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Analysis of consolidation settlements in a bauxite waste disposal system considering different constitutive models

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ABSTRACT: This research presents a study on the numerical modelling of a reinforcement landfill of a bauxite waste containment, in Brazil, built on a layer of clayey soil. The geometry of the cross section was discretized into finite elements, using the Soft Soil Creep and the Hardening Soil models to represent the mechanical behaviour of the soil layers, using input data in terms of both effective and total stresses. The numerical model was calibrated by comparing the results of the field instrumentation. The main objective of this research was to validate the choice of the constitutive models as well as the corresponding parameters estimated through field and laboratory tests.

Keywords: Embankment on soft soils; Numerical analysis; Primary consolidation.

1 INTRODUCTION

Bearing in mind the complexity of interpreting the loading on soft soil deposits using only analytical formulation, numerical models are often used in engineering projects involving the occurrence of clay layers. However, uncertainties in numerical analysis are still many, from the appropriate choice of constitutive models to represent soil behaviour to the parameter values used in these models. In this regard, it is a good opportunity to monitor the construction of a geotechnical work to assess, based on field instrumentation data, whether the material properties were adequately estimated through laboratory and field tests.

This research presents a numerical investigation involving the reinforcement landfill of a bauxite mining waste containment built on a layer of soft clayey soil. The analysis compares the predicted consolidation settlements with the displacement data obtained from field instrumentation (settlement plates).

2 BAUXITE WASTE DISPOSAL SYSTEM

The study area comprises a Bauxite Waste Disposal System (BWDS), situated in the Southeast region of Brazil, for which a numerical model was developed in order to estimate the soft soil displacements due to landfill overloads. Specifically, berms were built to reinforce the main dike to enable the implementation and operation of an upstream raising stage.

Based on a survey of available information about the area and available data from geotechnical tests, it was possible to define a stratigraphic profile for the cross section as shown in Figure 1. The profile is composed of layers of normally consolidated clayey and silty soils overlying a layer of more resistant sandy silt, which was verified both through the tactile-visual analysis of the SPT data and the classification system proposed by Robertson and Wride (1998) based on results of piezocone tests (CPTu).

3 CONSTITUTIVE SOIL MODELS

Two elastoplastic models, available in the software Plaxis 2D v. 2021, were used in this research: Soft Soil Creep model (SSC), for the clay layers and mining waste, and Hardening Soil Model (Schanz, Vermeer and Bonnier, 1999) for all other materials.

In order to gain better insight of the behaviour of the soil models and improve the quality of the input parameters, the SoilTest application from Plaxis 2D was used to reproduce the stress - strain curves previously obtained in laboratory tests.

Figures 2 and 3 show the results of consolidated undrained compression triaxial tests (CIU) for confined stress σ_3 = 50, 150 and 300 kPa carried out in the organic clay 5 indicated by soil number 14 in Figure 1. The continous curves (laboratory tests) are overlapped with the optimized curves obtained with Plaxis SoilTest considering the SSC model.

