Performance analysis of an embankment over a soft soil deposit through instrumentation and numerical simulation

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Abstract. The present paper describes the performance analysis of an embankment over a soft soil deposit. The embankment is part of the duplication of the Brazilian Federal highway, BR 470, a major infrastructure project that crosses several soft clay deposits in the State of Santa Catarina, Brazil. To increase stability the reference section was reinforcement with geogrids, equilibrium berms, and stage constructions of the landfill (slow construction) was specified combined with preloading and pre-fabricated vertical drains (PVDs) to accelerate consolidation. The performance during field construction was mainly evaluated through instrumentation considering the evolution of horizontal displacements and the stabilization of settlements. Given the limitations of the methods used for control during the construction process, and a discrepancy between field settlements and original project predictions, soil profile and geotechnical parameters were revaluated during construction, and used in a revised performance analysis. The revised performance analysis was done in finite element software PLAXIS, using Mohr Coulomb and Soft Soil Models. Some important considerations were observed when proceeding this back analysis that could have been used to modify the construction process during execution. Additionally, to analyze the effects of the inherit variability, a sensibility analysis to determine levels of possible settlements and time of consolidation was performed.

Keywords: soft soil, back analysis, finite element.

1 Introduction

The duplication of the Brazilian Federal highway, BR 470, is a major infrastructure project that crosses several soft clay deposits, between the municipalities of Navegantes and Gaspar, in the state of Santa Catarina, Brazil. These soft deposits, have approximately 20 m thick, are very soft, and are composed essentially by normally consolidated clay layers with superficial sandy silts layers, which can be subjected to excessive settlements and lateral movement and even failure (Cordeiro, [1]).

The present paper describes the performance analysis of a specific section of the duplication project of BR 470, Km 30 + 460 (UTM coordinates 706668.93, 7023483.22), in which the embankment was constructed considering geotechnical solutions as follows: basal reinforcement with geogrids; equilibrium berms; stage constructions of the landfill (slow construction), preloading and pre-fabricated vertical drains (PVDs).

Preloading with prefabricated vertical drains is and effective and low-cost solution (Indraratna et al. [2]) that is well established, and widely used and describe in literature (e.g. Carillo, [3]; Barron, [4]; Hansbo, [5]; Indraratna et al. [6, 2]; Bergado et al., [7, 8, 9]; Chai et al., [10]; Shen et al., [11]; Rowe and Taechakumthorn, [12]; Deng et al., [13]; Cascone and Biondi, [14]; Bari and Shahin, [15]; Liu et al., [16]; Lam, Bergado, and Hino [17], Stark et al., [18]). The challenge to adopt this solution consists in defining reliable predictions of the soil performance, consolidation rates and deformation levels, as design properties are generally obtained through an insufficient profile investigation, and even with a consistent campaign natural variability leads to uncertainties.

To handle soil inherent variability and insufficient soil profile investigation, instrumentation during the construction process and the life time of the structure can be an essential tool but must be combined with and appropriate control method. In this sense, the performance during field construction of section Km 30+460, was mainly evaluated through instrumentation considering the evolution of horizontal displacements and the stabilization of settlements. Given the limitations of the methods used for control during the construction process, and a discrepancy between field settlements and original project predictions, soil profile and geotechnical parameters were revaluated during construction, and used in a revised performance analysis, that was done in finite element software PLAXIS and will be discuss in the sequence.

2 Characteristic section

Figure 1 shows a general view of geometric and geotechnical section for Km 30+460. The geotechnical section was obtained after a carefully revision of in situ (Standard Penetration Test, piezocone and vane tests) and laboratory tests (triaxial UU - Undrained and Unconsolidated test, oedometer) defining a subdivision of layers as follow: from 0 – 2 m, clayey silt with gravel; 2 - 4.46 m, very soft light grey clay; 4.46 – 9.32 m, very soft dark grey clay; 9.32 -10.5 m, fine loose clayey sand; 10.5 – 12.12 m, very soft dark grey clay; 12.12 – 12.78 m, fine loose clayey sand; 12.78 – 16.10, very soft dark grey clay; 16.10 – 17.62, loose gray clayey sand; 17.62 - 18.36, very soft dark grey clay; and 18.36 – 20m, dense gravelly sand. Geotechnical parameters considering the presented subsoil sub-division are presented in Table 1.

