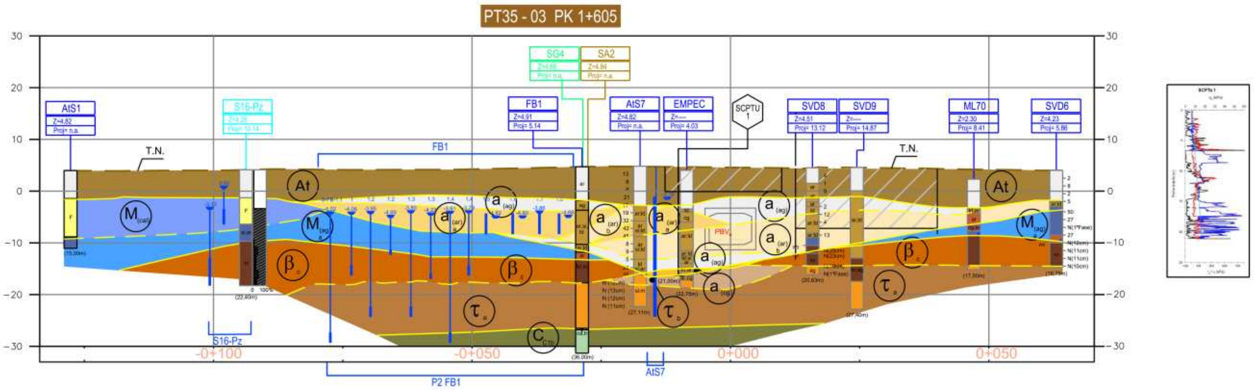


Figure 4. Profile PT35-03, geological model at pk 1+605, in the paleovalley area, in Bento et al. (2022). Particularly noteworthy are the piezometry results from borehole FB1 and the transversal variations of geologic strata. Profile PT35-05 location, at pk 1+600,5 signalled with black line in Figures 2 and 3.

The lithological units adopted in the geological model are part of the stratigraphic units, and different lithological units are considered depending on the type of soils found in each of the stratigraphic units. The geologic model was primarily developed using geotechnical survey results, geological mapping, as well as ancient cartography, engravings and tile paintings. The geotechnical zoning adopted was based mostly on the analysis of the results of the geotechnical surveys carried out in the various ground investigation survey campaigns. All this data was weighed up against the type of works to be built. Regarding the geological units adopted, following the analysis of all the available data, it was decided to subdivide some of the lithological units in order to best tailor and optimise the solutions designed. This approach can be considered as geological and geotechnical zoning, primarily based on lithological criteria.

Thus, the zoning first identified a total of 6 distinct geologic units (Landfills, Alluvium, Miocene Complex, Volcanic Complex-Basalts, Volcanic Complex-Tuffs, and Cretaceous - Bica Limestones), as a result of which the different lithostratigraphic complexes crossed by the works were separated. In turn, some of these units were further first subdivided into lithological units and then in geotechnical zones, differentiating them according to their distinct mechanical properties.

In this context, in the alluvium geological unit, the sandy alluvial lithological unit was subdivided into $a(ar)_a$ and $a(ar)_b$, depending on whether the N_{SPT} value is greater or less than 20 blows, respectively.

The clayey Miocene unit was also subdivided into $M(ag)_a$ and $M(ag)_b$, depending on whether the N_{SPT} value is greater or less than 40 blows, respectively.

The basalts and tuffs of the CVL were also subdivided into β_a, β_b, and β_c, and τ_a and τ_b, respectively. The β_a basalts are low to moderately-highly weathered (W_2 to $W_{3\text{-}4}$) with GSI values > 40, the β_b basalts, while partially maintaining the rock structure, are moderately to highly weathered to decomposed ($W_{3\text{-}4}$ to W_5) with GSI values < 25, and the β_c basalts consist of decomposed material (W_5) with N_{SPT} values $\leq$ 60 blows, commonly referred to as basaltic breccia in older campaign drillings. Tuffs τ_a and τ_b were subdivided based on N_{SPT} values equal to or less than 60 blows, respectively.

This process resulted in 17 geotechnical sub-zones, as presented in Figure 3.

2.2 Geotechnical survey

In addition to analysing and incorporating into the study the results obtained in 91 boreholes from previous campaigns, carried out both as part of previous phases of this project and other works in this densely populated area, a significant set of field and laboratory work was carried out as part of the studies. This included: 73 mechanical boreholes (17 geotechnical, 24 hydrogeological, and 32 geo-environmental); 13 core drillings; 3 inspection wells; 13 vane tests (FVT); 3 Marchetti dilatometric tests (DMT); 4 static penetration tests with seismic measurement (SCPTu); 39 Menard pressuremeter tests (PMT); 138 Lefranc permeability tests; 4 Lugeon permeability tests; collection of 30 undisturbed samples with a Shelby sampler for laboratory tests; Installation of 40 piezometers with a monitoring campaign; Laboratory tests for the physical and mechanical characterization of the soils, including particle size analyses, consistency limits, organic matter content, density of solid particles, volumetric weights, permeability tests, consolidation tests, and triaxial tests on soils; determination of bulk densities, unconfined compression tests, point load tests (PLT), and triaxial tests on rock; collection of water samples for laboratory analysis in environmental boreholes; pH, temperature, and electrical conductivity analyses in some piezometers; 4 pumping tests and 2 tide tests. The fieldwork is indicated on the plan in Figure 2.