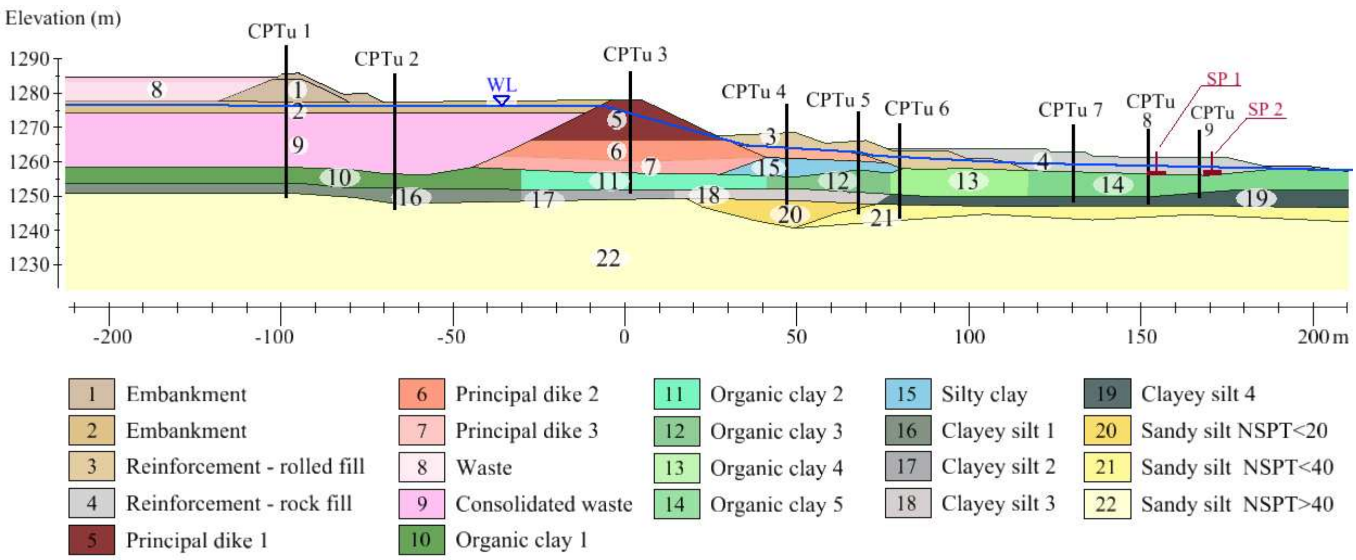

Figure 1. Cross section of the geotechnical profile

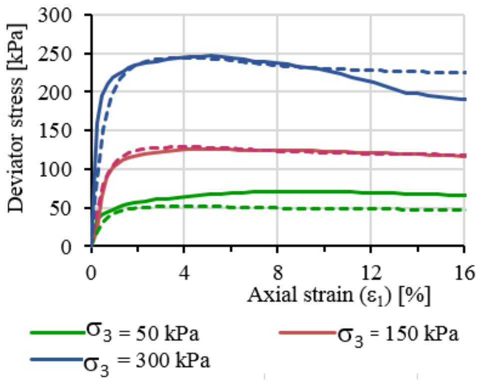

Figure 2. Stress vs. axial strain curves of CIU tests

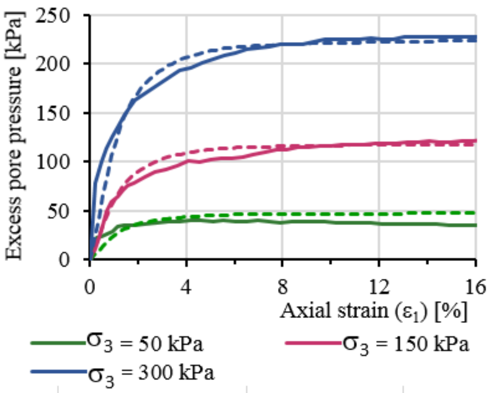

Figure 3. Excess pore pressure vs axial deformation curves of CIU tests

Since results from laboratory tests were not available for all soils, interpretation of piezocone penetration tests (CPTu) was also undertaken to assess geotechnical parameters along the boreholes shown in Figure 1.

To obtain the vertical permeability coefficient, it was used the correlation proposed by Robertson and Cabal (2015) as a function of the CPT-based soil behavior type classification system. At the points where dissipation tests were carried out, the horizontal consolidation coefficient was estimated according to Housby and Teh (1988), Jamiokowski et al. (1985) and the horizontal permeability coefficient was determined according to the empirical equation suggested by Baligh and Levadoux (1980). Values of the vertical permeability coefficients were also obtained directly from some 1D oedometric consolidation tests performed on saturated clays under different vertical loads.

For two boreholes (3 and 6 indicated in Figure 1), a comparison between estimates of the vertical permeability coefficients determined with different procedures is shown in Figure 4, where the grayish hatches represent clay layers and the greenish regions indicate the organic clay layers. The brownish regions correspond to layers of sandy silts.

Values of the oedometric modulus (E_{oed}) for the clay layers were calculated through the approach suggested by Lunne et al. (1997) where the CPTu corrected tip resistance (q_t) is multiplied by a factor α_m, whose value is a function of the soil characteristics. A similar correlation was presented by Robertson (2009), where the coefficient α_m varies with the normalized tip resistance (Q_t). Figure 5 shows the variation of E_{oed} for the clay layers along two CPTu boreholes (8 and 9).

