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THREE-DIMENSIONAL MODELLING OF AN EMBANKMENT BUILT ON A SOFT SOIL IMPROVED WITH PREFABRICATED VERTICAL DRAINS

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Abstract. This work compares the field measurements of a non-symmetric embankment built on a Portuguese soft soil improved with prefabricated vertical drains (PVDs), with the numerical predictions of a 3D modelling where the PVDs are simulated according to the field flow conditions. The change in the permeability with the void ratio and the effect of the smear zone are also included in the numerical analysis. The numerical predictions are compared with the field data in terms of settlement, horizontal displacement and excess pore pressure. In general, the numerical results are very close to the field measurements, namely in terms of vertical and horizontal deformations.

1 INTRODUCTION

The construction of embankments on soils with a low coefficient of permeability (clays and soft soils) frequently requires the installation of vertical drains within the soil foundation to accelerate the consolidation process. In general, the design of the vertical drain is still often based on Barron's [1] classical theoretical solution, which considers an infinite discharge capacity, a realistic condition for sand drains with a permeability greater than 500 m/year [2].

In general, with the use of prefabricated vertical drains (PVDs) the rate of consolidation is smaller than in the case of an infinite discharge capacity, since there is a reduction of the discharge capacity (well resistance), caused by [3]: (i) drain folding caused by settlement; (ii) reduction of the drain core, due to increased lateral earth pressure at depth; (iii) drain blockage due to the infiltration of fine soil particles. The efficacy of the PVDs may also be affected by disturbance of the surrounding soil (smear zone) due to the installation process, which depends on the shape of the mandrel and the anchor plate [4]. Some research studies have shown that the smear zone diameter is about 2-5 times the size of the equivalent mandrel diameter [2, 5, 6], while the horizontal permeability inside the smear zone decreases approximately 1.5-2 times [6, 7]. Some three-dimensional (3D) consolidation analyses of embankments have been performed over the last twenty years, using various constitutive models, such as: a linear elastic law [8], p-q-θ critical state model [9] and Modified Cam Clay model [10-12]. However, in general, the water flow into the system's soil-drains is still simulated using an equivalent permeability [11], ignoring the real water flow conditions.

This work compares the field data of a non-symmetric embankment built on a Portuguese soft soil, where significant horizontal displacement is expected, with the results of a 3D numerical analysis, where the PVDs are simulated according to the field flow conditions. The change in the permeability with the void ratio and the effect of the smear zone is included in the numerical analysis.

This investigation was carried out using a 3D FE code developed at the University of Coimbra [13-15], which allows elastoplastic analyses to be performed with coupled consolidation and several constitutive laws.

2 MAIN CHARACTERISTICS OF THE SITE AND THE EMBANKMENT

The embankment is located in Portugal, at km 7.775 on the A14 motorway. The geotechnical profile of the cross-section (Figure 1) shows alluvial sandy-silts and clay-silts over limestone [16]. The water table changes with time, between 0.5 meters above and 1.5 meters below the surface. The geotechnical characterization of the foundation soil is shown in Table 1 [4, 15-17]. To account for permeability anisotropy, a ratio of k_h/k_v equal to 3 was considered.

The embankment (Figure 1) was instrumented with: a sub-vertical inclinometer tube placed vertically at the foot of the main embankment slope, three settlement plates (T_1, T_2 and M_1) and three electric piezometers (P_2, P_3 and P_5). To accelerate the consolidation of the soft soil under the embankment and the additional berm, PVDs with a rectangular cross section of 100 mm x 3 mm were installed to the bottom of the soil foundation, in a 2.2 meter spaced square pattern. The PVDs drainage is upwards into a 0.5 meter thick sand layer, located at the bottom of the embankment. The PVDs used are band-shaped drains with a crenellated polypropylene core, wrapped in a non-woven polypropylene filter and have a discharge capacity of about 790 m^3/year for a confining pressure of 200 kPa (manufacturer's specifications).

The construction sequence for the embankment is described by six sub-layers with elevations of 1.1, 1.85, 3.45, 4.7, 7.55 and 8.1 meters, applied at 0, 80, 240, 290, 385 and 420 days, respectively.

