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GEOTECHNICS

SOIL IMPROVEMENT CHALLENGES ON ALLUVIAL ZONES

28-29 January 2019
Vila Franca de Xira, Portugal

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of extended abstracts

Edited by

Alexandre Pinto, António A. Cristóvão, António Alberto Correia, António Gomes Correia, Baldomiro Xavier, Eduardo Fortunato, Isabel Pinto, Jorge Barros, José Luis Antunes, José Mateus de Brito, José Neves, Madalena Barroso, Pedro Guedes Melo, Rui Tomásio, Vieira Simões

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PREFACE

Significant advances in soil improvement and reinforcement methods are expected in the near future to meet the challenges that the increasingly demanding market poses, in particular as regards both efficiency and speed of control of costs. Regarding the importance of ensuring the best practices in the indispensable soil improvement works, which will certainly be included in the investments planned in the country in transport infrastructures, this seminar intends to present the most recent national and international experience of geotechnical solutions for soil improvement in the field of road, rail, airport and maritime-port infrastructures, focusing on the experience acquired in the execution and observation of work cases in conjunction with recent developments of innovative techniques and with greater sustainability.

Lisbon, Portugal

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Audience of delegates during technical sessions.

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MODULE I

The problem of soil improvement. The Portuguese experience

SOFT SILTY-CLAYEY SOILS FROM PORTUGAL PARAMETERIZATION FOR GEOTECHNICAL DESIGN

Elisabete Esteves

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Figure 1 shows the correspondence between the zones of highest population density and the occurrence of soft soil deposits in Portugal. Soft silty-clayey soils are the foundation materials supporting infrastructure, such as roads, ports, logistics platforms, railways, bridges and tunnels.

This work was developed to establish an up-to-date framework of properties regarding physical characteristics and mechanical behavior of these soils, based on a thorough analysis of 587 geotechnical data from 120 important construction works built in Portugal, complemented by laboratory and *in situ* tests carried out on an experimental site (Esteves, 2014).

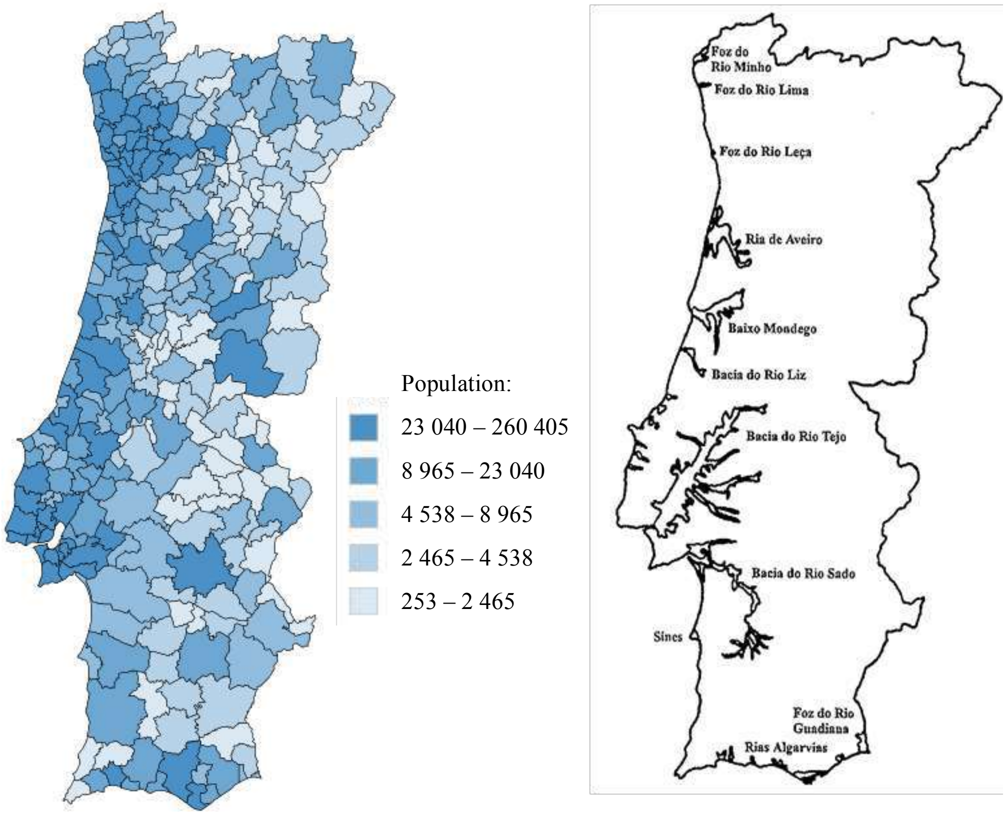

Figure 1. Comparison between population density and location of soft soil deposits in Continental Portugal: a) population density (INE-CENSOS2011); b) main soft soil deposits.

Physical and identification parameters

Figure 2 illustrates the granulometric curve of the soft silty-clayed soils from Portugal. The percentage of sand found in these formations may reach up to 50% and the remaining percentage is divided into clay and silt which may ascend to 90% of the soil.

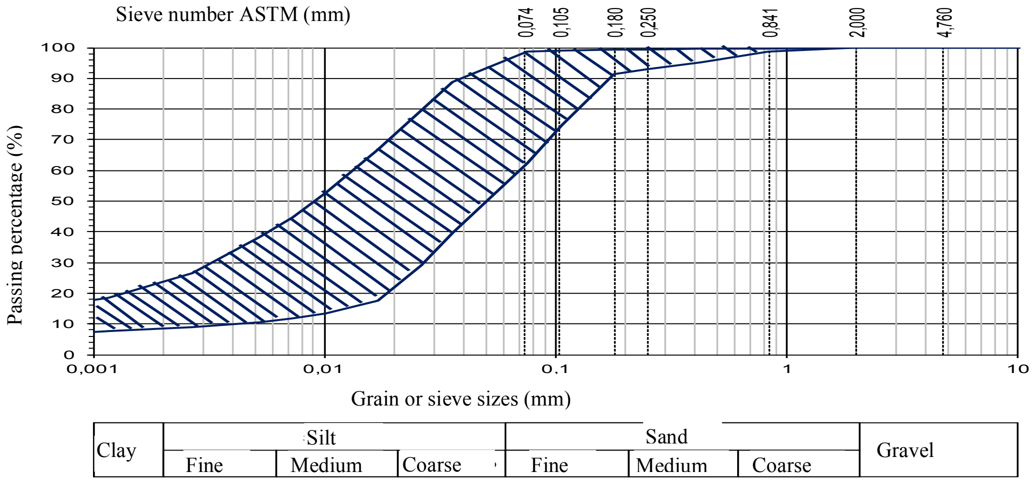

Figure 2. Granulometric curve of the soft silty-clayed soils from Portugal.

In these soils, the organic matter has a significant influence on the physical and plasticity characteristics that may reach up to 13%, being its variability very pronounced with depth, showing higher values near the surface.

Regarding the Atterberg limits, the water content and consistency indexes, Table 1 summarizes the range of variation, average value and standard deviation for this type of soils. Since it is verified that the organic matter can significantly change the Atterberg limits results it was made a separation in the preparation mode of the sample (natural state and air dry).

Table 1– Descriptive statistics for w_L, w_P, I_P, w e I_c of soft silty-clayed soils from Portugal.

Index	Sample preparation	Minimum	Maximum	Average	Standard deviation	Nº tests
w_L (%)	Natural state	70	110	82	12	9
	Air dry	27	98	59	13	361
w_P (%)	Natural state	33	45	38	5	9
	Air dry	13	69	31	7	360
I_P (%)	Natural state	33	66	44	10	9
	Air dry	6	63	28	10	360
w (%)		6	127	64	17	461
I_c	Natural state	-0,13	0,29	0,14	0,17	5
	Air dry	-3,05	0,50	-0,25	0,59	340

Table 2 summarizes the physical values found for alluvial silt-soft clay from Portugal. It should be noted that the variability of the water content, specific weight and void ratio is considerable.

Table 2– Proposed Physical indexes of soft silty-clayed soils from Portugal.

w (%)	γ (kN/m^3)	G_s	e
64 (± 17) n = 461	15,9 (± 1,4) n = 497	2,69 (± 0,08) n = 458	1,77 (± 0,56) n = 509

Average (± deviation)

n – number of tests

Overconsolidation ratio

Figure 3 shows the evolution in depth of the overconsolidation ratio. It should be noted that soft soil deposits are characterized by an overconsolidated crust that, in some cases, can reach 8.5 m. This desiccated crust exhibits a degree of overconsolidation decreasing in depth. For depths outside the influence zone of this crust this deposit is normally consolidated or very slightly overconsolidated.

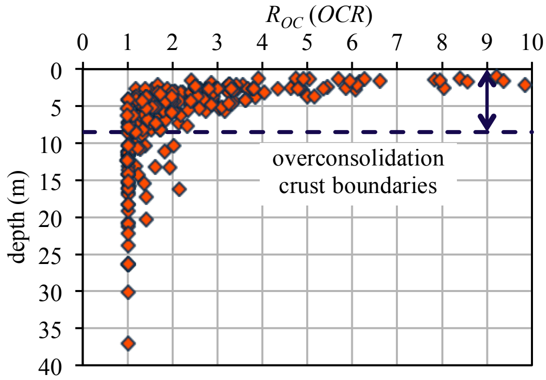

Figure 3. Variation of the overconsolidation ratio with depth.

Compressibility and consolidation parameters

The compressibility and consolidation parameters of soft silty-clayed soils from Portugal are summarized in Table 3.

Table 3– Average values of compressibility and consolidation parameters of soft silty-clayed soils from Portugal.

C_c	$C_c/(1+e)$	C_r	C_r / C_c
0,58 (± 0,24) n = 278	0,22 (± 0,06) n = 241	0,09 (± 0,04) n = 267	0,14 (± 0,04) n = 255

Average (± deviation)

n – number of tests

With a linear regression analysis it was concluded that the compressibility index can be estimated with high confidence through the variables water content (w) and void ratio (e). To determinate the compressibility index the following expressions are suggested:

$$C_c = 0{,}013 \times (w - 10{,}6) \qquad (1)$$
$$C_c/(1+e) = 0{,}002 \times (w + 42{,}0) \qquad (2)$$

with adjusted coefficients of determination of 0.868 and 0.628, respectively.

For the recompressibility index determination, the following expression is referred:

$$C_r = 0{,}14 \times C_c \qquad (3)$$

with adjusted coefficients of determination of 0,671.

The values of the secondary consolidation coefficient are not presented in Table 3 because the results of the framework oedometric tests came only from 24-hour stage loading tests, which were found to be inadequate to determine that parameter. Indeed, as Figure 4 shows, in the very long loading stages oedometric tests was consistently observed that the compression curves clearly show three linear response zones in the semi-logarithmic time scale, with different slopes, giving different values of C_α for different charging times. In the first period of time, after the primary consolidation, the slope of e-log(t) branch is relatively low and subsists more than 24 h. The determinations of C_α up to 24 h, provided by the classic tests, will tend to underestimate that parameter. In a second phase an acceleration of the creep rate is observed. Finally, in a third phase, that rate seems to be practically null (Esteves, 2014).

Regarding the variation of C_α with depth, this parameter assumes a constant value in the first period of time, when the slope of the e-log(t) is low, and, in contrast, presents higher values for smaller depths in the second period of time (which refers to the steeper slope of the e-log(t)). This is probably related to the organic matter content which also explains why the C_α determinations up to 24 h given by the classical tests do not show a strong relationship between the secondary consolidation coefficient and the organic matter content that would reasonably be expected.

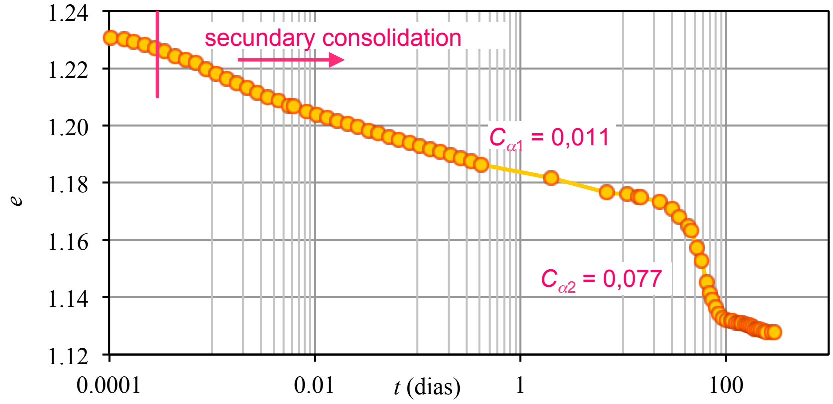

Figure 4. Obtained results for secondary consolidation coefficient on oedometric test.

Effective stress values and undrained resistance

Figure 5 shows the failure envelope obtained from triaxial compression tests. The undrained shear strength (c_u) is highly influenced by the stress consolidation adopted in the test. The variation of c_u with depth was established using the theoretical expression that relates c_u with ϕ', σ'_{v0}, K_0 and A_f. Table 4 presents a summary of the expressions that provide the lower and upper boundaries for c_u.

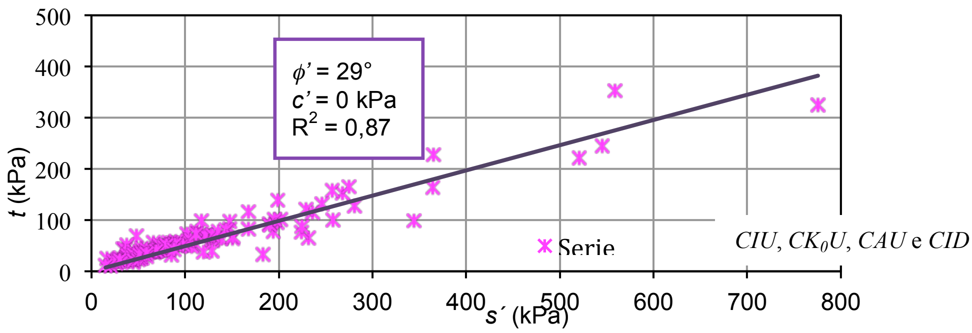

Figure 5. Failure envelope obtained from triaxial compression tests.

Table 4– Mean values of the compressibility and consolidation parameters.

Test	$c_{u,máx.}$ (kPa)	$c_{u,mín.}$ (kPa)
CIU	$c_{u,máx.} = 0{,}86\sigma'_{v0} + 0{,}05$	$c_{u,mín.} = 0{,}47\sigma'_{v0} + 0{,}03$
CAU, CK$_0$U	$c_{u,máx.} = 0{,}65\sigma'_{v0} + 0{,}04$	$c_{u,mín.} = 0{,}50\sigma'_{v0} + 0{,}03$

Below the desiccated crust the ratio c_u/σ'_{vo} varies between 0.20 and 0.50. These values are in agreement with the results published in the bibliography for this type of soils (Ladd *et al.*, 1977, Jamiolkowski *et al.*, 1985, Matos Fernandes, 2006).

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INNOVATIVE SOLUTIONS TO MITIGATE EARTHQUAKE INDUCED SOIL LIQUEFACTION DAMAGES (EILD)

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Following the works undergoing in H2020 Liquefact project, this presentation will focus on the advantages of less invasive EILD mitigation solutions, like dessaturating by lowering the water level by drainage systems or air injection, bio-system for generation of air bubbles and cementation as well as colloidal silica grouting in reducing that susceptibility in sands and silty sands, as an alternative to solutions like jet grouting or compaction grouting.

Ground improvement is usually associated to methods that introduce "*materials or energy to soils to affect a change in performance of the ground such that it performs more reliably and can be incorporated within the design process*". The use of ground improvement on projects has increased worldwide as its design and application becomes more common. However, such techniques are only to be used as remedial measures when there is an unexpected problem on site. Additionally, environmental issues may arise, resulting from the use of cements or chemicals associated to traditional ground improvement techniques via grouting. For these reason, new technologies are being developed and applied increasingly. These can rely on the decrease in water content, a key factor for the stability of sensitive soils under diverse loading actions- Alternatively, coupled interaction between the biological and chemical components of geomaterials which has been proved to create passively bonds between the solid particles, are being applied in small to large-scale.

In this area the improvement of natural granular liquefiable soils when subjected to earthquake actions by recurring to these innovative techniques have the advantage of, with lower costs and reducing the environmental impact when compared with traditional grouting, to mitigated the loss damages critical infrastructures, like embankments in transportation lines (roads and railways, or abutments for bridges' accesses, leaves in rivers and canals) or ports retaining structures and protection dikes.

New bio-mediated ground improvement techniques, low pressure grouting of nanosilicate and the techniques to decreasing the degree of saturation, by either lowering the water level with sub-surface horizontal drainage or by injection of air, inducing suction and increasing the pore fluid compressibility, are being successfully established.

The effectiveness of colloidal silica grouting in reducing the liquefaction potential of natural sands and silty sands is due to its low viscosity, wide range of gel times, nontoxicity, and low cost, which tends to stabilize effectively these granular materials. Colloidal silica treated and untreated sand specimens have revealed different pore pressure response and deformation behavior under cyclic loading in simple shear tests. The results indicate that, for a given initial relative density and initial effective vertical stress, liquefiable silty sand specimens stabilized with colloidal silica grout generally exhibit significant gain in liquefaction resistance compared with untreated specimens. It was also found that the colloidal silica grout reduces considerably the rates of pore pressure generation and shear strain of the silty sand specimens subjected to cyclic loading. Recent studies have been conducted by Hamderi and Gallagher (2015) where bypassive site stabilization was develop for in situ mitigation of the risk of liquefaction without surface disruption. It involves the injection of stabilizing materials into liqueable saturated sand, injecting a dilute colloidal silica stabilizer into liquefiable sand specimens. Different injection rates were used to investigate the optimal rate of grout delivery. In tests with low injection rates, the delivery performance was low due to sinking, while at higher injection rates, sinking was less noticeable. After the treatment, the degree of grout penetration was evaluated by excavating the model. The results of the strength testing demonstrated that as little as 1% by weight of the colloidal silica provides a significant improvement in strength after a month of curing. The increase in performance is due to the formation of aggregates as the particles rendered unstable by the accelerator or changes in pH. As these aggregates form and grow the mobility of the individual particles and aggregates decrease leading to the increase in the viscosity. The formation of such aggregates induces an increase of the viscosity where silica solution is priorily introduced as an accelerator where only single particles are present, In Fig.1 some ongoing works in University of Cassino and Southern Lazio illustrate well this evolution in the formation of small aggregates shortly after introduction of accelerator an subsquent formation of a gel network as the PoG is reached and the viscosity is increasing (Fig. 2).

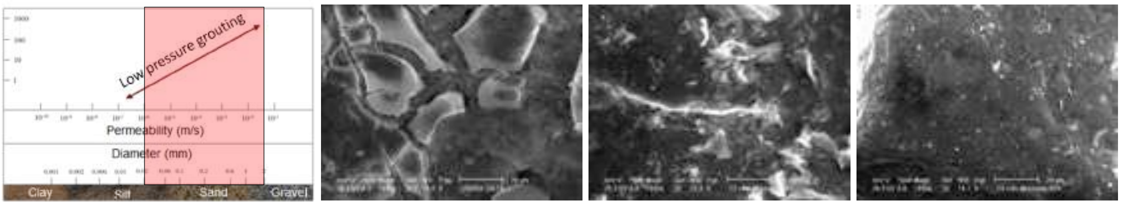

Figure 1. Colloidal silica low viscosity; SEM images of the material treated with decreasing concentration of silica

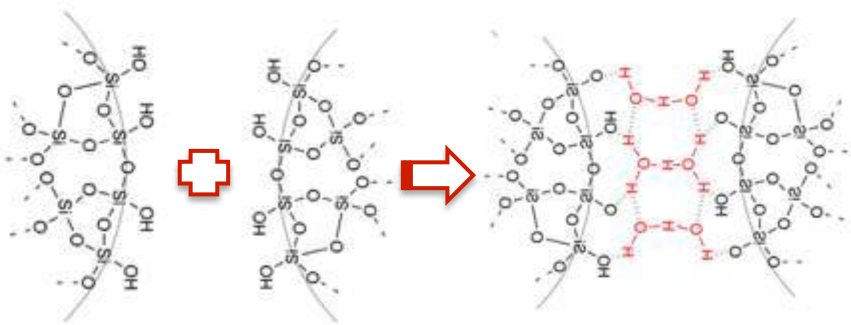

Figure 2. Viscosity and structure development during gelling

Soil liquefaction can cause serious damage to engineering structures and, as a consequence, many types of mitigation techniques (densification, drainage, addition of fines, etc.) have been developed. Among these, the 'induced partial saturation' (IPS) is considered one of the most innovative and promising ones. IPS increases the resistance to liquefaction of the liquefaction-susceptible soil by introducing a certain amount of air/gas into the voids. Laboratory results have shown that even a small reduction in the degree of saturation of an initially saturated sand can have a relevant effect. In fact, the presence of air in the voids increases the resistance against liquefaction in two ways: the first mechanism is connected to the very low volumetric stiffness of gases, because of which during undrained loading there is a volumetric reduction of the gas phase and therefore reduced excess pore pressures. This mechanism is the ruling one for high degrees of saturation (i.e. dispersed air bubbles).

The second mechanism is due to the matric suction of unsaturated soils, which increases the stiffness and strength of soils. This becomes relevant when the degree of saturation is low enough to have a continuous air phase. It is known that there is a unique relationship between the normalised liquefaction resistance (CRR) and the potential volumetric strain (i.e. the volumetric strain at which liquefaction is triggered): the liquefaction resistance of a partially saturated soil can be quantified from the one of the saturated soil by estimating the potential volumetric strain caused by the gas phase. Mele et al. (2018) have presented laboratory experimental results that confirm that desaturation increases liquefaction resistance and defend that the change in behaviour of the soil after liquefaction is due to an apparent viscosity (Fig.s 3). The liquefaction resistance ratio (LRR) is defined as the ratio between the CRR of the unsaturated soil (CRR_{unsat}) and the CRR of the saturated soil (CRR_{sat}) at the same relative density and at the same number of cycles ($LRR = CRR_{unsat}/CRR_{sat}$). This parameter summarises the positive effect of desaturation on liquefaction resistance. A link exists between the values of the apparent viscosity defining the different behaviours and the coefficient of uniformity of the soils: the more graded the material. Therefore, the higher the energy dissipation because of the particles' relative movements, the higher the values of the apparent viscosity.

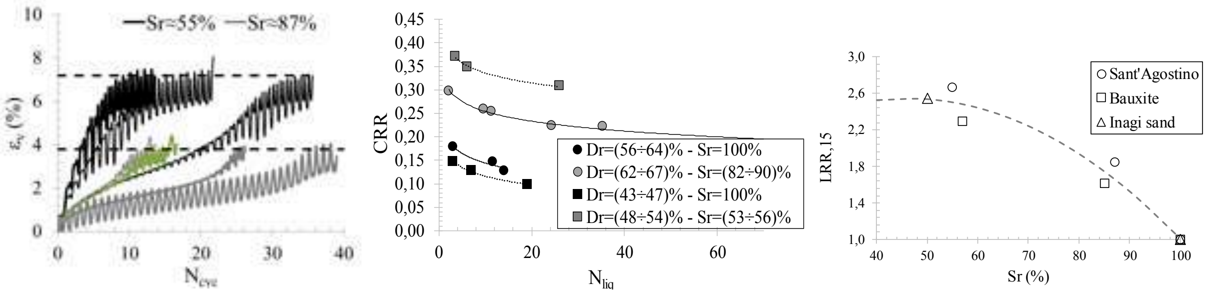

Figure 3. Volumetric strain vs number of cycles a sand with Sr= 55% and Sr= 87% (adapted, Mele et al. 2018)

It was proved that theoretical interpretation has conclused that this is consistent with the assumption that the cyclic resistance curve of unsaturated soils is a unique function of the volumetric component of the total specific energy spent to reach liquefaction. From a modelling point of view, however, this is extremely promising as it may lead to new and more general approaches to define the cyclic resistance of unsaturated sandy soils (Fig.4)

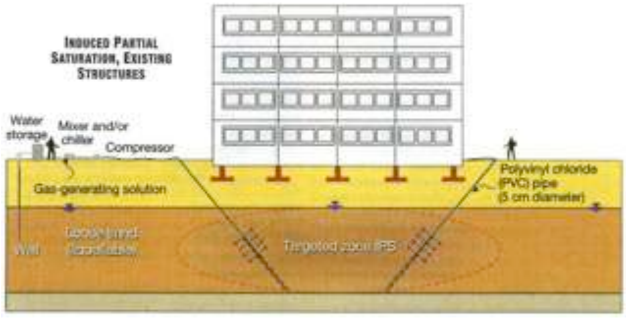

Figure 4. Inducing partial saturation in existing structures

Under activities of Liquefact H2020 project (www.liquefact.com) two of three different mitigation technologies (drains – vertical and horizontal -, Induced Partial Saturation – IPS - and compaction grout) were tested in Small Scale using a Geotechnical Centrifuge facility in ISMGEO, Bergamo, Italy. Horizontal and Vertical Drains and ai injection (IPS) were implemented (see Figure 5). The horizontal drains and IPS were the most interesting in terms of innovation and had excellent results. These were the first choice for the pilot site real scale tests.

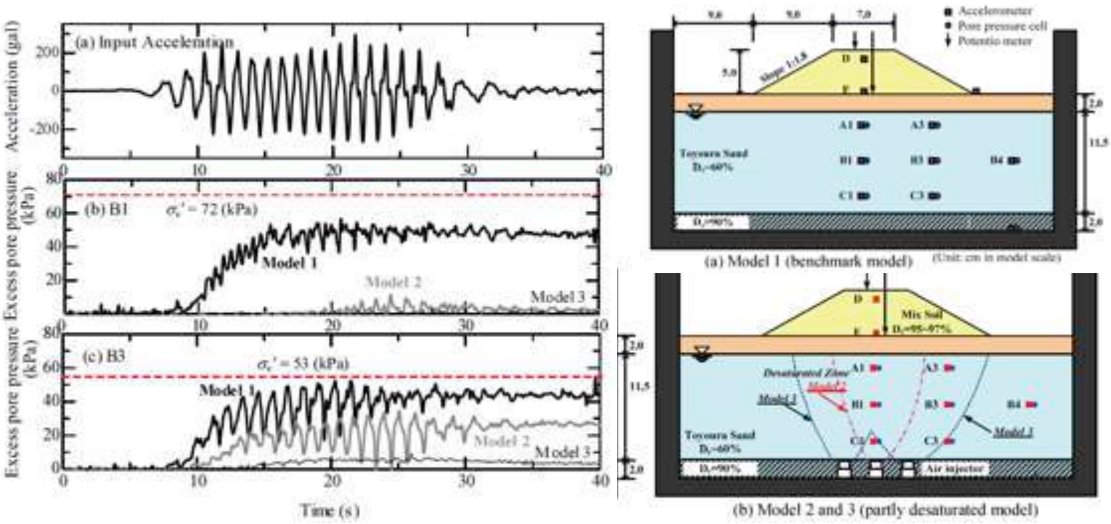

Figure 5 Centrifuge tests on ISMGEO for physical modelling of mitigation technologies (www.liquefact.com),

On February 2019 Liquefact H2020 project (www.liquefact.com), under the coordination of two of the partners (TREVI and UNINA) and the collaboration of others (ISMGEO, UNIPV and UPORTO) the Field Trial Pieve di Cento, in Emilia-Romagna, North of Italy, where EARTHQUAKE, was settled and prepared for these studies.
In situ tests, with relevance to CPTu and SCPT tests and Vs surveys, interpreted by current unified approaches, complemented with other important data coming from advanced laboratory test (lilke RC and CSST) with undisturbed sands specimens (taken with Gel-Push Sampler) have allowed a thorough characterization of the soil deposit. Relevant profiles were defined with the stratigraphy supported by deep downhole investigations. In Figure 6 CPTu are interpreted in equivalent soil profiles (ESP), a new approach for definition classes of

Liquefaction vulnerability (www.liquefact.com), taking into account the presence of clayey crust (its thickness), a liquefiable sand (its thickness and its cyclic resistance) (Fig. 6a). After a thorough characterization of the soils dominating the sub-surface liquefiable soils a geotechnical 3 layers model was considered for the numerical FLAC(R) analyses. Most effort was devoted to define the mechanical properties of the liquefiable sand layer (Fig. 6b).

The prediction of the behavior of the natural soil, for one side and with the induced desaturation conditions, for another, were studied. The parameters that were used, were derived from the previously described tests (in situ surveys, advanced laboratory tests with distinct degrees of saturation) and calibrated by the centrifuge physical models results. The techniques for induction of partial saturation (IPS) comprised Horizontal Directional Drilling with 3” steel rods, pulled-back with a revolutionary porous HDPE well screen designed to minimize flow resistance by providing great porosity compared to conventional well screens (Fig. 7 - www.liquefcat.eu). Shaking tests with Mega-Shaker will be performed in three (3) main areas: one on the virgin ground, one in the area treated with HD, the last one in the area treated with IPS.

