Field instrumentation of an embankment on stone columns

Instrumentation de un remblai sur colonnes ballastées

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ABSTRACT

The construction of a 10-metre-height embankment on a marsh area is presented. The foundation ground was improved with stone columns to reduce the final settlement and increase the settlement rate. Stone columns were constructed by the dry bottom-feed method with a diameter of 0.8 m and a spacing of 2.8 m in a triangular pattern. The soft soil treated is a more or less homogeneous clay layer of 9 m. Field instrumentation was used to study stone column behaviour. Total pressure cells on soil and column, piezometers and a 3-level extensometer were installed. Settlement and stress concentration factors are assessed, whereas the consolidation process was too fast to be analysed. The measurements are compared with the results of a three dimensional finite element model.

RÉSUMÉ

On présente la construction d'un remblai 10 m haut sur un sol mou. Le terrain a été amélioré pour réduire les tassements et accélérer la consolidation, avec colonnes ballastées jusqu'une profondeur de 9,0 m, avec 0,80 m de diamètre et séparées de 2,80 m. On a instrumenté le remblai, et on a mesuré les contraintes verticales totales dans les colonnes et dans le sol, les surpressions interstitielles et les tassements à trois profondeurs. Les résultats pur la situation finale ont été comparés avec des calculs par éléments finis.

Keywords : instrumentation, stone columns, settlement, finite elements

1 INTRODUCTION

Stone columns are one of the most common ground improvement techniques for foundation of embankments on soft soils. This paper presents an example where stone columns proved to be a successful and economical solution, reducing the final settlement and the consolidation time. The stone column performance during and after the embankment construction was monitored. The results of the observed behaviour and a numerical back calculation are presented.

The presented instrumentation is part of a wider research project on stone columns that include lab tests (Cimentada et al., submitted) and theoretical analyses (Castro and Sagaseta, 2008). Furthermore, an instrumentation of installation effects of stone columns in the presented field site was carried out (Castro and Sagaseta, in preparation).

2 SITE DESCRIPTION

The field trial was carried out in the construction of the ring road of Sueca, near Valencia (Spain). This road is close to a marsh area of high environmental value, called "La Albufera", crossing large areas of soft soils that had to be improved with stone columns, preloading and vertical drains. The instrumented area is located under the access embankment to a 10 m high overpass. In this area, the natural ground level was flat. Stone columns and 2 meters of preload were used to improve the ground.

Stone columns, 9 m deep with an average diameter of 0.8 m and a spacing of 2.8 m in a triangular patter were used. The columns were constructed by the dry bottom-feed method (vibro-displacement), using a vibrocat with a probe of 0.3 m of radius.

The design soil profile is shown in Figure 1. The most compressible unit is a sensitive soft clay layer, 2 m thick, with undrained shear strength of about 20-50 kPa, immediately below the 2 m crust, which is silt and silty clay with higher undrained shear strengths.

Below a depth of 4 m, the clay becomes slightly stiffer and less sensitive. This clay layer has some occasional thin sandy layers or lenses. Between 6 and 8 m depth, there is sand with a variable content of silt. From 8 to 10 m there is dense clayey silt and downwards the soil gets stiffer, sand with clay intercalations. The ground water table appears at 2.5-3. m depth, depending on the seasonal oscillations.

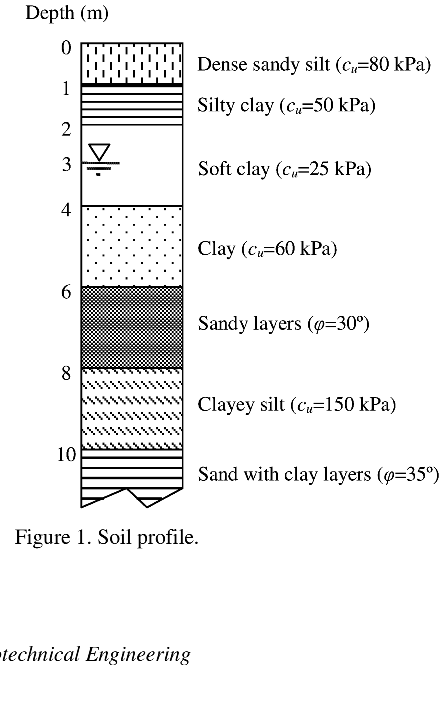

Figure 1. Soil profile.