3 NUMERICAL MODELLING OF THE EMBANKMENT

3.1 – Characteristics of the mesh

The FE mesh used in the 3D analyses consists of 10,406 nodal points and 1,917 twenty-noded isoparametric quadrilateral elements (Figure 2), with a refinement of the mesh close to the permeable boundaries. Below the water table, the FE mesh allows a coupled analysis of fluid flow and deformation to simulate the consolidation phenomenon. Due to the symmetrical conditions of the embankment, in the direction of the motorway axis, only one row of drains with half of their section is modelled to save computation time. Each 3D element has sixty-eight nodal degrees of freedom, which permits the evaluation of the displacement at twenty nodes and the excess pore pressure at eight corner nodes. The FE elements used provide quadratic interpolation of displacement and bilinear interpolation of pore pressure.

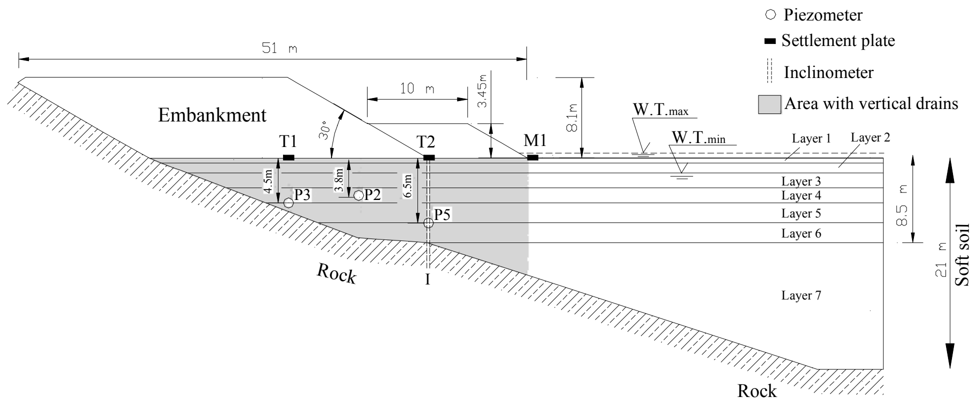

Figure 1: Geotechnical profile of the section of the embankment (based on [4, 15, 16])

Table 1: Characteristics of soil layers (based on [4, 15-17]).

Layer	Depth (m)	γ (kN/m^3)	OCR	K0	e_0	Parameters of MCC model				k_v (m/day)	k_h/k_v
						$e_{\lambda o}$	λ	κ	M		
Emb.	-	22.0	-	-	-	-	-	-	-	-	-
1	0.0 - 0.5	15.0	7.5	0.87	2.0	2.57	0.217	0.027		-	
2	0.5 - 1.5	15.0	5.5	0.76	2.0	2.72	0.217	0.027		1.30×10^{-4}	
3	1.5 - 3.0	14.8	3.5	0.62	2.0	2.88	0.261	0.035		5.20×10^{-4}	
4	3.0 - 4.5	14.5	1.8	0.47	2.2	3.05	0.304	0.060	1.48	8.64×10^{-4}	3.0
5	4.5 - 6.5	14.5	1.0	0.40	2.1	3.13	0.304	0.060		1.73×10^{-4}	
6	6.5 - 8.5	15.2	1.0	0.40	1.8	2.45	0.178	0.025		0.43×10^{-4}	
7	8.5-21.0	15.0	1.0	0.40	1.9	2.76	0.217	0.026		0.52×10^{-4}	

The boundary conditions are such that the right vertical side is restrained from horizontal movement, the bottom and left boundaries have zero displacement both vertically and horizontally, and the displacement parallel to the axis of the motorway is also restrained. Thus, the deformation pattern is close to a plane strain condition. The boundary condition defined between the rock foundation and the upper soil layers (soft soil and embankment) simulates the wrinkling induced by the significant weathering of the limestone near the surface, providing an effective bond between the rock and the soil [16].

In terms of hydraulic boundary conditions, the top boundary, corresponding to the water table (assumed at 0.5 meters of depth), is permeable (zero pore pressure boundary condition is prescribed). The remaining boundaries are impermeable (i.e., no water flow), since the rock foundation is impermeable and the right hand side boundary is sufficiently far from the embankment. Above the water table, no water flow was considered in the numerical analysis, that is, drained materials are used.