To define the friction angle based on CPTu tests it was applied the Norwegian Institute of Technology (NTH) method for normally consolidated clay layers, detailed by Senneset et al. (1989) and Mayne (2007). The friction angle for NC clays can be also estimated through the plasticity index (PI), according to the graphical correlation proposed by Bjerrum and Simons (1960). This correlation was applied to verify the suitability of the friction angles determined with CPTu and compression triaxial tests (Figure 6).

Following a conservative approach, a complementary estimate was made using the effective friction angle for clays based on piezocone penetration tests suggested by Ouyang and Mayne (2019) for soils with friction angle between 20° and 45°.

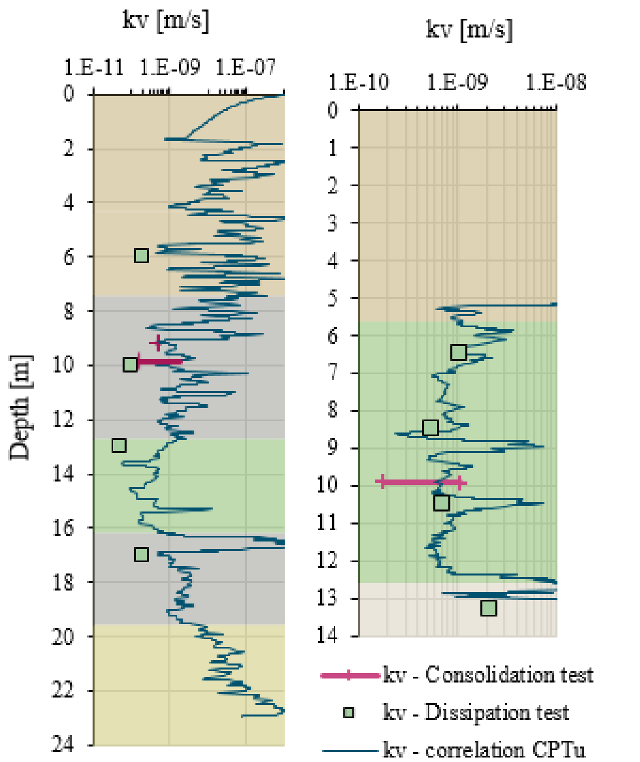

Figure 4. Vertical permeability coefficient (m/s) along borehole 3 (left) and 6 (right).

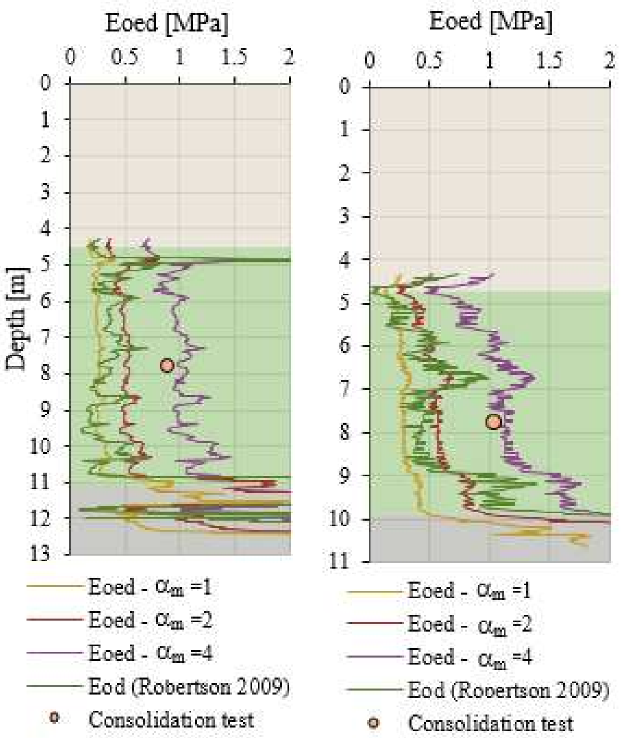

Figure 5. Oedometric modulus (kPa) along boreholes 8 (left) and 9 (right)

Cohesion of the clayey soils (Table 3) was obtained from triaxial tests and the estimation of their specific weights was done through laboratory tests and by the correlation proposed by Robertson (2010) for CPU tests, which also permitted an evaluation of the variation of the specific weight along the depth.