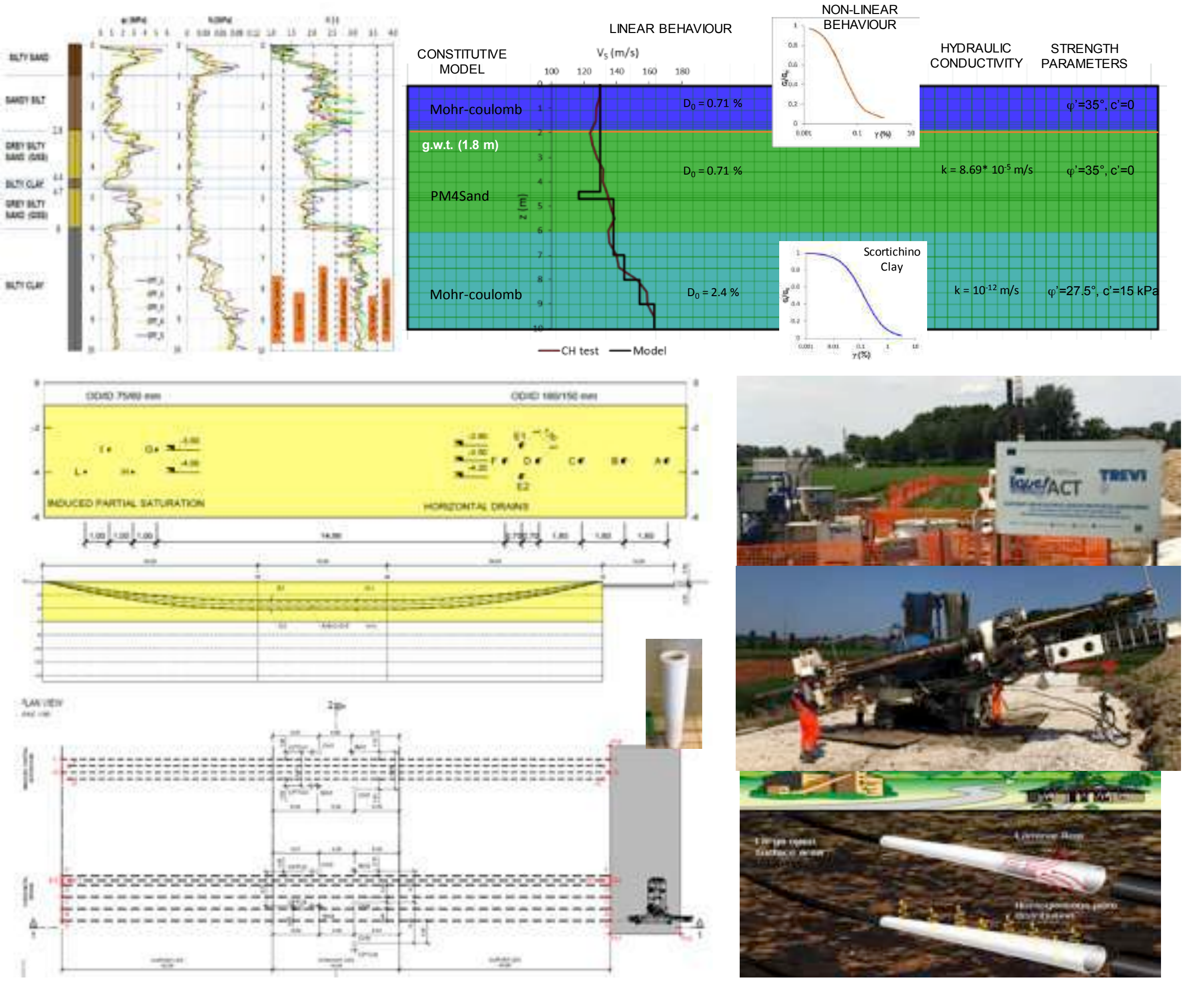

Figure 6. The pilot testing site (www.liquefact.eu)

A shear-wave vibrator (M13S/609 S-Wave) was used as a dynamic loading source at the ground surface. In each test, the static vertical loading of the machine was firstly applied and, after verifying that the consolidation process was completed, a dynamic loading at 10 Hz was applied for a duration of 100 s or 200 s, in the first and in the second phase of shaking, respectively. The results obtained are very positive and are being processed but confirm the previous results obtained by numerical analyses (www.liquefact.eu).

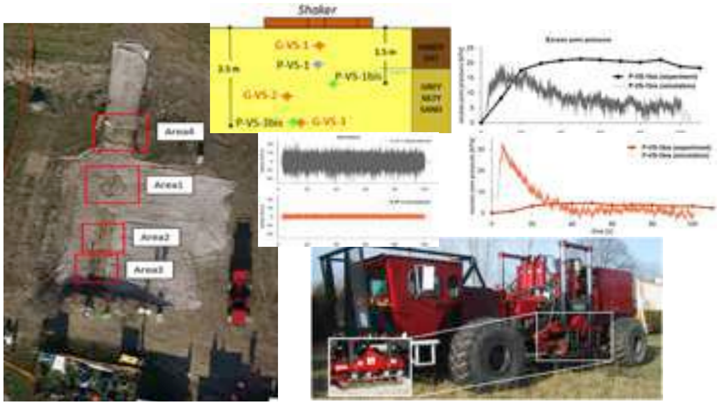

Figure 7. Pictures of the S-wave vibrator and vibrating base plate; the testing áreas; Inducing partial saturation in existing structures (www.liquefact.eu)

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FOUNDATION STRENGTHENING OF BASIN 7E OF CRESTUMA-LEVER DAM

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Crestuma-Lever dam (Figure 1), located 13 km upstream from Porto, is the last downstream hydropower scheme of a dam cascade system in the Douro River, which comprises 6 Spanish and 8 Portuguese dams. The catchment covers 97,603 km^2. The dam was designed and built by Electricity of Portugal (EDP) (completed in 1985) for hydropower purposes (105 MW), water supply and river navigation. The dam safety control has been followed-up closely by EDP, with technical support from the National Laboratory for Civil Engineering (LNEC).

Figure 1. Crestuma-Lever dam aerial view.

The dam must cope with a very wide range of flow discharges (the maximum flood discharge is 26,000 m3/s) and a considerable variation of water depths downstream (from a few meters to over 20 m), involving tidal effects. To deal with these hydraulic constraints, a gated dam was designed, composed of 8 concrete stilling basins (56 m long), equipped with double leaf vertical roller gates (28 m wide, 13.8 m high – Figure 2), supported by seven concrete piers, discharging floods by the crest and bottom. These piers reach the schist bedrock, and cross 40 m of sandy soils. The stilling basins rest in the alluvial riverbed. To reduce the seepage flow and hydraulic gradients, upstream and downstream partial concrete cut-off walls were adopted, reaching elevation -17.50 m.

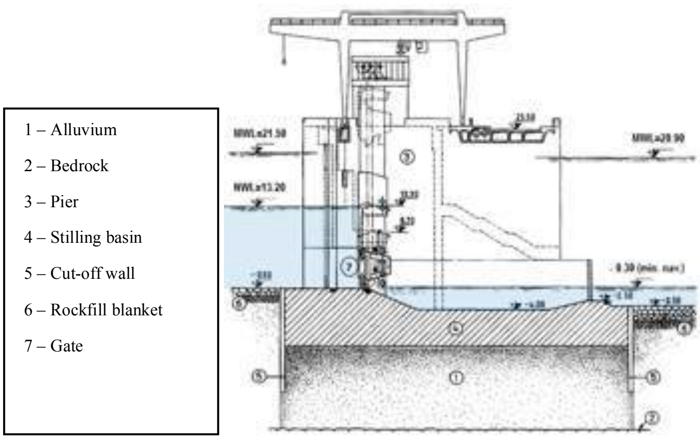

Figure 2. Crestuma-Lever cross section (Álvares Ribeiro *et al.*, 1982).

Regular surveying of the downstream rockfill stability has been carried out. The survey completed after the first significant flood (13,000 m3/s), in the 1989/90 winter, detected a 4 m deep scour cavity in the initial 10 to 20 m of rockfill, and a 1 to 2 m high bar around 30 m downstream of the spillway end sill in three out of the four basins surveyed. The following surveys showed a general scour of the rockfill protection close to the downstream concrete cut-off wall. In addition, visual inspections performed by divers showed that sediments formed most material in the scour depression, and that the protective rockfill and underlying filter layers, adjacent to the downstream cut-off wall zone, were severely damaged.

The corrective measures found to address foundation safety of stilling basins 1E and 3E, implemented in 2012 and 2013, were the construction of a watertight upstream solution, new filters beneath the rockfill protective layer and a new rockfill protection with larger blocks than those of the original design (Caldeira et al., 2013). The upstream watertight intervention consisted of a secant pile wall, embedded, in the bedrock formation, sealed at the top by a jet grout slab and laterally by grouting.

During those interventions, an upward flow of water was detected immediately downstream of the stilling basin 7E, under a large concrete block. The documentation concerning construction and operation of the dam and the results of underwater inspections of the stilling basin were analyzed and a geotechnical investigation was carried out. A set of boreholes was executed, with the measurement of pore water pressure along the alluvial formation, which revealed the presence of muddy soils, immediately underlying the stilling basin, in some location with null SPT values.

Analyzing the data collected and the results of the three-dimensional modeling of the seepage conditions under the basin (Caldeira et al., 2015), it was concluded that the observed boiling was caused by water passages through defects located in the upstream cut-off wall, some of which were confirmed later by underwater inspection. The hydraulic gradients associated with these anomalies, however, excluded any downstream heaving failure scenario, but not the occurrence of localized internal erosion, essentially at the upstream flow inlet areas and at the bottom of the upstream cut-off wall. The boiling detected at the surface was attributed to the flow concentration between the referred large concrete block and the adjacent concrete structures.

Following this study, several interventions were defined (Figure 3). These interventions included:

(i) secant pile wall of plastic concrete with a diameter of 1,5 m and a spacing of 1,0 m in a single row, embedded, at least 1.0 m in the schist formation;
(ii) lateral sealing of the pile curtain with the pre-existing structures by grouting;
(iii) a clay layer, placed between two layers of plastic concrete, for sealing of the top space between the cut-off wall and the pile curtain;
(iv) the sealing of the perimeter construction joint between basin 7E and cut-off walls.

Taking into account the mechanical characteristics of the foundation soils and the need to minimize the occurrence of settlements, it was recommended to reinforce its foundation using micro piles (Figure 3).

In the verification of the dimensioning of these elements, conditioned by the geological-geotechnical and hydrogeological scenario in question, it was necessary to resort to a three-dimensional model of finite elements of the basin and of its foundation (Carreto et al., 2018).

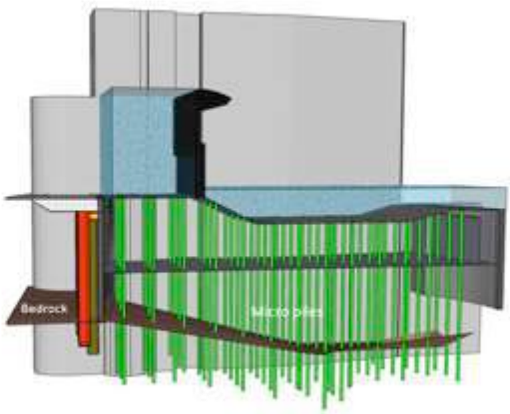

Figure 3. Representation of basin 7E upstream water tightness solution and foundation strengthening (Fernandes et al., 2018).

The geotechnical studies, carried out during the diagnosis phase of the boiling causes, the analysis of seepage through the foundation of basin 7E, the results of the stress-strain analysis of the stilling basin and its foundation are presented in this communication.

Acknowledgments

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STABILIZATION HIGHWAY EMBANKMENT OVER ALLUVIAL SOILS

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At February of 2010, the platform of the highway A8, at km92+600, connecting Lisbon to Leiria, was partial destroyed by a landslide of the embankment which was executed over soft clayey soils.

The platform was built about 8 years ago with part on excavation and part on embankment. The lower part of the platform was stabilized with a gabions wall in order to protect some small houses, located very close from the platform slope base (Figure 1)

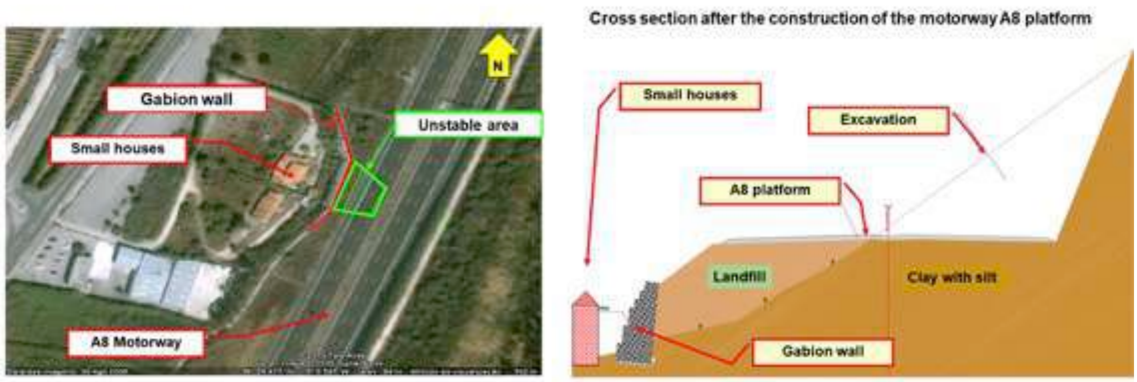

Figure 1. Site location and geological conditions.

The embankment was built over soft clayed and silty soils, with cohesion and resistance increasing with depth. After the landslide it was possible to confirm by the analysis of the deformations and cracks that a circular slip surface was formed, passing under the gabion wall foundation. The movement intersected a mass of soil with about 60m wide and 12m height, including 3 lanes of the highway platform (Figure 2).

Figure 2. Slip surface bellow the gabion wall foundation.

After the landslide, it was necessary to design a solution in order to rapidly stabilize the platform, minimizing the impact on the highway traffic. Other main issues were the preservation of the small houses, as well as the schedule for the stabilization works, in order to minimize the impact on the highway traffic.

Taking into account the existent conditions, mainly the small working area, as well as the works schedule, a solution based on jet grouting and steel tubular micropiles was adopted, performed using small weight and versatile equipment. The adopted solution consisted on the execution of jet grouting vertical wall, columns ∅1,2m spaced 1,0m, stiffened by buttresses, spaced 3m. Each buttress was formed by 3x∅1,2m inclined jet grouting columns and one back micropile. The vertical column located at the buttress alignment was also reinforced with one micropile. Over the jet grouting wall, a reinforced concrete cantilever upper wall was built in order to allow the partial demolishment of the gabion wall, as well as the excavation of the existent fill, in order to decrease the unstable gravity loads. Behind the upper wall, a light weight aggregates backfill, over a lower sandy backfill layer, was installed in order to allow also the reduction of the unstable self-weight, as well as, to improve both the wall and the highway platform drainage. At the top of the LWA backfill one layer of polypropylene biaxial geogrid was installed in order to minimize the probability of differential settlements between the new and the existent platforms (figures 3 and 4).

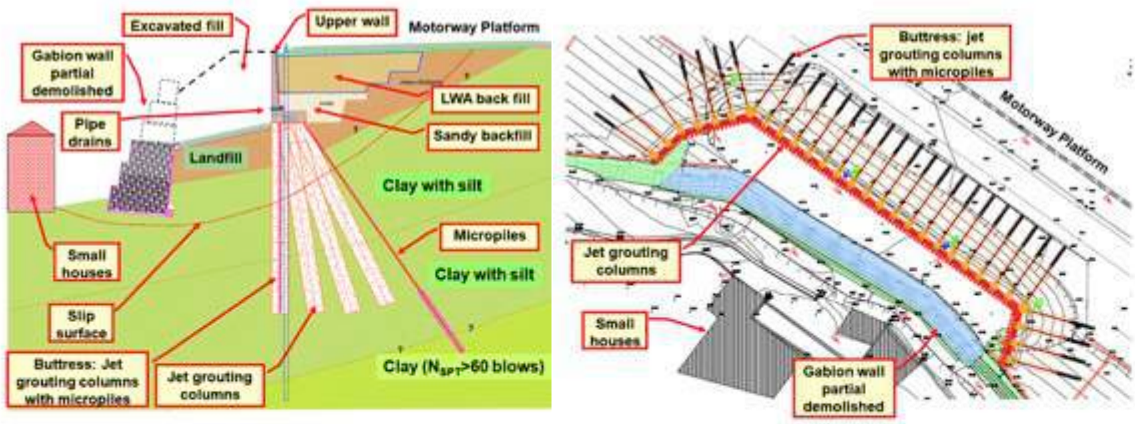

Figure 3. Cross section and plan of the adopted solutions.

For the execution of the jet grouting columns it was necessary to increase the existent landfill horizontal confinement, in order to protect the small houses located at the slope base. For this purpose, geotextile bags filled with sand were used. After the execution of the upper wall, the sand was used for the lower backfill layer (figure 5).

Figure 4. Views of the jet grouting works.

Figure 5. Views of the upper wall backfill works.

For the design of the adopted solution equilibrium limit (overall stability for static and seismic loads) and FEM (deformations) analysis were carried out, using Slide and Plaxis software. The maximum estimated horizontal displacements at the highway platform after the conclusion of the earth retaining works was about 23mm, much higher than the ones obtained through the instrumentation readings, about one rainy winter after the conclusion of the stabilization works (figure 6).

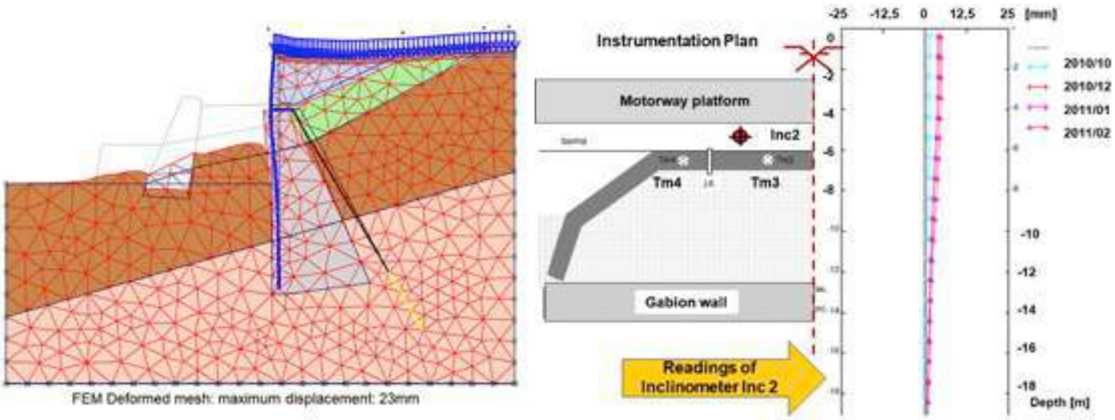

Figure 6. View of the adopted FEM model and horizontal displacements.

The execution of the jet grouting columns was complemented by a tight quality control and quality assurance, allowing the confirmation of both the resistance and geometry of the columns. For this purpose, test columns were built and cores from test and final columns were collected in order to access the geometry and to perform laboratorial tests, mainly Unconfined Compression Strength, at different ages, including the measurement of the Young Modulus (figure 7). During the execution of the jet grouting columns a permanent registration of all the adopted parameters was also performed using an LT3 device.

The solution overall performance was accessed through an instrumentation plan during and after the jet grouting works. The instrumentation plan comprised the installation of inclinometers and tiltmeters, at the highway platform, as well as topographic marks, at the gabion wall and neighbor houses (figures 7 and 8).

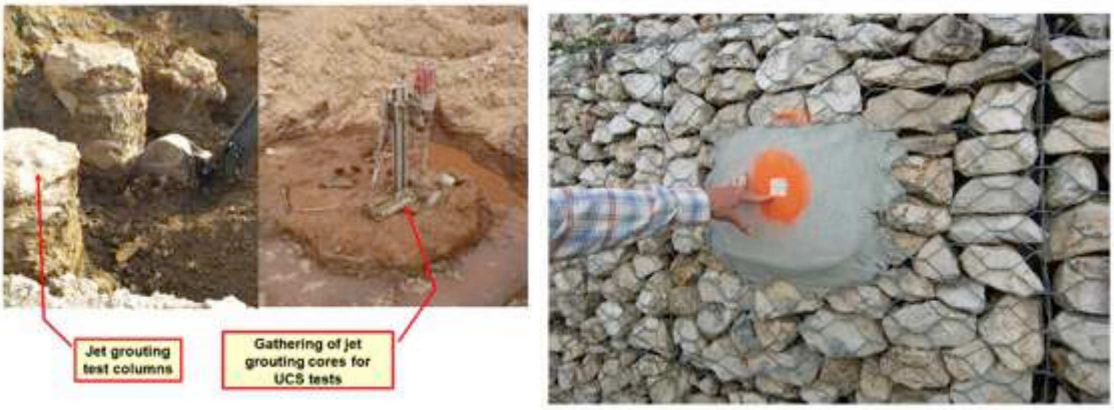

Figure 7. Jet grouting columns quality control and gabions wall instrumentation.

The adopted solution allowed the fulfilment of all the main objectives: technical (low deformations at the highway platform and at the neighbourhood small houses and slope stabilization) and control of both costs and construction schedule, confirming the good performance of the solution (figure 8).

Figure 8. View of the finished stabilization works.

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SOIL TREATMENT WITH JET-GROUTING ASSOCIATED TO RETAINING STRUCTURES OF THE MACHICO-CANIÇAL EXPRESSWAY IN MADEIRA ISLAND

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The Machico-Caniçal section, with about 8 km long, constitutes the final segment of the expressway that connects the city of Funchal to Caniçal along the Madeira island South coast. The entire liaison has been constructed by stages during the last 30 years, according to the island expressway development program. The expressway is developed along an extremely mountainous region, characterised by two very heterogeneous geological volcanic complexes, which are generally covered by thick and unstable slope deposits or by alluvial deposits.

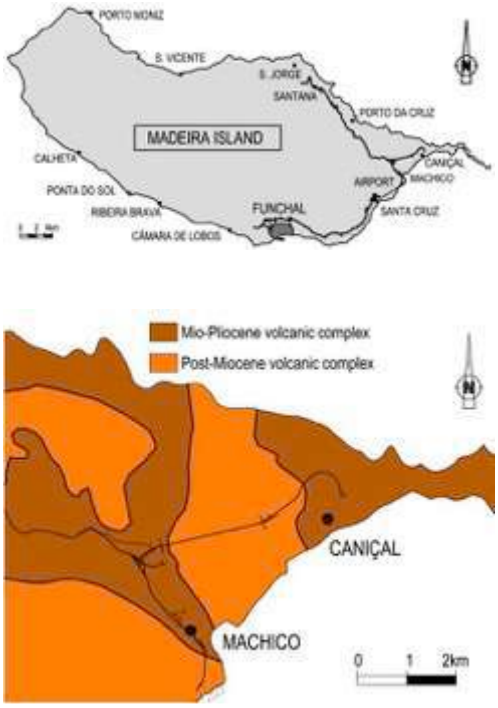
The constraints related to the layout, topography, high urban occupancy and very unfavourable geological and geotechnical conditions, specially the presence of slope deposits with significant thickness and very weak strength characteristics, lead to the need of conceiving different types of retaining structures (Figure 1).

This presentation focuses on the description of the most relevant geological and geotechnical conditions related with the above mentioned slope deposits and on the main aspects related with the design and construction of the special foundations with jet-grouting which were considered necessary in order to allow the construction of several retaining structures, namely gravity walls, reinforced earth retaining walls and sheet pile walls.

In the first stage of this presentation, the geotechnical characterization of the aforementioned deposits will be addressed. They result from landslides and rockfalls due to the declivous and escarped rocky slopes of the Machico stream valley and the Portais valley, as well as its accumulation along and at the base of those slopes. These deposits are very heterogeneous, comprising small angular or sub rolled fragments and blocks that can reach 2 m in diameter, predominantly of basalt, in a clay-silt-sandy matrix of a brown-reddish dark colour. This matrix consists essentially of highly plastic clays. Another characteristic is the higher concentration of rocky blocks at the base. In most places the water table is located at the base of the deposits, in the contact with the volcanic substrate.

In many cases these deposits are in limit equilibrium conditions due to their depth and geotechnical characteristics, to the bedrock's inclination and, above all, to the water table variations. They easily become unstable, provoking landslides with more or less significant areas. They cover most of the slopes where the expressway was implemented and gradually increase their thickness to the base of the slope. These deposits have been largely surveyed and their thickness ranges from about 0,3 m to 21,0 m.

Concerning the previous aspect, mention must be made to the fine fraction which controls the behaviour of these deposits. The lowest value of particles passing the #200 sieve was around 66%, the lowest value of particles smaller than 2mm was 22% and the lowest value of particles smaller 2μ was 38%. Liquidity limit (LL) values between 56 and 122% were also obtained as well as plasticity index (PI) values between 21 and 80%. The water content (w) and unit weight (γ) values obtained were on average 28% and 18,4 kN/m^3, respectively. For the specific gravity of solid particles (G) values between 2,53 and 2,90 were obtained. Sand equivalent (SE) ranged between 14 and 16%. Carried out analysis on the matrix using the x-ray diffraction method showed that the clay fraction is essentially comprised of montmorillonite (around 95%).



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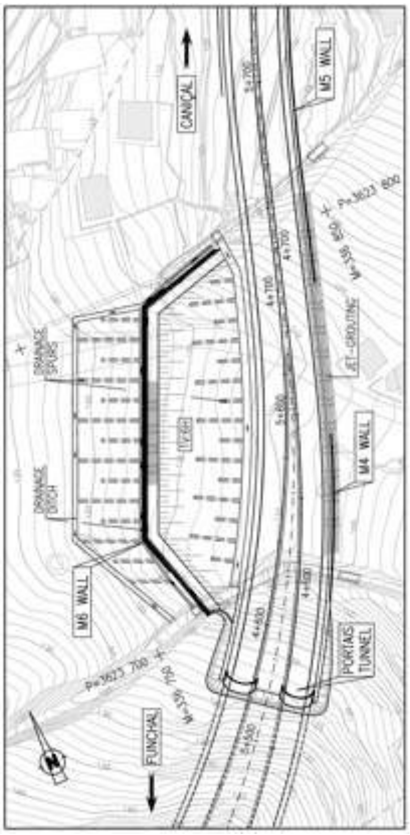

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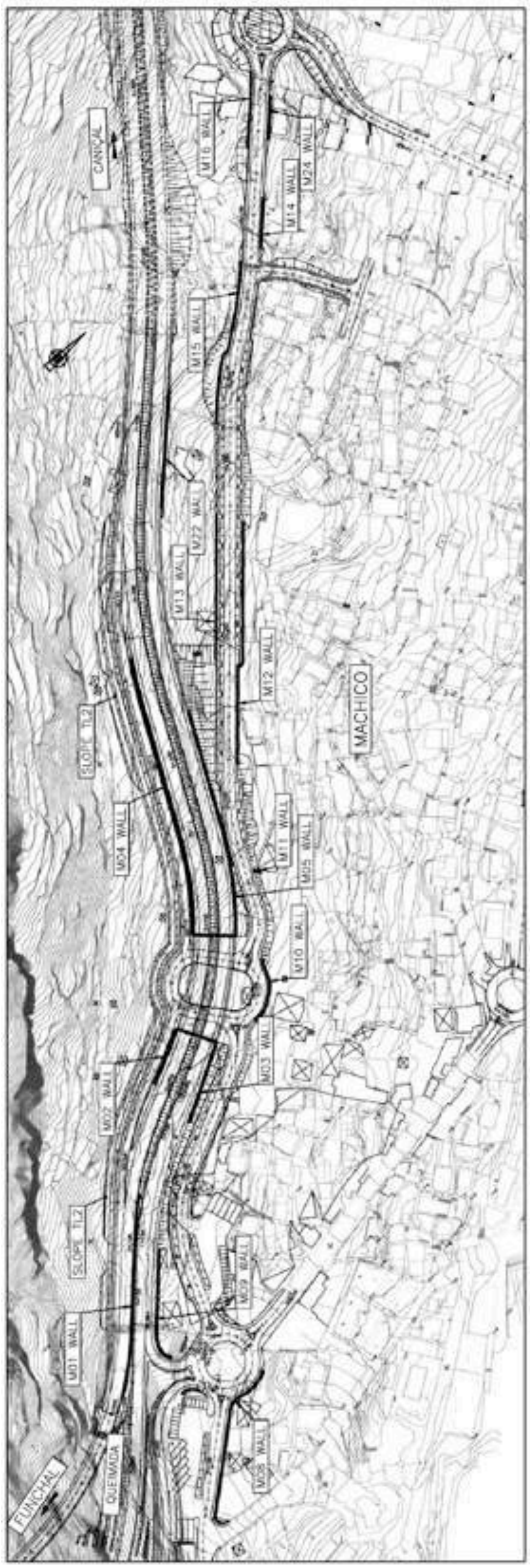

c)

Figure 1. a) Location plan; b) Retaining structures in the East Portal of Portais Tunnel; c) Retaining structures in the Machico Sul Interchange

The undrained residual strength was estimated from the residual strength parameters in terms of the normal acting stress, according to the Mohr-Coulomb failure criterion. Thus, the following parameters were obtained: $cu_{res} = 14$ kPa, $c'_{res} = 0$ kPa and $ø'_{res} = 14º$. However, to take into account the favourable contribution of gravel and blocks and the irregularity of the bedrock to the shear strength, it was considered that more favourable parameters could be adopted. As a result of this analysis the following parameters were adopted $cu_{res} = 20$ kPa, $c'_{res} = 0$ kPa and $ø'_{res} = 16º$.

After establishing the geotechnical characterization of the deposits, the main aspects related with the design and the construction of the special foundations with jet-grouting will be presented. These works were held in order to allow the construction of several retaining structures, namely gravity walls, reinforced earth retaining walls and sheet pile walls.

Indeed, in situations where the expressway or the various branch roads were located over quite thick deposits, making it impossible to completely remove them, it was necessary to assure suitable conditions for the establishment of the different retaining structures' foundations. Thus, for structures built in these locations, it was necessary to use soil improvement technics, as illustrated in Figure 2. In these situations, the selected solution was the use of soil improvement guaranteed by Ø1000 mm diameter jet-grouting piles, generally crossing the deposits all the way to the bedrock.

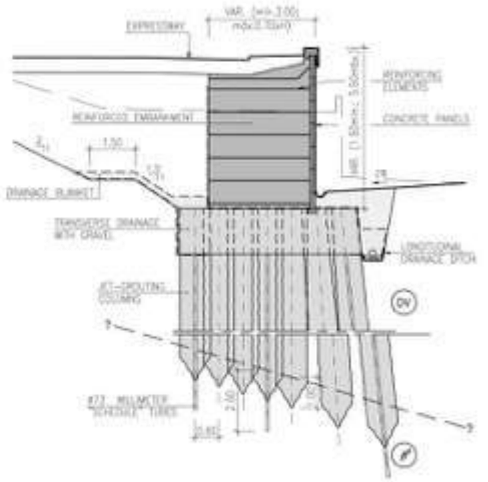

Figure 2. Reinforced earth retaining walls with jet-grouting foundation.

In this presentation, the solution implemented immediately after the East portal of the Portais Tunnel is highlighted. In fact, that location of the expressway runs along an area in which the slope deposits occur with significant thickness, sometimes up to 10 m thick. There is evidence that these deposits reach the buildings at the village of Barro, close to the construction site, which show significant damages that can be attributed to the movement of this mass of slope deposits. For this reason, in addition to ensuring the stability of the retaining structures necessary to the expressway, an attempt was made to substantially improve the overall stability of the mass of existing slope deposits, thus minimizing future damage to these buildings.

Therefore, the solution found was to build the retaining structure away from the expressway, a little further up the slope, where the thickness of the deposits was smaller. Thus, the slope deposits to be supported were reduced as well as the potentially instable mass, being the latter partially removed. Due to the deposits' thickness in this area it was impossible to have shallow foundations for the retaining structures, making it necessary the use of deep foundations through soil improvement technics.

For this up the hill structure, an anchored pile wall was defined reducing the excavation of the slope deposits to a little over 3.0 meters. The anchored pile wall was made of Ø800 mm reinforced concrete piles, placed with 1,0 m intervals, driven from the surface of the natural terrain and had 1 or 2 levels of anchors. Along the wall, where the depth of the bedrock was greater than the bottom of the excavation, it was necessary to improve ground properties to ensure an adequate behaviour of the structure, in particular its ability to mobilize the passive earth pressure.

Given that jet-grouting technology was available on site, slope deposits were thus treated using this technology along a section of about 30 m long and 4 m wide. Jet-grouting piles were executed through the deposits into the bedrock. ∅1000 mm diameter columns of jet grouting were set in a 1m (longitudinal to the wall) x 7m (transverse to the wall) grid. Two areas of the soil mass, each with about 2m, were left untreated at the base of the treated area to allow natural percolation in the soil and thus prevent the groundwater level raising behind the wall (Figure 1-b).