3 GEOTECHNICAL PARAMETRISATION

3.1 Introduction

When possible, the estimation of the design geotechnical parameters for each geotechnical unit took into account the results of the work carried out in the various ground investigation campaigns available. When necessary, correlations established in the specialized literature were employed to convert the parameters obtained in the tests into design parameters.

The geotechnical field survey was parameterised using the following *in situ* tests: static cone penetration tests with seismic module (SCPTu's), Marchetti dilatometer tests with seismic module (SDMT's), Menard pressuremeter tests (PMT's), LNEC dilatometer tests (BHD's), Vane Tests (FVT) and permeability tests of the Lugeon and Lefranc types. Some results from laboratory soil tests were also used, namely uniaxial compression tests, determination of bulk density, and permeability determination obtained through triaxial tests. In addition, laboratory test results on rock samples were used, particularly uniaxial compression tests (UCS), point load tests (PLT), and determination of bulk density.

3.2 Methodology

Based on the mentioned tests, a statistical estimation of the possible characteristic values of the geotechnical parameters for the material of each considered unit was carried out.

Eurocode 7 recommends that "the characteristic value of a geotechnical parameter should be chosen to constitute a cautious estimate of the value that conditions the occurrence of the limit state under consideration." However, it is not specified what constitutes a "cautious estimate" of the value and how to calculate it.

As a contribution to the ongoing revision of EC7, Orr (2017), in Shen *et al* (2018), proposed the simplified equation (1), which was used as a preliminary calculation for the characteristic value of the geotechnical parameters:

$$X_k = X_{mean}\left(1 - 3aCOV/\sqrt{L_v}\right) \qquad (1)$$

After testing several approaches to determine the characteristic values, the project opted for equation (2), adapted from the simplified equation (1) proposed by Orr (2017):

$$X_k = X_{mean}\left(1 \pm 3aCOV/\sqrt{L_v}\right) \qquad (2)$$

where:

X_k –characteristic value of geotechnical parameter X;

X_{mean} –mean value of geotechnical parameter X;

a – factor responsible for the quality and extent of laboratory and field investigations. For high-quality and extensive investigations, a = 0.5; for average quality and extent of investigations, a = 0.75; and for values estimated from general experience or tabulated values, a = 1.

COV – coefficient of variation of geotechnical parameter X;

L_v –vertical dimension of the zone of influence.

The mentioned equation (2) was used, with a negative sign, whenever the number and consistency of tests allowed it, to calculate the characteristic values of the parameters undrained cohesion (c_u), effective friction angle (ø'), deformation modulus (E), and coefficient K_0, as these geotechnical parameters are more critical for the project the lower their values. On the other hand, for the unit weight (γ), saturated unit weight (γ_{sat}), and coefficient of permeability (k), which should be more critical for the project the higher their results, the equation was chosen with a positive sign.

The factor "a" and the coefficient of variation COV are important factors in these equations, as the first considers the quantity and quality of sampling, and the second, being a function of the standard deviation, reflects the dispersion of the data. For larger "a" and COV values, more cautious will the characteristic value obtained be.

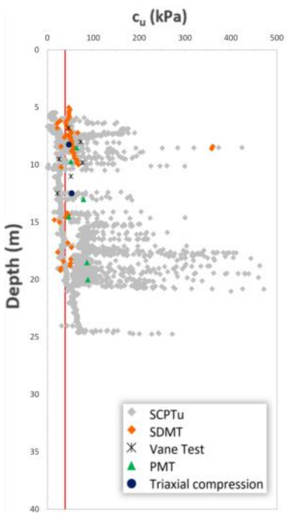
A factor a = 0.5 was considered when the number of tests is greater than or equal to four, and a factor a = 0.75 when the number of tests is less than four.

The factor L_v is variable for each geotechnical unit, depending on the weighted thickness of the respective considered units, and also reflects the dispersion of the results. For a specific parameter with a certain range of results, the smaller the area in profile where this dispersion occurs, the lower the degree of confidence that these results provide. In other words, the lower the value of L_v, the more conservative the characteristic value obtained will be.