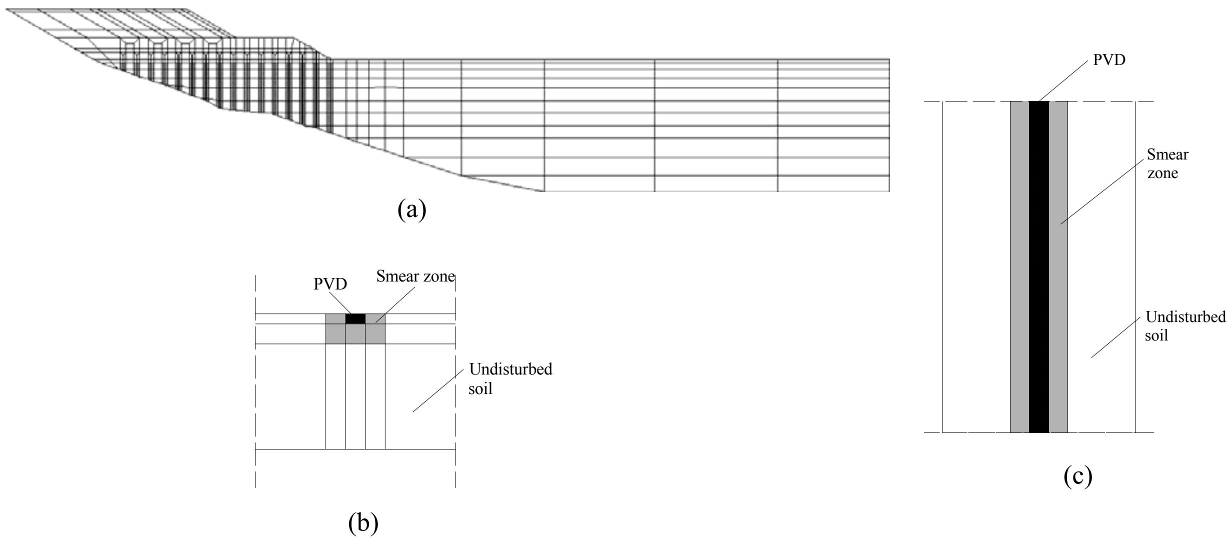

Figure 2: 3D FE mesh (based on [4, 14]): (a) side view; (b) plan view of the PVD; (c) section view of the PVD.

3.2 Characteristics of the PVDs

In the 3D analysis, a smear zone, with a radius of 16.5 centimeters (r_s/r_w =5) is considered, with a reduction in the horizontal permeability of the soil surrounding the drain to k_s=k_h/2 [18], while the vertical permeability remains the same as the natural soil [6, 7].

The geotextile strip drains are simulated by equivalent square drains with a 52 mm [(a+b)/2] edge (Figure 2b), calculated to obtain the same draining surface as the PVDs. The smear zone is considered as a square area around the PVD, corresponding to a relation of r_s/r_w of 5. The well resistance is not considered, since PVDs have a high discharge capacity (about 790 m3/year), thus, the PVDs are modelled by highly permeable FE elements (k = 1000 m/day).

During the calculation, the permeability coefficients change with the void ratio, in accordance with the expression [19]:

$$k = \mathrm{k}_0 x 10^{\frac{e-e_0}{C_k}} \tag{1}$$

where e_0 represents the initial void ratio, k_0 is the coefficient of permeability corresponding to e_0, k is the corrected permeability coefficient relative to the current void ratio e and C_k is a constant equal to $e_0/2$ [20].

3.3 Mechanical properties of the materials

The material of the embankment is characterized by an isotropic linear elastic law, with ν' equal to 0.3 and E' assuming the values of 30, 25, 20, 15,10 and 5 MPa, from the bottom to the upper layer to reflect the lesser confinement of the upper layers [16].

The Modified Cam Clay (MCC) model is used to predict the behavior of the soil foundation (and PVDs), whose parameters are shown in Table 1. The behavior inside the yield surface of the MCC model is simulated by a linear elastic law defined by a Poisson ratio (ν’) of 0.3 and a Young modulus (E') derived from the following expression [21]:

$$E' = \frac{3(1+e_0)(1-2\upsilon')}{\kappa} p'_0 \tag{2}$$

where κ represents the swell-recompression index (in scale) and p'_0 the initial volumetric stress.