Regarding the structures to be built along the right side of the expressway (down slope) to support the road embankments, the recommendation was to build reinforced concrete walls with Ø1000 mm diameter jet-grouting piles, driven through the deposits into the bedrock to ensure adequate foundation, since the slope deposits were so thick that it was impossible to place the foundations directly into the bedrock (Figure 3).

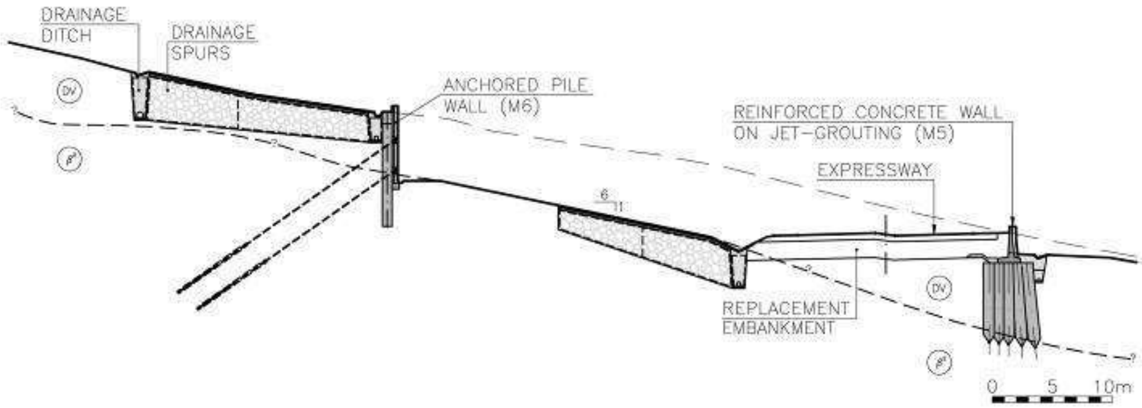

Figure 3. Typical cross section for the road in the Portais valley

In this presentation the main conclusions regarding the works are also highlighted, in particular the fact that the design of the soil improvement solutions associated with the structures of the Machico-Caniçal section of the expressway was subject to constraints of several nature, mainly due to the existence of areas with thick slope deposits with very poor strength characteristics and densely occupied areas. It should be stressed out how essential was to fully understand the geological conditions during the conception/design phase, this allowing an appropriate geomechanical characterization of the slope deposits and thus ensuring the optimization of the designed solutions.

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GROUND IMPROVEMENT SOLUTIONS AT MYRIAD SANA HOTEL

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ABSTRACT

The aim of this abstract is to present the main design and execution criteria related with ground improvement solutions, using jet grouting columns as foundation and cofferdam solutions, at the MYRIAD SANA HOTEL , located next to the Vasco da Gama Tower, at the Tagus river right bank, in Lisbon, Portugal. The hotel , with two underwater floors and twenty two upper floors, is located at the river, over alluvial soils with a thickness of about 17m, resting over Miocene soils. This complex scenario demanded the use of some unusual ground improvement solutions, related and interconnected with foundation and cofferdam solutions. The main results of the adopted finite element models are presented, as well as of the monitoring and survey plan.

INTRODUCTION

The MYRIAD SANA HOTEL has 24 floors and a plan area of approximately 1.500m2, with a semi-elliptical geometry, at the place of the existent and previously demolished building, which held the European Union pavilion at EXPO'98. The building had three upper floors. Its foundations consisted on driven steel piles, filled with concrete. The Vasco da Gama tower foundations consists on a massive peripheral box, made from diaphragm walls and filled in with plain concrete, resting over Miocene soils, with approximately 16m height and a total plan area of about 18x26m2 (Figure 1).

Figure 1. Bird eye view of the Hotel excavation works.

Due to the geotechnical and geological conditions of the site, with more than 17m of alluvial soils, as well as the occupation of the ground with the foundations of the existing building, it was necessary to study and apply some unusual solutions for ground improvement, integrated with foundations and cofferdam solutions.

MAIN CONDITIONS

Geological and geotechnical conditions

The ground at the site is composed by alluvium soils with an average thickness of about 17 m, resting over the Miocene heterogeneous materials, composed by sands and limestones.

The alluvium soils are essentially formed by silts and clays with NSPT blows not bigger than 3. The undrained shear resistance, assessed by Vane tests, range with depth between 8,5 and 22,0kPa.

Due to the site location, near a previous riverside wall, the site land side has heterogeneous sandy fills and a protection rip rap prism, over the original alluvium mud fills. Table 1 presents the main geotechnical parameters, obtained by the interpretation of the SPT tests and both UCS and triaxial laboratory tests.

Table 1. Main geotechnical parameters.

Material	N_{spt}	ϕ' (º)	c' (kPa)	C_u (kPa)	g (kN/m^3)	E' (MPa)
Alluvium	<9	18	0	8-22	16	4-10
Miocene	>60	35	40	200	20	50
Jet Grout.	-	-	200-400	500	18	1.000

Other conditions

The land side of the peripheral wall, as well as some new foundations were located at the border of the Vasco da Gama tower's foundation, leading to a solution able to compensate the partially and temporary loss of confinement of the tower's foundation, due to the excavation works.

The foundation of the previously existent building was composed by 69 circular driven piles with permanent steel casing Ǿ630 × 8mm. The construction of the working platform for the driven works demanded the execution of sheet pile wall, braced by steel ties. A protection riprap was placed at the external side of the sheet pile wall (Figure 2).

Demolishing procedures of the pre-existing foundations were performed taking into account the need for compatible solutions between the new and the existent construction, mainly for: foundations, excavation and peripheral walls.

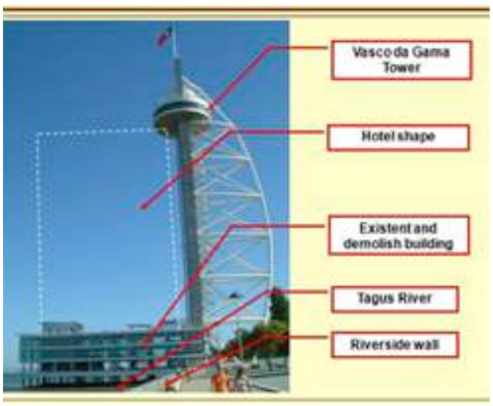

Figure 2. Main conditions.

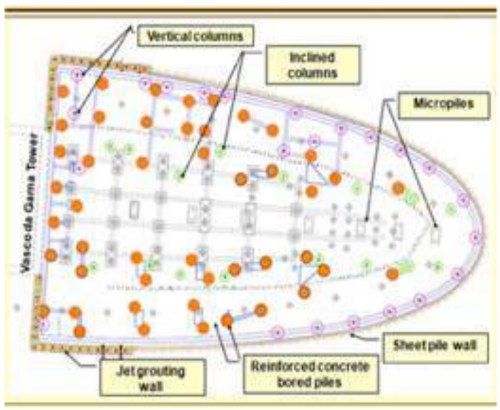

Figure 3. Plan of the main adopted solutions.

MAIN SOLUTIONS

Introduction

The main adopted solutions for the foundation and cofferdam retaining walls were the following (Figure 3):

- A temporary "hybrid" cofferdam, composed, at the riverside main perimeter, by a tied / propped sheet piles wall and, at the land side, by an anchored jet grouting columns wall;
- A bottom jet grouting sealing slab, to stiff horizontally the cofferdam and to prevent the water in flow from the excavation base;
- Large diameter reinforced concrete bored piles as foundation of the main structural vertical elements.
- Steel micropiles to nail the jet grouting sealing slab and as foundation of the lightest structural elements, in this case sealed inside jet grouting columns.

In order to obtain safe and permanent working conditions, above water level, the works started with the execution of a temporary working platform, formed by a granular fill, resting over a biaxial geogrid, made from polypropylene (tension resistance of 20kN/m), resulting on a load transfer platform towards the existing foundations (driven and sheet piles), in order to support all the equipment necessary to the main geotechnical works.

Among the equipment, this fill supported a 100ton crane in order to allow the sheet piles driven from the site central area, correspondent to the location of the existent building, about 10m of distance from the driven point.

Figure 4. View of sheet pile driven operations from the site central area.

"Hybrid" cofferdam

The execution of the temporary cofferdam comprised, due to geotechnical restraints, two main solutions: sheet piles at the riverside and main perimeter, with a semi-elliptical plan shape, and jet grouting columns on the land side, adjacent to Vasco da Gama tower. In order to decrease the water in flow and to stiff the cofferdam toe, its base rested at the Miocene layer. The sheet piles were driven using a guiding beam (Figure 4).

The cofferdam bracing system consisted in hot rolled steel profiles distributed on a grid shape, supported by distribution beam, at the cofferdam extremities. This steel grid allowed not only the confinement of the working platform fill, performed at 1,70m height (acting as ties during the working platform works), but also allowed the support of the cofferdam walls (acting as props during the excavation works), enabling, together with the jet grouting sealing slab, the stability of the cofferdam during all the construction phases (Figure 5).

In order to facilitate the excavation works the bracing system was vertically supported on steel micropiles, sealed at the jet grouting sealing slab, and located at a minimum height of 6m from the excavation base.

According to the architectural solution, it was necessary to build a permanent reinforced concrete wall, integrated on the final reinforced concrete structure, about 1m for the inside of the temporary cofferdam. This solution demand on a second stage the demolition and cut of both the jet grouting columns and the sheet piles located above the minimum water level.

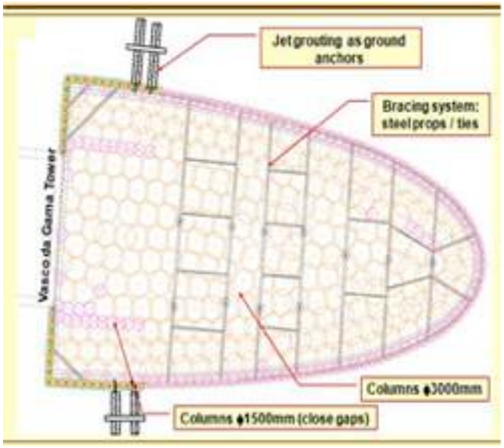

Figure 5. Plan of jet grouting solutions and cofferdam bracing systems.

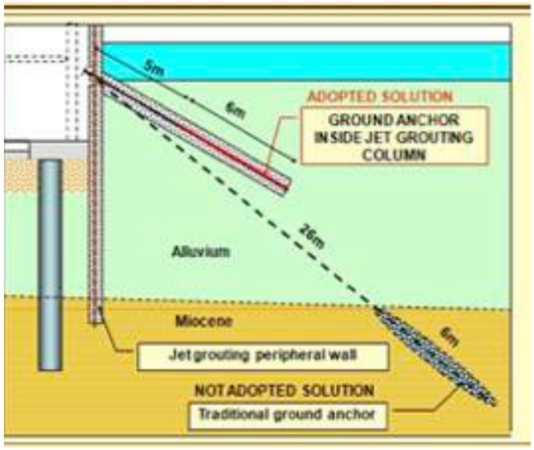

Figure 6. Ground anchor using jet grouting columns.

Jet grouting: horizontal sealing slab, cofferdam wall and foundation

The jet grouting technology was generally applied, as ground improvement technique, for different solutions.

Below the excavation level, the cofferdam wall was previously braced by one horizontal sealing slab, composed by jet grouting columns ϕ3000 mm and ∅1500mm with a minimum thickness of 2 m, and also inclined jet grouting columns ∅1200 mm. This sealing slab allowed also to increase the cofferdam overall horizontal stiffness, as well as to reduce the water in flow from the excavation base (Figure 5). In order to reduce the overall thickness and the bending stress the sealing slab was nailed to the Miocene layer through both the existent and the new foundations: bored piles and micropiles.

The use of jet grouting columns as cofferdam walls was justified by the presence of pre-existing diaphragm walls, as foundation element for the Vasco da Gama tower, as well as the existence of heterogeneous fills, with boulders, not allowing the driving of sheet piles. Therefore, the solution was materialized by a jet grouting columns wall, ∅1000m spaced 0,60m, and reinforced with hollow steel tubes N80 ∅177,8 × 9 mm (API 5A), resting over the limestone substratum.

The bracing of the jet grouting cofferdam walls was assured by an anchoring system, were the steel wires were sealed inside jet grouting columns, taking the advantage of an existing alluvium with more sand and boulders at the land side. Comparing with the traditional procedure, the adopted one allowed the reduction of the ground anchors overall length, as well as the increase of the anchor stiffness (figure 6).

Foundation: bored piles and micropiles

Taking into account the geological restraints and the carrying loads magnitude the main foundations of the Hotel are reinforced concrete bored piles ∅1500mm, built with top retrievable steel casing. The piles bottom length, including the embedment of about 4,5m at the Miocene layer, was drilled using stabilizing polymers. The equipment required to perform the drilling operations was a rotary “Kelly” ring, allowing both to drill and to clean the pile hole. The integrity of all bored piles was tested through cross hole tests.

As stated, steel micropiles with external couplers were used to nail the jet grouting sealing slab, accommodating tension axial loads, mainly during the excavation phase.

Steel micropiles were also used as foundation of the lightest structural elements, accommodating compression axial loads. In this case the micropiles were sealed inside jet grouting columns, in order to decrease its overall difference of stiffness, comparing to the bored piles one, as well as to allow a better confinement and corrosion protection.

Seismic loads are accommodated both through the bending of large diameter bored piles, as well as through some inclined steel micropiles, sealed inside jet grouting columns.

All the foundation elements, bored piles and micropiles, were capped by a reinforced concrete mat slab, cast against the horizontal jet grouting sealing slab

DESIGN

The design of the cofferdam walls, including its bracing systems, was performed using a 2D finite element numerical model (Plaxis Professional V.8).

Internal forces and displacements at the cofferdam walls, as well as on the supported soil, were analyzed and predicted for all the main construction stages (Figure 7). All the main geotechnical parameters were established taking into account the results of previous laboratorial tests (Table 1).

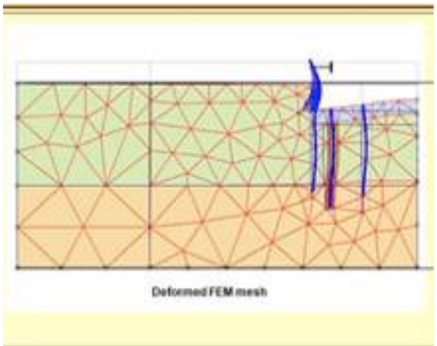

Figure 7 – View of the adopted 2D FEM model.

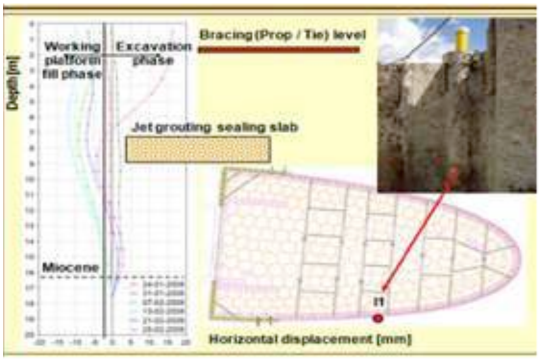

Figure 8 – Monitoring and survey plan: inclinometer displacements.

MONITORING AND SURVEY PLAN

Considering the context and the complexity of the described solutions, a monitoring and survey plan was applied taking into account the need to perform the construction in safe and economic conditions for both the site and the Vasco da Gama tower.

In order to accomplish this goal, the following equipments/devices were installed: inclinometers (7un.), topographic targets (21 un.) and topographic marks (5un.). Measurements were commonly performed, at least, twice a week until the cast of the ground level structural slab. The hotel structure were monitored during its construction in order to access the foundations behavior during the loading process. Figure 8 shows the results of inclinometer I1, where is possible to point out the importance of the jet grouting sealing slab for the cofferdam horizontal stiffness.

MAIN CONCLUSIONS

In this extended abstract the main design and performance issues of complex cofferdam and foundation solutions, for the MYRIAD SANA HOTEL, using mainly jet grouting as ground improvement technique, were presented. It should be pointed out the versatility and the good interconnection between the adopted solutions, ranging from jet grouting to bored piles, micropiles and sheet piles, allowing the construction of the cofferdam and the foundations using the safest method: from the working platform, previously from any excavation works.

The main conditions and adopted solutions were described. Some FEM models and monitoring results were also presented. Those results show that the obtained displacements were in general lesser than the predicted ones, confirming, together with the respect for the initial schedule and predicted quantities, the overall suitability of the adopted solutions.

SOIL IMPROVEMENT BY PRECAST DRIVEN PILE RIGID INCLUSIONS FOR EMBANKMENTS ON VERY SOFT SOILS

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The use of prefabricated driven piles as a rigid inclusion is increasingly common in the control of embankment settlements, due to the growing demands for quality of the foundation element. Associated or not to a vertical drainage, this type of solution is an effective way to solve the problems derived from the foundation in soft and alluvial soils.

In this presentation we will talk about two construction sites, the first of them located near Lisbon, in Sacavém, within works for new bridge over Trancão river. The second site is part of the project of high speed train line "SUD EUROPE ATLANTIQUE" (LGV SEA), which involves the construction of a new infrastructure of double railway track of 300 km between Tours and Bordeaux, in France. In particular, lot 15 (section G), shortly before reaching Bordeaux, must cross through floodplain of the Dordogne River, specifically, the area of Virvée swamp, with the execution of three embankments between 2 and 6 m in height and a viaduct. Treatment of the foundation of these embankments will be the topic of the presentation.

In the case of the works for the new bridge over Trancão River, the improvement treatment of transition platform between the road and the structure was carried out.

General scheme of the solution designed by TRIEDE can be seen in figure 1 and consisted of the driving of a mesh of prefabricated piles. For correct working of the solution, a cushion or distribution layer must be placed on the prefabricated piles. This part is fundamental for effectiveness of the treatment, since it works as a transmitting element of the loads towards the piles, absorbs the punching that occurs in head of inclusions when compression load acts and homogenizes settlements, guaranteeing an adequate behavior of the system (also limits material plasticizing). In this case, on the piles were placed small slabs and above them, within a load transfer layer of 60 cm of thickness, two geogrids, SS30 type for the lower and SS20 for the upper one. Above this layer the different subbases of the road were arranged.

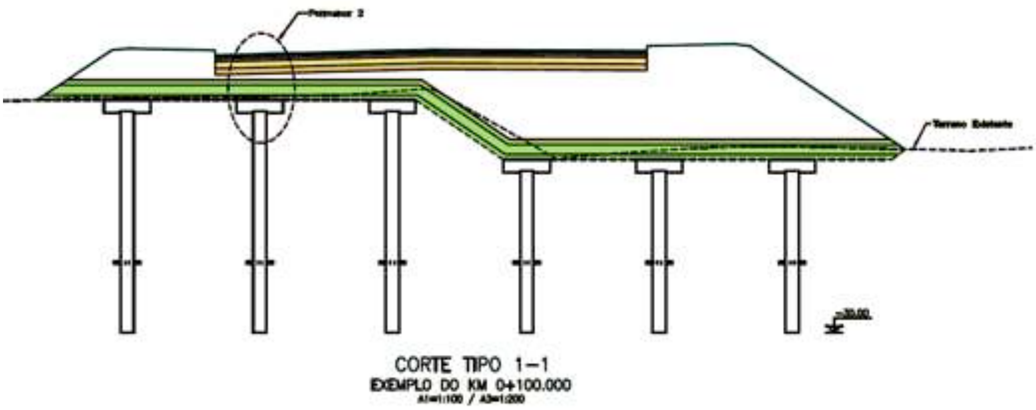

Figure 1. TRIEDE Solution

Typical geotechnical profile can be seen in figure 2. In depth we can find Miocene soils covered by alluvial deposits of variable thickness and fill materials of approximately two meters thick.

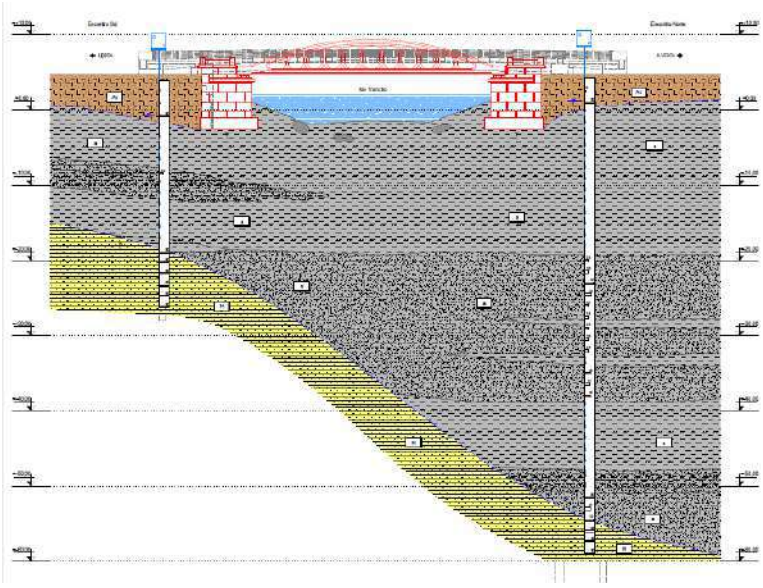

Figure 2. Geotechnical profile

A total of 102 piles of type CK-400 Class I, according to Standard EN 12794, were driven. These piles are manufactured by RODIO in permanent facilities with a class C50/60 concrete and an elastic limit for reinforcement of not less than 500 N/mm^2.

The final average length was 20 m, although depths of 31 m were reached in some piles. For the connection of the pieces, Class A steel joints according to EN 12794 Standard were used.

In the case of the foundation for embankments that cross the area of the Virvée swamp, geotechnical conditions were characterized by the presence of compressible soils up to a thickness of 10 m, where metric levels of peat are included (W = 230- 500%, MO = 61-196%, NH4 +> 100 mg / l) and/or muddy clays with a undrained shear resistance of less than 15 kPa.

Figure 3 shows undrained shear strengths measured in situ as a function of depth.

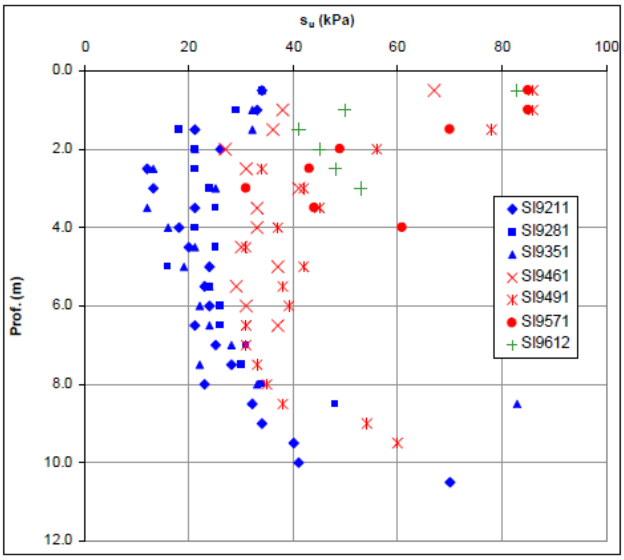

Figure 3. Undrained shear resistances as a function of depth

In this complex geotechnical context and taking into account the demands of a high-speed railway line, different solutions were studied and the improvement of the ground was made with prefabricated driven piles, working as rigid inclusions, associated with a vertical drainage.

Figure 4 shows geometric model of the solution.

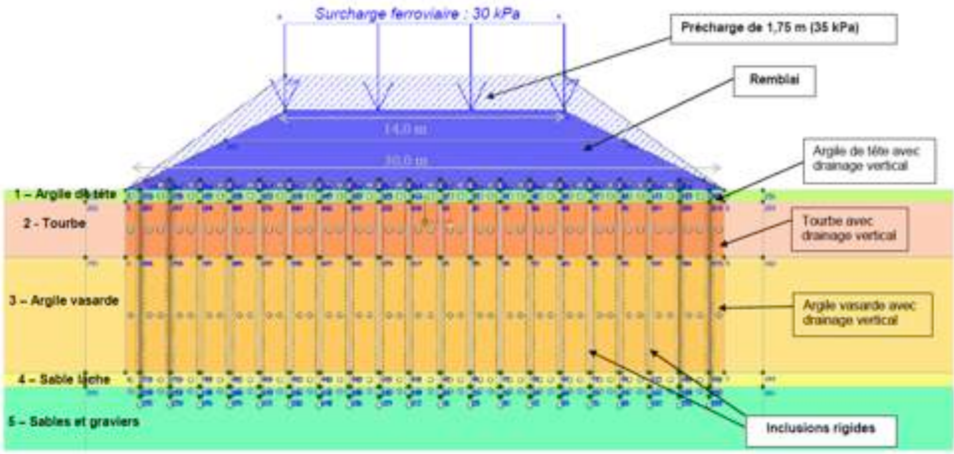

Figure 4. Geometric model of the solution with soil profile

To validate proposal solution, a real scale experimental embankment was designed by TERRASOL and ANTEA GROUP. Four different test cells with the characteristics shown in Table 1 were executed on this embankment.

Table 1. Characteristics of the cells in the experimental plot

Sección	Malla	Elementos suplementarios de refuerzo
1	1,6 m × 1,6 m	Coronación con capiteles de 0,7 m × 0,7 m
2	1,7 m × 1,7 m	Refuerzo con dos geomallas
3	1,6 m × 1,6 m	Refuerzo con dos geomallas
4	1,8 m × 1,8 m	Refuerzo con dos geomallas

This test allowed to reproduce the conditions of the different situations in the center of each one of the sections. The upper part of the embankment was inscribed in a 13.5 m square side and lateral slope was arranged at 2H/1V. In figure 5, an aerial view of this embankment can be contemplated.

Figure 5. Experimental plot in la Virvée. ©COSEA/Pascal Le Doare

Regarding measured settlements, as you can see in figure 6, they remain below the calculated values, although this is reasonable since soil compressibility parameters taken in the calculation were worse than those measured in experimental plot. In August 2013, the driving of piles around experimental plot caused an increase of settlements in several sections. Although these new movements were stabilized, additional settlements generated by piling were not considered as representative of behavior of the embankment in real conditions.

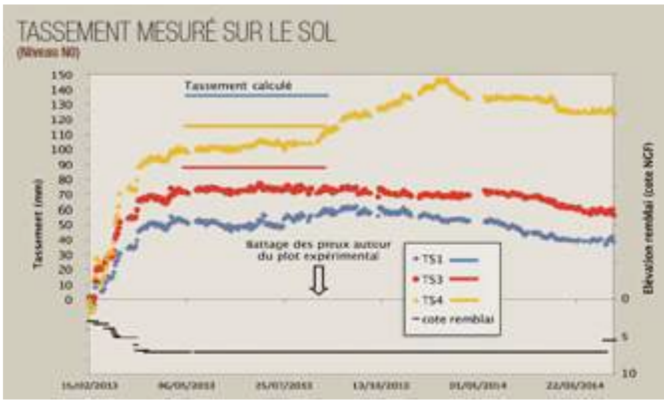

Figure 6. Measured settlements by section

Taking into account these results, the solution was validated. Depending on the level of appearance of lower granular layer, the final length adopted in the piles was 13 or 14 m, with a load transfer platform reinforced with two geogrids and a pile separation of 1.8 m × 1.8 m as seen in figure 7. Finally, 8400 units of type CK-270 were driven with a total measurement of 115920 ml.

Figure 7. View of pile mesh from experimental embankment

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LANDFILLS ON SOFT SOILS. ONE OR TWO LAYERS OF GEOGRIDS. INFLUENCE OF THE MODULUS OF ELASTICITY OF THE GEOGRID. CASES OF SUCCESS AND FAILURE

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In order to increase the factor of safety of an embankment over soft soils, during construction (short term) and/or operation (long term), geogrids or reinforced geosynthetics are very often used.

To design the type of geogrid required, Limit Equilibrium Methods and Finite Elements Methods can be used.

Limit Equilibrium Methods frequently assume that collapse will follow a particular assumed geometry: plane, circular, log spiral. With this assumption, each of the stability problems is reduced to one of finding the most dangerous position of the failure or slip surface of the shape chosen.

Finite Elements Methods are a numerical analysis with which obtains solutions for the different conditions of balance, compatibility, structural behaviour and boundary conditions, both forces and displacements. This type of method generates a mesh consisting of finite elements connected to each other by nodes. Each of these discretization elements is called a finite element. This type of analysis usually does not assume any type of collapse geometry.

In this case both methods, LEM and FEM, will be used to study the behaviour of an embankment over soft soil and the influence of using one or two layers of geogrids and the raw material used in the manufacturing of the geogrids. In this analysis two raw materials are considered, PET Polyester and PVA Polyvinyl Alcohol. The main differences between them are, PET has a strain at nominal tensile strength about 10-12% while PVA has 5-6%, besides PVA the range of application is 2<pH<13 and PET 4<pH<9.

Be supposed an embankment of 3 m height and slopes 2(H):1(V) and 10 m width at the top, with a surcharge of 10 kN/m2. Figure 1 shows the natural materials and the complete geometry of this hypothesis.

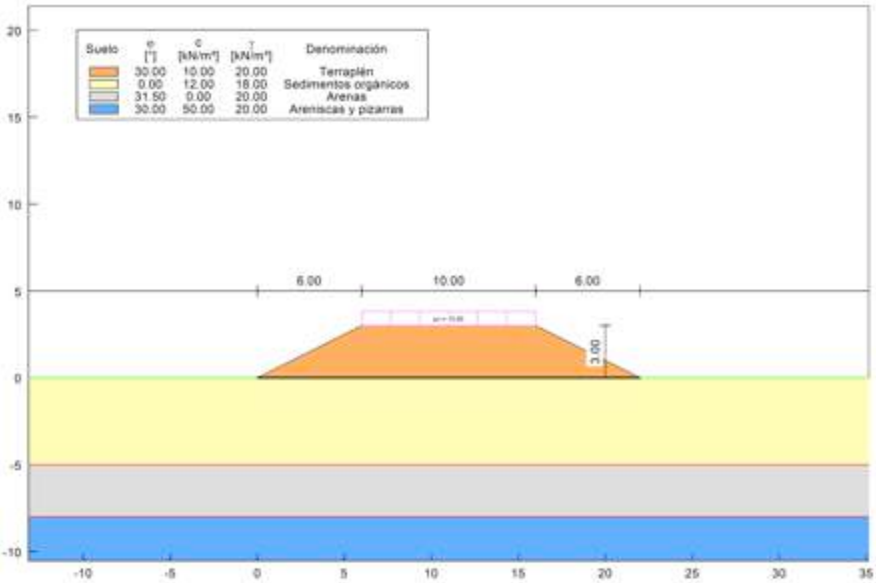

Figure 1. Embankment over soft soils

Using a LEM, in this case Bishop's method, the following Factor of Safety in are obtained for one geogrid at the bottom of the embankment, Figure 2, and for two layers of geogrids with a distance between them de 0.5 m, Figure 3. The geogrid chosen in the case of one layer has and Ultimate Tensile Strength (UTS) of 234.7 kN/m.