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The soil characterization is described in detail by Castro (2008); a summary is presented here for completeness.

In the design stage, penetration tests were used to characterize the site. Dynamic penetration tests ("Borros" type) show the presence of a very soft layer around 2 and 4 m (c_u=20-50 kPa) and that the soil profile gets much stiffer below 7-8 m (c_u >100 kPa). Therefore, stone columns are assumed to be end-bearing at 9 m depth. The settlement-time behaviour of the different embankments was analysed based on several static penetrations tests ("piezocones" with dissipation tests, CPTU). Unfortunately, none of them was exactly in the instrumented area; the closest ones were 100 metres far. This fact together with some anomalies in the pore pressure measures, probably due to the unsaturation of the porous filter, made difficult an accurate identification of the interbedded sandy layers. Two representative dynamic and static penetration tests are shown in Figure 2 and Figure 3, respectively.

As part of the research project and after the column installation, 2 boreholes were drilled, one in the midpoint between columns (B1) and another close to the instrumented area but outside the column treated area (B2). Samples each 2 metres were taken and several lab tests were performed. The index properties, such as density, water content and Atterberg limits are shown in Table 1. The results of the oedometer, triaxial compression C-U and unconfined compression tests are summarized in Table 2.

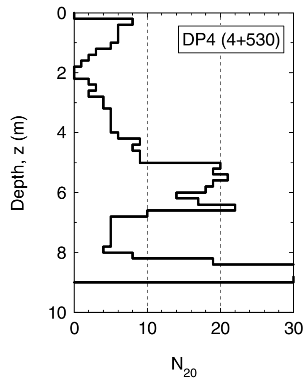

Figure 2. A typical dynamic penetration test in the area.

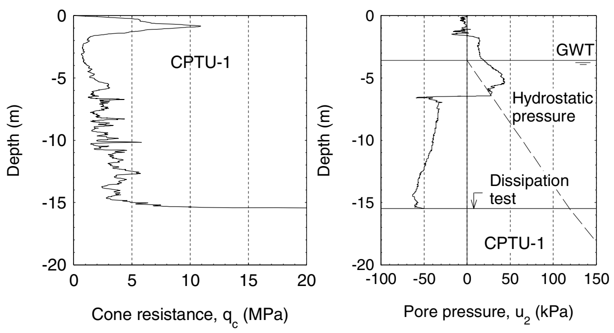

Figure 3. Static penetration test ("piezocone").

Table 1. Summary of soil index properties.

Borehole	*Depth(m)*	$\gamma_d(kN/m^3)$	*w(%)*	$w_L(\%)$	$w_P(\%)$	*LI*
Between columns (B1)	4-4.6	17.0	21	35.4	19.5	0.09
	8-8.6	17.2	22	34.2	16.8	0.30
	10.1-10.7	16.4	22	32.2	16.3	0.36
Outside columns (B2)	2-2.6	17.8	18	21.8	16.7	0.25
	4-4.6	17.0	22	33.0	18.3	0.25
	6-6.6	17.4	19	33.6	18.1	0.06
	8.6-9.2	17.3	21	26.6	8.4	0.69

Table 2. Oedometer, triaxial and unconfined compression tests.

Borehole	*Depth(m)*	*c'(kPa)*	$\phi(^o)$	$c_u(kPa)$	e_0	C_c	C_s
Between columns (B1)	4-4.6	25	26	29.5	0.60	0.130	0.020
	8-8.6	20	25	30.5	0.58	0.110	0.017
	10.1-10.7	15	28		0.62	0.109	0.012
Outside columns (B2)	2-2.6	70	32		0.52	0.086	0.008
	4-4.6	23	29	78.0	0.58	0.123	0.016
	6-6.6	31	26	166.5	0.55	0.118	0.012
	8.6-9.2	30	30	57.0	0.56	0.065	0.006

3 INSTRUMENTATION

The stone column behaviour was monitored using the following measuring devices:

a) 6 wire vibrating piezometers (Pz) installed in 3 boreholes, which were located in the midpoint between columns. In each borehole, 2 piezometers were set in the middle of a less permeable layer and below the groundwater table, namely at 4 and 7 m depth. The piezometers were used to control the consolidation process.
b) 6 total pressure cells (TPC). 3 of them were located on stone columns and the other 3 on the midpoint between columns. In this way, the vertical pressure distribution between soil and column could be analyzed.
c) An extensometer (Ext) with 3 levels of measurement (at 4.5, 9 and 16 m depth) in the midpoint between columns. The extensometer gave not only the ground settlement with a higher accuracy and reliability than the settlement gauges used by the constructor in the normal quality control of construction but also the compression of the different soil layers. The displacement sensors of the different depths were design for 250, 500 y 800 mm respectively and in accordance with the settlements predicted by the design project. The settlements below 16 m depth were assumed negligible.