As for the geometric section the original project defined a final road embankment height of 1.6 m, plus 1.5 m landfill temporarily surcharge and 1.0 m landfill compensation. Figure 1 also displays the equilibrium berm with 12.5 m wide and 2.0 m thick, allied to three mono directional geogrids to increase stability: two geogrids with tensile strength of 1000 kN/m and one with 600 kN/m. Stage construction of the landfill (slow construction) was recommended in the original project and adjusted according the evolution of the working front, that is, no predefined heights were specified for the stages of construction. A PVD triangular mesh of 2.5 m spacing was considered with a rectangular section of 10 cm x 0.5 cm with an equivalent diameter (dw) of 0.067 m.

For the evaluation of the landfill performance during and after construction phase, a monitoring program was elaborated and instruments for measuring vertical and lateral movement, and pore pressures were installed. Inclinometers were placed adjacent to the equilibrium berm, piezometers and settlement plates (benchmarks) were placed in the middle of the transversal section, with a spacing in a longitudinal section around 10 m (see Figure 1).

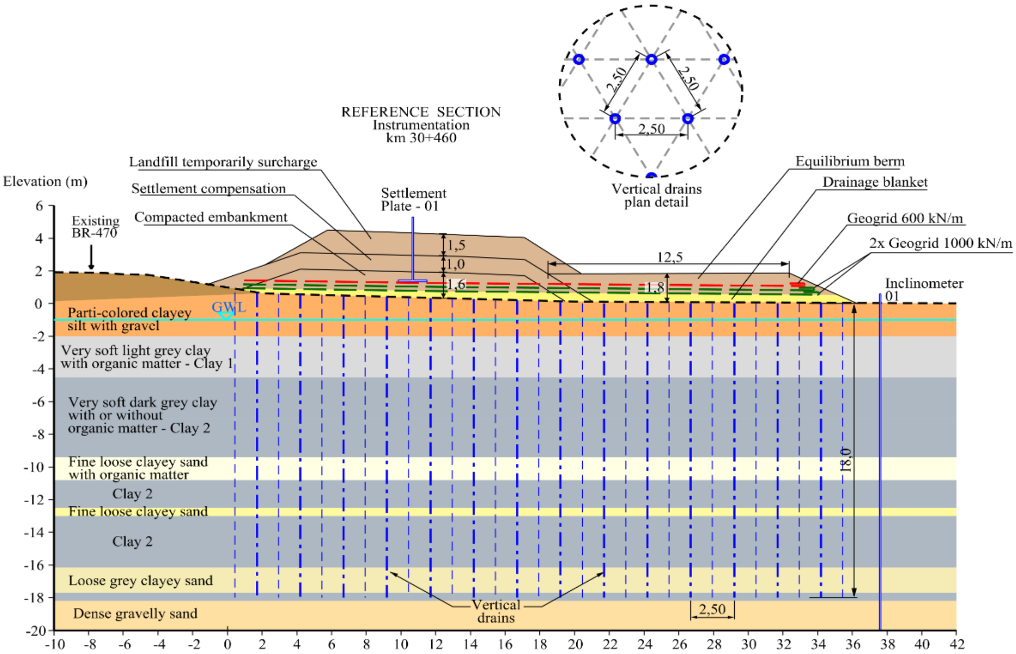


Figure 1. Reference section – KM 30 + 460

Table 1. Materials identification for section Km 30 +460 and constitutive parameters

