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Ground improvement and reinforcement in four dikes on soft soil

Amélioration et renfort du sol dans quatre digues sur sol mou

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ABSTRACT

The final stretch of the Low *Guadalquivir* Canal includes an irrigation control pond, closed by four homogeneous embankments, three of which are placed on marshland. Figure 2 shows the final design that incorporates the following elements: a geotextile with a stiffness of 8333 kPa at the base, band shaped drains and upper sand blankets, a chimney drain and widened berms. During 1157 days measurements have been taken at topographical stakes, settlement plates, piezometers and inclinometers placed within and below the embankments. Several rheological models have been employed to interpret the measured displacements and pore pressures.

RÉSUMÉ

La section finale du Canal du Bas *Guadalquivir* inclut un étang de control pour l'irrigation, fermé par quatre remblais homogènes, dont trois sont placés sur le marais. La Figure 2 montre la conception finale qui incorpore les éléments suivants : un géotextile avec une rigidité de 8333 kPa à la base, des drains de bande et des couches supérieures de sable, un drain de cheminée et des bermes élargies. Pendant 1157 jours des mesures ont été prises à des points de repère, plaques de tassement, piezometres et inclinomètres placés dans et au-dessous des remblais. Plusieurs modèles rhéologiques ont été utilisés pour interpréter les déplacements et les pressions interstitielles mesurés.

1 INTRODUCTION

The final stretch of the low *Guadalquivir* Canal includes an irrigation control pond with a capacity of 7.9 hm^3, closed by four embankments with initial heights over foundation of 11.2, 6.2, 9.0 and 5.0 m respectively. Its object is to assure the irrigation of 14,600 ha of soil, providing flexibility to the demand. The work has been integrated in the environment, as it constitutes a rich damp area that provides biodiversity and ecological stability to a surrounding, where autochthonous species develop (Fig. 1). It permits the nesting and hibernation of birds. To help to this task autochthonous species will be planted on the banks and floating islands for white storks will be installed. The pond is located at the south of the province of Seville, near other damp areas, such as Doñana Park.

Figure 1. Pond view showing flamingos and other bird species

Dikes No. 1, 2 and 3 are placed on marshland. The channelling of a 2727 m stream provided one part of the embankment materials.

Figure 2 shows the final design of the closure dike No. 1.