Each of the geogrids chosen in the case of two layers have and UTS of 111.5 kN/m. In this analysis only is taken into account the strength of the geogrid.

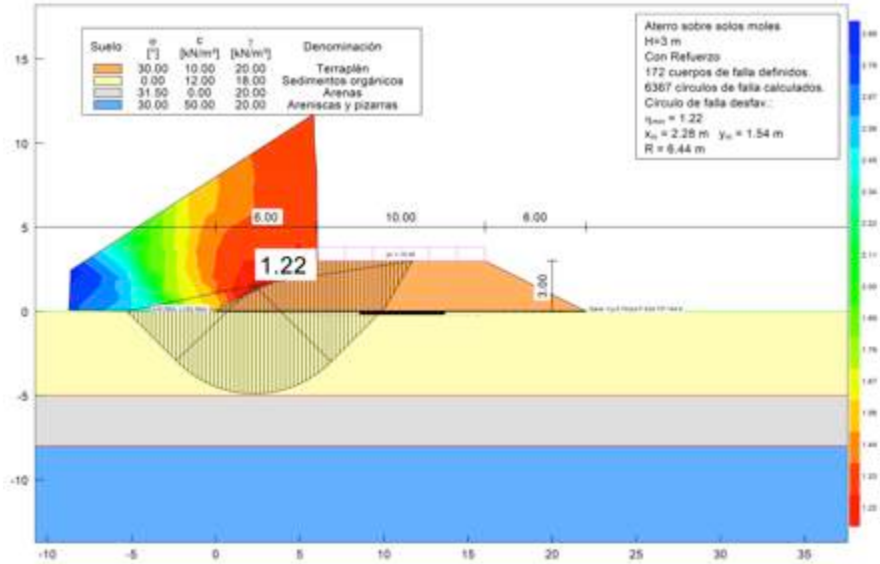

Figure 2. FS one geogrid using LEM

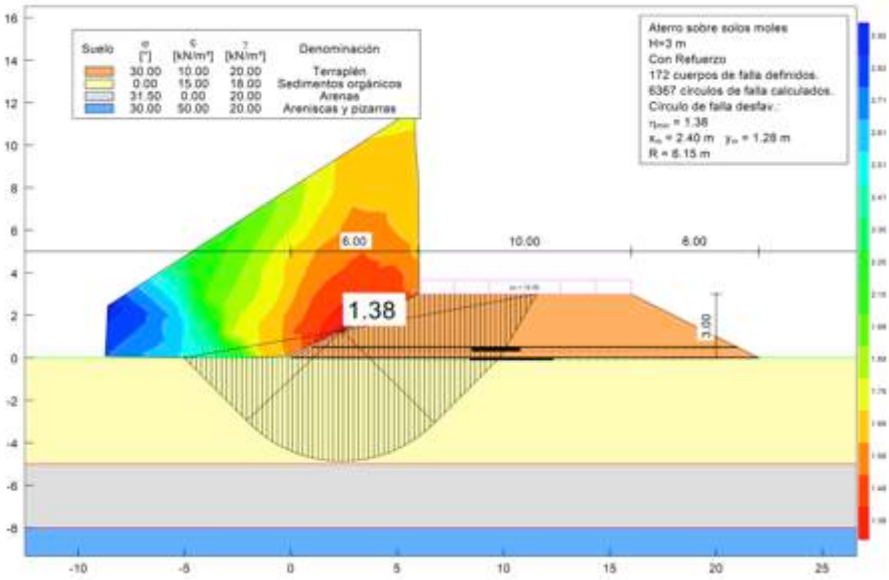

Figure 3. FS two geogrids using LEM

With the FEM, four analyses will be done. First design will be with one geogrid of PET with and UTS of 234.7 kN/m, Figure 4. Second analysis will be with two layers of PET geogrids with a distance between them of 0.5 m and an UTS of 111.5 kN/m, Figure 5. Third design will be again with two layers of geogrids with a distance between them of 0.5 m, the one at the bottom with PET and the other inside the embankment with PVA, both geogrids with an UTS of 111.5 kN/m, Figure 6. Finally, fourth analysis will be with one geogrid of PVA with and UTS of 234.7 kN/m, Figure 7.

Table 1 shows a summary of all previous FEM analysis.

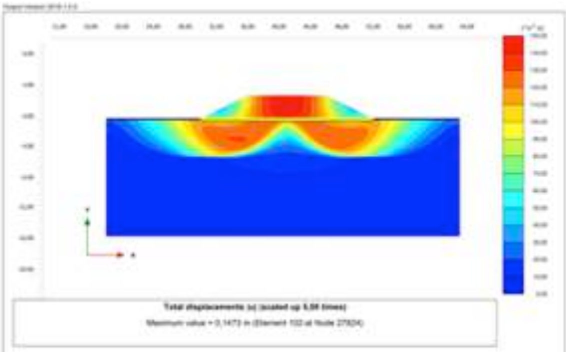

Figure 4. FEM analysis one PET geogrid

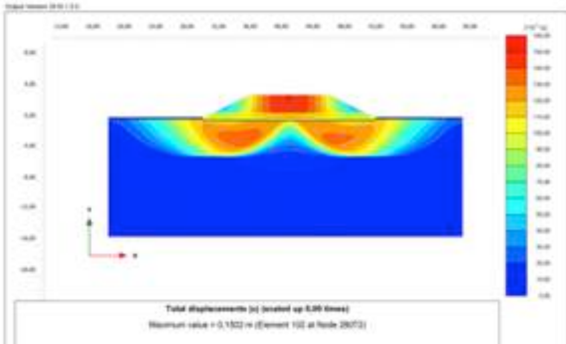

Figure 5. FEM analysis two PET geogrids

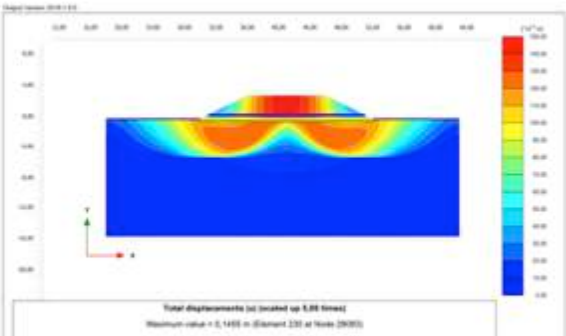

Figure 6. FEM analysis two geogrids PET and PVA

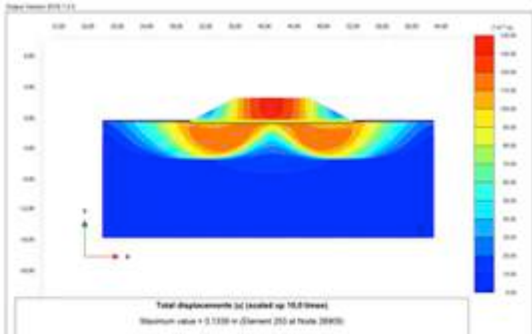

Figure 7. FEM analysis one PVA geogrid

Table 1

<table>
<tr><th></th><th></th><th>UTS (kN/m)</th><th>J (kN/m)</th><th>Geogrid Tensile Strength from FEM Analysis (kN/m)</th><th>Settlement (m)</th><th>FS from FEM Analysis</th></tr>
<tr><td colspan="2">One PET Geogrid</td><td>234.7</td><td>3400</td><td>174.2</td><td>0.147</td><td>1.22</td></tr>
<tr><td colspan="2" rowspan="2">Two PET Geogrids</td><td rowspan="2">111.5</td><td rowspan="2">1700</td><td>Bottom 111.5</td><td rowspan="2">0.15</td><td rowspan="2">1.21</td></tr>
<tr><td>Top 78.54</td></tr>
<tr><td rowspan="2">Two Geogrids</td><td>Bottom PET</td><td>111.5</td><td>1700</td><td>92.66</td><td rowspan="2">0.145</td><td rowspan="2">1.21</td></tr>
<tr><td>Top PVA</td><td>111.5</td><td>3400</td><td>111.5</td></tr>
<tr><td colspan="2">One PVA Geogrid</td><td>234.7</td><td>6800</td><td>178.7</td><td>0.133</td><td>1.22</td></tr>
</table>

Conclusions:

- Factor of Safety in the case of one geogrid at the bottom of the embankment, with PET or PVA, for both methods, LEM and FEM, is the same, 1.22.
- There is a big difference in Factor of Safety between LEM and FEM in the case of two geogrids. For LEM the FS is 1.38. In FEM, in the case of two layers of PET geogrids FS is 1.21, and the same for the case of one PET geogrid + one PVA geogrid. The previous difference is due to fact LEM has to be given the tensile strength of the geogrid (UTS). In FEM the tensile strength is connected with the strain of the geogrids and the movements of the materials surrounding, so FEM analysis will take into account the stiffness of the natural materials present.
- There is a significant difference in the geogrid tensile strength, ≈30%, obtained from FEM analysis in the case of two layers of PET Geogrids. In this way the bottom geogrid reaches strength of 111.5 kN/m and the strength of the top geogrid is 78.54 kN/m. This is due to the differences in the stiffness of the natural soil surrounding. In the case of the bottom geogrid the soft soil below it is much less rigid than the soil at the top, and allows the geogrid to deform more and to reach more strength. So for the geogrid at the top the natural soil surrounding does not allow it to get a high strain, high strength. This

means that is totally recommended to use only one geogrids located at the bottom of the embankment instead of two layers.

- In the case PET geogrid is at the bottom and PVA geogrid is at the top, using a raw material like PVA with a higher elasticity module, allows the top geogrid to reach a higher strength, 111.5 kN/m in PVA and 92.66 kN/m for PET.
- Obviously, if the elasticity module of the geogrids is increased, using PVA instead of PET, the settlements of the embankment are reduced and the geogrid tensile strength obtained from FEM increases. In this case the FS remains in 1.22. So when settlements are important and/or low embankments is totally recommended to use PVA geogrids.

All the previous conclusion will be seen in a real case the Autobahn A26 Hamburg-Stade, Figure 8.

Figure 8. Plan view Autobahn A26 Hamburg-Stade

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MODULE II

Latest soil improvement techniques

SOIL IMPROVEMENT BY JET GROUTING FOR THE CONSTRUCTION OF THE ACCESS TO THE BARCELONA AIRPORT APPLICATION OF THE RECENT TECHNOLOGIES

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INTRODUCTION

With its famous football team and history as an Olympic city, Barcelona is no stranger to breaking records. And the jet grouting project at Barcelona airport is just one more on the list for this unique centre of culture and sport.

This very ambitious project consists of the construction of a new railway platform that will provide optimum accessibility to the airport, which will translate into further increase in passenger numbers of the Barcelona-El Prat Airport.

A large part of the 4,5 km of the route will run through a tunnel measuring 2,8 km long, excavated using an EPB-type tunnel-boring machine. The rest of the tunnel and intermodal station will be formed by two parallel D-walls and a jet grouting slab to avoid water inflow during excavation and to reduce D-wall depth.

In this presentation will be exposed the most important details of jet grouting application in this project, including design focus, field tests results and quality assurance. Equipment and latest technologies developed by Keller will be also presented.

JET GROUTING PROCESS

Jet grouting is a well-known method which was introduced to the field of geotechnical engineering more than 40 years ago; it primarily acts in the ground either as a mean of stabilization or as a sealing structure. With the aid of high pressure cutting jets of water or cement suspension, having a nozzle exit with a velocity ≥100 m/sec (Kimpritis et. al. 2014), sometimes air-shrouded, the soil around the borehole is eroded. The eroded soil is rearranged and mixed with the cement suspension. The result is a structured element 'Soilcrete' column, which has improved mechanical characteristics compared with the original soil.

Jet grouting has been introduced to the European market in the 70ies of the last century. Its versatility and flexibility together with its applications, in almost every soil formation, makes it a perfect solution for complex geotechnical problems. It is effective in open field as well as in confined space with limited headroom, since the column diameter does not correspond to the size of the rig. The salient feature of this technology is that from relatively small boreholes, columns in the range of several metres can be formed and almost arbitrary geometries can be composed out of them.

Over this long period of application, a lot of experience was gained with this technology and enormous progress was made pushing back boundaries and limitations for its application. Nowadays, it is used for various depths, even more than 50 m, and column diameters of more than 3 m are often common.

In the last decade, the unique features of this technology were used in almost all high profile transportation and infrastructure projects in Europe in order to facilitate the construction process and to improve the level of safety and efficiency.

Due to the drilling (293.400 m) and jetting lengths (aprox. 85.000 m), and cement grout pumping, this jet grouting project is a record in Spain and in fact, it is one of the largest ever performed in Europe.

ADOPTED SOLUTION

In order to limit water inflow during excavation, reduce D-wall depth and guaranty excavation stability along 1.000 m of the tunnel and within the whole area of the underground train terminal and runway area, Keller proposed and executed soil improvement by jet grouting. The treatment was performed mainly in fine soils to a depths up to 35,0 m according to the geometry and depth of the corresponding underground structures.

Considering various factors as the soil characteristics, depth, geometrical efficiency and possible deviation during drilling, column diameter of 1,80 m was adopted and triangular grid of 1,35 x 1,35m. Based on analytical and FEM analysis the characteristic strength value was calculated to be 3,5 MPa.

The first part of the project required the construction of a jet grouting sealing and strutting slab of 2,5 to 5,5 m between parallel D-walls constructed previously (see fig. 1a); in total 19.230 m of jetting and 68.400 m of drilling. The second part of the project was carried out along the airport runway area with objective to improve soil conditions for the posterior tunnel construction by TBM (see fig. 1b and 1c); in total 17.362 m of jetting and 39.000 m of drilling.

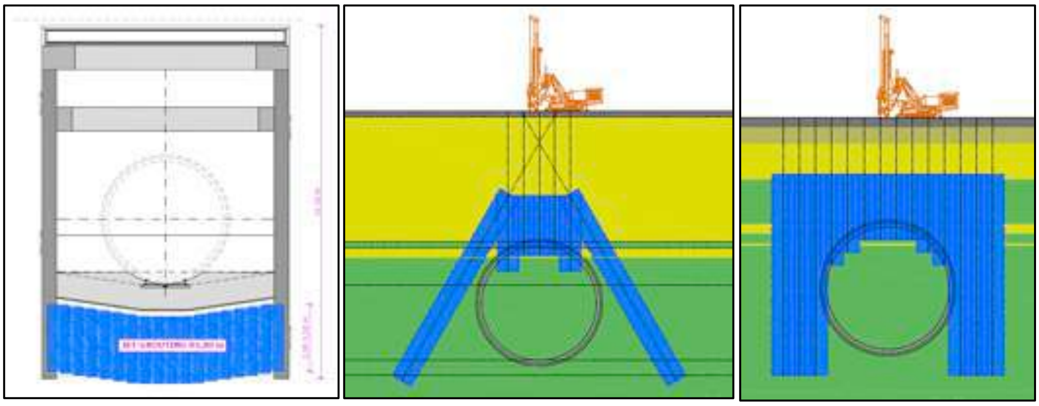

Figure 1. Typical sections jet grouting treatment: a) parallel D-wall tunnel area; b) and c) airport runway area.

The final part comprised the construction of the jet grouting strutting and sealing slab beneath the new intermodal underground station (see fig. 2); in total 48.400 m of jetting and 186.600 m of drilling.

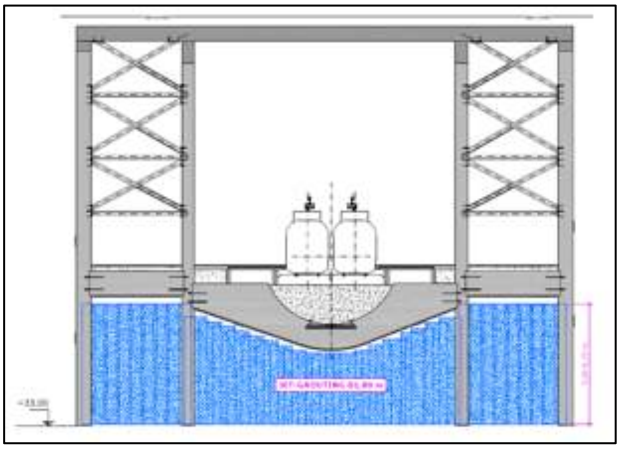

Figure 2. Typical sections jet grouting treatment new intermodal station.

FIELD TEST AND QUALITY CONTROL

Before the commencement of the works, a specific field test programme was executed in order to define the parameters of the jet grouting technique (jet grouting system, grout pressure, monitors and nozzle, flow rate, lifting speed of the monitor, rotations of the monitor per minute and the water cement ratio, the diameter of the columns, deviation control and the required strength and permeability). Numerous trial columns were performed and latest technologies developed by Keller were applied: ACI® (Acoustic Column Inspector for diameter control), IncliJet® (control of deviation or verticality), wet sampler, DX and DS monitors, high capacity pumps and plants, etc.

The diameter of the proposed jet grouting elements was confirmed using the ACI®, and at a later stage core drilling method was implemented (core drillings through the jet grouting body). Regarding the strength, the characteristic value was checked by testing wet samples (fresh jet grouting material from inside the column) and cores. Since the wet samples were obtained directing after the construction of the jet grouting elements, there was the opportunity to check the compression strength of the jet grouting body in an early phase, for instance at 3 and 7 days and estimate final strength after 28 days or more.

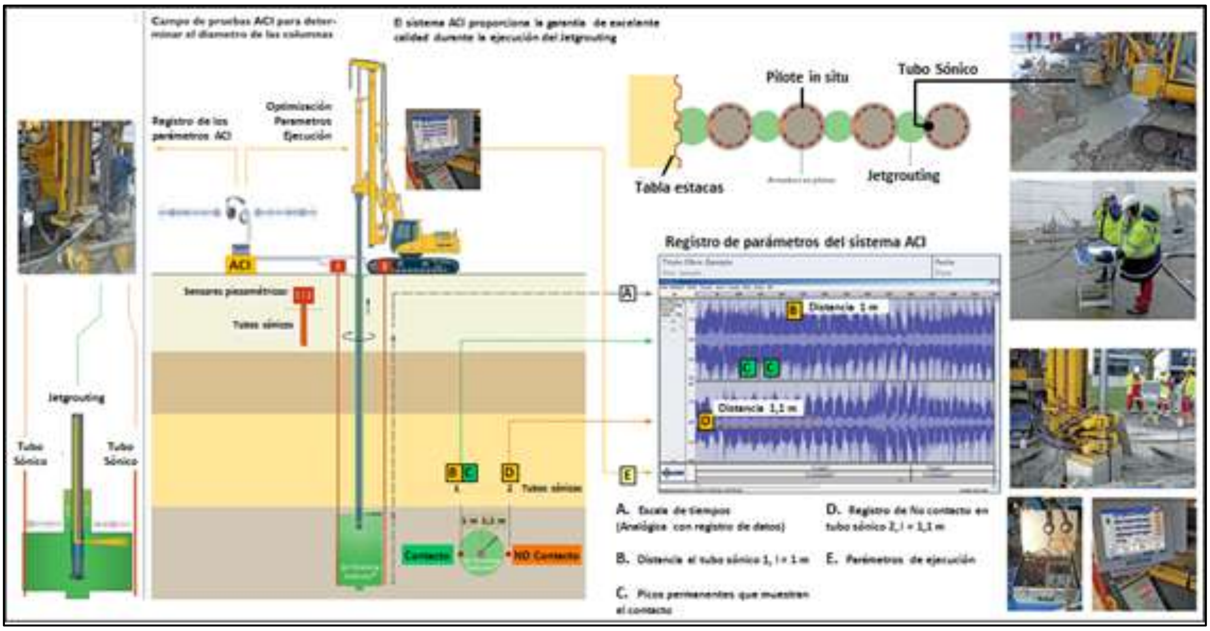

Figure 3. Jet grouting column diameter control by ACI-Acoustic Column Inspector System (Vukotić et. al. 2016).

During the execution of the project, a detailed and extensive quality control programme was implemented in order to ensure the safe tunnel construction and passage of the TBM. Four trial field test in different areas of the project were performed. It is important to emphasize that the drilling deviation by IncliJet was measured in more than 50% of the total drilling length in order to verify on one hand the actual position of the jet grouting elements at the base level of the jet grouting slab and on the other hand to determine the position for possible additional columns if necessary.

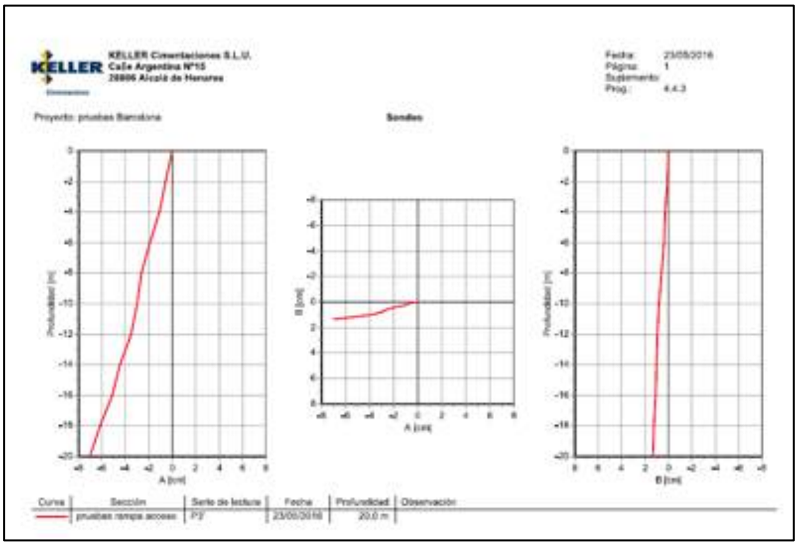

Figure 4. Deviation control by IncliJet.

The best confirmation of well performed works and quality control was the excavation process and tunnel construction in general; it was confirmed that all results were within allowable established limits. It is worthy to mention that water inflow was not detected at any point as the jet grouting sealing slab was formed adequately.

EQUIPMENT AND EXECUTION

Jet grouting work began in May 2016 and was completed in April 2018. During that time, Keller drilled 293.400 m, jetted 85.000 m and deployed four rigs on double shifts, six days per week, alongside four modern and high capacity Tecniwell pumps and plants. Spoil management and materials were supplied by our client, totalling over 170.000 tonnes of cement.

Photo 1 and 2. Keller equipment for jet grouting with the must of aprox. 35 m.

Once more it is confirmed that jet grouting is a very efficient and effective technique to solve complex geotechnical problems in transportation infrastructure. Decades of experience with this method allow for design and execution on a high level delivering a reliable and impressive level of quality and safety. There is also always space for improvement and further development which constantly occurs within Keller Group.

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SUCCESSFUL MENARD VACUUM TRIAL AREA IN THE NEW MEXICO CITY AIRPORT

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Mexico City's New International Airport (NAICM) is being constructed *(*)* and is intended to replace Benito Juárez International Airport, which is at full capacity. At its first phase in operation in October 2020, the new airport will include one large X-shaped terminal of 743,000 m^2 and three runways over a total area of 4 430 hectares. The new airport is being built to the northeast of Mexico City, in the Zona Federal del Lago de Texcoco (part of the dry lake bed of Lake Texcoco). Given the proximity to the current airport (AICM), about 3 miles away, the opening of NAICM requires the complete shutdown of AICM and the immediate transfer of operations to the new airport.

The soil conditions in the lakebed of Lake Texcoco are particularly poor with a cumulated thickness of roughly 40 m of soft clay exhibiting water content up to 275%, void ratio up to 9.5 and Young modulus from 0.3 to 0.5 MPa. Ground improvement works have thus been required to achieve the technical specifications for runways and connecting areas.

After several trial areas testing different ground improvement solutions (sand drains, pre-fabricated wick drains, rigid inclusions), it has been decided to build the runways after classical preloading including the use of Prefabricated Vertical Drains (PVD). Runways 2 and 3 have already been consolidated using this technique.

Classical preloading requires a large amount of fill material to apply the surcharge and compensate the high settlements induced by the soft soils. To save material and time, and also because it was not possible to install all the material preloading due to lack of space near the terminal building, Vacuum preloading was then envisaged for the connection areas between the runways and the terminal which is built on deep piles.

Vacuum preloading utilizes the atmospheric pressure to consolidate soft saturated silts and clays by similar principles as those used in surcharge pre-loading by vertical drains and embankment. The pre-load is applied by Vacuum load (typically 0.6 to 0.8 bars) equivalent to a 3 to 4 m embankment (γ = 20 kN/m^3).

Two trial test areas were then performed in order to compare the two main different ways of applying Vacuum:

1. The Menard Vacuum Consolidation (MVC) method. It consists of associating a vertical and horizontal draining system under an airtight impervious membrane with a Vacuum pumping system (Figures 1 and 2). It should be mentioned that the horizontal drains do not have a direct mechanical connection with the vertical drains; the connection is made through the draining layer below the membrane, which gives freedom of movements without any risk of disconnections;
2. The drain to drain (membrane less) method. It consists of connecting horizontal impervious plastic pipes to vertical drains, the first 3 m of the vertical drains being sealed by an impermeable plastic tape to reduce the risk of air inflow from ground surface. Horizontal pipes are linked to Vacuum pumps (Figure 1).

The only difference between the two trial areas lies in the way of applying the Vacuum pressure (MVC or drain to drain method), all the remaining conditions being the same ones for both trial test areas. Indeed, the soil conditions are similar as the trial test areas are very close. Footprint areas don't differ much: 76 m x 56 m for the drain to drain method and 70 m x 50 m for the MVC method. In both cases, Prefabricated Vertical Drains are installed at 1.2 m interval on a triangular grid and to a depth of 28 m from Ground Level, the loading fill consists of 2 m thick low density volcanic material ("Tezontle") corresponding to an overall vertical pressure around 25 kPa and the Vacuum pressure is applied during 180 days.

Both trial tests areas were extensively monitored in order to follow-up Vacuum pressure at the pumps and inside the soft soil, pore water pressures, settlements and lateral displacements. It should be mentioned that the atmospheric pressure at the elevation of the Texcoco Lake (2228 m a.s.l.) is 78 kPa leading to a maximal Vacuum pressure of 78 kPa assuming no head loss in the system.

Menard Vacuum Consolidation method — Drain to Drain method (membrane less method)

Figure 1. Two Vacuum trial areas in NAICM

The main results observed for the Menard Vacuum Consolidation trial area are summarized hereafter.

Vacuum pressure

The sensors for depression measurements were distributed over the entire surface of the test area and installed directly under the membrane. They were connected to a Vacuum gauge located outside the area, which allowed to know during 180 days the Vacuum pressure applied directly to the ground. At the beginning, the pumping system had an efficiency of more than 92% (72 kPa of 78 kPa possible). Subsequently, the values tended to stabilize, maintaining a homogeneous pressure on the entire platform of around 63 kPa. The decrease of the Vacuum pressure was caused by the settlement of the ground, generating head losses between the pump (outside the area, not settling and remaining at 75 kPa all along the test) and the ground.

During the 6 months duration of the test, the average Vacuum pressure applied was therefore more than 60 kPa, which would be equivalent to the weight of a 3 meters high basalt embankment (γ = 20 kN/m^3). Compared to classical embankment preloading, the Menard Vacuum Consolidation method has the advantage that the pressure is applied in all directions (isotropic), and without loss against depth (Figure 2).

Pore Water Pressures

The electric piezometers allowed to monitor the pore water pressures inside the treatment area. The Vacuum pressure generated a decrease of the pore water pressure with time, inducing an increase of the effective stresses in the same proportion. The rate of the water pore pressure decrease is directly linked to the PVD spacing, the distance of the sensor to the PVD and the soil layer permeability; some variations of the decrease magnitude with depth were therefore observed, whereas the Vacuum pressure was constant over the whole depth.
Once the pump system was shut off and the outlet valves of the horizontal drains were closed, it was observed that the pore water pressures continued to decrease in all the piezometers, this being a proof of the impermeability of the system. The system was capable of maintaining the Vacuum which was trapped under the membrane and the soil continued to consolidate.
It should also be mentioned that the water table level and the pore water pressures did not vary outside the 70 m x 50 m area of Vacuum application.

Lateral displacements

Inclinometers were installed at the edge of the trial test area in order to record the lateral displacements during all the trial test period. During the installation of the "Tezontle" filling (2.0 m), the inclinometers registered outwards lateral movements (towards the outside of the trial area), which is the expected natural behaviour of soft soil under embankment. On the contrary, the inclinometers registered inwards lateral deformations (towards the inside of the trial area) during the Vacuum consolidation, highlighting the isotropic benefit from the Vacuum system (Figure 2).

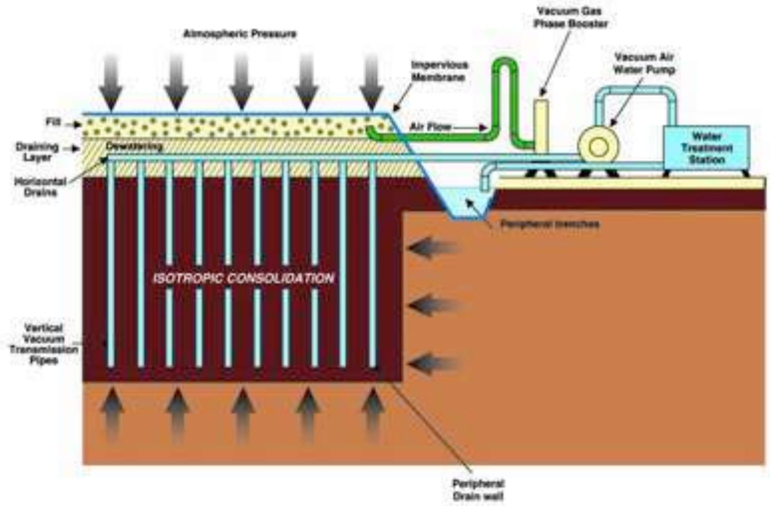

Figure 2. Menard Vacuum Consolidation method – Consolidation is isotropic

Settlements

The settlement plates allowed to record the total settlements at different points inside and outside the trial area. At the beginning of the Vacuum application, the settlements reached speeds of up to 5 cm per day, and then the settlement speed decreased with time. That is perfectly in agreement with the consolidation theory.

Compared to classical embankment preloading, the settlement was almost flat, with no significant differential settlement between the centre and the edges of the trial test area. It is because the Vacuum pressure acts homogeneously throughout the soil mass, without any reduction in depth or width.

Magnetic extensometers were located at different depths to be able to interpret the behaviour of the different sub-layers and to know the profile of deformation with depth. This allowed verifying that the layers with the highest water content present the largest settlements, as well as verifying that the cumulative settlement of all the sub-layers coincided with that recorded in the surface settlement plates.