The instrumentation was gathered in a hexagon cluster of columns, as it is shown in Figure 4. The readings of the instrumentation were scheduled according to the embankment elevation rate. The coordination between these two tasks was an important issue that not always could be successfully accomplished. Once the embankment reached its preloading height (12 m), 4 readings were done. Additionally, another reading was done for the final embankment height without preload (10 m).

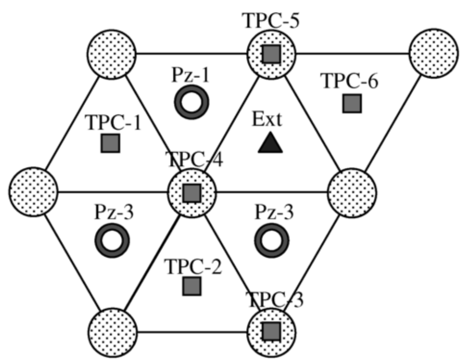

Figure 4. Instrumentation layout.

4 OBSERVED BEHAVIOUR

4.1 *Embankment elevation rate*

The embankment elevation rate is shown in Figure 5 and was imposed by the requirements of the construction site. The embankment construction started in September 2006, but due to the lack of material, the construction was really slow until January 2007. The preload was kept during nearly 2 months.

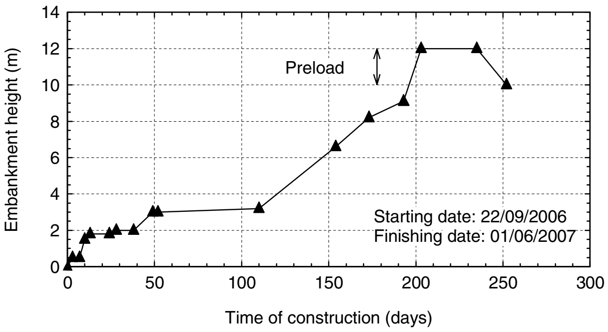

Figure 5. Embankment elevation rate.

4.2 *Pore pressures*

The piezometers measured very low excess pore water pressures during the embankment construction, as an example Figure 6 shows the values measured by the 2 piezometers located in the borehole 1 (Pz 1). This fast dissipation is caused by the stone columns that connect the different horizontal sandy layers and allow for a vertical drainage path.

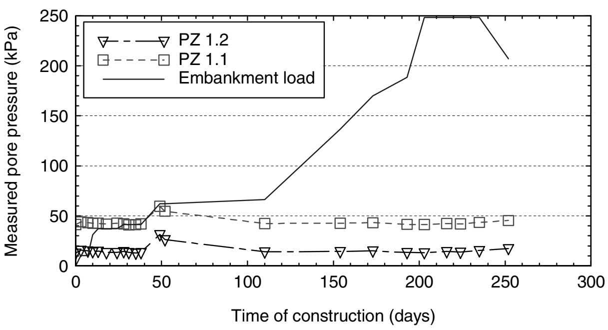

Figure 6. Measured pore pressures.

4.3 *Total vertical pressures*

The total vertical pressures measured by the total pressure cells (TPC) are shown in Figure 7. The TPC 3 was damaged at the beginning of the construction and therefore it is not shown in the graph. However, the value of the vertical pressure on the stone columns is very reliable because of the excellent agreement between the other two cells located on stone columns (TPC 4 and 5). Contrary, there is scatter in the value of the total vertical pressure on the soil.

The pattern of variation of the vertical pressures is reasonable similar to the embankment elevation. However, during the first 120 days the vertical pressures measured were negligible for an embankment height of 3 m. The gravel blanket on the improved ground was replaced in the instrumented area by a bed of sand to avoid any damage to the pressure cells. This could produce an arching of the embankment load around the instrumented zone and explain the low values measured.

The stress concentration factor, the relation between the vertical stress on the column and on the soil, is in a reasonable range between 3 and 6, depending on the pressure cell on the soil used to calculate the value.

