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Importance of field test and FEM analysis in embankment construction on soft ground

Carraro, S. & Sartorel, E.
Sogen S.r.l., Engineering Company, Padova, Italy

Fantini, P. & Ferraro, V.
Huesker S.r.l, Trieste, Italy

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ABSTRACT: The construction of embankment on soft ground needs a well-known characterization of soil parameters in order to define an accurate underground geotechnical model. Furthermore, the reconstruction of "tensional history" of concerned soil, allows to obtain reliable results, during the executive phase of design. The following case deals an application of geosynthetic reinforced soil systems on vertical sand columns drainage. The study of the physical behaviour of embankment has been useful to obtain an exact definition of the resistance of soil reinforcement geotextile, to verify the diameter and the disposition of vertical drains, the consolidation time, and to control horizontal displacements and settlements, with regard to construction plan time. Then a 2D numerical analysis has been implemented with finite element method (FEM), to calibrate design parameters of soil, taking into account the soil stress path ("tensional history").

1 INTRODUCTION

The general project of perimetral hydraulic dams in Cà Rossa Landfill (Chioggia, Venezia – ITALY) has required a field test to verify the behaviour of a new embankment on soft ground (clay and peat).

Construction on soft ground needs to solve stability and deformation problems. If these problems are not solved, it is necessary to improve soil characteristics. In the following case of embankment on soft ground, an application of geosynthetic reinforced soil systems on vertical sand columns drainage, during the embankment design, has permitted to optimized the design parameters of soil foundation.

The construction time scheduled by plan, with the respect of stability, and the necessity to accelerate the time of consolidation, has required the use of vibrated sand columns, ϕ 40 cm, especially with drainage function. Above these columns there is a sand layer, confined in a geotextile with specific mechanic characteristics.

The test field of the embankment is a small-scale construction with a monitoring system, that control its behaviour.

In a second stage, experimental results were reproduced with a numerical code finite element program PLAXIS (Ver.8.02), in order to make a back-analysis and to validate the soil parameters, extending results, and the FEM analyses, to all the plan.

2 GROUND IMPROVEMENT METHOD

In the presence of soft soil, especially clay and peat, vibrated columns of sand and gravel are a typical method of ground consolidation that improves mechanic characteristics of strenghtness and deformability; also the consolidation time becomes comparable to the construction time.

We have one of the calculation method from Van Impe e De Beer (1983). Columns are considered as a continuous diaphragm wall, with equivalent area in plant.

Analytical solutions were obtained by authors according to equilibrium and congruence conditions of soil-columns system, and with a presence of a firm subsoil for the columns. The friction between soil and columns and self-weight of each columns has not been taken into account. Columns have elasto-plastic properties, instead soil has elastic behaviour. The solution is given in a graphic form according to the following parameters

- $m = F_1/F_{tot}$
- F_{tot} = total load on area $a \times b$
- F_1 = single column load
- $\kappa = s_v/s_{v,0}$
- $s_{v,0}$ = original soil settlement
- s_v = settlement of soil-columns system.

The geometry of the problem, in particular in area "$a \times b$", is the most important factor ("dominant factor") in the analytical solution; soil and columns deformability parameters have little influence.

For example, in our case, with very poor deformability parameters (Young Modulus E = 1 Mpa), and a spatial axes from 2,5 × 2,5 m to 3 × 3 m, and ϕ columns of 40 cm, settlements reduction is about 5-10%.

The best performance is the consolidation time reduction. Time consolidation problem, with the presence of vertical drains, was solved with radial consolidation theory (Barron 1948), according to monodimensional consolidation hypothesis of Terzaghi. The solution of the problem, expressed in form of consolidation degree, assumed an expression like that

$$U_h = 1 - e^{\frac{8T_h}{F(n)}}$$

where
- $T_h = c_h t/D^2$
- $F(n) = ((n^2)/(n^2 - 1)) \times \ln(n) - (3n^2 - 1)/4n^2$
- $n = D/d$

Wide is the range of time consolidation results, for the variability of c_h from in situ test (piezocone). By simple calculation the previous consolidation time (degree of 90% of consolidation) is included from 0,5 to 2 months.

3 EXPERIMENTAL PROGRAM (FIELD TRIAL)

The results from field test embankment are represented to follow.