A final settlement of 2.95 m (Figure 3) was recorded at the end of the period test (6 months), corresponding to an average degree of consolidation of 46 % (a back-calculation based on the measurements leads to a long-term settlement of 6.3 m). Settlements measured both on the ground surface and at depth were in good agreement with predictions based on analytical analysis (the degree of consolidation was expected to be 53%).

Outside the trial test area, settlements were negligible highlighting that the Menard Vacuum Consolidation method applied fully and only over the PVD treated area.

Conclusions

The Menard Vacuum Consolidation method uses the site atmospheric pressure to pre-load an impermeable soil in order to anticipate long-term surface deformations and to "prepare" the soil to withstand the future service loads. The Menard Vacuum Consolidation trial area proved the efficiency and effectiveness of this ground improvement technique and allowed to highlight the membrane solution advantages over the drain to drain method which generated not more than 2.0 m of settlement in absolutely the same conditions (Figure 3).

The Menard Vacuum Consolidation method is currently (*) applied to all the connecting areas between the runways and the terminal on the New Mexico City International Airport (NAICM).

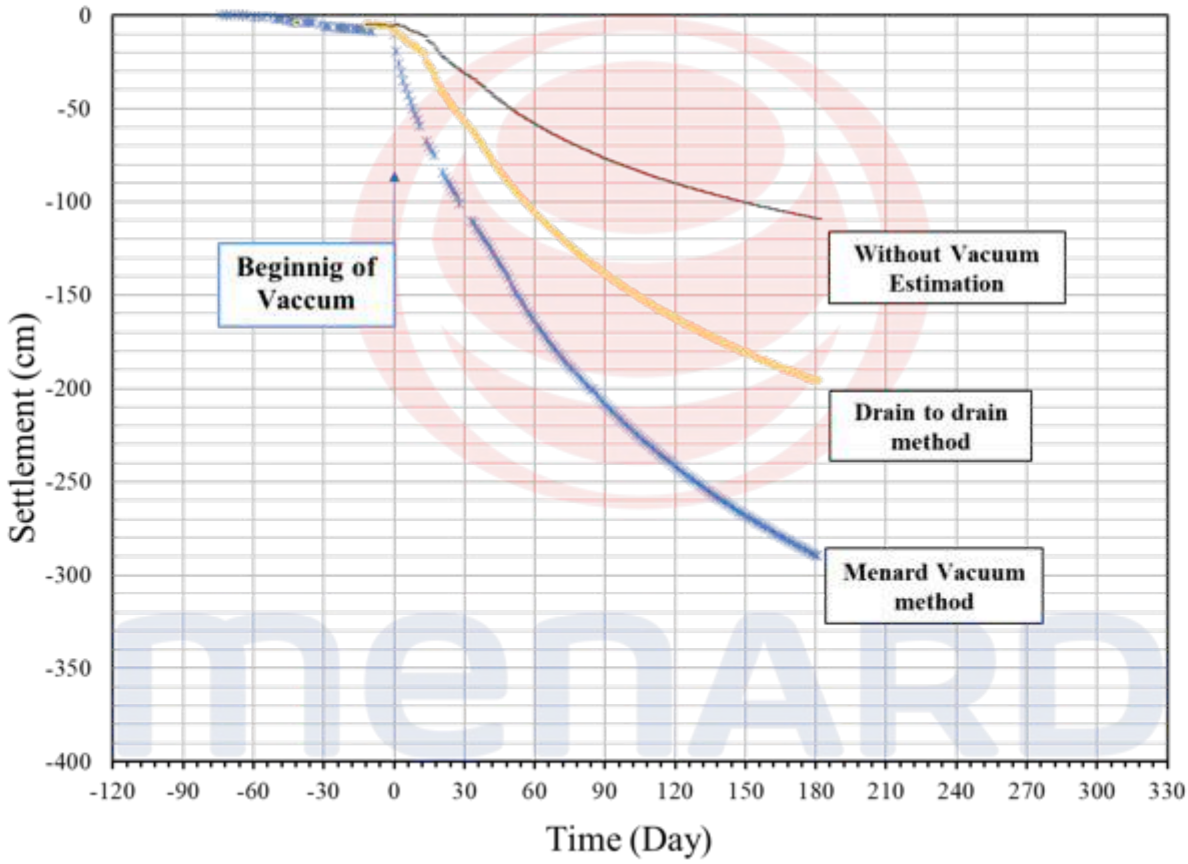

Figure 3. Menard Vacuum Consolidation method – Consolidation is isotropic

(*) On October 30, 2018, the project was set to be cancelled after a popular consultation but this decision has not yet been formally made and works continue even after the consultation. In the consultation it was decided that a new airport will be built on the grounds of Santa Lucía Air Force Base instead. As of January 17, 2019, the airport is still under construction and work has not been cancelled.

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CHALLENGES IN GROUND IMPROVEMENT RESEARCH

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Many ground improvement methods were applied successfully in the last decades. However, a lot of details are still not known. Some research ideas and projects which have been started are presented. Topics are vibro compaction, vibro stone columns and jet grouting.

1. Optimization of vibro compaction

Vibro compaction is used to compact loose sands by means of a depth vibrator. The success of vibro compaction depends on the quality control (motor amperage) and on the skill of the crane driver. There is no automatic control of amperage or compaction time.

In this research project the frequency of the motor is controlled automatically to maintain large amplitudes by means of resonance. Based on Fellin 1999 calculations with a mass-spring-damping system of a Keller S-vibrator yielded that resonance depends on the depth of the vibrator and on soil density (Wehr 2005). Resonance is not possible for frequencies smaller than 20 Hz because the soil density cannot be looser than a certain limit. On the upper end frequencies larger than 30 Hz are not possible because this is the maximum working frequency of the depth vibrator. Attempts to control the resonance frequency by the crane operator failed. Therefore a sophisticated control algorithm is developed.

To elaborate this algorithm a model test device was constructed and a model vibrator was developed, see fig 1. Loose sand is pumped from one container to the other one.

Figure 1: model test device at Erfurt university of applied sciences

The model vibrator and the soil is equipped with different sensors in order to measure accelerations and the phase angle. The aim is to compact soil more effectively to save time and money. First results display higher vibrator amplitudes in the soil than in the air, if the phase angle is approximately 90 degrees which means resonance.

2. Grain crushing due to depth vibrators

Vibro stone columns are installed since the 1950ies successfully but the influence of grain crushing has never been subject to systematic research. The amount of grain crushing is important because it may influence the soil properties like friction angle and Youngs modulus.

There are very suitable material laws like hypoplasticity to describe the soil behaviour. However, grain crushing cannot be take into account realistically. In this research simple laboratory test with different sands are carried out varying systematically the average grain diameter d50 and the uniformity coefficient U. In this way a database is created to propose modifications of the material law.

The main idea is to extract crushed gravel from already constructed stone columns first and to compare the grain size distribution with the original distribution from the gravel delivered to site. The original gravel was treated in the laboratory with different Proctor energies until the grain size distribution of the stone column was reached. By modifying the compaction energy with a special Proctor device in the laboratory it was therefore possible to simulate the compaction energy of a Keller M-Vibrator successfully. This energy was found to be 3.24 MNm/m³. Further tests will be reported elsewhere.

3. Filter stability of vibro stone columns

In previous research field tests were evaluated measuring the pore water pressure around stone columns. If the pore water pressure becomes very high small soil particles of the cohesive soil around the columns may be eroded and the pore volume of the column may be filled with these particles blocking the drainage paths of the columns. The question is which is the critical hydraulic gradient where erosion starts (Zou et al 2010).

Permeability tests were executed in the laboratory in order to measure the critical hydraulic gradient. With various test devices using a boundary between cohesive and granular soils it was difficult to detect the exact hydraulic gradient.

Therefore a new test device was developed with an acrylic glass window just on top of the soil sample. If a considerable amount of small soil particles could be detected be behind the glass the critical hydraulic gradient is reached, see fig. 2.

Figure 2. Many small soil particles behind the glass: critical hydraulic gradient

Two series of tests were designed using A) a horizontal boundary with clay at the bottom and sand on top and B) a vertical boundary with a clay column in the middle and sand around the clay. The water pressure was increased in steps until the critical hydraulic gradient was reached. It was possible as well to change the surcharge from 10 kN/m² to 600 kN/m² simulating long columns deep in the soil. After the tests the results were compared to the analytical solution by Zou 2010. The detailed test results will be reported elsewhere.

4. Design of the diameter of jet grouting columns

Jet grouting is used to construct soil-cement columns in the soil. The erosion process in the soil with high pressure is very complex and no design formula to estimate the diameter exists so far. Extensive laboratory tests were made by Bergschneider 2002 and Stein 2004 developing different approaches to estimate the column diameter.

Bergschneider considered only the parameters influencing the maximum possible column diameter. However, he did not consider the influence of pore water pressure. Stein was more interested in the development of the diameter vs. time. Both researchers concentrated on jet grouting in sands.

In this research both approaches are combined and missing parameters like pore water pressure, permeability and cohesion were included.

Calculating the pore water pressure the conventional theory of consolidation was modified. It is important to consider soil permeability, stiffness and drainage conditions. It is calculated how long the built up of pore water pressure takes to predict the beginning of the erosion process. Without this knowledge the delay time to start erosion and therefore the diameter cannot be predicted in fine sand and cohesive soils.

Cohesion is taken into account along the shear zones in fig.3 which Bergschneider has proposed. Model tests are planned to confirm the theoretical findings.

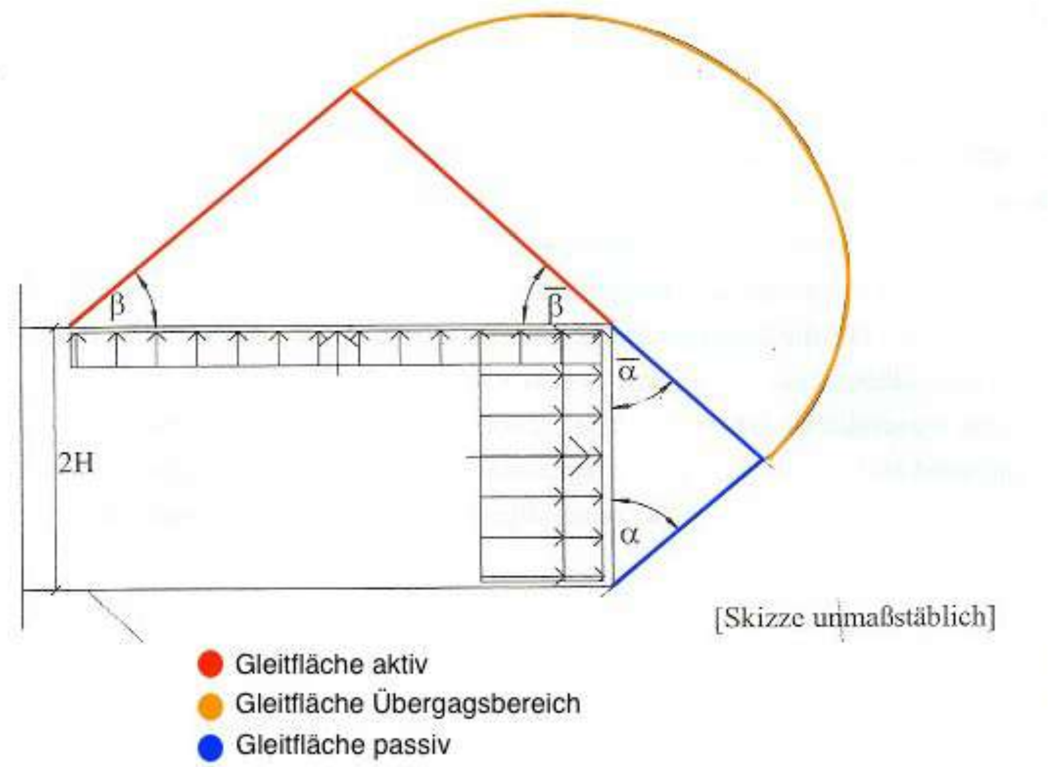

Fig 3. Considering cohesion in the shear zones (Gleitflächen)

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APPLICATION OF GEOTEXTILE ENCASED COLUMNS (GECS) IN EMBANKMENT OVER SOFT SOILS

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Embankments over soft soils supported by piles or stone columns have several advantages over the classical unsupported embankment foundation when compared in terms of bearing capacity, serviceability, and duration of construction stage. In extremely soft soil conditions, lateral support can be problematic for stone columns (Figure 1). An alternative system, which can both provide the required lateral support and increase bearing capacity, is the "Geosynthetic Encased Columns" (GECs). This system includes a high-modulus, creep resistant geotextile encasement called Ringtrac® that confines the compacted sand or gravel column thereby providing constructability and bearing capacity even in extremely soft soil. This paper provides a system description and the required design procedure.

Compacted gravel column techniques are usually limited to soft soils with undrained cohesion (undrained, unconsolidated shear strength) cu or su ≥ 15 kN/m². The problem was solved by confining the compacted sand or gravel column in a high-modulus geosynthetic encasement (Huesker's Ringtrac® GECs). The general idea of Geosynthetic Encased Columns (GECs) is shown in Figure 2. Development of the technology, design procedures and appropriate geosynthetics went hand-in-hand throughout the 1990's. The first project was built successfully in Germany around 1995. From the beginning of GEC's, a lot of successful projects have been completed in different countries.

The GEC system is now accepted as a proven foundation solution and is now included within German design recommendations.

Figure 1. Installation of GECs (Ringtrac) over water and extremely soft mud

The general concept remains the same as for conventional piled embankments: to take the load from the embankment and to transfer it directly through the soft soil down to a firm stratum. The main difference is that embankments on concrete, steel, and wooden piles are nearly settlement-free. If the design is appropriate, the compression stiffness of the piles is so high, that practically no settlement occurs at the level of pile tops or caps. High strength horizontal geosynthetic reinforcement is typically installed above the piles to bridge over the soft soil between piles and homogenize the embankment's deformations.

The vertical compressive behaviour of the GEC's is less rigid. The compacted vertical sand or gravel column starts to settle under load mainly due to radial outward deformation. The geosynthetic encasement, and to some extent the surrounding soft soil, provides a confining radial inward resistance acting similar to the confining ring in an oedometer, but being more extensible. The mobilization of ring-forces requires some radial extension of the encasement (usually in the range of 1 to 4 % strain in the ring direction) leading to some radial "spreading" deformation in the sand (gravel) columns and resulting consequently in vertical settlement of their top.

The GEC system therefore cannot be completely settlement-free. Fortunately, most of the settlement occurs during the construction stage and is compensated by some increase of embankment height. Finally, ensured by the strength and stiffness of sand or gravel, confining ring-force in the encasement and soft soil radial counter-pressure, a state of equilibrium is reached.

The specific characteristics of the GEC system are:

1. The primary function of the high-modular high-strength geotextile encasement is the radial confining reinforcement of the bearing (sand or gravel) column.

2. The secondary functions of the encasement are separation, filtration and drainage.

3. The system is not completely settlement-free.

4. The GEC is typically an end-bearing element transferring the loads to a firm underlying stratum.

5. The GECs are water-permeable; they practically do not influence the flow of groundwater streams, which has potential ecological advantages.

6. The GECs may also perform as high-capacity vertical drains, although it is not their primary function.

7. The geotextile encasement is a key bearing / reinforcing element, capable of meeting high quality engineered design standards and specifications.

8. It is strongly recommended to install horizontal geosynthetic reinforcement on top of the GECs (at the base of the embankment). The horizontal reinforcement is used for general stability, for transferring spreading forces or to facilitate load transfer into the columns, as well as to equalize settlements.

The geotextile-encased column (GECs) foundation system was specially developed for earthwork structures built on weak and very weak subsoil. It comprises uniformly arranged columns, filled with non-cohesive material and enclosed in a geosynthetic sleeve, which transmit the structural loads to the bearing stratum (Figure 2).

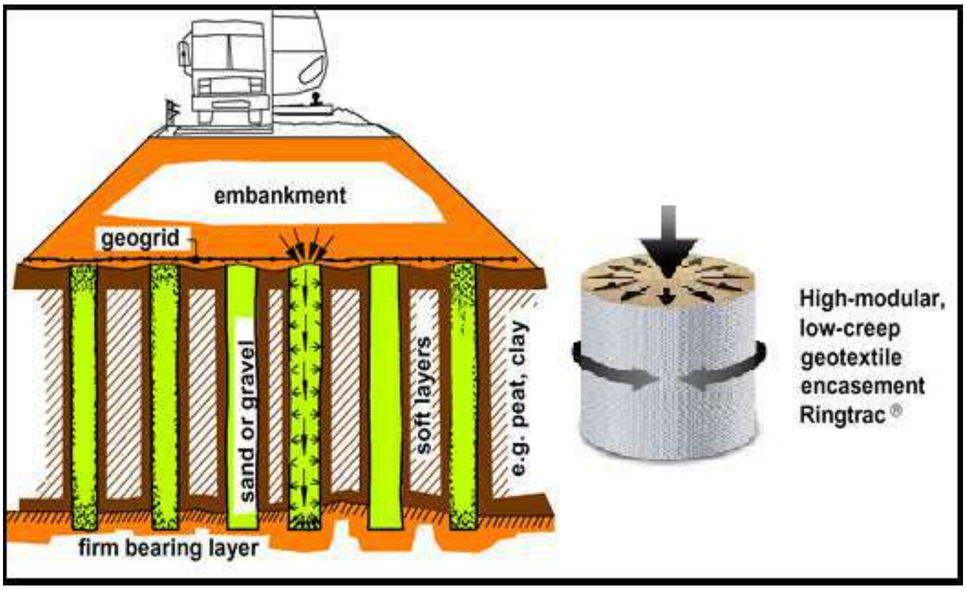

Figure 2. General idea of embankment on soft soil set on Geosynthetic Encased Columns (GECs)

The overall loads and stress concentrations above the column heads induce outwardly directed radial horizontal stresses in the columns. The particularity of the GEC system is that these stresses are counteracted not only by the inwardly acting pressure of the soft soil, but also – most importantly – by the radial resistance of the geotextile casing of high tensile stiffness (low radial extension). The substantial circumferential tensile forces generated in the casing provide radial support to the columns and ultimately safeguard the equilibrium of the system, thereby allowing its use even in very soft soils (and strictly speaking even in air with zero lateral soil support, Figure 3). The arrangement of geotextile-encased columns produces a ductile bearing system that is immune to buckling under the incident column loads. The use of GECs considerably reduces both absolute and differential settlement, while enhancing structural stability both during construction and after that.

Figure 3. Demonstration of the confining capability of high strength geotextile encasement for GEC "in air"

Two different options are generally available with regards to the GECs construction technology. The first option is the displacement method (Figure 4) where a closed-tip steel pipe is driven down into the soft soil followed by the insertion of the circular weave geotextile and sand or gravel backfill. The tip opens, the pipe is pulled upwards under optimized vibration designed to compact the column. The displacement method is commonly used for extremely soft soils (e.g. su < 15 kN/m²).

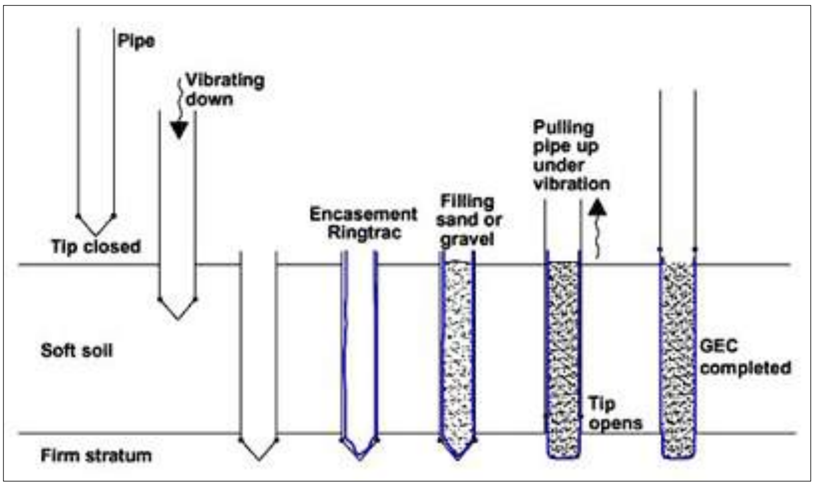

Figure 4. Displacement method of construction

The second construction option is the replacement method (Figure 5) with excavation of the soft soil inside the pipe. This method uses an open pipe where special tools remove the soil during or after driving the pipe down into the ground. The rest of the operation is identical to the displacement method. The excavation method is likely to be preferred with soils with high penetration resistance or when vibration effects on nearby buildings and road installations have to be minimised.

The advantage of the displacement method compared to the excavation method is based on the faster and more economical column installation and the effects of pre-stressing the soft soil. Furthermore, it is not necessary to excavate and dispose soil. The excess pore water pressure, the vibrations and deformations have to be considered.

Figure 5. Replacement method of construction

There are two options available when selecting the diameter of the circular weave geotextile (Ringtrac®). In the first option the diameter of the circular geotextile is slightly larger than the diameter of the steel pipe, thus allowing for a better mobilization of soft soil radial counter-pressure after extracting the pipe. The disadvantage is a larger column settlement based on the larger radial deformation due to an "unfolding" phase prior to mobilization of the geotextile's tensile modulus. In the second option, the diameter of the geotextile and the pipe are the same. This provides for a quick strain–tensile ring force mobilization, which results in less soft soil mobilization and higher ring- tensile forces, but in reduced settlement. The equal diameter option is preferred at present.

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DREDGING AND REUSE OF CONTAMINATED SEDIMENTS AT EMBRAPORT, SANTOS, BRAZIL

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Embraport is an 850,000 m2 container and bulk goods terminal, being constructed in the Port of Santos, Brazil. When completed, Embraport will not only be the largest privately owned port facility in Brazil with over 600,000 m2 dedicated to container storage alone, but it will be also the largest in South America. Further, the terminal will be able to turn over 2 million TEU (20 ft container equivalent units) and 2 billion litres of bulk liquids per year. The terminal is located on the North shore of the Estuário de Santos opposite the City of Santos in the State of São Paulo in an area that is primarily tidal flats.

In 2007 Embraport was granted a Government approval to develop the site for the terminal and port facility. The initial survey and soil borings indicated that more than 50% of the site was in the tidal flats with an elevation between -1.0 m and +1.0 m. All of the proposed terminal area was located over approximately 22 to 25 m of soft to medium sandy silt. To achieve the design platform elevation of +3.5 m and to account for anticipated settlements, it was determined that 1.5 million m3 of select fill would have to be imported to the site.

Further, as a condition of the Government approval the Embraport owners were also required to remove, dewater and dispose of 600,000 m3 of contaminated sediments that were located within the planned entrance channel and turning basin of the port. The source of the sediment contamination is due to heavy industrialization in the Santos region. Testing has shown a wide variety of contaminants in the sediments ranging from heavy metals (lead, copper, nickel, chromium and mercury) to PAH's and PCB's in varying concentrations.

The importation of 1.5 million m3 of select fill in conjunction with the removal of 600,000 m3 of contaminated sediments comprised a major cost implication for the Embraport owners and impacted the whole financial viability of the project. A highly innovative solution to this problem was found whereby Geotube® containers would be used to dewater and encapsulate the dredged contaminated sediments within the earthfill platform of the port. Once dewatered, the encapsulated sediments would then comprise the base for the container storage area. This solution proved highly attractive because not only did it significantly reduce the amount of imported fll required, but also saved on the cost of transporting the dredged contaminated sediments to an offsite disposal facility, or using part of the Embraport footprint for a disposal facility.

To evaluate the feasibility of using the dewatered contaminated sediments as part of the earthfill for the port platform a series of small scale dewatering tests were carried out on contaminated sediment samples to determine dewatering effectiveness, final solids concentration values and water effluent quality. The results of the tests demonstrated that with the correct dosage of chemical dewatering accelerant the contaminated sediments could be dewatered to a condition that would render them stable under the imposed surcharge loads and that the effluent water would remain consistent and require small additional treatment before release back into the environment.

A primary containment dyke was constructed around the port earthfill platform area to a level of +4.5 m. Within this, a woven geotextile separator was laid across the surface of the soft sandy silt foundation with a 0.5 m thick gravel drainage blanket placed over the top. This gravel layer performed two functions. First, it acted as a drainage layer to convey the effluent water from the dewatering contaminated sediments to an internal collection channel. Second, it acted as a drainage layer for the pore water drainage from the soft foundation during consolidation following placement of the surcharge. The dewatering area was then divided into two large cells by the construction of a secondary dyke to a level of +2.5 m.

The Geotube® dewatering containers were laid out in the first dewatering cell and connected the sediment slurry pipeline from the dredge. Dredging delivered the contaminated sediment slurry to the Geotube® containers at a rate of 1,400 m3/hr. Each Geotube® container was filled to a control height of 2.2 m before filling was stopped and the unit allowed to drawdown in height. This filling and drawdown process was repeated several times until the design objectives were achieved.

The effluent water flowing from the Geotube® units drained into a collection basin from where it was pumped to a water treatment basin. The pH of the effluent water was initially raised to precipitate out any dissolved solids. Next, the water was transferred to a second basin where the pH of the water was neutralized. The water was then passed through activated carbon filters and then released back into the environment.

The Geotube® units had a final dewatered height of about 1.8 m with each containing an average of 2,145 m3 of dewatered sediment. The Geotube® dewatering proved very effective with a 65% dewatered solids concentration achieved within 30 days after filling completion.

Once the dewatering operation was completed in the first cell the same dewatering procedure was repeated in the second cell. While dewatering was being performed in the second cell a gravel layer was placed around and over the filled Geotube® units in the first cell and then this was covered with surcharge to a level of +7.0 m. This surcharging consolidated the soft sandy silt foundation and the dewatered contaminated sediments to a level that met the settlement performance objectives of the earthfill platform. Once dewatering had been completed in the second cell the same surcharging procedure was repeated there.

Once foundation and sediment consolidation had reduced to an acceptable level the surcharge was removed down to a level of +4.1 m which coincided with the subgrade surface for the container terminal pavement. Following this, the container terminal pavement was constructed.

It was estimated that the 600,000 m3 of contaminated sediments, when dewatered and consolidated, provided a saving of 400,000 m3 of imported fill costs. This resulted in an overall saving of 20% to 30% of the earthfill platform cost for the terminal.

Client: Empresa Brasileira de Terminal Portuários SA, Santos, Brazil.

Dredging Contractor: Jan de Nul do Brasil Dragagem Ltda, Rio de Janeiro, Brazil.

Environmental Engineering and Project Management: Allonda Geossinteticos Ambientais Ltda, São Paulo, Brazil.

Figure 1. Geotube® dewatering of contaminated sediments

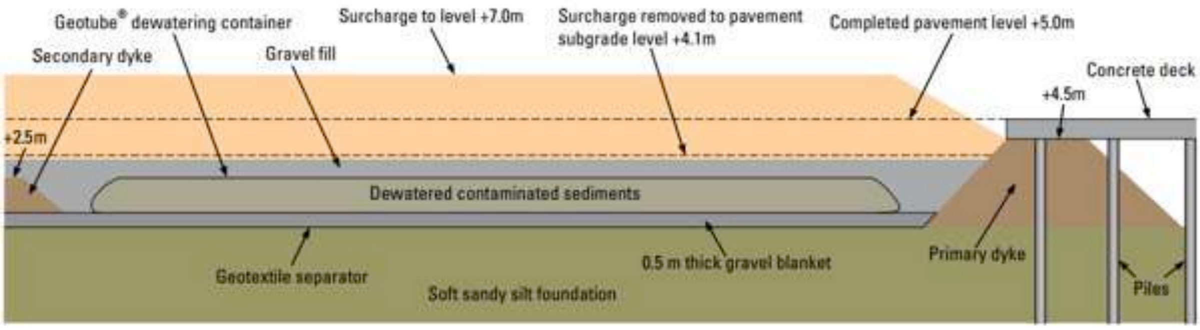

Figure 2. Section through the Embraport earthfill platform showing location of the insitu Geotube® dewatering of contaminated sediments

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THE USE OF 16 TON CDC COMPACTION FOR THE COMPACTION OF THE TRANSPORTATION ROUTE OF 13500 TON RAILWAY BRIDGE

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Introduction

A considerable stretch of one of the major highways in The Netherlands, the A1 between Amsterdam and Muiderberg, has been upgraded with additional lanes to a 5x5 highway. Part of the project was the span enlargement and replacement of a railway bridge crossing the highway. As the busy railway (12 trains an hour each direction) and highway (over 200.000 cars each day) were required to remain operational during construction, a steel bridge was assembled on a temporary location adjacent to the highway near its final location. After partial completion, the bridge with at that time a design load of 13500 ton, almost twice the steel weight of the Eiffel tower, was transported using two 500m^2 groups of Self Propelled Modular Transporters (SPMT's) over a length of 380m to a temporary foundation next to the old bridge where the concrete deck and railway infrastructure was added. From this position, the bridge was horizontally jacked to its permanent position to become part of the existing railway track. Figure 1 present the route of the bridge.

Figure 1. Overview of the site to be compacted

Soil conditions and requirements

The soil profile below the route consisted of 3 to 4 meters of Holocene peat underlain by a thick glacial sand deposit. In the past the peat at the location of the old highway was already replaced by loosely packed sand.

A soil replacement was designed for the sections where peat layers were still present to provide a solid base for the transport. On the bases of a critical state soil mechanics framework, a minimum compaction requirement was formulated for the sandy base underneath the path. The very high loads induced by the two SPMT-groups together with the low tolerances in allowable differential deformation during transportation resulted in stringent requirements with a strong dilative response of the sand. A requirement of 85% relative density over the top 6.5m was set for the compaction.

Compaction operations

The heavy 16 tons Rapid Impact Compaction technique (RIC) called Cofra Dynamic Compaction (CDC) with a spherical zone of influence in depth up to 9m was used. The method was chosen as most suitable for this project because of the following reasons:

- The depth of influence coincided with the requirements (i.e. 85% RD over the top 6.5m).
- The ability to create homogeneous compaction by the use of a stop criteria based on the reaction of the foot to the impact energy.

- The high production rates of the equipment. These were required for the compaction of the highway stretches in weekend closures.
- The safe work method with no free falling objects as compaction neede to be performed within 2 meter of the operational main highway.
- The good quality control system in place to monitor the compaction.

Figure 2. Compaction operations next and on stretches of the highway

Pore pressures

Compaction often took place just after the soil replacement, in which very loose sand was placed under the water level with a high void ratio and water content. Due to the limited extent of the soil replacements, the loose method of placement (dumped backwards into waterfilled holes), the peat at the sides being left in place and the large strains due to very fast compaction operations (a location takes less than a minute to compact) several issues arose with large exces spore pressures in the sand. Under normal conditions a special work method would be chosen leaving time to dissipate the exces spore pressures, in this project this was not possible and therefore Prefabricated Vertical Drains (PVD's) were succesfully used to dissipate the excess water.

Figure 3 shows a drain that is discharging water at a high rate due to nearby compaction. The drains continued with the high discharge rate hours after compaction was performed.