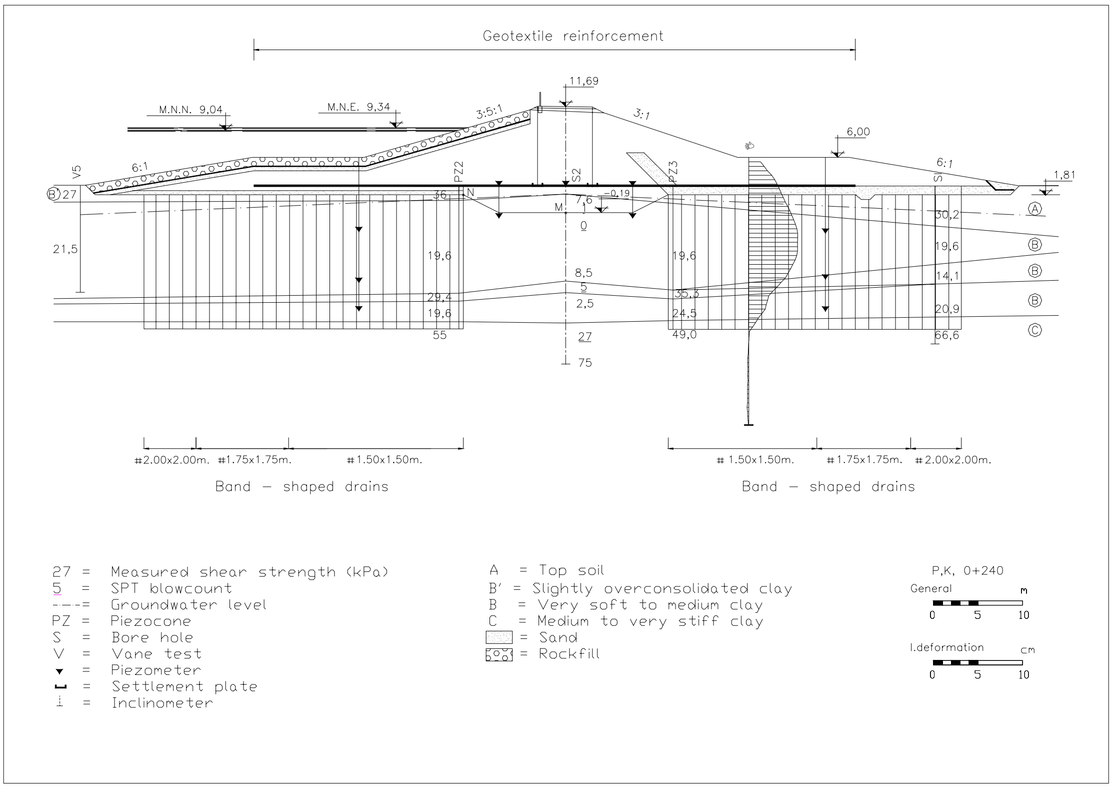

Figure 2. Final cross-section of dike No. 1.

The four dikes are homogeneous and were initially designed with a horizontal downstream drain and with slopes indicated in Figure 2 but with 2.5 m wide berms. The dikes were founded at a depth of 1 m, and 3 m at the place of the cut-off (2.5 m at the place of dike No. 4).

Preliminary stability calculations were carried out based upon the shear strength obtained from the unconfined compressive strength (v. equation 4) and assuming that the dikes might crack due to the differences of rigidity between embankment and foundation (v. Nakase, 1970). The results (Justo et al., 2000) indicated that the factors of safety obtained were unacceptable at the central part of dike No. 1 (0.29). The stability calculations were repeated using as a parameter the shear strength obtained from consolidated- undrained shear tests under the effective overburden pressure. Then a factor of safety of 1.55 was obtained. As the parameters obtained from this test are usually unsafe, due to the decrease of the water content of the soil during recompression at the consolidation step, this factor of safety was not considered safe, and it was decided to carry out a new improved site investigation.

2 NEW SITE INVESTIGATION

The new site investigation was based upon piezocone, vane and Marchetti dilatometer tests, and excellent piston samples that provided, among other things, the undrained shear strength, the coefficient of consolidation under horizontal flow and the effective stress parameters. Figure 2 shows the situation of the in situ tests in the central section of dike No. 1 and the measured undrained shear strength.

3 SOIL PROFILE

The following soil types appear from top to bottom:

A. Top soil
B'. Organic clay and silt, with high liquid limit, slightly overconsolidated.
B. Very soft to medium blue organic clay and silt of high liquid limit.
C. Medium to very stiff yellow clay of high liquid limit.
D. Green marl like clay, very stiff to hard.

Figure 3 shows a geotechnical profile along the axis of dike 1. The average properties of the layers are collected in Table 1.

4 CALCULATION PROGRAM AND RHEOLOGICAL MODELS

Plaxis FE program and the sliding surface method have been used throughout in the calculations. The following Plaxis models have been employed:

1. *Mohr-Coulomb: e*lasto-plastic model with non-associated flow rule to take into account dilatancy. Three varieties of this model have been used for undrained analysis, depending upon the use of effective or total stresses, and drained or undrained parameters:
   a) Effective stresses and drained parameters.
   b) Effective stresses and undrained parameters.
   c) Total stresses and undrained parameters.
2. *Soft Soi*l: similar to cam-clay and using effective stresses and drained parameters.
3. *Soft-Soil-Creep*: like *Soft Soi*l but with time effects