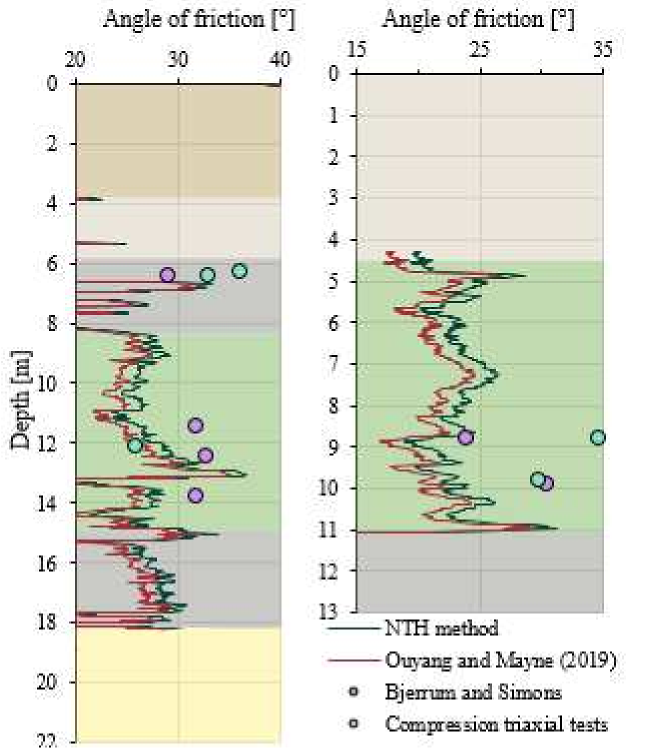

Figure 6. Friction angle (degrees) along boreholes 5 (left) and 8 (right)

The full interpretation of data from laboratory and field tests allowed the definition of a set of parameters representative of the hydro-mechanical behavior of all soils. Additionally, adjustments were also made with the Plaxis SoilTest module considering the stress-strain curves of triaxial compression tests (whenever available) as well as the settlement measurements over more than 500 days (plates SP 1 and SP 2 in Figure 1) due to the construction of the reinforcement berm (soil 4 in Figure 1). Table 1 indicates the parameters for the organic clay 5 obtained with Plaxis SoilTest and the best fit considering the consolidation settlements measured in field.

The final parameters for modeling the embankment, dyke and sandy soils (HSM) are given in Table 2 while Table 3 presents the corresponding parameters for modeling the waste and clayey soils (SSC model).

Table 1. Parameters of the organic clay layer 5

Parameter	Plaxis SoilTest	Back-analysis with SP data
λ^*	0.14	0.13
κ^*	0.014	0.017
ϕ' (°)	31	29
K_0	0.6	0.6
k_h (m/s)	8.1×10^{-10}	2.3×10^{-10}
k_v (m/s)	4.1×10^{-10}	1.2×10^{-10}

Table 2. Parameters for the HSM constitutive model

Layer	γ (kN/m³)	e₀	k_h (m/s)	k_v (m/s)
Embankment	19	0.5	1×10^{-7}	1×10^{-7}
Embankment 2	19	0.5	1×10^{-7}	1×10^{-7}
Rolled fill	18	0.5	5×10^{-8}	5×10^{-8}
Rock fill	20	0.5	0.5	0.5
Dike 1	19	1.0	1×10^{-8}	5×10^{-9}
Dike 2	19	1.0	1×10^{-8}	5×10^{-9}
Dike 3	19	1.0	1×10^{-9}	6×10^{-10}
Sandy silt (N_{SPT}<20)	18	0.4	1×10^{-7}	1×10^{-7}
Sandy silt (N_{SPT}>20)	19	0.4	4×10^{-8}	4×10^{-8}
Sandy silt (N_{SPT}>40)	20	0.3	1×10^{-6}	1×10^{-6}

Table 2, cont.