4 ANALYSIS OF RESULTS

4.1 Settlement

The evolution over time of the settlement under the embankment (plate T_2) is shown in Figure 3. In general, the 3D numerical results reproduce the embankment's behavior very satisfactorily. The major discrepancy between the numerical predictions and the field data occurs between 200 and 400 days, perhaps due to a poor simulation of the evolution of the embankment's height over time. Another discrepancy is observed for the final settlement, where the value measured is slightly higher than that obtained numerically.

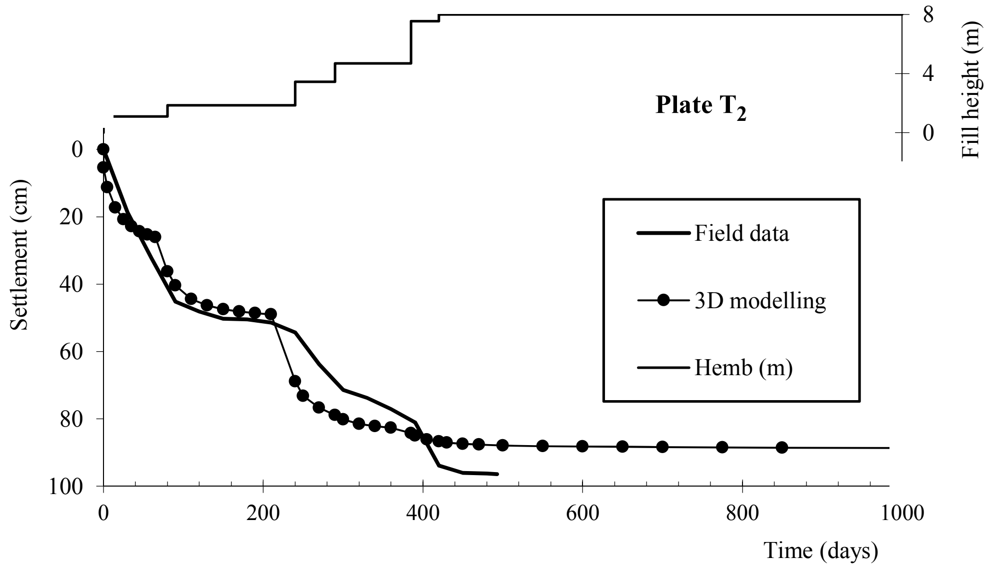

Figure 3: Time evolution of the settlement for plate T_2

The transversal evolution of the settlement measurements for 290 and 500 days is illustrated in Figure 4. A poor numerical prediction is observed for plate T_1, with smaller settlements than those measured in the field. This fact can be due to a faulty assessment of the alluvium thickness in that zone, due to limestone weathering and the imposition of null displacement at the embankment-rock boundary in the calculations [15]. Thus, the numerical results originate a more closed settlement profile than that found in situ.

4.2 Horizontal displacements

Figure 5 shows the evolution at depth of the horizontal displacement under the foot of the main embankment slope, for 290 and 500 days. In general, the numerical predictions reproduce qualitatively the evolution of the horizontal displacements, although there are some quantitive discrepancies in relation to the situ behavior. Indeed, the 3D analysis predicts slightly less horizontal displacement than the field data, where the major discrepancies are near the surface and for 500 days.