Material	Constitutive Model	Parameter	Value
Embenkment and equilibrium berm	Mohr-Coulomb	Specific Weight (γ)	20 kN/m³
		Friction Angle (ϕ')	35°
		Cohesion (c')	5 kPa
		Young Modulus (E)	2.5×10^4 kPa
		Poisson Coefficient (υ)	0.3
		Permeability - k_h e k_v (m/day)	1.0 / 1.0
Clayey silt with gravel (old embankment)	Mohr-Coulomb	Specific Weight (γ)	18 kN/m³
		Friction Angle (ϕ')	28º
		Cohesion (c')	3 kPa
		Young Modulus (E)	7×10^3 kPa
		Poisson Coefficient (υ)	0.4
		Permeability - k_h e k_v (m/day)	1.0 / 1.0
Very soft light grey clay with organic matter (Clay 1)	MohrCoulomb and Soft Soil	Specific Weight (γ)	15 kN/m³
		Initial Void Ratio (e_0)	2.03
		Coefficient of Compressibility (C_c)	0.68
		Coefficient of Recompression (C_r)	0.13
		Friction Angle (ϕ')	25º
		Cohesion (c')	2 kPa
		Undrained Shear Strenght (S_u)	23.0 kPa
		Young Modulus (E)	1×10^3 kPa
		Poisson Coefficient (υ)	0.45
		Permeability - k_h e k_v (m/day)	$3x78.10^{-4}$ / 1.89×10^{-4}
Very soft dark grey clay with or without organic matter (Clay 2)	MohrCoulomb and Soft Soil	Specific Weight (γ)	16 kN/m³
		Initial Void Ratio (e_0)	1.66
		Coefficient of Compressibility (C_c)	0.46
		Coefficient of Recompression (C_r)	0.13
		Friction Angle (ϕ')	26º
		Cohesion (c')	2 kPa
		Undrained Shear Strenght (S_u)	27.0 kPa
		Young Modulus (E)	1.3×10^3 kPa
		Poisson Coefficient (υ)	0.45
		Permeability - k_h e k_v (m/day)	9.0×10^{-5} / 4.5×10^{-5}
Sands	Mohr-Coulomb	Specific Weight (γ)	16 kN/m³
		Friction Angle (ϕ')	25º
		Cohesion (c')	0 kPa
		Young Modulus (E)	4×10^3 kPa
		Poisson Coefficient (υ)	0.35
		Permeability - k_h e k_v (m/day)	1.0 / 1.0

3 Performance Analysis

3.1 Original project

This section briefly describes the original conceptual project defined by a consulting company. In this sense, only a brief description of geotechnical considerations and summarized general results of predicted settlement and safety factor will be presented.

The geometric section of the embankment considered in the original project is the same as that display in Figure 1 but initially only projecting an embankment with 1.6m height over a clay deposit of a total thickness of 16 m, with no subdivision. Parameters adopted for the clay material were: Specific Weight (γ) of 15 kN/m^3; Initial Void Ratio (e_0) of 2.7; Coefficient of Compressibility (C_c) of 1.50; Coefficient do Recompression (C_r) of 0.05 (resulting in compression and recompression ratio of 0.41 and 0.013, respectively); Overconsolidation ratio (OCR) of 1.25; Vertical coefficient of settlement (c_v) of 1.3x10-3cm^2/s and Undrained Shear Strength (Su) of 12 kN/m^2.

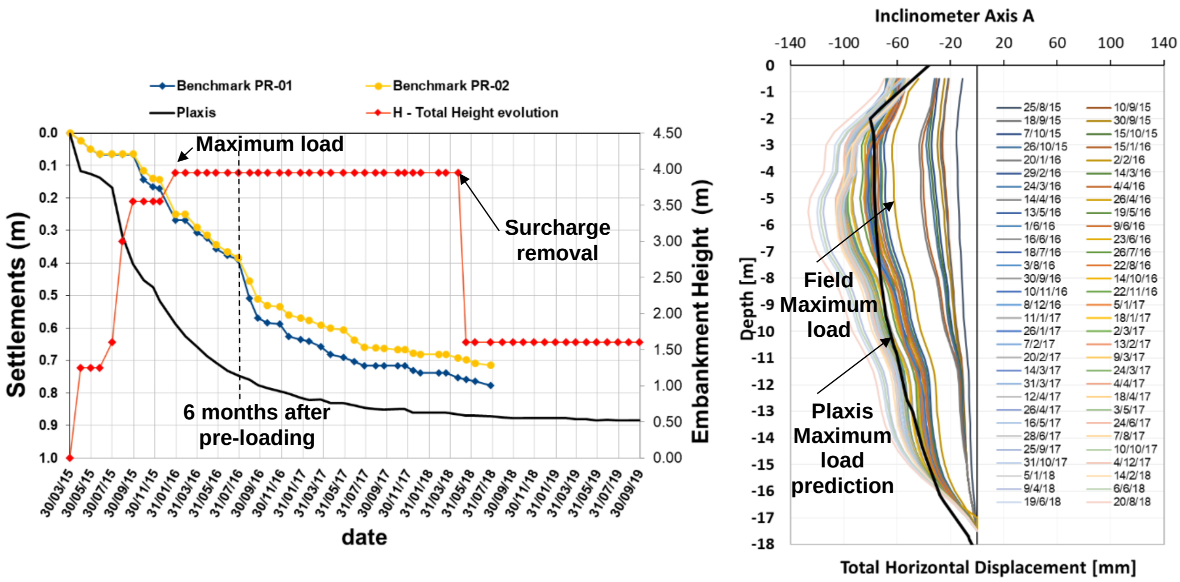
From this set of parameters a total vertical settlement of approximately 1m was characterized, thus letting the adoption of a preloading surcharge of 1.5m combined with PVD to accelerate displacements. A PVD triangular mesh with a 2.5 m spacement was considered to reaches with the pre-loading a work displacement of 1m in six months, so additionally projecting and 1m field compensation. To guarantee stability two geogrids of 1000 kN/m and one of 600 kN/m, and an equilibrium berms are also conceived. With the aforementioned considerations a safety factor FS of 1.20 was defined.