Layer	ϕ' (°)	c' (kPa)	E_{50}^{ref} (MPa)	E_{oed}^{ref} (MPa)	m
Embankment	35	8	40	20	0.6
Embankment 2	35	8	15	8	0.6
Rolled fill	35	8	24	12	0.6
Rock fill	40	0.5	80	45	0.45
Dike 1	32	8	20	10	0.6
Dike 2	27	8	40	20	0.6
Dike 3	25	8	24	12	0.6
Sandy silt (N_{SPT}<20)	34	8	40	22	0.55
Sandy silt (N_{SPT}>20)	35	8	30	15	0.55
Sandy silt (N_{SPT}>40)	38	2	80	50	0.45

Table 3. Parameters for the SSC constitutive model

Layer	γ (kN/m³)	e0	k_h (m/s)	K_0
Waste	15	1.0	6×10^{-10}	0.58
Organic clay 1	17	1.5	2×10^{-9}	0.6
Organic clay 2	16	1.5	3×10^{-10}	0.6
Organic clay 3	14	2.5	4.6×10^{-10}	0.6
Organic clay 4	13	3.5	2.3×10^{-10}	0.6
Organic clay 5	13	3.7	2.3×10^{-10}	0.6
Silty clay	17	0.6	4×10^{-9}	0.5
Clayey silt 1	19	0.7	4×10^{-8}	0.5
Clayey silt 2	18	0.7	1.2×10^{-8}	0.5
Clayey silt 3	18	1.0	9.3×10^{-9}	0.5
Clayey silt 4	18	1.0	4.6×10^{-9}	0.48

Table 3, cont.

Layer	c' (kPa)	ϕ' (°)	λ*	κ*	μ*
Waste	2	25	0.06	0.019	0.002
Organic clay 1	2	30	0.08	0.027	0.003
Organic clay 2	2	25	0.08	0.027	0.005
Organic clay 3	2	25	0.12	0.042	0.009
Organic clay 4	2	28	0.13	0.020	0.008
Organic clay 5	2	29	0.13	0.017	0.016
Silty clay	4	30	0.17	0.011	0.002
Clayey silt 1	4	30	0.08	0.019	0.001
Clayey silt 2	2	30	0.08	0.019	0.001
Clayey silt 3	2	30	0.08	0.019	0.002
Clayey silt 4	2	30	0.17	0.019	0.001

4 FIELD MONITORING AND NUMERICAL ANALYSIS

The main dyke (soils 5, 6 and 7 in Figure 1) reached its full storage capacity (soil 9) in 2011. In the following year a compacted soil dyke, approximately 8.0 m height (soils 1 and 2) was raised on the BWDS plateau. Years later, reinforcement berms (soil 3 in 2014 and soil 4 in 2019) were built near the downstream slope of the main dyke to improve the global stability of the structure.

The water level position is also shown in Figure 1, under condition of steady state flow, and the boundary conditions in the finite element mesh were prescribed as null horizontal and vertical displacements at the lower boundary and null horizontal displacements at the lateral boundaries.

The finite element consolidation analysis was carried out considering as input for the SSC model the shear strength and stiffness parameters in terms of effective stresses. Figures 7 and 8 show a comparison between the plate consolidation settlements and the curves obtained by numerical analysis considering the optimized parameters determined by Plaxis SoilTest and the adjusted parameters through back-prediction of field settlements.

An additional numerical analysis was also carried out representing the behaviour of the clayey soils by the HSM model. This constitutive model is formulated in effective stresses, but the input parameters may be given in two alternative ways: effective stress parameters to describe stiffness and shear strength (type A) or stiffness parameters given in effective stresses but shear strength in total stresses (type B). In the first case the undrained shear strength is an output of the analysis whose value must be continuously checked, observing if the computed shear stress $(\sigma_1 - \sigma_3)/2$ does not violate the undrained shear strength determined from field or laboratory tests. In analysis type B, although the pore pressures are calculated, the modulus of deformability is no longer dependent on the stress state. Not every constitutive soil model allows these alternative types of analysis in Plaxis. The SSC is available only for analysis type A.