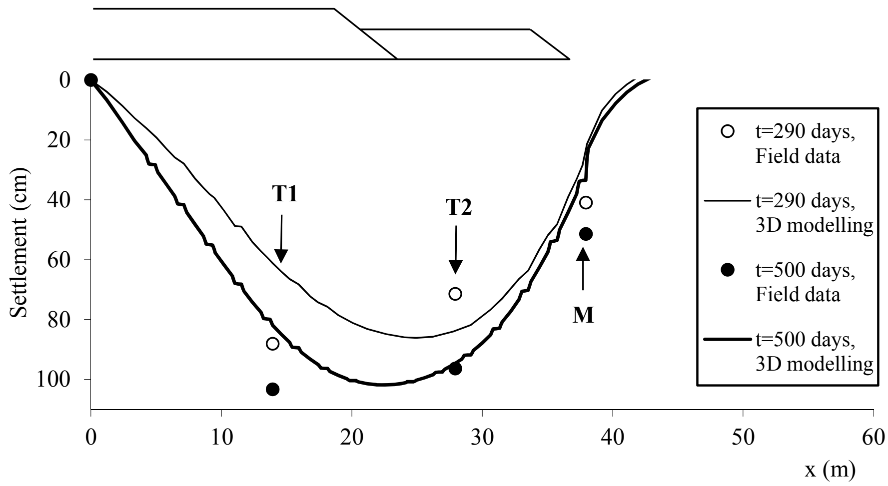

Figure 4: Transversal variation of settlement for 290 and 500 days

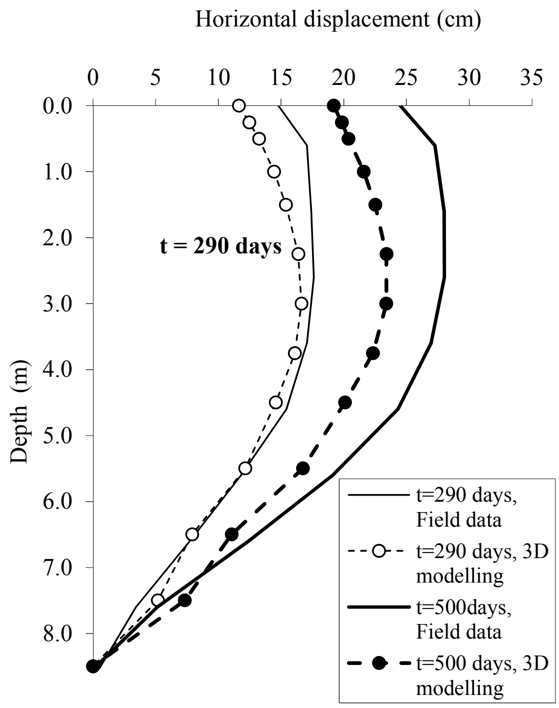

Figure 5: Horizontal displacement for 290 and 500 days

4.2 Excess pore pressure

Figure 6 depicts the predicted and observed evolution of excess pore pressure (Δu) at piezometer P_5. Although the numerical results show the general pattern of the field data, the values predicted numerically, namely the peaks of the excess pore pressure, are higher than the field data. These differences can be attributed to a poor agreement between the times of the piezometer readings and the construction phase and the omission of the time variation of the water table in the numerical prediction.

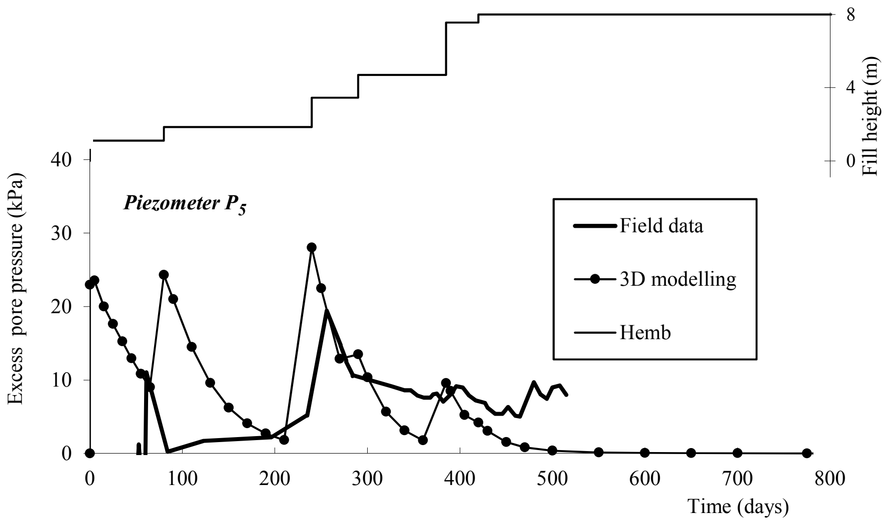

Figure 6: Time evolution of the excess pore pressure at piezometer P5

5 CONCLUSION

The 3D calculation performed for this particular embankment, which has a non-symmetric geometry, reveals that the numerical results reproduce qualitatively the general pattern of the field behavior, namely the evolution of the settlement, horizontal displacement and excess pore pressures. However, in quantitative terms, some slight discrepancies are observed. In fact, the predicted settlement and horizontal displacement tend to be slightly lower than those measured in-situ, while the "peaks" of excess pore pressure obtained numerically are higher than those measured in the field. This discrepancy may be attributed to a poor agreement between the times of the piezometer readings and the construction phase and a non-consideration of the time variation of the water table in the numerical analysis.

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