Figure 3. Vertical drains discharging compaction induced excess pore water

Quality control

The onboard GPS guided logger logs the different parameters during operations. The settlement per blow can be correlated to the end resistance of the subsoil. This was used to identify soft spots and monitor the compaction levels over the transport route. In order to reach a similar behavior more effort is put into softer sections. This is identified by the total settlement of the foot measured during the compaction. Figure 4 shows a 3D image of the settlement that occured during the compaction. Remarkable were the high settlements at the location of the old highway, where on some sections a total settlement of almost a meter occured.

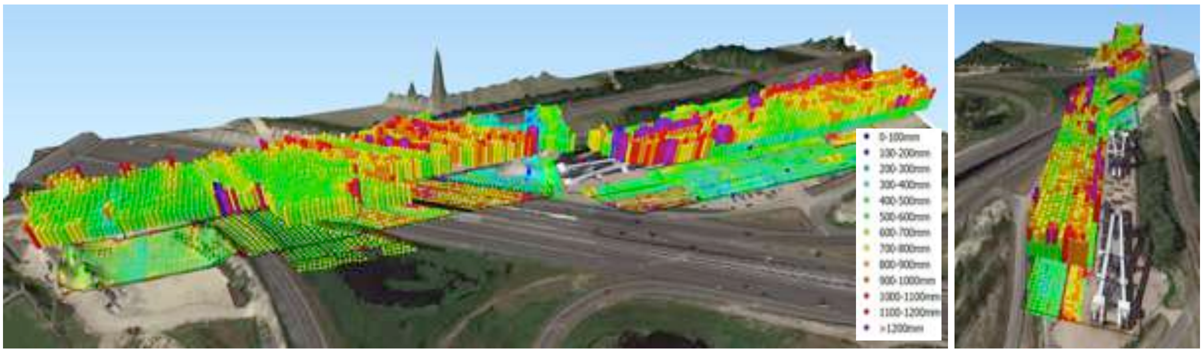

Figure 4. Total settlement of the foot

CPT Results

The compaction induced excess pore pressures in the sand influenced CPT cone tip resistance results in case the CPT was performed shortly after compaction, predominantly in areas where a very recent soil replacement was performed. Especially in the top layer the requirements were sometimes not directly reached. A short trial was performed to investigate the results and even after 2 or 3 compaction passes (with the PVD's in place) no large improvement was measured in the newly placed sands. The CPT's are given in figure 5 (left) which illustrates the CPT results just after the first (AC1), the second (AC2) and even a third pass (AC3) with CDC – represented by the red, the green and the black line respectively. As can be observed from the graph there is little to no improvement in the newly placed sand in the top 4m. During these CPT tests the water was still flowing up through the surface and vertical drains, clearly indicating excess pore water pressures. The graph also shows a test on another section that was performed after a resting period of approx. 2 days, clearly showing an increase of 5-10Mpa in the top layer which is caused by the dissipation of water pressures (green line to black line). Due to the weekend closures of the highway where only a limited time window of 8 to 10 hours was available to compact the section, the time was not available on some sections and the loggerdata together with the trials was used to validate the compaction, supported by the CPT's. On all locations the requirements of 85% relative density up to 6.5m below the surface was met.

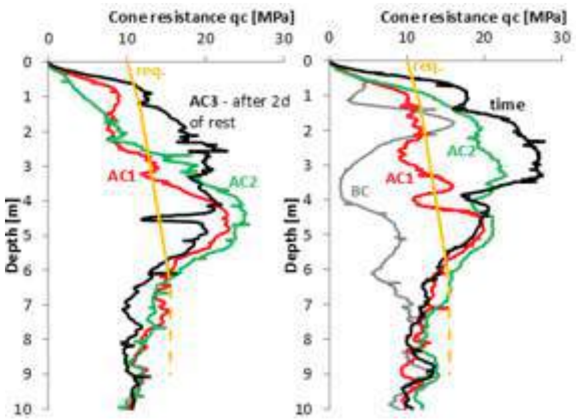

Figure 5. CPT results just after soil replacement and compaction and after a period of rest

Conclusion

The ground improvement of the transport path of the bridge has been successfully performed using 16 ton CDC compaction. The compaction operations mitigated the most predominant risks. The remaining possible failure mechanisms could be controlled with the observational method by means of an extensive monitoring program during the transport.

The system behavior, comprising of CDC compacted subsoil, pavement and SPMT-groups, behaved very well resulting in marginal deformations on the path and relative under pressures in the piezometers indicating a dilative soil behavior under the rapid loading during movement of the bridge.

Figure 6: The bridge after placement on its temporary location with the old highway still in place (http://www.nevesbu.nl)

REINFORCEMENT AND GROUND IMPROVEMENT GEOPIER® SOLUTIONS

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1. Geopier® system description

GEOPIER® System for reinforcement and ground improvement consist in of the execution of rammed aggregate piers or rigid inclusions, to increase the bearing capacity of the soil and reduce the seating of the supported structures, as well as to increase the shear strength in the overall stability of embankments and retaining walls, and for the mitigation of liquefaction in soils of seismicity areas.

These are intermediate foundation solutions, alternatives to traditional excavation and ground substitution solutions, structural fillers, foundation wells and preloads, for the support of structures, footings, foundation slabs, tanks, wind turbines, embankments, etc.

They are constructed by replacing and/or displacing the soil in columns made up of successive layers of compacted gravel aggregates, using specially patented tools to apply high vertical compaction energy, with high frequency and low impact amplitude, thus achieving high internal friction angles, which vary between 48-52º, while the elasticity modules of the column reach values vary between 150 and 250 MPa, higher than those reached with gravel columns performed by means of vibro-substitution.

The vertical compaction action increases the lateral pressure and improves the capacity and resistance of the surrounding soils resulting in an over-consolidation of the soil around the column which, together with the high rigidity of the constructed element, allows the reduction and control of the seats in a very effective way. It can be applied in loose soils, soft cohesive or compressible soils: soft clays and silts, loose sands, in uncontrolled landfills, rigid clays and silts and sands of medium to dense density, which require improvement to reduce or avoid differential seating.

For very soft and stiff soils, where the lateral tension is not enough to contain and confine the column of rammed aggregates, a very high stiffness modulus may be executed by adding a cement slurry during the rammed of the gravel or by constructing a concrete column, compacted and enlarged at the tip in potentially upgradeable soils, in order to increase the geotechnical capacity of the column.

Rammed Aggregate Piers (RAP) were developed in the United States in the 1980s and have since been used in countless projects around the world.

2. Geopier® solutions

These are techniques that cover practically the entire spectrum of foundation solutions where ground improvement is required to increase bearing capacity, reduce seats or limit differential seats, as alternative solutions technically and economically to deep foundations (See Figure 1).

The rammed aggregates piers are:

- ***Geopier System (GP3)*****:** up to 5-7 m deep, where the bearing capacity of the soil requires a previous perforation, for its later filling and compaction with the aggregate of gravel for the conformation of the column.
- ***X1 System (X1):*** up to 15-17 m deep, as in the previous case, in soils where bearing capacity requires prior drilling and filling and compaction of the column with the addition of gravel.
- ***Geopier Impact (IMPACT)*****:** up to 25-27 m deep in saturated or cohesive sandy, potentially cavity soil, where the column is constructed by displacing the soil and compacting the gravel aggregate.

In very weak and compressible soils, GEOPIER® solutions include rigid inclusions:

- ***Grouted Impact Pier (GIP)*:** is the same solution as ***IMPACT*** but introducing a cement slurry which is mixed with the rammed aggregate, resulting in a column with a high stiffness modulus.
- ***Geo-Concrete Columns (GCC*):** is a concrete column up to 25-27 m deep, with a high stiffness modulus, which is constructed by displacing the ground by installing a base or tip with a larger diameter than the shaft of the column, by compacting the concrete and by lateral displacement and deformation of the surrounding ground. In this way, the geotechnical resistance of the column is increased, growing the bearing capacity in soft and very deformable soils, transferring the loads to the immediately lower layer that has been improved during the construction of the column.

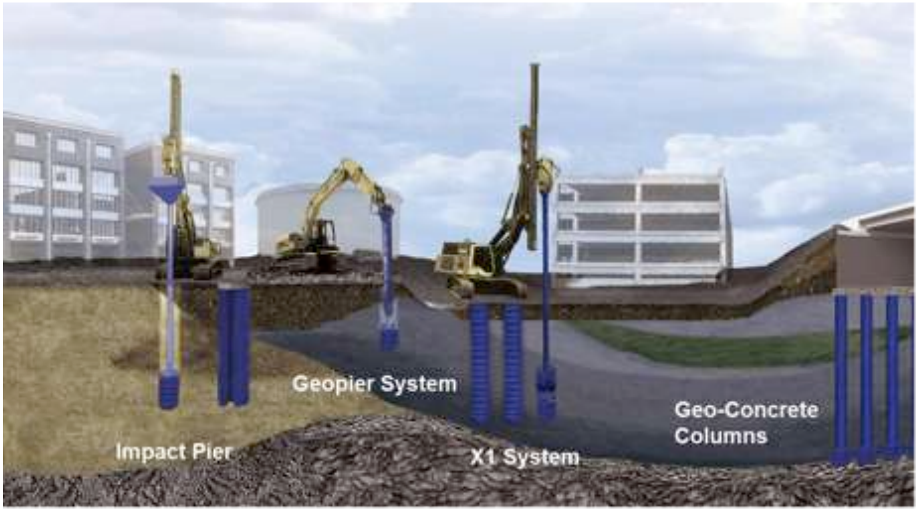

Figure 1. GEOPIER® System Intermediate Soils Foundations

In all cases, GEOPIER® systems make it possible to reduce execution times; these are fast and safe solutions, with high quality control, the results of which are verified with static load tests to check the stiffness modulus of the column and guarantee the estimated seats.

The design of the rammed aggregate piers is based on the classical principles of soil mechanics and geotechnical analysis techniques. The settlement calculation is performed by subdividing the stratigraphic soil profile into two layers. The first, called upper zone, involves the reinforced strata with the rammed aggregated piers, while the second, called lower zone, refers to the strata below the reinforced zone, but is located at a depth where the effort received is greater than 10% of the total effort applied at the level of the superficial foundation.

The settlement in the upper zone (SZS) or reinforced zone, will depend on three factors: (a) the stiffness of the rammed aggregated piers, (b) the original stiffness of the original soil, and (c) the replacement area occupied by the rammed aggregated piers under the foundation slab or footing.

The settlement in the lower zone (SZi) or the unreinforced zone can be estimated using classical soil mechanics techniques, including the analysis and selection of the compressibility parameters of the lower zone strata and the concept of stress distribution under foundations, using conventional theories of soil elasticity.

As these are elements of high rigidity compared to the surrounding soils, a concentration of loads on the head of the columns will occur, which can be improved by installing a load transfer layer with or without reinforcement geogrids. Therefore, the design must consider the magnitude and means of transfer of loads and the level of admissible seats, according to the tolerances of the structure, and depending on this, the number, the spacing of the mesh and the dimensioning of the columns will be established.

3. Performance process

The traditional method (***Geopier System - GP3***) involves drilling boreholes from 60 to 90 cm in diameter, in soils with a certain bearing capacity, free of phreatic level, where once the design depth of the column has been reached, the installation and compaction of successive layers of gravel aggregates with a thickness of approximately 30 cm is carried out by means of a tool specially beveled, a *Tamper*, to which a high vertical compaction energy is applied by means of a hydraulic hammer.

During the construction of the element, the high energy applied with the Tamper, in combination with its beveled shape, leads to the vertical densification of the gravel aggregate, causing a lateral displacement of the gravel, pre-stressing and pre-deforming the surrounding soils, resulting in an increase in lateral pressures soil matrix leading it to the mobilization of its passive.

The ***GEOPIER IMPACT®*** system is used in soils with less rigidity, in soft or granular soils without cohesion, or under the water table; in soils susceptible to collapse during pre-drilling of the column. The mandrel includes a sacrificial cap or the provision of internal flow restrictors to prevent soil from entering the tamper foot and mandrel during driving.

The process displaces soils laterally, resulting in densification and reinforcement, driven by a vibrating hammer located in the head of the element to the maximum depth of design to go in withdrawal, gradually, pouring and compacting the gravel inside the cavity, in layers of about 30 cm thick, until the total conformation of the column.

During the driving of the mandrel, a first improvement of the ground occurs due to the driving process and displacement of the surrounding ground. A second process of reinforcement of the soil matrix takes place during the process of pouring and compacting the gravel. The process densifies aggregate laterally into cavity sidewalls, increasing the diameter of the column and pre-stressing and pre-deforming the surrounding soils. This results in excellent coupling with surrounding soils and reliable settlement control with superior strength and stiffness, therefore an increase in the resistance to the stresses applied by the acting loads of the surface foundation.

Rammed aggregated piers are typically designed to cover the area under the footprint of the foundations, with a 25-40% substitution surface for replacement systems and 10-15% for displacement systems. Foundations supported on soils reinforced with rammed aggregated piers can withstand stresses of 200 to 450 kN/m^2. The permissible bearing capacity will depend on the rigidity of the compacted aggregate columns, the consistency of the matrix soil and the percentage of coverage of the columns, the ratio of the area of the columns (Ac) versus the area of the footing (As), reaching a load per column between 200 and 750 kN.

Rammed aggregated piers are used to improve and stiffen the most superficial layers of the ground to comply with the design criteria, and not to support the loads directly as independent rigid or structural elements. Therefore, they are not considered elements that transmit their loads to the tip, as is the case with piles, but rather that the loads are adsorbed by the shaft, for which reason it is not necessary for them to reach a layer of soil competent for use as a foundation element.

In seismic events, as rammed aggregated piers are considerably stiffer than the surrounding soil, they will take a higher percentage of shear strength, thus reducing the load on the ground. Additionally, due to the high permeability of the element, they will provide a radial drainage to dissipate the excess pore pressure that could be generated during the earthquake.

In the case of very soft and compressible cohesive soils, even with organic material contents, GEOPIER® systems offer a rigid inclusion by means of ***GEO-CONCRETE COLUMNS® (GCC®)***. The construction process is like the ***IMPACT®*** system, by driving a casing pipe or mandrel driven by a vibrator installed at the head, while pumping concrete into the ground, which results in lateral displacement of the ground without any removal of detritus.

At the end of the drive, once the practical rejection has been reached, an enlarged tip is constructed, with a larger diameter than the column shaft, which allows greater resistance to be exploited from a geotechnical point of view. Not only because of the larger diameter of the tip, but also for the reduction in the comprehensibility of the ground matrix at the tip, by lateral deformation carried out by compacting the concrete, creating a bottom bulb.

Subsequently, the tool is removed while simultaneously pumping the concrete, controlling the injection pressure in the column shaft, to avoid cuts in the concreting and ensure the continuity of the column.

Therefore, ***GEO-CONCRETE COLUMNS®*** are an intermediate solution between surface foundation and deep foundation (piles), where soil improvement is performed at the base or tip of the column. Due to the high stiffness ratio between the column and the soil, there will be a concentration of load on the column, transmitting much of it to the deepest substrate, so that there is a discharge of the compressible soil reducing the magnitude of the seats.

The loads supported by the column will depend on its diameter and the characteristic strength of the concrete; while the strength of the column will depend fundamentally on the diameter of the tip and the contribution of the underlying layers improved during the construction of the bulb, so the loads per column can vary between 400 and 1,500 kN.

4. Conclusions

GEOPIER® reinforcement and ground improvement solutions are used to increase bearing capacity or foundation in loose soils, soft cohesive or compressible soils, soft clays and silts, loose sands, in uncontrolled landfills, rigid clays and silts and medium to dense sands, requiring improvement to reduce or avoid differential seating.

They are the result of continuous development and research to offer foundation and seat control solutions, providing significant increases in the permissible bearing capacity of the soils or limiting the seating of supported structures according to the requirements of the project. In a seismic event, the gravel aggregate piers will take a greater percentage of the shear stresses, since they are considerably stiffer than the surrounding soil, and will help radially drain excess interstitial pressures. Improved soils around the enlarged tip of the Geo-Concrete Columns allow to increase the loads received by the column while its seat is reduced.

These technologies are fully proven, presenting advantages due to their cost-effectiveness and savings in construction times.

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COMPACTION GROUTING – A SOIL IMPROVEMENT TECHNOLOGY (ALMOST) UNKNOWN IN PORTUGAL

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Introduction

Soil improvement by compaction grouting was developed in the 1950s in the United States as a remedial measure for building settlement. Since then this procedure has been used to solve a wide range of other underground problems, such as to reduce liquefaction potential, decrease settlements, increase bearing capacity, stabilize sinkholes or reduce sinkhole potential, among others, and started to be utilized commonly in Europe in the 1990s.

The compaction grouting method is based on the injection into the soil of a mortar of low mobility, so that the injected mixture does not flow through the soil, remaining concentrated around the injection point. The injected material fills the voids and produces a lateral displacement causing densification and stabilization of the soil surrounding the injected area. The vertical tension of the treated layer must guarantee that the low mobility mortar displaces the ground horizontally without lifting on the surface.

In this paper are described the applications, common uses, design parameters, execution and quality control of compaction grouting.

Applications

Compaction grouting is suitable for wet to saturated sand, silt and clay (fig. 1). It consists of pressure grouting the ground with a high viscosity mortar in a pattern designed to suit the specific features of the site. Low mobility grouting densifies loose granular soils, reinforces fine grained soils, stabilizes subsurface voids or sinkholes, reduce liquefaction potential, increase bearing capacity, all that through the staged injection of low-slump, low mobility grout.

Low mobility grouting often offers an economic advantage over conventional approaches such as soil removal and replacement, or piling, and can be accomplished where access is difficult and space is limited. Low mobility grouting for treatment beneath existing structures is often selected because the low mobility grout columns do not require structural connection to the foundations.

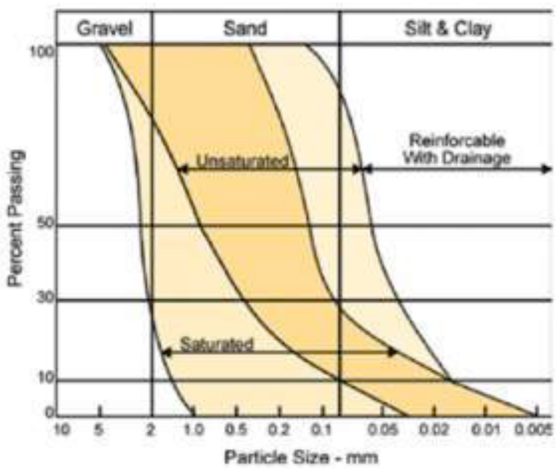

Figure 1. Range of soils suitable for compaction grouting

Design parameters

- Injection Volume: 5 to 15%
- Injection pressure: between 5 and 30 bar
- High consistency mortar: Cone Abrams between 3 and 8 cm slump
- Sequence of the perforations/injections designed to obtain the maximum possible confinement of the treated soil
- Perforation mesh: 1.0 to 3.5 m
- Steps or stages of injection: 0.20 - 1.00 m

Execution/Construction

When performing soil improvement, an injection pipe is first inserted typically to the maximum treatment depth. The grout is then injected as the pipe is slowly removed in lifts, creating a column of overlapping grout bulbs (fig. 2). The expansion of the low mobility grout bulbs displaces surrounding soils. Compaction grouting increases the density, friction angle, and stiffness of surrounding granular soils. The effectiveness of the improvement can be increased by sequencing the low mobility (compaction) grouting work from primary to secondary to tertiary locations. In all soils, the high modulus grout column reinforces the treatment zone.

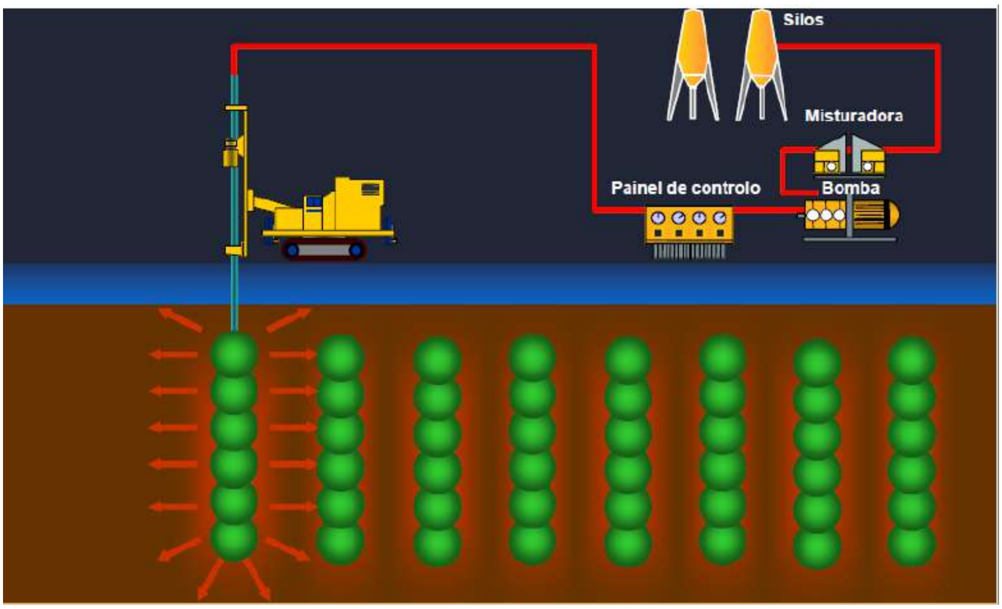

Figure 2. Typical layout of a compaction grouting site

Quality control

- Systematic control of the mortar through cone Abrams test
- Control of the performing parameters (fig. 3)
- Control of the working platform level and/or the structure foundation
- Control of the final improvement result

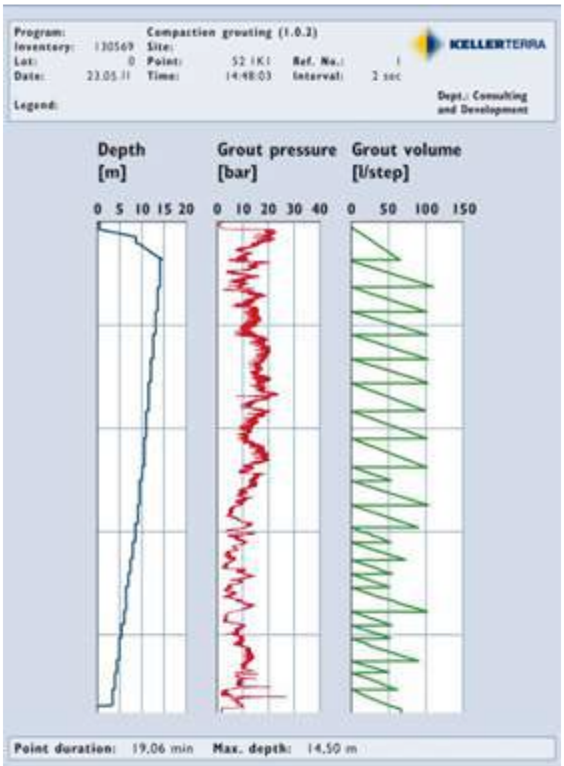

Figure 3. Record of performing parameters

Examples

Ex. 1 – Change of the original load plan in an industrial unit

Ex. 2 – Change of the surrounding conditions of an industrial warehouse due to the loss of fine material by water percolation

Conclusions

- Wide range of treatable soils
- Treatment of soil in localized areas
- Quick installation and execution, from the inside and/or outside of a structure
- Feasible in places hard to reach
- Possibility of working in confined spaces (headroom <3,0 m)
- Non-destructive process adaptable to the existing foundation type
- Clean Process - does not generate significant spoil material
- Does not require structural connection with existing foundation
- Soils may be treated at depths not reachable with some other techniques
- Economic alternative when compared to soil removal or the execution of deep foundations
- It is a proven, economical and versatile technology, reducing the risk of liquefaction during an earthquake.
- It can be used to improve the foundations of existing structures.

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SOIL TREATMENT UNDER SLABS BY RIG INCLUSSIONS OF SOIL-MIXING COLUMNS. SPRINGSOL® METHOD

José Luis Arcos

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This article describes a new technique for the execution of rigid inclusions in the form of soil-mixing columns by means of a rotary system to deconstruct the soil structure and mix with a binder, by means of the mechanical action of folding cutting blades.

The concept of mixing soil and a binder by mechanical beating is having an important boom in recent times due to the advantages it provides thanks to the implementation of technification to these tools and their sustainability (use the land itself as construction material).

All the different methods for Soil-Mixing execution have in common a series of characteristics:

Basic Process:

- The soil is deconstructed using a mechanical tool.
- A hydraulic binder is incorporated into the soil.
- The mixture of the soil with the binder is produced.

Binders:

- Cement, lime or a binder specially designed to fulfill a special mission.
- The binder is applied in powder or as slurry (premixed with water), which differentiates the dry method from the wet method.

Result:

- A rigid inclusion of soil-binder is generated into the soil.

Among the advantages that soil-mixing provides compared to other methods of treatment by injection can be cited the following:

- Economy: the consumption of binder material are much lower than those required in the other techniques, for the same result. In particular, compared to jet-grouting, the consumption to achieve the same column of soil-cement can be around a 20%.
- Sustainability: It uses the soil itself as a construction material. It reduces the waste of unnecessary binder and, for example in the case of jet-grouting, the excess of detritus that has to be removed.
- Geometry well known with accuracy. (unlike the other treatments by injection, the geometry of the treated element is known. In the case of grouting by pressure and jet-grouting, the diameter of affectation is always unknown).
- Possibility of execution parameters recording and automation of procedures.
- Control of the scarce detritus generated, by collection systems designed for this purpose.

Rigid inclusions as soil-cement columns – type Springsol®

Rigid inclusions inside the ground in the form of columns are performed to make qualitative improvements of soil in extensive areas under distributed loads. For example, land reclamations in dredging terrain, underpinning of foundation slabs, road embankments.

Rigid inclusions may consist of columns of concrete, mortar or soil-mixing with addition of a hydraulic binder such as cement.

Springsol is a relative new method to create soil-mixing columns by means of rotative rods armed with perpendicular cutting blades, able to be folded to reduce de perforation diameter where the column formation is not necessary. The rods are hollow inside to permit the intake of a cement slurry or other binder slurry.

This basic procedure has gained in technification in recent years with a new generation of tools capable of folding the mixing blades, able to pass the first meters in depth with small diameter, thus equating the advantage is this aspect that jet-grouting has. Likewise, the equipment has been equipped with a register of parameters and automatic pump control systems and capture and conduction of the detritus generated. An example of these new tools is the named Springsol®.

The final product is a soil-cement column equivalent to that of jet-grouting; however, the execution of both techniques has important differences:

- In jet-grouting, the destructuring of the soil is carried out by means of a jet ("jet") of a fluid (usually cement slurry) which, thanks to very high-pressure pumps (> 400bar), manages to shoot the jet through a nozzle at a speed close to 300m/s. The destructuring of the soil by the jet depends on this hydraulic energy and the erodibility of the soil, so that the column diameter obtained is not known with accuracy. However, the soil-mixing uses mechanical energy, from the rotation of its blades, for the destructuring of the ground.
- On the other hand, the high pressures necessary to generate the jet-grouting jet carries the risk that, when a plugging occurs in the evacuation of the detritus, very high overpressures are generated capable of lifting the ground and structures that exist on the surface. In the soil-mixing, the necessary pump pressures are the minimum necessary to manage the flow rate that turns out to be around 10 times lower than that required for jet-grouting.
- Against soilmixing, it can be argued that, in soft soils, the columns diameter is limited by the torque that can be supported by the tool and that thick gravel layers prevent the advance and rotation of the tool.

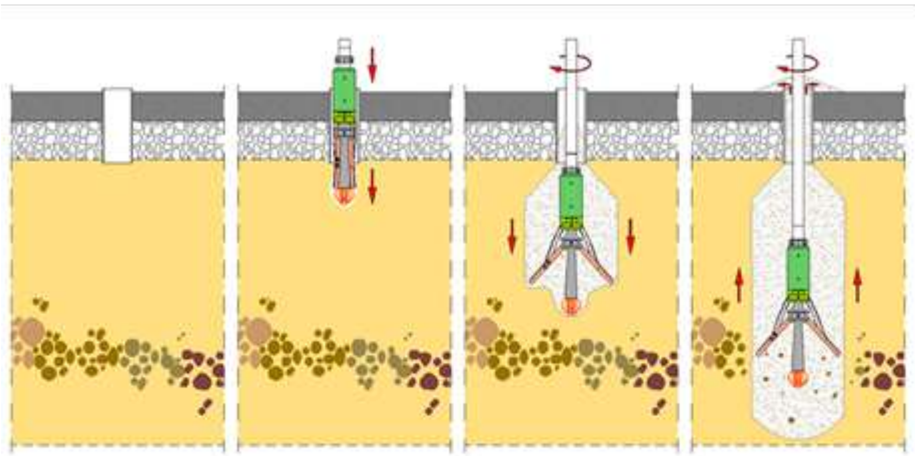

Figure 1. Springsol® - Passage through the slab with small diameter and opening of the blades in depth.

The Sprinsol® system was initially designed for the treatment of railway embankments, thanks to the advantage of passing throuh the upper soil layers with small diameter, which allowed the execution between the sleepers, and the detritus catchment system, which allowed the non-contamination of the ballast with grout.

Figure 2. Springsol® treatment in a railway embankment.

For the same reason Springsol® is a technique proposed in numerous road embankment treatment projects, where the small equipment used allows traffic to be maintained in some of the lanes and with the only damage to the road surface due to the small holes of 160mm in a grid of $2x2m^2$ or $3x3m^2$.

Foundation slabs with postconstructive settlement have also been consolidated, where the work under lowheadroom, the cleaning of the detritus and the minimum affection to the pavement are very much appreciated.

Parámetros que gobiernan el proceso de ejecución del soilmixing en columnas.

El procedimiento de soilmixing en columnas del tipo Sprinsol® conlleva la mezcla del suelo con el cemento en descenso, durante la perforación. Este hecho hace que el aporte de cemento se haya de realizar con un caudal variable en función de la velocidad de avance de la perforación.

Parameters that govern the execution process of the soilmixing in columns

The procedure of soilmixing in columns by Sprinsol® involves the mixing of the soil with the cement downwards, during perforation. This fact means that the contribution of cement must be made with a variable flow depending on the speed of advance of the perforation.

An indicator called "Mixing factor" is defined to control the appropriated mixing of the soil with the cement. This factor considers the number of blades of the tool, the speed of drilling advance and the speed of rotation. The minimum Mixing Factor depends on the composition of the soil, the more cohesive the soil the higher should be the Mixing Factor.

Likewise, we use an “Incorporation Factor” (also called “Binder Factor”) to define the amount of cement to be supplied per cubic meter of land to be treated. This amount of cement will be related to the resistance of the soil-cement material finally obtained.

However, although the amount of detritus generated by this procedure is relatively small, we must consider what part of cement is finally evacuated with the detritus to truly know what is the amount of cement that remains inside the column. For this reason, it is enough to make a mass balance to estimate the final content of cement in the column, with a series of hypotheses. In our case we make the assumptions that the air of the soil is totally removed and that the coarse aggregate above a certain size is not evacuable with the detritus.