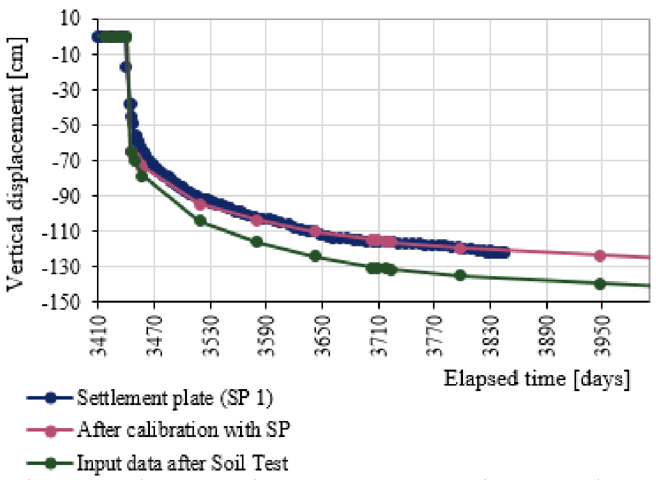

Figure 7. Consolidation settlement measured by plate SP 1 compared to curves from numerical analysis.

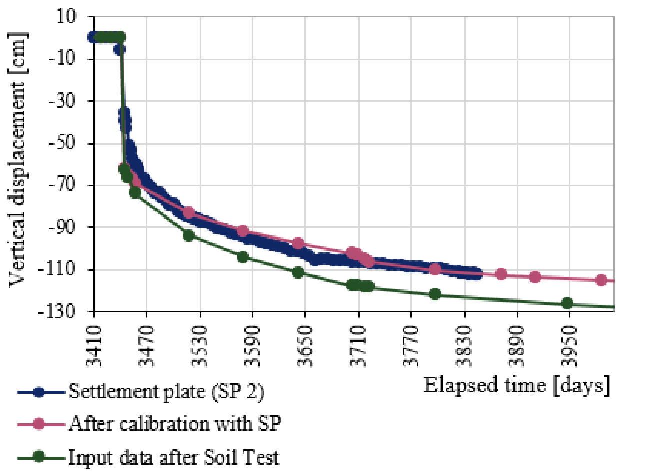

Figure 8. Consolidation settlement measured by plate SP 2 compared to curves from numerical analysis.

For analysis HSM type A, the strength parameters were the same as indicated in Table 3 while the stiffness parameters for the clayey soils $(E_{50}^{ref}, E_{oed}^{ref}, m)$ are shown in Table 4. Values of the deformation moduli were obtained from results of Plaxis SoilTest and correlations with CPU tests proposed by Lunne et al. (1997) and Robertson (1990), as shown in Figure 5. The parameter m was estimated from calibration with Plaxis SoilTest. For analysis HSM type B the undrained shear strength S_u, shown in the last column of Table 4, was determined from CPTu tests.

As can be seen in Figure 9, analysis carried out with HSM model, considering the type B option, yielded consolidation settlements much smaller than those calculated by SSC and HSM (type A) models. A similar behaviour was observed by Karstunen and Amavasai (2017). The HSM model is able to carry out simulations of oedometric and triaxial compression tests but with different sets of parameters, but it is not capable to represent the behavior of a soft soil predominantly subjected to shear or compression loads with the same parameter values. This fact also explains why the slope of the critical state line (CSL) was described differently in the theoretical development of both constitutive models (SSC and HSM).

Table 4 Parameters of HSM model for clayey soils

Layer	E_{50}^{ref} (MPa)	E_{oed}^{ref} (MPa)	m	S_u (kPa)
Waste	2054	1643	0.8	40
Organic clay 1	2500	1250	0.8	60
Organic clay 2	2500	1250	0.8	30
Organic clay 3	986.5	789.2	0.8	25
Organic clay 4	1530	765	0.8	25
Organic clay 5	1480	740	0.8	15
Silty clay	744.1	595.3	0.8	32
Clayey silt 1	1577	1261	0.7	100
Clayey silt 2	1582	1266	0.7	130
Clayey silt 3	1582	1266	0.7	80
Clayey silt 4	718	575	0.7	50

Although the results calculated with HSM (type A) may be seen as satisfactory when compared to those determined with the SSC model, there is no justification for using HSM in problems involving soft soils under compression, even because the input parameters must be determined by the same laboratory tests (triaxial compression and oedometer) required by the SSC model.