3.2 Field Monitoring analysis

For the evaluation of the landfill performance during and after construction phase, a monitoring program was elaborated and instruments for measuring vertical and lateral movement, and pore pressures were installed, as already shown in Figure 1. Stage construction of the landfill was recommended in the original project and adjusted according the evolution of the working front. For Km 30 + 460, the advancement of the front work can be verified with loading dates from April 2015 until January 2016 (Figure 2a). A total vertical displacement around 27cm was measured in 28 of January of 2016, when the maximum load was reached (1.6m embankment height, plus 1.5m surcharge and 1m compensation). According the original project the surcharge should have been maintained in the field until a 1m settlement was reached, after six months. However, in this indicated time the field vertical displacements were around 40 cm, observation that made the executor decide to keep the surcharge in field until the settlements stabilized. The maximum load was left in the field for a period of 26 months. In this time period the evolution of vertical displacement reached 75cm. In April of 2018 preloading surcharge and over plus field compensation were removed after an analysis of displacement stabilization (Asaoka Method, 1978, see application in Cordeiro, [1]).

Horizontal displacements evolution can be verified from inclinometer field measurements displayed in Figure 2 b. From the related figure a maximum horizontal displacement of 60mm was verified when the maximum load was reached, continuing with an evolution until reaching 124 mm in August/2018. Although, no signs of an abrupt horizontal displacement can be observed, an additional analysis of distortion and distortion rates was performed indicated stability (distortion rates less than 0,01 % per day, see Cordeiro, [1]).

It is important to note that due to problems in the field, the pore pressure measurements were disregarded already in the initial loading phases, not being taken into account in the stabilization analysis.



(a) (b)

Figure 2. Instrumentation results and numerical predictions (a) vertical displacements - benchmarks readings and evolution of embankment height detailed; and (b) horizontal displacements

3.3 Back analysis

A structured back analysis was developed right after the removal of the preloading surcharge. Initially, after a careful review of geotechnical design parameters and the executed geometry, numerical analyses in plane-strain, using Finite Element (FE) software Plaxis were performed to evaluate stability (short term analysis) and to analyse consolidation (long term analysis).

According the type of analysis, the following constitutive models were used:

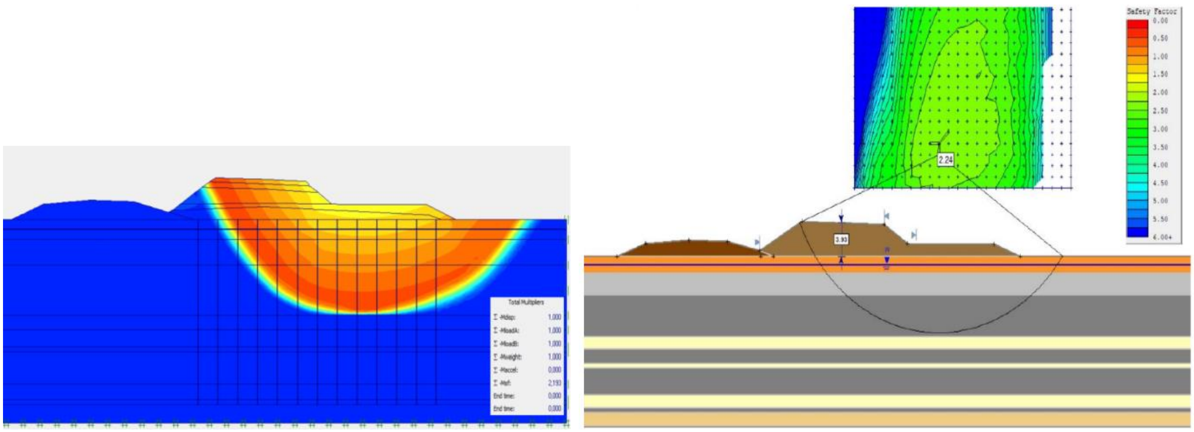
- for short-term analyses all layers were modelled using Mohr Coulomb's elastoplastic model;
- for long-term analysis the Mohr Coulomb elastoplastic model was used to represent sandy materials, embankment, equilibrium berm and silty layers, while the Soft-Soil model, a modified Cam-Clay model, was used to represent the clay layers, very soft light grey clay with organic matter and very soft dark grey clay with or without organic matter.