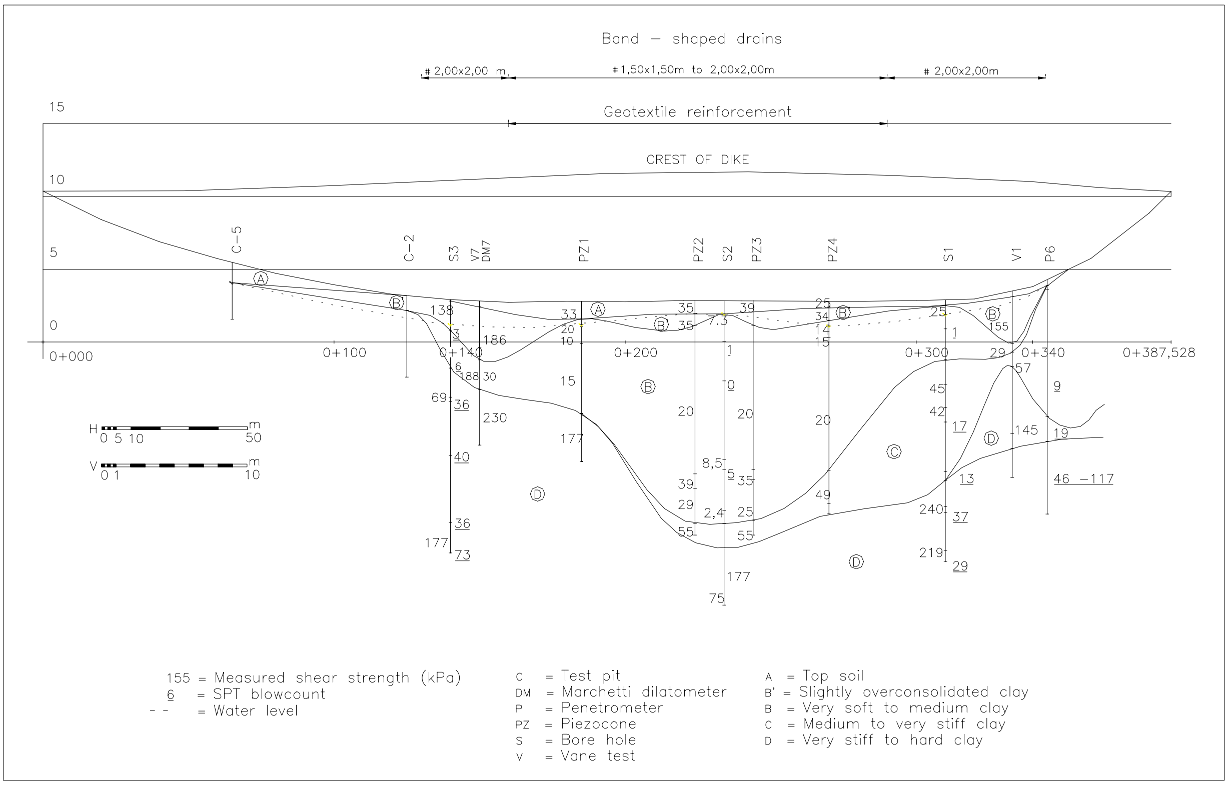

Figure 3. Geotechnical profile along centreline of dike No. 1.

Table 1. Parameters for calculation in the different soil layers

Soil layer	Direct shear				Triaxial		q_u	<80 μ	w_L	I_P	I_L	N	ρ_d	γ	USCS
	c' kPa	ϕ'	c_{cu} kPa	ϕ_{cu}	c' kPa	ϕ'	kPa	%				*blows/30 cm*	kg/m^3	kN/m^3	
B'							196	86.0	75.3	40.7	0.04	9	1290	17.4	CH MH OH
B	12	16.1º	13	15.4º	5.4	23.8º	41	96.1	71.1	38.6	0.61	4	1100	16.6	CH MH
C			8.8	26.7º			84	90.5	66.2	45.5	0.47	15	1180	16.5	CH
D	41	18.1º	102	17.9º			21	91.9	65.2	27.7	-0.19	40	1420	18.3	MH

In models *1a, 2* and *3*, the drained parameters are introduced in the calculations and the materials are put to undrained behaviour. The excess pore pressures are obtained assuming an undrained Poisson's ratio, ν, of 0.49 5 in saturated materials to avoid numerical problems. The following equation relates the bulk moduli of water, K_w, and the soil skeleton, K:

$$\frac{K_w}{n} = \frac{3K(\nu_u - \nu)}{(1-2\nu_u)(1+\nu)} = \frac{300K(0.495-\nu)}{(1+\nu)} \qquad (1)$$

To ensure realistic computations, K_w must be high, compared with K. If $\nu \leq 0.35$ then $K_w/n > 30$.

The rate of excess pore pressure is calculated from the small volumetric strain rate, according to:

$$\dot{u}_e = (K_w/n)\,\dot{\varepsilon}_v \qquad (2)$$

In model *1c* total stresses are employed and the undrained parameters are introduced in the calculations. The generation of both steady and excess pore pressures are prevented, setting the water weight to zero and the material to drained behaviour.

5 PARAMETERS FOR UNDRAINED CALCULATIONS

The piezocone point resistance was corrected according to Baligh et al. (1981) and Campanella et al. (1982). The undrained shear strength was calculated according to the equation:

$$c_u = (q_c - \sigma_v) / 16 \qquad (3)$$

where q_c is the piezocone point resistance and σ_v the total vertical stress at point level.

The values obtained according to the equation given by Senneset et al. (1982) are larger. Figures 2 and 3 show the average values of the undrained shear strength obtained with different procedures under dike No. 1.

Many authors have indicated that the factor of safety in short term slides when the undrained strength is obtained from vane tests may be above one, and the calculated factor of safety increases with the plasticity index (Jioménez Salas et al. 1981; Lade, 2001). So the shear strength was divided by a coefficient larger than one increasing with plasticity index.