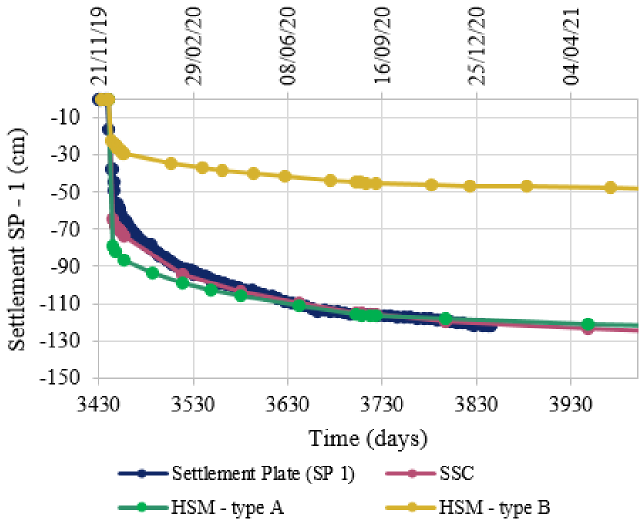

Figure 9. Consolidation settlement measured by plate SP 1 compared to curves from numerical analysis considering clayey layers modelled with HSM and SSC.

For the two settlement plates SP 1 and SP 2, predictions of the final settlement were also made by the Asaoka's (1978) graphic method, as shown in Figure 10, based on the field measurements observed over time.

The values of the final settlement for SP 1 and SP 2 were predicted as 129.5 cm and 115.5 cm, respectively, approximately 60 cm lower than settlements estimated by numerical analysis. The main factor of this difference comes from the fact that in Asaoka's method (op. cit.) the secondary compression is not taken into account, which is an important key factor when clays with high content of organic matter are present, such as in the BWDS subsoil.

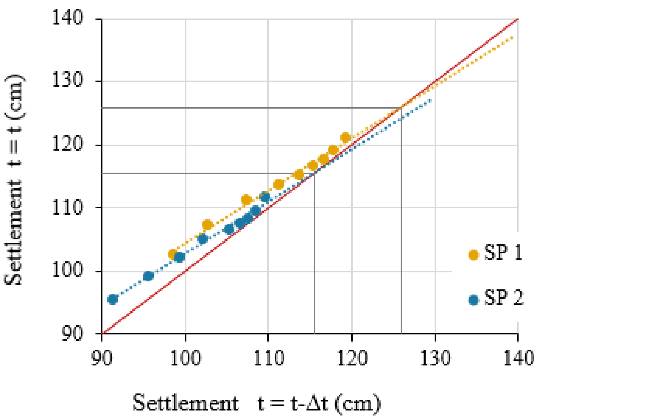

Figure 10 Application of Asaoka method (1978) for prediction of the final consolidation settlement of plates SP 1 and SP 2.

5 CONCLUSIONS

In this paper the consolidation settlement due to the construction of a reinforcement berm on layers of organic clay was numerically computed through finite element analysis using the software Plaxis 2D, v. 2021.

Several laboratory and field test results were available for geotechnical characterization of the clayey soils. The correlations from the literature, especially with CPTu data, were generally satisfactory when compared to results obtained in laboratory tests. The redundancy of parameters proved to be quite interesting in this research, allowing the definition of coherent ranges of input parameter values for the numerical simulations.

With respect to the constitutive models used to represent the hydro-mechanical behavior of the organic clays, the SSC model yielded the most consistent results when compared with displacements measured along more than 500 days. An attempt was made to use the HSM model to represent the behaviour of organic clays, but the results from analysis type B resulted quite different than those obtained with SSC and HSM (type A) models.

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