Table 1 presents a summary of the geotechnical parameters of the materials involved, as well as the constitutive models adopted in the numerical modelling according the above mentioned types of analysis.

It is important to highlight that the plane strain condition considered in the present paper is a simplification that allows a general view of the phenomenon and further analysis considering a 3D model are indicated by the authors.

3.3.1 Stability – short term analysis

This section presents the results of the numerical analysis for the sort term evaluation, considering a fast construction and undrained behaviour of the clays. Therefore, undrained shear resistance values (Su) were considered representative for the clay layers (see Table 1). Results of this analysis are directly display in Figure 3, as surface failures. Additionally to the analysis in the FE software Plaxis, Figure 3 also presents a complementary analysis performed in limit equilibrium using Slide 5.0 software and Janbu model (Figure 3b). From the presented results it can be verified a good agreement between the dimension of the surface failure predicted by both models and a good agreement between safety factors (FS): the analysis by FE resulted in a safety factor, FS, of 2.19, while the limit equilibrium analysis provided FS of 2.24. In both models no geogrid was considered, since there is no recommendation in cases of FS greater than 1.5. It is important to highlight that the original project had provided a considerable lower safety factor (FS=1.2) but considering a distinct geotechnical profile and parameters (see item 3.1), thus demonstrating the importance of revision and discussion of soil profile and geotechnical section and parameters.



(a) **(b)**

Figure 3. Surface failure (a) Modeling in FE, FS = 2.19; (b) Limit Equilibrium, FS = 2.24

3.3.2 Settlements – long term analysis

To complement the performance analysis, a set of numerical predictions considering the executed section and times was done in FE (long term analysis). The first evaluation considered the characteristics geotechnical values defined for section KM 30+460. In this analysis, additionally to settlements, horizontal displacements are evaluated, aiming to validate the subsoil layers definition. In the subsequent analysis compressibility and permeability parameters are varied to define possible ranges of settlements and times for stabilization.

Characteristic values

Results of predicted settlements and horizontal displacements when characteristic values for section KM 30+460 were adopted (parameters Table 1) are already display in Figure 2. In Figure 2a, the evolution of settlements with time can be observed, showing that in general numerical evolution of settlements agrees with field data, although predicting a higher value of settlements. On the date of pre-loading surcharge removal (April 2018), the PR-01 plate had 0.76 m and the PR-02 plate, 0.69 m of settlement, compared to 0.86 m predicted by FE. In this time of evaluation a 97 % of consolidation was defined by FE. Three (3) months after removing pre-loading, last reading presented in Figure 2 a, consolidation reached 98 % in FE evaluation, with 0.87 m of settlements. Comparing with field measurements, PR-01 and PR-02 had 0.78 and 0.72 m of settlements, respectively, providing differences from predicted values and measurements of 10 and 17%.

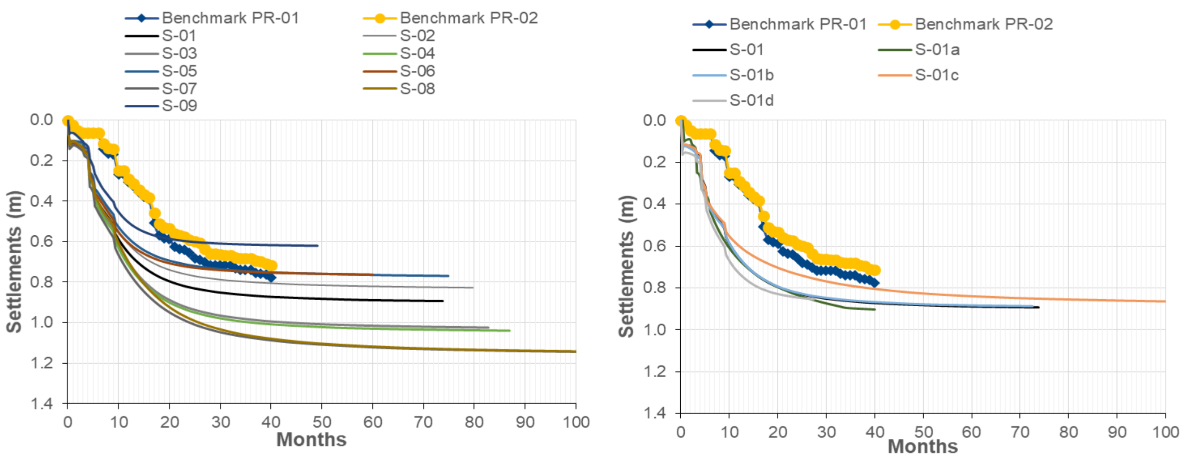
Predicted values and field results of horizontal displacements are presented in Figure 2 b. In general, there is a good agreement between the evolution of horizontal displacements with depth, although, maximum predicted horizontal displacements for the maximum load are higher than the field readings - 70 mm horizontal displacements are characterized by the numerical analysis while field results are 60 mm.