The undrained shear strength obtained from piezocones (PZ), vanes (V & SI) and the Marchetti dilatometer are quite similar and larger than the one obtained from the unconfined compression strength, in boreholes (S), using the well known equation:

$$c_u = q_u / 2 \tag{4}$$

The average value of the coefficient of consolidation for horizontal flow (1.1×10^{-7} m^2/s) obtained from piezocones is only 3.7 times larger than the coefficient of consolidation for vertical flow obtained from oedometer tests.

The values of c_u / σ'_0 obtained with the piezocone have been compared with those suggested by Hansbo (1957):

$$c_u / \sigma'_0 = 0.45\ w_L / 100 \tag{5}$$

The overconsolidation ratio (OCR) has been obtained as:

$$OCR = c_u / \sigma'_0 \text{ (piezocone)} / c_u / \sigma'_0 \text{ (Hansbo)} \tag{6}$$

In this way, it was found that soil type B has an OCR = 1.2 up to 5.25 m depth and is normally consolidated below. Soil type C has an OCR =1.3.

The *Soft-Soil-Creep* model has been introduced recently in the calculations and no final results have yet been reached. From the rest of the models it is assumed that the *Soft Soi*l model is the best for settlement calculations. In *Mohr-Coulomb* models the Elasticity modulus has been obtained from the well known correlation $E = \alpha\ c_u$. In order to reach similar undrained settlements the following values must be assigned to coefficient α :

α = 75 for model *1a*
α = 170 for model *1b*
α = 135 for model *1c*

On the other hand, the pore water pressures obtained in *Mohr-Coulomb* models *1a* and *1b* and in the *Soft Soi*l model are similar.

The calculation is performed in total stresses, introducing the undrained Poisson's ratio ν_u and undrained modulus $E_u = E(1+\nu_u) / (1+\nu)$, in all models except *1c*. For ν=0.35, E_u=1.107E. The following moduli are used in the calculations:

E_u = 80 c_u for model *1a*
E_u= 190 c_u for model *1b*
E = 145 c_u for model *1c*

6 SAFETY FACTORS AND MODIFICATIONS IN THE DIKES

The safety factor (SF) in finite element calculations is obtained reducing the strength parameters by a coefficient up to the moment when this value reaches a maximum. This maximum is the factor of safety. On the other hand, the sliding surface method gives an upper bound of the SF (v. Jiménez Sala et al. 1981). In this case, the short-term SF using model *1a* is somewhat smaller than using the modified Bishop method.

The long-term safety factor was 2.03, but in order to reach an adequate short term SF the following modifications were introduced in the dike (v. Fig. 2 & 3):

- A reinforcement geotextile with a stiffness of 8333 kN and a strength of 1000 kN/m was placed at the base of the embankment in dike No. 1.
- Band-shaped drains, with spacing from 1.5 to 2.0 m, and upper sand blanket in dikes 1 and 3.
- The horizontal downstream drain was transformed into a chimney drain.
- The berm width was extended up to 12.5 m.
- Topographical stakes, settlement plates, piezometers, and inclinometers were introduced in the four dikes.
- Owing to the large foreseen settlements, the maxima heights over foundation were increased to 11.9, 7.2 and 9.2 m for dikes 1, 2 and 3.

The calculated stresses in the geotextile were not very different in all the *Mohr- Coulomb* and *Soft Soil* models. The SF in models *1b* and*1c* is similar.

Construction has been carried out along one year. At the end of construction, the pore pressure measurements indicate an average degree of consolidation of 63% in the zone occupied by the band-shaped drains and 30% where there are no drains at the central part of the dike. At present the water has been raised up to level 8 m.

7 CONCLUSIONS

- In soft clay similar values of the undrained shear strength are obtained using vane tests (corrected for plasticity index), piezocone tests (eq. 3) or the Marchetti dilatometer.
- The use of the unconfined compression strength and equation 4 may lead to very low values of the shear strength. Owing to that the method is not recommended.
- If the parameters of the consolidated-undrained shear test are used to correct the shear strength, this strength is overestimated by a factor of 1.6.
- The three Mohr- *Coulomb* models indicated in § 4 may be used to reach acceptable values of the settlement as indicated in § 5.
- The pore pressures are acceptably predicted with all models.
- The SF obtained with the FE and the sliding surface methods are alike, but in sections in which a reinforcement geotextile is included only the FE method may be duly employed.
- Only a *Soft-Soil-Creep* model, has allowed reproducing in the calculations the very high horizontal displacements measured at the inclinometers. This model will be treated in a forthcoming paper.

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