Mixing factor, I_m (1/m)	$I_m = N \cdot \frac{\omega_{rot}}{U_{perf}}$; • N = number of Blades of the tool • ω_{rot} = rotation speed of the tool (1/s) • U_{perf} = speed of drilling downwards (m/s)
Incorporation factor, I_i (kg/m^3)	$I_i = \frac{Wc}{Vs} = \frac{Cco \cdot Q}{\frac{\pi \cdot \Phi^2}{4} \cdot U_{perf}}$; • Wc = Consumption of cement per unit of time (kg/s) • Vs ω_{rot} = Volume treated per unit of time (m^3/h) • Cco = kg dof cement per m^3 of slurry (kg/m^3) • Q = Debit of delivered slurry (m^3/h) • Φ = diameter of the treated column (m) • U_{perf} = speed of drilling downwards (m/h)

The Incorporation factor (Binder factor) depends on the type of soil and the target of resistance to achieve.

In order to guarantee the good execution of a soil-mixing, in general, the automation of the pumps is recommended to supply the flow of consigned slurry: systems that stop the pumps when the perforation does not advance and regulate the flow according to the speed of drilling.

Figure 2. Springsol® columns executed in different soil types.

BIOCEMENTATION BY BIOCALCIS, FROM DESIGN TO SITE IMPLEMENTATION

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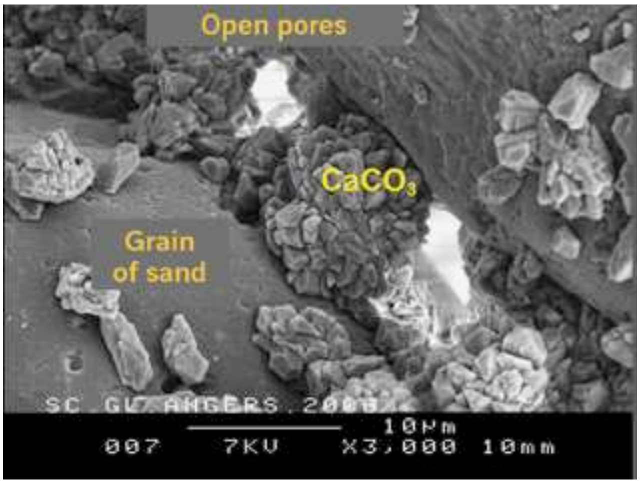
PRINCIPLE

Biocementation is a new injection principle used for ground improvement. Soletanche Bachy holds several patents for implementing the industrial process based on the use of non-pathogenic bacteria in a natural environment. These bacteria will act on a calcium rich solution, leading to the formation of calcite – a mineral form of calcium carbonate – that will act in situ to create a stable and durable biological bond over time.

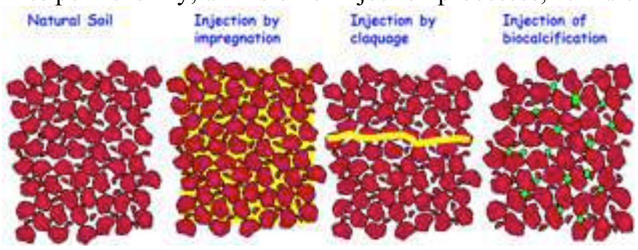
The injected fluids, a bacterial suspension and a calcifying solution, are injected under low pressure as their viscosity is close to that of water.

By creating calcite bridges between the grains, Biocalcis brings a new cohesion to a granular soil, without any significant change to the original porosity, as indicated on the scanning electron microscope (SEM) in Picture 1. The Biocalcis process results in the transformation of granular soil into a cohesive mass similar to sandstone or limestone. Varying degrees of calcification can be achieved through differing formulations, and/or repeated applications, to achieve a UCS in the range of 100-500kPa.



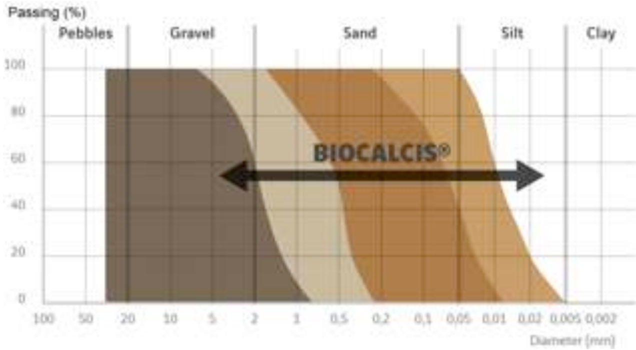
Picture 9 : Image of $CaCO_3$ calcite with Scanning Electron Microscope (SEM)

An interesting characteristic of the process is that after treatment the soil porosity is maintained, without any significant changes to the mass permeability, unlike other injection processes, as indicated in Picture 2.



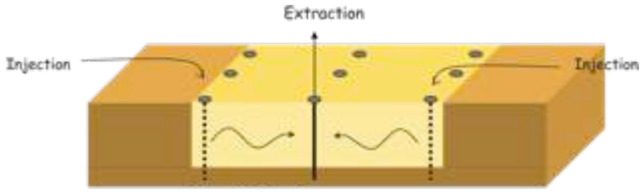
Picture 10 : Different injection methods

As indicated in picture 3, Biocalcis can be used for a range of soil particle sizes, including soils of mixed composition, including gravels, sands and silts.



Picture 11 : Areas when Biocalcis is applied

The execution of the Biocalcis process involves several stages, from the preparation of the bacterial culture, its injection on site, and the management and the control of all the calcification fluids. A computerised modelling tool (Comsol), has been specifically developed, to optimise the process in line with the site characteristics and treatment objectives. An example is indicated in Picture 4, carried out below the water table through an injection and extraction well network. The use of a digital tool allows the modelling of the bio-calcification process, which is an important step in defining the site specific treatment programme.



Picture 12 : Principle of setting-up the injection and extract process under the (water) table

TYPICAL APPLICATIONS

The typical applications for the process are mainly:

- Treatment of loose soils to combat liquefaction. It is possible to reduce the risk of liquefaction of loose sands, by increasing their shear strength (Cu).. For this application, Cu of some tens of kPa would be sufficient. and these values are easily achieved by biocalcification. As the permeability is unchanged after the treatment, there is no resistance to the dissipation of excess pore water pressure in case of seismic shock.
- Treatments against the risk of internal erosion of embankments or their foundation.
- Ground treatment to reduce earth pressure, increase support, or for all other strengthening applications necessitating the treatment of lose soils: e.g. for deepening the foundations of existing quay walls, biocalcification is an interesting alternative treatment for areas with restricted access or partly underground. To treat a homogenous block of soil, the values of c'/UCS required, are much lower than those required for soil-mixing, for example. Moreover, to increase the support of the quay wall, treatment in contact with the wall would not require any other structural element.
- The results obtained from these geotechnical applications have enabled the company to test the effectiveness of Biocalcis in a completely different domain, that of mechanical reinforcement of limestone columns of a historic monument. This solution would guarantee a permanent solution by regenerating the limestone mass. It respects the architectural environment because it does not modify the appearance of the stone.

The presentation will focus on the mechanical properties of bio-calcified soils, the methodology for implementing the technology on site and the description of a field application for strengthening a reinforced earth wall.

ANCHORED HIGH PERFORMANCE TURF REINFORCEMENT MAT FOR SLOPE STABILIZATION

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Many times the shallow slough of a steep slope is ignored by the geotechnical community as it is incorrectly viewed as erosion. The United States Army Corps of Engineers (USACE) defines a slough as "a shear failure in which a surficial portion of the embankment moves downslope". While surficial slope failures are often considered maintenance problems, if not addressed, they can become progressively larger and effect the stability of the total slope. The purpose of this paper is to give importance to the proper identification of shallow plane slope failures and an innovative repair method that is more environmentally friendly and cost effective than traditionally used hard measures, such as rock rip or concrete based solutions. Upon completion of this paper, the reader will have a greater understanding of the shallow plane slope stability design and installation of an anchored high performance turf reinforcement mat (HPTRM), such as ARMORMAX by Propex GeoSolutions. With over 16 million square meters of ARMORMAX installed worldwide, engineers can feel confident designing with this solution as they balance the security of older, traditional solutions with innovations that promise to improve long term performance at a lower cost to the environment and budgets.

Often, failures of slope faces are misdiagnosed as erosion and and are repaired with erosion control measures. Because the true failure mechenism has not been identified, the slope face continues to slough despite the implemented repair measures. An example of this can be seen in Figue1 which shows two slopes where engineers have used typical erosion control solutions to attempt to solve a geotechnical failure and have seen continued failure as a result. Both slopes are globally stable and the observed soil movements are within the first three meters of the slope face, which are typical of a shallow plane slope failure. While this can be viewed as a low priority of maintenance problem, the Texas Department of Transportation's "experience has shown that most exposed side slopes failures begin as shallow slides and then deepen with time." To determine the depth of the failure plane, a geotechnical analysis must be performed.

Figure 1. Surficial slope instability incorrectly repaired with turf reinforcement mats

As a matter of practice, engineers will evaluate the global stability (global stabilty definition deep seated filures.) the purpose of this discussion is to encourage engineers to complete a seperate analysis to define failure planes within approximately the first 3 meters of the slope face. These are failures that are more involved than plain erosion and are traditionally reppaired with rock rip rap, but may be suitable for a better approach such as an anchored HPTRM. This consists of a High Performance Turf Reinforcement Mat (HPTRM) in combination with Engineered Earth Anchors™ and is designed to meet a project's hydraulic, geotechnical, design life, environmental and economic needs. In introducing the force from an engineered earth anchor into the fundamaental slope stability equation shown in Figure 2, a seperate analysis is performed near the slope face to degtermine the existence of a failure plane and the size, frequency, and pullout resistance of the anchors required to stabilize the slope. The analysis uses vertical slice limit equilibrium methods to determine slope stability following the Spencer method for the given geometry, soil properties, and earth anchoring parameters. The calculation is approriate for slopes consisting of alluvial soils and all other soils as long as the soil properties can be accurately defined.

The strength and durability of the HPTRM is critical in order to connect the earth anchors together into a passive system that resists movement. Figure 3 conceptually shows how the components of the ARMORMAX System can be used to stabilize an embankment against surficial sloughing. The slope face has reinforced vegetation

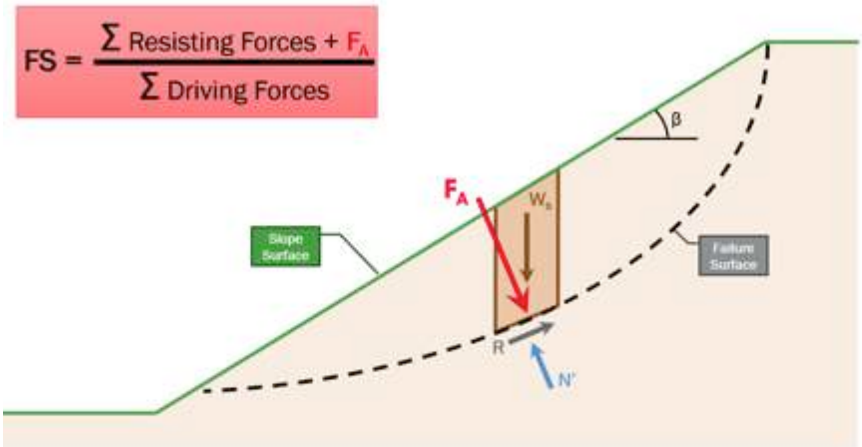

Figure 2, Free body diagram for the shallow plane slope instability calculation

having the benefits of being aesthetically pleasing, providing pollutant removal from overland flow, and protecting against wind or water induced erosion. Additionally, the ARMORMAX solution is generally half the cost of a a traditional "hard" slope treatments and is faster to install with smaller equipment. To approriately design with an anchored HPTRM, like ARMORMAX, the engineer should require:

- All system components to be provided by the HPTRM manufacturer,
- For the system selected, a portfolio of performance in like conditions of > 650,000 sy,
- Third-party UV stability test results for the HPTRM polypropylene,
- A geotechnical stability analysis for project conditions.

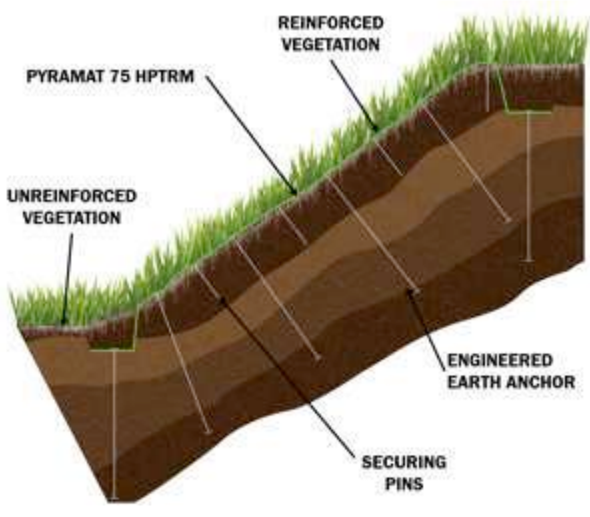

Figure 3. Cross section of an ARMORMAX reinforced slope

The brief intent of this paper is to draw attention to the need for the proper identification and design procedure for a shallow plane slope failures. While the engineer has many traditional hard rock or concrete based solutions, an anchored HPTRM is an innovative method that is environmetally friendlier and more cost effective than these traditionally used hard measures. Designing with an anchored HPTRM uses the Spencer method and is dependent upon slope geometry, earth anchor paramaters, and soil properties. Alluival soils with their weak profile, may be especially suited for stabilization with an anchored HPTRM. With over 16 million square meters installed worldwide, the designer can be confident in using ARMORMAX to stabilize the face of steep slopes and channel banks.

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MODULE III

Learning from (in) success in soil improvement

GROUND IMPROVEMENT FOR CONCRETE TANKS BUILT ON RECLAIMED SAND

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Palm Jumeira was reclaimed off the coast of Dubai, UAE using carbonate sand that was dredged from the Persian Gulf and placed by bottom dump barges and rainbow discharge. The reclamation was in the shape of a tree trunk, a crown with 17 fronds, three surrounding crescent islands that form an 11 km long breakwater and two smaller islands on the sides of the trunk that resemble the logo of the project's developer. The reclamation is approximately 5 km by 5 km, and has added about 78 km to Dubai's coastline (Hamidi and Varaksin, 2015).

Sladen and Hewitt (1989), Lee et al. (1999), Lee et al. (2000), Lee (2001) and Na et al. (2005) have studied the effects of sand fill placement methods in the sea. The density of sand that is dumped by trucks and then pushed into the sea by a bulldozer is usually low, with relative density of about 20%. Exceptions can be thin layers that have been compacted by traffic of earthmoving equipment. Hydraulic placement can be subaqueous by hoppers or bottom dump barges. If possible, sand is discharged via a big door located on the bottom of the hull, but when the water is shallow alternative methods such as pipeline or sub-aerial rainbow discharge are used. In pipeline discharge low velocity water-sand slurry is pumped; however, in the latter method the dredger sprays a high velocity water-sand mixture onto the reclamation.

At Palm Jumeira, 94 million m^3 of sand was used for the reclamation. The material was dredged from up to about 11 km away from the seabed using trailing suction hopper dredgers (Dowdall Stapleton, 2008; Lees et al., 2013). The dredgers initially lowered the suction pipes on both sides of the ships all the way to the seabed. Sand pumps transferred the sand dredged up by the suction head into the hold or hopper, excess water was drained off by means of the overflow pipes, and the ship sailed off to the reclamation area when the hopper was full. Sand was discharged by means of a big door located on the bottom of the hull onto the relatively uniform natural seabed at about -10.3 m DMD (Dubai Maritime Datum). Dumping continued to about -3 m DMD where water depth became too shallow. The dredger then sprayed sand and water mixture onto the reclamation by rainbow discharge. Redistribution and levelling of fill material above water level was undertaken with bulldozers to a mean final level of +4.1 m DMD.

At Palm Jumeira, cone penetration test (CPT) cone resistance, q_c, of the deposited sand above water level was very high and in the range of 20 to 40 MPa. The soil then became very loose in the rainbow discharged layer below water level with q_c as low as 1 MPa in the next 4 to 5 m of soil. Loose to medium dense sand with q_c varying from 4 to 8 MPa was encountered down to the depth of about 12 to 14 m where the soil became very dense and was soon followed by slightly to moderately weathered calcarenite and sandstone. Carbonate content of the sand that was measured as $CaCO_3$ varied from about 60% to 90%.

Two sewage treatment plants (SPT), called Lot A-A and Lot G-G SPTs at the time of construction, are respectively located on the upper ends of the two side crescents of Palm Jumeira (Hamidi and Varaksin, 2015). Each Lot has a reinforced concrete tank with a diameter of 35.1 m. The design of each tank was based on the assumption that the dead and live loads would be respectively 49 kPa and 71 kPa.

Two CPTs were carried out during the preliminary geotechnical investigation not very far from Lot A-A's tank. Ground level at the test locations was approximately at +4 m DMD and groundwater was 3 m below ground surface. CPT readings indicated that the upper 2 m of soil was composed of very dense sand with q_c as high as 25 MPa. The soil then became loose with q_c as low as 3 to 4 MPa to the depth of approximately 13 m where great resistance was encountered, and testing was terminated.

Four standard penetration tests (SPT) with samples collected from the split spoon were performed in the centre and three sides of Lot A-A's tank. These tests indicated that the upper 3 m of sand was very dense, then the soil became very loose to medium dense at groundwater level. SPT blow counts, N, at depths of 3 to 8 m varied from as low as 4 to as high as 14. N values were then measured to be in the range of 11 to 20 to depths of approximately 12 to 13 m where the ground became very dense and N exceeded 50. Fines content of the 38 samples that were extracted from the four boreholes ranged from 10% to 30%. Although no silt pockets were identified under the tank, as fines content was observed to be more than 20% in almost half of the samples, it was understood that the tank's location was probably one of the siltiest areas of the reclamation.

Later, two pressuremeter tests profiles (PMT) were also carried out in the tank area. As it was already established that the ground was very dense above water table level, testing was done at 1 m intervals below sea level. These tests reconfirmed that the submerged soil was in a loose state. In this zone PMT limit pressure, P_{LM}, was less than 100 kPa to about 700 kPa, and Menard modulus, E_M, was measured to be from less than 1 MPa to 6 MPa.

The presence of loose compressible sand raised geotechnical concerns and indicated that implementation of specific foundation measures may have been required.

Hamidi et al. (2011) have concluded that ground acceptance criteria based on design criteria and accompanied with proper testing is able to achieve optimized performances. In particular, Hamidi et al. (2017) reviewed acceptable uniform settlement, rigid body tilting and out of plane settlements of tank structures. ACI 376 (2011) that is applicable to concrete tanks for the containment of refrigerated liquefied gases allows uniform settlement of shallow foundations, provided that the other provisions of the standard are met, and the connecting piping system accommodates the settlement. The same standard limits planar tilting and dishing settlement measured along a radial line from the outer perimeter to the tank center respectively to 1/500 and 1/300. The standard notes that restricting the dishing settlement to 1/300 maintains the bending curvatures within acceptable limits so that insulation materials are not damaged. ACI 376 also limits footing settlement around the perimeter of the tank to the lesser of 1/500 and the maximum settlement limit calculated for the planar tilting of the tank.

Piling was the commonly used method for supporting heavy structures at Palm Jumeira; however, it was estimated to be costly and difficult to support on a small undeveloped island (at the time, the crescents were disconnected from the mainland and marine transport was the only means of access to the project); hence, ground improvement was considered as a viable option. Acceptance criteria for ground improvement at tank foundation level of +2.5 m DMD were specified by the project's designer to be:

- Bearing capacity: 160 kPa with a safety factor of 3
- Differential settlement: 1/750 for a uniformly distributed load of 120 kPa

Several ground improvement options were considered for treatment of the reclamation, including vibro compaction that was the commonly used technique at Palm Jumeira. However, the presence of highly silty sand beyond the acceptable limits of vibro compaction (Mitchell, 1981; Massarsch and Heppel, 1991; Woodward, 2005) disfavoured the application of that technique. Alternatively, stone columns (vibro replacement) were considered as a feasible option, but the importation of large quantities of stone to the island would have made that technique too expensive. In the end, the ground improvement works of the project were awarded to a geotechnical specialist contractor who had proposed the application of dynamic compaction and dynamic surcharging. Verification was proposed to be by PMT with interpretation of the results being by Menard's (Centre d'Etudes Menard, 1975) method.

Dynamic compaction is a ground improvement technique in which a very pounder is dropped from a significant height in a grid pattern to compact and improve the properties of soil (Hamidi et al., 2009). Pounder weight and drop height can be calculated using the method proposed by Menard and Broise (1975).

Most engineers are well informed that surcharging is applicable to fine soils, but are not confidently aware of the reason that the same technique is not used for improving granular soils. Although the latter type of soils will also settle under static surcharge, as dynamic shear modulus has been found to decrease significantly with increasing values of shear strain amplitude (Silver and Seed, 1971), it can be expected that vibrations will increase the amount of settlement under static loading.

Dynamic surcharging is a technique in which the effects of static loading and dynamic compaction are combined to accelerate the consolidation process in silty soils that have reached consolidation of 50 to 70% by regenerating pore pressure (Hamidi, 2014). In this project, the combined effects of static loading and high energy impacts were also used to produce a shearing process around the surcharge fill and to reduce the spreading of the load that was initially caused by the high strength of the upper layers. Generation of vibrations and increasing of the pore pressure under the tank was to reduce the intergranular friction of the soil and ultimately result in the collapse of the foundation soil under the influence of dynamic surcharging.

A 4 m high surcharge was initially placed at the tank location and allowed to consolidate the foundation for five days. Then dynamic compaction was performed on a ring at the periphery of the surcharged area using a 15-ton pounder with a total of 30 blows per print in 6 cycles. Each print location was pre-excavated to the depth of 1 m to increase the depth of treatment influence. In the next step, the surcharge and 1.2 m of ground were removed, and dynamic compaction was applied using the same pounder within a circular area with 41.2 m diameter at working platform level. Dynamic compaction was carried out on 4 concentric rings and a central print. Approximately 150 m^3 of crushed rock and cobbles were also added to the prints to increase soil permeability.

Measurement of subsidence values of settlement plates that had been installed in the centre and periphery of the Lot A-A's tank showed that the plates' settlement rates were similar to the surcharge placement rate. Sustaining the surcharge for an additional 5 days resulted in additional settlements of 60% in the central plate and 24 to 45% in the periphery plates. Application of dynamic surcharging further increased ground settlement by 1.6 to 5.2 times compared to static loading. With consideration of the added quantity of crushed rock, final measurement of ground levels after completion of dynamic compaction indicated that, on average, the ground had settled an additional 0.68 m due to the application of this technique. While the magnitude of dynamic compaction induced settlements were much larger than what was induced by dynamic surcharging, this does not reduce the importance of the effect of dynamic surcharging that was intended to reduce the settlements of the deeper and siltier layers.

Upon completion of ground improvement works and levelling of the site, 4 PMTs were carried out within the treatment area of Tank A-A. It was observed that the most amount of improvement occurred within depths of about 8 m where, excluding the highest values of one of the tests, P_{LM} ranged from approximately 2,000 to 4,000 kPa. E_M ranged from 23 to 30 MPa at depths of about 4 m to 8 m. It was also seen that due to the combination of dynamic surcharging and dynamic compaction the soil parameters continued to improve from 0 to 180% to the depth of testing termination at 14 m, which is quite significant.

Bearing capacity of the tank was confirmed using Menard's method (Centre d'Etude Menard, 1975). Settlements were calculated using three-dimensional finite element analyses with input values based on the PMT results. The analyses indicated that maximum settlement at the centre of the tank was expected to be 21.35 mm. Minimum tank settlement at the shell was calculated to be 10.91 mm. Consequently, differential settlement over the radius length of 17.55 m was 10.44 mm or less than 1/1,681, which was much smaller than the allowed value of 1/750. Differential settlement from one side to the other side of the tank was calculated to be 3.13 mm or less than 1/11,200.

This project has demonstrated the effectiveness of combining dynamic surcharging and dynamic. Dynamic compaction induced large subsidence of the ground and significantly improved the soil properties in the upper 8 m; however, dynamic surcharging was able to consolidate the deep silty layers and induce additional settlements compared to what was achievable by static loading.

The authors would like to express their gratitude to Menard for providing the data and information that has been used.

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IMPACT OF CREEP PHENOMENON ON THE BEHAVIOUR OF THE SOFT SOIL OF BAIXO MONDEGO

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When soft soils contain a high organic matter (OM) content, the soil's properties are considerably affected (Venda Oliveira et al., 2012) and consequently significant creep deformations are expected to occur in the long term after the end of the primary consolidation phenomenon. The creep phenomenon assumes a notable importance in the presence of soft soils, even when mitigation techniques are used. Two approaches are usually used to mitigate the creep deformations: (i) the use of a preloading technique to anticipate the creep deformations; (ii) or installing rigid vertical inclusions with better characteristics in the soil foundation, such as: concrete piles, stone columns and deep soil mixing columns (DMC).

The effects of time on the rheological behaviour of soft soils are very important and it should incorporated creep phenomenon (Correia et al., 2009a). The creep behaviour of a chemically stabilized soft soil with binders depends on: the OM content, binder quantity, binder composition, stress level and curing conditions (Venda Oliveira et al., 2012, 2013). Moreover, the transference of stresses from the soft soil to the DMCs, induced by the arching effect, potentiates the occurrence of creep settlements of embankments built on soft soils reinforced with DMCs, in consequence of the higher stress level on the DMCs (Venda Oliveira et al., 2011, 2017).

The main aim of this work is to study the ability of two creep laws to simulate the creep behaviour of a soft soil, whether unstabilised or chemically stabilised with binders. Firstly, the creep laws are used to simulate the creep behaviour obtained in eodometer and triaxial creep tests (validation stage). Then, the constitutive models and creep laws are used to predict the long-term behaviour of an infinite embankment built on a soft soil reinforced with DMCs, in order to study the influence of the creep phenomenon of the stabilized material.

Description of creep laws and constitutive model

Material creep is the plastic deformation that occurs in time when material is subjected to a constant effective stress state. The phenomenon is irreversible and time dependent, so in terms of macro-analysis, creep is a visco-plastic phenomenon (Venda Oliveira 2000). The creep deformations in soils could be described a sum of two distinct, but interdependent, components (Kavazanjian & Mitchell 1980): volumetric and deviatoric components. Creep volumetric strains are calculated on the basis of the secondary consolidation equation (based on Taylor's law) from oedometer tests:

$$\varepsilon_v^c = \int_{t_i}^{t_i+\Delta t} \frac{C_{\alpha e}}{2.3\left(1+e_0\right)t_v} dt \qquad (1)$$

where $C_{\alpha e}$ ($\Delta e/\Delta \log t$) is the index of secondary consolidation, e the void ratio and t_v the volumetric age of the soil in relation to the reference volumetric time (t_{vi}). Creep deviatoric strains are calculated based on Singh and Mitchell (1968), where the axial strain is evaluated from creep triaxial tests:

$$\varepsilon_{axd}^c = \int_{t_i}^{t_i+\Delta t} A.e^{\overline{\alpha}.\overline{D}} \left(\frac{t_{di}}{t_d}\right)^m dt \qquad (2)$$

where A, $\overline{\alpha}$ and m are soil creep parameters, $\overline{D}$ (q/q_{ult}) is the deviatoric stress level and t_d the deviatoric age of the soil in relation to the deviatoric reference time (t_{di}). Parameter A reflects the influence of the composition, structure and stress history. Parameter $\overline{\alpha}$ reproduces the influence of the shear stress level, while, parameter m controls the creep strain rate decrease in time (Singh & Mitchell 1968). Parameter m is dependent of the overconsolidation ratio, varying from 0.7 to 1.25, being in general less than one (Singh & Mitchell 1968).

Two constitutive models are used in this work, the Modified Cam Clay (MCC) and MCC/Von Mises (VM) models, both associated with a creep law, to simulate the behaviour of the natural soft soil and the stabilized soil, respectively. The MCC is an elastoplastic soil model based on isotropic conditions described by a yield function:

$$F(\sigma'_{ij}, e_k, t_v) = \underbrace{(\lambda - \kappa).ln\left[p'\left(1+\frac{\eta^2}{M^2}\right)\right]}_{f(\sigma'_{ij})} - \left[\underbrace{e_{\lambda o} - \underbrace{(e+\kappa.\ln p')}_{e_\kappa}}_{h(e_\kappa)} + \underbrace{\frac{C_{\alpha e}}{2.3}.\ln\left(\frac{t_v}{t_{vi}}\right)}_{\Delta h(t_v)}\right] \quad (3)$$

where M is the slope of a critical state line, λ and κ are, respectively, the slope of the virgin consolidation line (VCL) and the slope of the overconsolidation line in the plot e-lnp', η is the ratio q/p' and $e_{\lambda 0}$ is the void ratio for p'=1. The coupling of the VM model with the MCC model is intended to reproduce the non-linearity within the yield surface of the MCC model. In this case, the MCC/VM model is expressed by two yield functions, described by equations (3) for the MCC model, and equation (4) for the VM model:

$$G(\sigma'_{ij}, \gamma^p, \varepsilon_v^p, t_v, t_d) = \underbrace{q}_{g(\sigma'_{ij})} - \underbrace{\frac{\gamma\, p'_c}{a+b\gamma} R_f}_{h(\gamma^p, \varepsilon_v^p, t_v, t_d)} \quad (4)$$

where $g(\sigma'_{ij})$ is the load function and $h(\gamma^p, \varepsilon_v^p, t_v, t_d)$ the hardening rule, which represents the trace of the yield surface on plane q-γ considering the Konder's hyperbola; a and b are normalized hyperbolic parameters, and R_f ($q_{failure}/q_{ult}$) is the failure ratio.

Parameters for creep laws/models

The soil used is a soft soil located in the "*Baixo Mondego*" area, along the side of the A14 motorway (Portugal). It is a silty soil classified as OH, due to its high organic matter content (9.3%), which affects the main properties of the soil (Correia et al., 2009b; Venda Oliveira et al., 2017). Thus, although this soil has a low clay content (less than 12 %), it exhibits low unit weight, high void ratio and plasticity, low undrained shear strength and high compressibility (C_c, C_r and $C_{\alpha e}$).

Table 1 shows the parameters of the constitutive models and creep laws used to reproduce the behaviour of the soft soil and stabilized soil, evaluated based on the results of several triaxial and oedometer tests (Correia, 2011).