Sensitivity analysis

To access a variation of settlements and times for stabilization for the region of BR 470 the compression ratio $CR=Cc/(1+e_0)$, and permeability ranges, obtain from odometers tests and dissipation were evaluated and a set of 12 complementary numerical simulations, S-02 to S-09 (varying compression ratio from 0.135 to 0.416), S-01a to S-01d (varying permeability from 4.30×10^{-6} to 8.50×10^{-4} m/day) were executed. Results of settlements evolution with time are directly compared to the field measurements of PR-01 and PR-02 in Figure 4.

From Figure 4a it is possible to verify that the numerical modeling curves showed behaviors similar to those obtained from the settlement plates featuring maximum vertical displacements from 0.62 to 1.15 m. The numerical curves that best represent field results correspond to simulations S-01; S-02; S-05 and S-06; with maximum settlements varying from 0.76 to 0.89. Those results were obtained for compression ratios (CR) of 0.135 to 0.225.

When permeability was varied, Figure 4b, it can be verified that according to the permeability combinations for the soft clay layers, the times for the occurrence of total settlements ranged from 27.4 months (analysis S-01d) to 422 months (analysis S-01c). Observing the plates field readings and numerical curves, the S-01c simulation was the one that came closest to the field results, when the plates PR-01 and PR-02 presented 0.78 and 0.72 m of settlement in a period of 40 months. Considering that S-01c simulation is representative of the field behavior a complete dissipation is reached only in 422 months, thus characterizing that when field measurements was taken at 40 months the dissipation was around 90%.



(a) (b)

Figure 4. Vertical settlement with time (a) compression ratio evaluation; and (b) vertical coefficient of consolidation analysis

4 Conclusions

The performance during field construction of BR 470 duplication was mainly evaluated through the instrumentation considering the evolution of horizontal displacements and the stabilization of settlements. Given the limitations of the methods used for control during the construction process, and the discrepancy between field settlements and original project predictions, soil profile and geotechnical parameters were revaluated during construction, and used in a revised performance analysis. This back analysis was performed in FE after the removal of the temporary load (preloading surcharge), to verify the validation of the maximum displacements and even security levels during loading.

The first step of the back analysis was an evaluation of stability which indicated, for section KM 30+460, considerable higher safety factor without geogrids. To complement the performance analysis, a set of numerical predictions of settlements considering the executed section and times was done in FE. One evaluation considered the characteristics geotechnical values defined for section KM 30+460, and in the subsequent analysis compressibility and permeability parameters are varied to define possible ranges of settlements and times for stabilization. Obtained results indicate that when characteristic values of revised section of KM 30+460 are used the numerical evolution of settlements and horizontal displacements are slightly higher than field readings, but provide a better approximation than that characterized by the original project.

The sensitivity analyzes of compressibility and permeability parameters identified possible ranges of settlements variation and stabilization times. The variation range adopted for CR characterized settlements in the range of 0.62 to 1.15 m, providing a better approximation to field readings when CR values of 0.135 to 0.225 where considered. As for the permeability parameters, the variation range characterized periods of stabilization of the settlements (complete dissipation of excess pore pressure) varying between 27.4 to 422 months. When comparing numerical results and field data of benchmarks the simulation that characterizes a stabilization time of 422 months provides the best approximation with the field curves, and from this consideration a consolidation of at least 90% can be inferred for the time when the temporary load was removed.

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