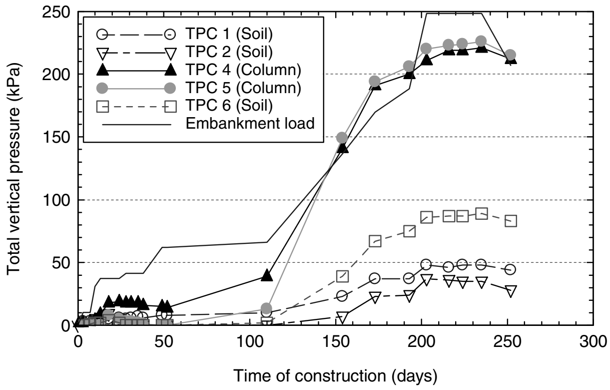

Figure 7. Total vertical pressures measured.

4.4 *Settlements*

The settlements at different depths measured by the extensometer are plotted and compared with the embankment height in Figure 8. Two remarkable results from these measures are that the settlement at 9 m depth is quite important and that the deformation of the soil layer between 0 and 4.5 m depth is negligible until 150 days of construction.

The small deformation of the upper layer at the beginning of the construction process is due to its overconsolidation pressure that it is not exceed in the first 150 days of construction. The arching effect around the instrumented area, commented above, may also explain this small deformation. On the other hand, stone columns reduce the deformation of the treated ground but transfer an important load below their length (9 m), making relatively important the deformation of the lower part (9-16 m).

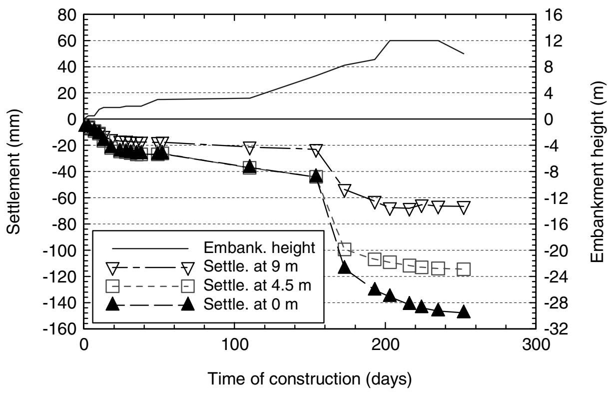

Figure 8. Settlements measured by the extensometer.

5 FINITE ELEMENT MODEL

A back calculation of the instrumented problem was done using a three dimensional finite element model. A 3D analysis was chosen to avoid the conversion of the columns in gravel trenches for a plane strain analysis. This conversion is inevitably attached to some simplifying assumptions. However, a manageable 3D model was achieved studying only a row of columns (Figure 9).

The finite element code was Plaxis 3D Foundation v1.6, using the Hardening Soil Model (Brinkgreve and Broere, 2006) to model all the materials. The chosen model parameters for the different soils are summarized in Table 3. Some parameters were adjusted for a better agreement with the measurements and their new values are between parentheses. The permeability of the clay was drastically increased to account for the interbedded sandy layers. Besides, the reference oedometric modulus of the clay was also increased because the initial one was obtained from the oedometer tests and it seems a bit conservative. A less conservative value was worked out from correlations with the penetration tests.

A comparison between the measured settlements and the values computed with the finite element model is shown in Figure 10. A reasonable agreement of the settlements at different depths is achieved.

The value of the vertical stresses on soil and columns is reasonable well reproduced but for the first 120 days. Afterwards, the measured stress increases are nearly the same than in the model (Figure 11). The stress concentration factor in the model decreases from 5 to 3 during the embankment elevation. Nevertheless, the discharge of vertical stress on the columns is much higher in the model than in reality after removing the preload.

6 CONCLUSIONS

The instrumentation of an embankment founded on stone columns shows the settlement reduction, the fast pore pressure dissipation and the stress concentration on the columns that occur with this ground improvement technique.

The stress concentration factor is in the range 3-6 and gives an idea of the applied load that is released from the soft soil. However, this load is transferred again to the soil under the column length, and a relatively important settlement of the not treated soil took place.

A back calculation of the problem by means of a 3D finite element model agrees reasonably well with the measured values.

Table 3. Summary of parameters used in the FE calculation.