Dimensions of embankment in plant are 31,4 × 11,4 m with height of 3 m. In a part of plant, for a development of 12,50 m, the spacing axial of vertical columns (drains) is 2,5 × 2,5 m; in the remaining area the spacing axial of vertical columns (drains) is 3,0 × 3,0 m. The embankment was monitored with n.2 vertical inclinometer, n.5 fixed extensometer (superficial settlement platform), n.1 magnet extensometer (multibase extensometer), n.2 vibrating wire piezometer (see figure below).

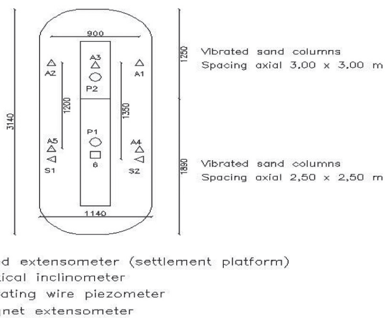

Figure 1. Embankment plant with geotechnical instrumentation.

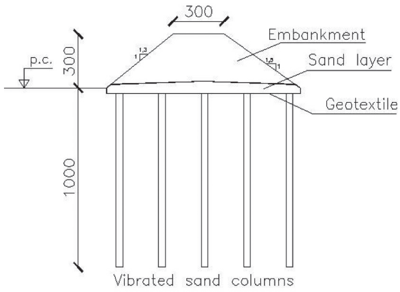

Figure 2. Field test section.

The vertical stratigraphy interested by the presence of the embankment is rappresented in Table 1.

Table 1. Vertical stratigraphy.

Material (M) (description)	Depth (m)
Superficial layer (SL)	0,0–1,6
Peaty sandy silt (PSS)	1,6–4,2
Sandy silt 1 (SS1)	4,2–5,2
Peaty silty clay (PSC)	5,2–7,8
Sandy silt 2 (SS2)	7,8–10,0
Silty clay (SC)	10,0–11,3
Overconsolidated sandy silt (OSS)	11,3–15,0

Now the experimental results are represented. Points A2 and A5 are near existing landfill (like vertical S1) and the value of their settlements are less then points A1 and A4 ones. In fact vertical A2 and A5 reflect the influence of the load of the landfill, and the follow consolidation caused by the same landfill.

This effect is clear also in the results of the vertical inclinometer. Through landfill, the consolidated soil "obstructs" the horizontal displacement in the soft (peat) layer.

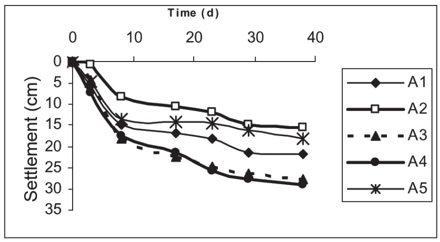

Figure 3. Settlements Vs Time.

4 FEM ANALYSIS

To calibrate design parameters of soil, reproducing the embankment field test, numerical analyses was carried out with a well-known commercial codes, PLAXIS 2D (2004).

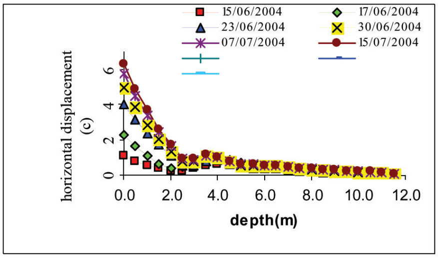

Figure 4. Horizontal displacements inclinometer S1.

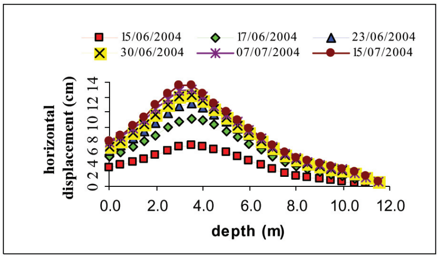

Figure 5. Horizontal displacements inclinometer S2.

Plaxis is a finite element computer program suitable fot the analysis of deformation problems in soli and rock. The embankment with drains and geotextile was schematized with the mesh of Figure 6.

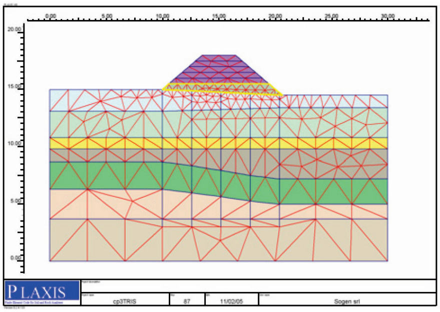

Figure 6. FEM mesh of the embankment.