In the calculation of characteristic values, any results that significantly deviated from all others, i.e., those outside the overall pattern of a distribution (outliers), were excluded. For this purpose, the common method based on the interquartile range (IQR) was considered. This method involves identifying a result as an outlier if it is more than 1.5 times the IQR above the 3rd quartile (Q3) or more than 1.5 times the IQR below the 1st quartile (Q1)

It is important to note that, for almost all units, despite having few results to determine the characteristic value of some geotechnical parameters, which makes the estimate less valid from a statistical point of view, it was still decided to carry out this assessment, which serves as another element of careful analysis.

The data processing results for determining the characteristic value of undrained cohesion (cu) for the clayey alluvial unit a(ag) that occurs in the upper zone of the lower section of the alignment are presented in Figure 5. This unit is depicted in Figure 4 and is more extensively found in the alluvial deposits towards the end of the alignment.



No. of data with outliers (n)	3556
No. of data without outliers (n)	3216
Average (A_x)	49,2
Standard deviation (S_x)	25,70
Coefficient of variation (V_x)	0,52
Minimum	0,1
Lower Quartile (Q_1)	30,2
Median	49,1
Upper Quartile (Q_3)	72,7
Maximum	471,9
Interquartile range (IQR)	42,6
Upper fence	136,5
Lower fence	-33,7
a	0,50
L_v	14,0
Characteristic Value (X_k)	39

Figure 5. Alluvium a(ag) – Estimate of undrained cohesion (c_u).

For reference, the unit a(ag) is a clay, sometimes silty, with a characteristic value of N_{SPT} of 4 blows.

Table 1. Characteristic values for undrained cohesion (c_u) by test type in the clayey alluvium unit.

a(ag) unit	SDMT	SCPTu	PMT	Vane Test	Tri-axial
c_u (kPa)	41	39	62	40	47

The adopted characteristic value for c_u was 40 kPa. For this unit in particular, the need to consider different values in function of the depth, was also considered, being this variation obtained by the expression below the table in Figure 6.

Geotechnical Unit	γ (kN/m³)	γ_{sat} (kN/m³)	c_u (kPa)	E_u (MPa)	c' (kPa)	ø' (º)	E' (MPa)	K_0	k (m/s)	ν	σ (MPa) [rock]	E' (GPa) [rock]
At	18	20	---	---	0	29	5	0,5	10^{-5}	0,35	---	---
a(ag)	17	19	* (40)	20	0	28	4	0,5 - 0,7 ***	10^{-8}	0,46	---	---
$a(ar)_a$	19	21	---	---	0	37	78	0,5	10^{-4}	0,30	---	---
$a(ar)_b$	18	20	---	---	0	33	22	0,4	10^{-5}	0,32	---	---
a(cg)	20	22	---	---	0	35	75	0,5	10^{-6}	0,30	---	---
$M(ag)_a$	22	23	338	98	10	33	61	0,8 - 1,0 ***	10^{-8}	0,33	7	1
$M(ag)_b$	21	22	186	39	0	28	19	0,8 - 1,0 ***	10^{-8}	0,38	---	---
M(cal)	24	24	---	---	75	34	400	0,8 - 1,0 ***	10^{-5}	0,25	7	7
β_a	27	27	---	---	160	40	2000	0,7 - 1,0 ***	10^{-8}	0,25	30	16
β_b	25	25	---	---	60	38	500	0,5	10^{-6}	0,26	6	3
β_c	23	24	---	---	50	34	254	0,5	10^{-6}	0,28	---	---
τ_a	21	22	---	---	60	35	228	0,5 -1,0 ***	10^{-7}	0,27	5	0,5
τ_b	20	21	---	---	30	30	83	0,5	10^{-7}	0,3	---	---
Cc1a	23	24	---	---	50	32	60	0,5 - 1,0 ***	10^{-6}	0,23	4	1
Cc1b	24	24	---	---	100	38	400	0,5 - 1,0 ***	10^{-7}	0,21	9	3
Cc1c	25	25	---	---	300	40	3900	0,7 - 1,0 ***	10^{-6}	0,21	53	27

* c_u=2,157z+5,886 ***Advisable to perform sensitivity analysis

Figure 6. Table of design parameters adopted for geotechnical safety checks, in Bento at al. (2022)

3.3 Design geotechnical parameters adopted.

Following the methodology explained above, naturally adapted to each testing type, geological unit material, location in depth, test quantity and previous experiences, it was possible to establish the geotechnical parameters presented in the table in Figure 6.

4 CONCLUSIONS

The extent of the ground investigation campaign, which complements previous campaigns, made it possible to gather the data that led to the development of the geological model adopted. At the date of the publication, this model has proven to be accurate in the ongoing works. The method of adopting characteristic values eased the systematic treatment of data (of varied nature and quantity) necessary for defining the design geotechnical parameters used in the development and safety verification of the construction works. Based on these characteristic values, the final adoption of parameters was naturally guided by the engineering practice and expertise, which are crucial in the process. developments.

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