	Embank.	*Blanket*	*Columns*	*Softer clay*	*Stiffer clay*
γ(kN/m^3)	20	20	20	20	20
γ_{sat}(kN/m^3)	20	20	22	20.5	20.5
p'_{ref}(kPa)	100	100	100	100	100
m	0.5	0	0.3	0.9	0.9 (1)
E^{ref}_{oed}(MPa)	25	100	75	4 (7.5)	4.5 (7.5)
E^{ref}_{50}(MPa)	25	100	75	7 (7.5)	7 (7.5)
E^{ref}_{ur}(MPa)	100	400	300	20 (22)	20 (22)
ν_{ur}	0.2	0.2	0.2	0.2	0.2
c'(kPa)	10	0.1	0.1	20	20
ϕ'(°)	30	45	40	25	29
ψ(°)	0	15	10	0	0
k_v(m/day)	-	-	-	10^{-5} (0.01)	10^{-5} (0.01)
k_h(m/day)	-	-	-	10^{-4} (0.1)	10^{-4} (0.1)
σ'_p(kPa)	-	-	-	170	170

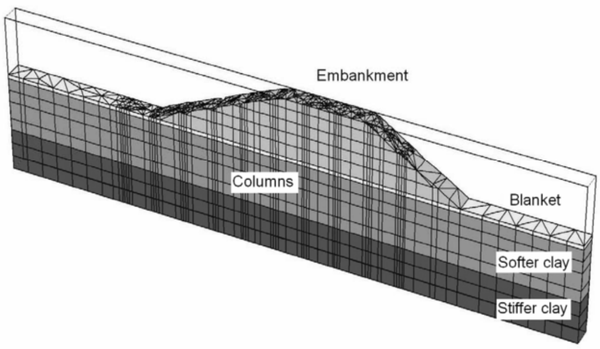

Figure 9. Overview of the 3D finite element model.

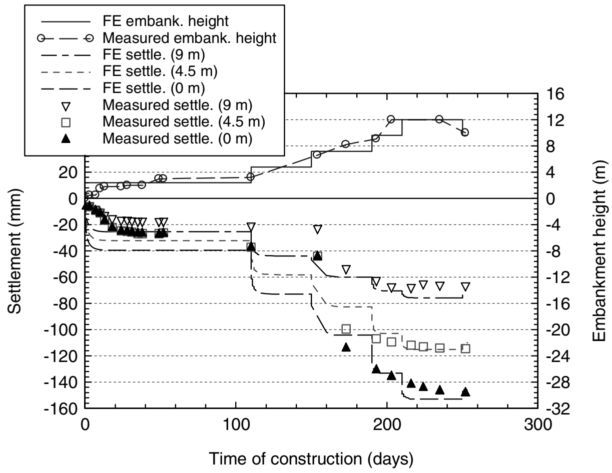

Figure 10. Settlement comparison.

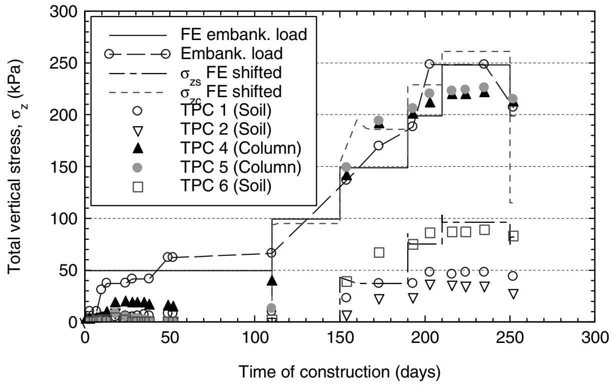

Figure 11. Total vertical stress comparison.

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REFERENCES

Brinkgreve, R.B.J. and Broere, W. 2006. Plaxis 3D Foundation Manual. A.A. Balkema Publishers.

Castro, J. 2008. Análisis teórico de la consolidación y deformación alrededor de columnas de grava. Doctor's Dissert. Univ. Cantabria.

Castro, J. and Sagaseta, C. 2008. Consolidation around stone columns. Influence of column deformation. International Journal for Numerical and Analytical Methods in Geomechanics. DOI: 10.1002/nag.745

Castro, J. and Sagaseta, C. Pore pressures during stone column installation (in preparation).

Cimentada, A., Sagaseta, C., Cañizal, J. and Da Costa, A. A laboratory testing system for consolidation around granular columns. Geotechnical Testing Journal (submitted).