According to the back-analysis, the soil parameters have been defined in order to reproduce experimental effects by numerical code.

The soil model in numerical analyses are two, Mohr-Coulomb Model (MC), and Hardening Soil Model (HSM). HSM is an elastoplastic type of hyperbolic model, formulated in the framework of friction hardening plasticity. Moreover, the model involves compression hardening to simulate irreversible compaction of soil under primary compression. This second-order model can be used to simulate the behaviour of sands and gravel as well as softer types of soil such as clays and silts.

Table 2. Material parameters.

M	YC	γs	E_{50}	E_{oed}	E_{ur}	m	ν	c′	ϕ'
SL	MC	19	2				0,35	10	24
PSS	HSM	15	1	1	3	0,5		10	16
SS1	HSM	19	4,5	4,5	13,5	0,5		8	25
PSC	HSM	17	1,6	1,6	4,8	0,5		8	18
SS2	HSM	19	5,5	5,5	16,5	0,5		10	27
SC	HSM	19	3	3	9	0,5		15	25
OSS	HSM	19	10	10	30	0,5		100	27

E50/Eoed/Eur in Mpa, c′ in kPa, γs in kN/m^3

The properties of the embankment material and the underlying layer are the following:

Table 3. Material parameters.

M	YC	γs	E_{50}	ν	c′	ϕ'
Embankment	MC	20	6	0,25	50	26
Sand layer	MC	20	10	0,2	10	30

For the basic geotextile: elastic behaviour, stiffness 3333 kN/m.

Since the only drainage capability was taken in account, the sand columns were simulated uniquely as a vertical drains, thus omitting the vertical settlement controlling function (without mass and stiffness property).

The embankment was constructed in three stages: one day for sand columns and the layer above; five days for the below part of embankment (inferior part) and two days for the above one.

Afterwards the numerical code simulated the consolidation time (settlement development) until a degree of 90%.

In order to calculate safety factor (FS), during the construction phases and at the end of consolidation time, also safety analyses was executed (Phi-c reduction analysis)

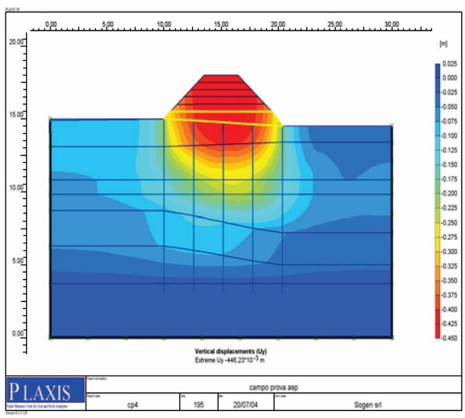

Figure 7. Settlements after consolidation.

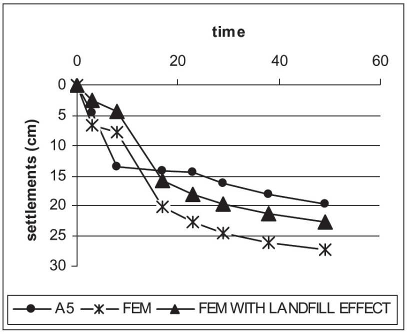

Figure 8. Settlements of A5.

5 CONCLUSIONS

The general project of perimetral hydraulic dams in Cà Rossa Landfill (Chioggia, Venezia – ITALY) has required a field test to verify the behaviour of a new embankment on soft ground (clay and peat).

Numerical analyses, with calibrated soil parameters (properties), have confirmed the results of embankment field test:

- after 40 days from the end of the embankment construction, the degree of consolidation is about 80-90%;
- also by numerical analyses it is possible to take into account the effects of landfill loads, consolidating significant volume of soil (influenced volume by consolidation effects);
- There are non substantial differences in in the consolidation time between a choice of sand columns with spacing axial of 2,5 × 2,5 m rather than 3,0 × 3,0 m;
- According to a preliminary design of ground geotextile, the choice of a T_{max} = 400 kN/m (Nominal Tensile Strength) is correct, because it always allows for a (safety factor) FS > 1,3 during embankment construction and during consolidation time.

After that it was possible to design all embankments in that area according to the information acquired by back-analysis.

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