Table 1. Parameters used in the numerical predictions (based on Venda Oliveira et al., 2017)

	Soil type	Soft soil	Stabilised soil/DMC(c)
Constitutive model		MCC	MCC/VM
Elastic parameters	E'	(a)	164.7 MPa
	v	0.3	0.3
MCC model	$e_{\lambda 0}$	2.315	5.07
	e_0	(a)	(a)
	λ	0.204	0.435
	κ	0.030	0.0074
	M	1.50	1.50
VM model	a	---	0.0013
	b	---	1.683
	R_f	---	1.0
Volumetric creep law	C_α	0.0236	(a)
	t_{vi}	1.0 min	(b)
Deviatoric creep law	A	0.001148 %/min	0.0000244 %/min
	m	0.978	0.28
	$\bar{\alpha}$	0.734	0.671
	t_{di}	1.0 min	(b)
	a	0.007	0.0013
	b	1.1033	1.683
	R_f	0.95	1.0

(a) Depends on the stress level; (b) The creep time is equal to the current time; (c) Evaluated for a curing time of 28 days.

Numerical predictions – Oedometer and triaxial creep tests

The experimental and the computed results obtained during the creep phase of oedometers and triaxial tests for the soft soil and stabilized soil are shown in Figure 1. The numerical results clearly reveal that the constitutive models and creep laws used are suitable for predicting the evolution of the creep strain under restrained lateral conditions (i.e., oedometer conditions) and under confined conditions (triaxial for a normally consolidated sample isotropically consolidated for a p'$_o$ of 100 kPa)

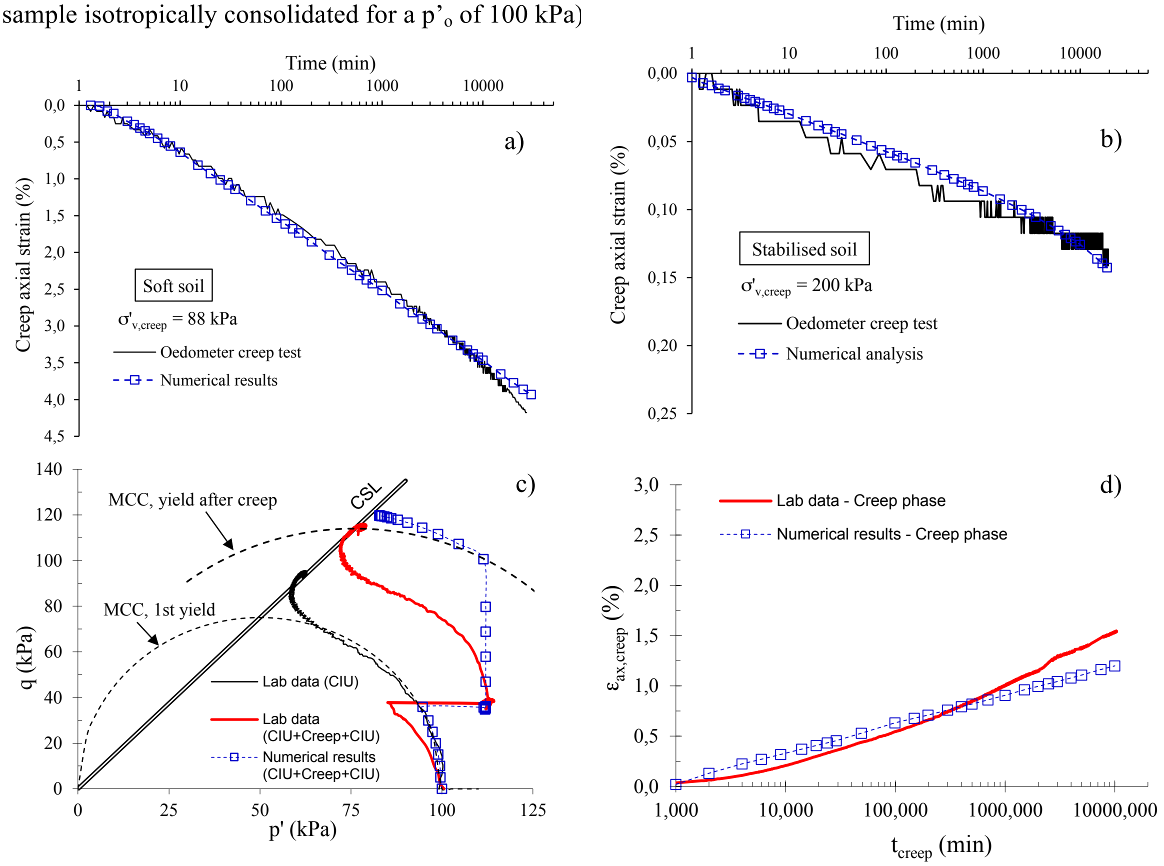

Figure 1. Laboratory results of creep tests and numerical predictions: a) Soft soil - eodometer; b) Stabilized soft soil - eodometer; c) and d) Soft soil – triaxial (based on Venda Oliveira et al., 2017)

Numerical prediction of the long-term behaviour of an embankment

The problem studied consists of a large-scale embankment with a height of 3 metres built on a soil foundation composed by 0.5 meter thick layer of sand and 7.5 metres of soft soil, lying on a rigid and permeable stratum. The water table coincides with the surface of the soil foundation. The construction of the embankment consists of three sub-layers, each one with a thickness of 1.0 metre applied with a time delay of one day. The DMC-soil system is simulated using an axisymmetric cylindrical unit cell (Figure 2a), consisting of half of one column and its radius of influence. The radius of the DMC and the unit cell are 0.4 and 1.0 metre, respectively. The finite element mesh used in the axisymmetric analysis consists of 130 eight-noded isoparametric quadrilateral elements and 355 nodal points.

Table 2 presents the initial state, elastic, strength and permeability parameters used to characterise the embankment, sand layer, soft soil and stabilised soil. A ratio of k_h/k_v equal to 3 was used for the soft soil.

Table 2. Material properties of the embankment and soil foundation (based on Venda Oliveira et al., 2017)

Material	Initial state			Elastic parameters		Permeability		Mohr-Coulomb	
	K_0	γ (kN/m^3)	OCR	ν'	E' (MPa)	k_v (m/s)	k_h/k_v	c'(kPa)	ϕ' (º)
Soft soil	0.4$^{(1)}$	15.0	1.0	0.3	$^{(2)}$	10^{-9}	3.0	MCC model	
DMCs	0.8	16.0	---	0.3	164.7	3×10^{-10}	1.0	MCC/VM model	
Sand layer	0.4	15.0	---	0.3	2.0	10^{-4}	1.0	---	---
Embankment	---	22.0	---	0.3	1.0/7.5/15.0$^{(3)}$	----	----	10.0	35.0

$^{(1)}$ From triaxial test (Correia et al., 2011). $^{(2)}$ Variable, $E' = \frac{3(1+e_0)(1-2\nu')}{\kappa} p'_0$. $^{(3)}$ Increases from the top to the bottom sub-layer.

Five numerical analyses were carried out in this study. Case A deals with a soft soil without creep, while in case

B the creep of the soft soil is considered. Cases C to E are intended to simulate the DMCs and the surrounding soft soil, thus, in case C, the analysis does not consider creep, case D includes creep only for the soft soil, while case E considers creep for both the soft soil and the DMCs.

The time evolution of the settlement under the embankment is illustrated in Figure 2. In general, the results show that the consideration of the creep phenomenon has a significant effect on the results obtained, which is more marked when the soil foundation of the embankment is reinforced by DMCs. These results also reveal that when the soil foundation is reinforced with DMCs the consideration of the creep phenomenon of the two materials (soft soil and DMCs) has a much greater impact on the settlement obtained (Venda Oliveira et al., 2017).

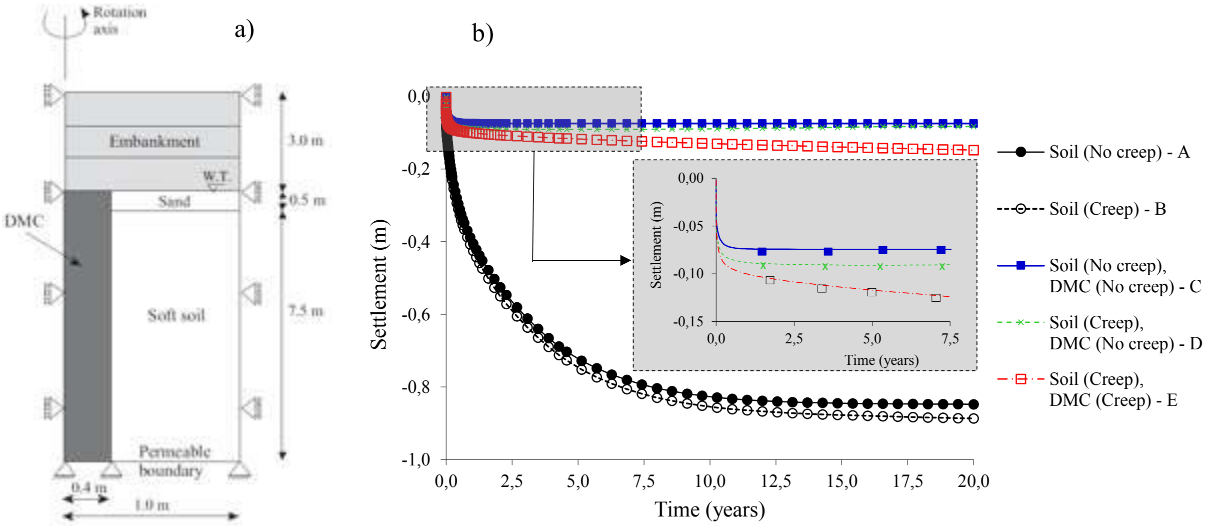

Figure 2. Numerical predictions. a) Unit cell; b) Time evolution of the settlement at the surface of the soil foundation (Venda Oliveira et al., 2017)

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EUROPEAN STANDARDIZATION FOR THE SHALLOW TREATMENT OF SOILS AND GRANULAR MATERIALS

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1. INTRODUCTION

This presentation dealt with the European Standardization for the treatment of soils and granular materials in the context of shallow improvement of geomaterials for earthworks and pavements in transport infrastructures. It was divided in two parts: the soil treatment in earthworks based on the standard series of EN 16907; the soil and granular materials treatment in pavements covered by the standard series of EN 14227. In both applications, the presentation has described the general framework of these standard series and some examples of application were also presented.

The presentation was focused only to the soil and granular materials treatment in layers, produced in situ or from a mixing plant, as opposed to the deep treatment, by columns for example. The objective of treatment is to enhance the properties of materials with poor geotechnical characteristics for use as a construction material in earth structures or in specific applications like capping layers, sub-base and base layers of pavements. The treatment is applied for two main purposes: improvement when the short term performance is the objective, or stabilisation if medium and long term performance are to be accomplished mainly through a couple of mechanical properties associated with strength and stiffness.

Although the soil treatment technique has been used for a long time, its application at a large scale, for the construction of earth structures, started in the 1960s. Its application in Europe was more oriented for soil improvement while in the USA more oriented for soil stabilisation.

Regarding the general scope of the activities of the Technical Committee on Transportation Geotechnics (CPGT), the soil treatment in earthworks is integrated in the activities of the Working Group Nº3 – Earthworks design, technology and management. The case of soil and granular materials treatment is covered by the activities of the Working Group Nº2 – Reinforcement of geomaterials and its implications in pavement and rail track design. CPGT is a technical committee of the Portuguese Geotechnical Society (SPG) that ensures, by delegation of SPG, the representation in Portugal of the ISSMGE TC 202 – Transportation Geotechnics, the development of activities associated with the terms of reference, and to enable the creation of working groups like those on the TC 202 (CPGT 2019).

2. TREATMENT IN EARTHWORKS

In the past, standardisation for earthworks has been mainly organized at national level. National experience was adapted to local grounds and climates, thus covering a part only of all existing situations. The idea of creating standards covering the whole field of earthworks at European level was suggested about fifteen years ago and it first met some resistance, each national community being convinced that their own traditions were the best and nothing could be anticipated from international cooperation (Gomes Correia and Magnan 2012). Fortunately, a CEN Committee (CEN 396 Earthworks) has now been working for eight years. The works aim at sharing national practices and creating a common framework, which keeps easy access to national experience and enables to share possibly better practices (Gomes Correia and Magnan 2012).

With the creation of this Technical Committee, several European standards within the framework series of EN 16907 on earthworks will be published covering:

- Part 1: Principles and general rules;
- Part 2: Classification of materials;
- Part 3: Construction procedures;
- Part 4: Soil treatment with lime and/or hydraulic binders (this document);

- Part 5: Quality control;
- Part 6: Land reclamation with dredged hydraulic fill;
- Part 7: Hydraulic placement of mineral waste.

The part 4 developed by the CEN Committee actually covers both improvement and stabilisation and is presented in PrEN 16907-4:2015 “Earthworks. Part 4. Soil Treatment with Lime and/or Hydraulic Binders”. It covers mainly the following items: (1) constituents and mixtures; (2) laboratory testing methodology; (3) performance classification of the mixtures; and (4) execution and control. The materials covered are: soils, weak and intermediate rocks (including chalk), recycled materials, and artificial materials. They can also be mixes of these different types: soils, weak and intermediate rocks (including chalk), recycled materials, artificial materials. The hydraulic binders covered are: cement, granulated blast furnace slag and hydraulic road binder.

The content aims to describe the requirements to the success of a treatment operation, as well as good practices that closely depend on local geological and climatic conditions. Thus, in addition to those requirements, notes of guidelines of good practices are also included in or presented in some of the 14 annexes.

This presentation was focused more in the performance based tests for stabilised soils, mainly advanced nondestructive seismic test methods for mechanical performance evaluation, in laboratory and in field (Silva et al. 2014, 2018, Santos et al. 2015). It was also highlighted the international experience using roller integrated continuous compaction control. Finally, recommendations were proposed to assure that the solution of soil treatment will be based in sustainable principles (Gomes Correia et al. 2016).

3. TREATMENT IN PAVEMENTS

The series of European standards EN 14227 covers the several specifications available for hydraulically bound mixtures to be used for construction and maintenance of roads, airfields and other trafficked areas. These standards are being developed by the Working Group WG 4 – Hydraulic bound and unbound mixtures (including byproducts and waste materials) – of the Technical Committee TC 227 – Road materials – of European Committee for Standardization (CEN). In Portugal, the national mirror subcommittee is SC 4 – Hydraulic bound and unbound mixtures (“Misturas de agregados não ligados ou ligados hidraulicamente”) – through the Technical Committee CT 129 – Pavement materials (“Materiais para pavimentação”) (IPQ 2019).

Depending on the binder type and the material to be treated (soils or granular mixtures), the following parts compose the general framework of the series of standards EN 14227:

- Part 1: Cement bound granular mixtures (this document);
- Part 2: Slag bound mixtures;
- Part 3: Fly ash bound mixtures;
- Part 4: Fly ash for hydraulically bound mixtures;
- Part 5: Hydraulic road binder bound mixtures;
- Part 15: Hydraulically stabilized soils (this document).

The most recent Part 15 has replaced the Parts 10 to 14 related to soils treated by cement, lime, slag, hydraulic road binder, and fly ash, respectively.

The standards series of EN 13286 is related to the test methods for the manufacture of test specimens and the determination of the fresh and hardened material properties. The mechanical performance is available in terms of the strength and modulus of elasticity of the hardened material. The test methods for the determination of the strength are specified in the following parts:

- Part 40: Direct tensile strength;
- Part 41: Compressive strength;
- Part 42: Indirect tensile strength.

The modulus of elasticity is determined accorded the Part 43 of EN 13286. The tests for the strength and modulus of elasticity determination shall be carried out in cubic or cylindrical specimens manufactured according the following test methods (EN 13286):

- Part 50: Proctor equipment or vibrating table compaction;
- Part 51: Vibrating hammer compaction;

- Part 52: Vibrocompression;
- Part 53: Axial compression.

The Part 1 of EN 14221 specifies the characteristics of cement bound granular mixtures, covering the properties of the constituents (aggregate, cement, water and retarders) and the mixture. The aggregates can have natural, artificial and recycled origins, or a combination of them. Five types of cement bound granular mixtures (1 to 5) are considered based on grading, compacity, and immediate bearing index of the mixtures.

The mixture requirements are specified for fresh and hardened material. Compacity, immediate bearing index and workability period are requirements of the fresh mixture. The immediate bearing capacity index of the fresh mixture should be determined in accordance with EN 13286-47 using the modified Proctor compaction. Workability period, taking into account the application and the meteorological conditions, is determined according the EN 13286-47.

For cement bound granular mixtures, EN 14221-1 presents two classification methods of the mechanical performance based on the following characteristics:

- the compressive strength (R_c);
- the combination of the tensile strength (R_t) and the modulus of elasticity (E). The tensile strength can be evaluated by direct or indirect tensile strengths. If indirect tensile strength (R_{it}) tests are performed, the following relation can be used: $R_t = 0.80\ R_{it}$.

The Part 15 of EN 14221 is related to the requirements of hydraulically stabilized soils using the following binders: cement, fly ash, blast-furnace slag, hydraulic road binder, and lime. This part covers requirements regarding the constituents (binder, soil, water and other constituents) and the fresh mixture: water content, degree of pulverization (EN 13286-48), immediate bearing index (EN 13286-47), moisture condition value (EN 13286-46) and, when required for the intended use and the weather conditions, the workability period (EN 13286-45). In addition to these parameters, others requirements related to mechanical performance and durability, in the medium and long term, should be included in the formulation study. The mechanical performance can be characterized and classified by the following methods:

- Californian bearing ratio (*CBR*) (EN 13286-47);
- compressive strength (R_c) (EN 13286-41);
- the combination of the tensile strength (R_t) and the modulus of elasticity (E).

The durability of the materials can be evaluated through resistance to water tests: strength after immersion in water, linear swelling after soaking in water (EN 13286-47) or volumetric swelling after immersion in water (EN 13286-49).

The Annex A of EN 14221-15 presents examples of “age of classification” and curing regimes for R_c, R_t and E testing of treated soils including resistance to water testing.

Besides the description of this framework of standard series EN 14221 (Parts 1 and 15), some examples of application concerning the strength resistance will be described during the presentation (Neves 2019, Neves and Gomes Correia 1995).

4. CONCLUSION

This presentation described the European Standardization for the shallow treatment of soils and granular materials in the context of earthworks and pavements in transport infrastructures. The focus was the soil and granular materials treatment in layers, produced in situ or from a mixing plant, as opposed to the deep treatment, by columns for example.

The soil treatment in earthworks (improvement and stabilisation) is based on the standard PrEN 16907-4:2015 “Earthworks. Part 4. Soil Treatment with Lime and/or Hydraulic Binders”. The standard content aims to describe the requirements to the success of a treatment operation, as well as good practices that closely depend on local geological and climatic conditions.

The requirements of cement bound granular mixtures and stabilized soils for pavement purposes are available in EN 14221-1 and EN 14221-15, respectively. These specifications are useful for pavement construction and maintenance of roads, airfields and other trafficked areas.

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SOIL IMPROVEMENT! WHERE OR WHEN?

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Soil Improvement is an old technic that is widely used everywhere. Despite being ruled by very simple empiric knowledge, comprehensive and accessible to everyone, it is normally used by a small number of engineers. The technic's concepts are basically simple and most engineers can get into it by using simple logic relations in Soil Mechanics rather than sophisticated computer software with several encapsulated black boxes.

When a large area is to be loaded with heavy distributed loads and the existing ground does not have the necessary load capacity, there is a need for a strengthening intervention in order to prevent the soil from collapse or the occurrence of unacceptable settlements. In those situations soil improvement is known to be the most efficient/competitive solution. But, just like in other business nowadays, there is no winner in advance. Depending on the requirements and specifications or ground conditions, many situations are beyond reach of strengthening solutions, leaving room to more conventional solutions.

Basically, the most efficient way to promote densification of a soil is by submitting it to an isotropic stress field,

$$\sigma_I = \sigma_{II} = \sigma_{III} = p$$

$$\sigma_{ij} = 0; \text{ for i not EQ j}$$

Under these conditions the soil can be overstressed without the risk of collapsing. The grains of soil have nowhere to go except getting closer to each other.

Unfortunately there is no developed technology available using this principle in full. The closer to this lies under the drums of the common vibrating roller, provided that it is rolling small layers of selected granular soil, away from the water table, layer by layer into a high performance embankment.

On the other hand, there is Dynamic Compaction, where a small portion of the energy of each drop is used to promote densification of the soil, right under the footprint of the dropping weight, but most of the energy is lost due to lack of lateral and surface confinement. Depending on the grain size distribution, it is possible to determine the necessary energy or the number of drops on each spot, before switching to another one. In practice its effect does not goes beyond 5 or 6 meters deep. The technology is cheap and fast to implement over large areas, but don´t expect great results. Usually a soil is loose because it is poorly graded and it can be improved only to some extent. A soil with SPT of 5 blows can possibly become a 10 blows soil, but achieving an improvement grade of even 30 blows is most likely a very hard and time consuming issue, and can turn out to be impossible with excess of fines particles or with high water levels.

Vibro Compaction lies in between the technologies mentioned above in what efficiency is concerned, the energy source can travel in depth, reaching the layers to be compacted. The vibration is generated by two eccentric masses, the amplitude and the frequency can be adjusted to suit each type of granular soil. By liquefying locally the soil, the weight of soil above does promote densification in a way similar to an isotropic pressure. But, once again, it is known that under a seismic event, liquefaction occurs normally in loose granular soils, at shallow depth, leaving this technology with limited results. Test ranges can be implemented with different meshes, parameter settings and travel speeds, but the outcome depends very much of the soil itself and the overburden pressure.

Nevertheless the technology remains useful when the specifications are not too tight, such as a container yard build over reclaim land, which will accept settlements of 20 cm over 5 years. It can be used alongside with pre-loading, unlike Dynamic Compaction.

Fine soils cannot be improved by using dynamic energy. The most common way to promote its densification is by pre-loading the soil without taking it to collapse, as pore pressure will gradually be released and the load transferred to the structure of the soil. In this process the soil will get stiffer and be able to carry more loads, meaning a staging construction that takes a lot of time. Vertical wick drains are normally used to shorten the way out for pore pressure, saving time. Wick drains are also a technology fast and easy to implement, in a fine working day a rig can drive several kilometers of drains, provided that the soil is really soft and without debris of any kind, otherwise the production will drop and the objectives turn out to be unattainable. Like other equivalent technologies, the improvement expressed in measure settlements does not always correspond in gain of resistance it promises, leaving this technology for the less demanding jobs.

When soil improvement is known to be inadequate, soil reinforcement is the strengthening method that follows. By introducing external elements into the soil, the ground will benefit from the resistance of the inclusions and gain an improvement on its capacity due to soil/inclusion interaction. Existing technology allows the use of several types of inclusions such as, sand columns, stone columns, soil-cement columns, mortar or concrete columns, etc.

Depending on how they are introduced into the ground, besides forming stiff structural elements, to some extent they can also promote soil densification in between those columns, such as displacement piles.

Like setup in piles, skin friction can improve several times in driven piles, but relaxation over time will do the opposite, so it is not wise to rely on setup in the case of piles, working as single elements. But here, in what soil reinforcement is concerned, there is a mesh of elements working together stressing the soil in between those elements. However, the horizontal stresses can be several times higher than the vertical ones causing heaving (uplifting) as the soil always chooses the easiest way to rearrange. A significant portion of the invested energy is lost during the installation process. On the other hand, as soil becomes denser, it will limit the usability of some technologies like, for instance, full displacement inclusions, putting the lower layers of soil to be improved, beyond reach of the treatment.

Sand columns and stone columns are the most common methods in soil reinforcement design. As a matter of fact this solution relies on a peculiar symbiotic equilibrium. The soil to be improved does not have the necessary resistance and deformability to bear the loads transmitted to it. The stone columns that are supposed to contribute to improve the soil disintegrates itself without the confinement offer by soil to be improved and they normally support the external loads nearly at equal shares.

In the case of very loose soils under heavy loads, it is necessary to rely on a denser mesh of columns. On the other hand, the column's capacity is inversely proportional to its length, so in the case of a thick layer to be improved stone columns are no longer an efficient option.

During a cast in situ driven piles process, to avoid damaging the recently installed piles it is common practice not to install piles next to them, until the concrete gains its resistance. Without the contribution of cement, arguably most stone columns are damaged by the installation of its neighbours. There is no practical way to control the dimensions of each column, differential and total settlements are difficult to predict, 10 to 20 cm is taken as a good result. This is an expensive solution, for it requires a high rate of substitution to achieve only acceptable results, meaning a dense mesh and a huge amount of crushed stones. It is an alternative to the wick drains solution in soft soil, trading consolidation time with a lot of money.

By adding cement to the columns or even by cementing the soil, one can easily get self-sustained columns with a tenfold increase in resistance, for a reasonable increase in cost, by turning them to a much more reliable/rigid type of inclusions. Under these circumstances, the columns with an enhanced modulus of deformability will take almost all the loads with much less overall settlements. Good results can be achieved by putting the soil and columns to work together under a simple but comprehensive rule, through the equilibrium of skin friction over the soil-column interface. Whenever the soil is weak, it will transfer the loads to the column through negative skin friction, so that the columns can carry the loads to the stiffer layers of soil, underneath. The inclusions cannot be too far apart and a well compacted interposing thick layer between the loads and columns is convenient, known as a transfer platform, in order to carry the surface loads to the inclusions through the Arc Effect rather than Bow and String, performed by a geosynthetic grid.

There are a lot of high strength geosynthetics on offer, but it is the stiffness, the modulus of deformability that matters. Despite its high resistance if the geosynthetic extends like a rubber band the surface loads will be transferred to the soil underneath, not directly to the inclusions.

Historically the first inclusions used as soil improvement elements were timber piles, driven into the top layers of soil in order to promote local densification, especially in wetlands. It can be found in downtown Lisbon under the heavy walls of six story buildings that stand over a thick layer of alluvial sand with the water table just three meters below the surrounding streets. Thanks to this ground improvement procedure many historical buildings, rebuild after the massive 1755 earthquake, still endure remarkably well.

Nowadays there are many types of rigid inclusions, from jet-grouting, colmix, displacement piles, full displacement piles, auger piles and a few more variants. To distinguish its functionality from the piles they are named columns. As far as the material's resistance is concerned each solution fits between soil-cement and concrete, giving a wide range of options available to each design. The technology for the installation of the inclusions plays an important role in the improvement process; in some cases it is convenient to use different procedures to cover the full scope of the job.

An inclusion of colmix is probably more precise in shape than a column of jet grouting, but the high energy of the jet is more favourable in densifying the soil. If precision and control are required, concrete columns should be the correct option. With the modulus of deformability and dimensions well established they can either be driven, installed as full displacement elements or as CFA bored piles. Auger piles can reach the design depth without major concerns, but full displacement piles or even driven piles are frequently limited by the capacity of the installation rigs, leaving part of the design requirements unfulfilled.

Auger piles do not promote densification of the soil as driven piles or full displacement piles do. Even for small diameter piles, let's say 0,5 m, in full displacement variant, it will take a 100 ton rig to drive through 10 m of loose soil, to reach a 10 blows soil, while a pre-cast concrete pile can reach a 30 blows soil by using an impact hammer. If the layer of soil to be improve is around 10m thick this technology can be used efficiently, but with thicker layers there are other solutions. Like in driven piles, if one cannot drive through it, doesn't mean that the soil is stiff enough. But is it necessary to promote densification of the soil in a context that it shares no more than 10% of the total loading?

Like designing a bridge one can use a mesh of piles supporting a light deck, or a small number of strong pylons supporting a heavy deck, with larger spans. If the bed-rock lays at shallow depth a light deck could be the solution, but an expensive heavier deck with a small number of long piles, can turn into a winner if the bridge require very long piles.

In the case of a tank farm for the petroleum industry, conforming to API650 the differential settlement cannot exceed 13 mm per 10 m of circumference, or a uniform settlement of 50 mm. To achieve this results, soil improvement or reinforcement cannot solve this issue by itself, a buffer, usually a reinforced concrete slab, is required in order to smooth the individual behaviour of each column. For tanks of 40 m in diameter and 20 m height, transmitting 200kN/m2 live loads, the traditional solution calls for a reinforced concrete slab sitting on conventional piles.

In the case of settlement sensible structures like gas tanks, buildings or bridges, soil improvement or reinforcement are not a valid alternatives to conventional piling, due to heavy concentrated loads and very small settlements acceptance. Under these circumstances they turn out to be less cost-effective. A 40 MPa concrete is easy to obtain with 350 kg of cement and the right aggregates, but in muddy soils even with 700 kg of cement, one can hardly achieve a resistance of just 4 MPa with jet-grouting or other similar technology.

Injections are also used in soil improvement. They are only valid for granular soils. Injection means filling the voids in between the particles that forms the soil, with cement or other chemicals and, recently, even with biological products in order to improve its resistance or to reduce its permeability. Cement is the most common solution with proven reliability and durability, but even with micro cement, due to its grain size, it can only be used in high permeability soils. For lower permeability soils there are chemicals that are injected in liquid form, and after a while will turn into solid, adding cohesion to the soil, as cement does. To spread the product uniformly through the soil to be treated require a dense mesh of injection pipes, introduced into the ground. It is a very expensive and time consuming activity, even so a significant part of the soil remains unreachable to the treatment. For this reason this technology is more often view as a corrective method in singular situations, rather than a soil improvement technology in full.

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CONSTRUCTION INSURANCE (CAR) GUARANTEES AND EXCLUSIONS CHALLENGES IN THE GEOTECHNICAL AREA – CASE STUDY

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INTRODUCTION

In this document the liability of the construction insurances (CAR) in the Geotechnical area is addressed with the example of a real case study were a circular sliding surface at a earth fill slope was mobilised and considered as the main reason for the slope loss of stability.

CASE STUDY

Description of the contract works

Within a road construction contract, the works under question covered an extension of about 300m.

The main works implied the several earth works, including excavations and earth fills. The 10 m high earth fill was implanted in an area crossed by a water line, being constructed a hydraulic underpass.

It should be pointed out that at the loaction area of the mentioned earth fill, at the initial design stage, no geological and geotechnical investigation works were proposed.

Foundation solution

The solution defined at the final design stage, already during the construction schedule on site, after the execution of geological and geotechnical investigation shafts, was the vegetation removal and the direct application of a draining layer (stone material) with around 0,5m of thickness.

The loss

The loss corresponded to the mobilized circular earth fill slope sliding, occurred at the final stage of the fill execution, when it was just about 1m to achieve the crest level. The sliding surface at the earth fill slope was assumed as circular, as showed in Figure 1.

Figure 1. View of the sliding surface

Slope overal loss of stability

The earth fill foundation, with low bearing capacity, which was not properly accessed at the design stage, induced the mobilizing a circular sliding surface, and was considered as the main reason for the slope loss of stability. This shows a CONCEPTION ERROR of the solution adopted in the earth fill foundation.

Main repairing works

A slope and earth filling repairing solution was defined, including the following main works:

a) Site investigation bore holes to access the geological and geotechnical scenario;
b) Removal of the backfill body, including the hydraulic underpass;
c) Removal of the drainage layer at the foundation level;
d) Replacement of the foundation natural and soft ground by crushed stone material to a depth of 5m;
e) Execution of a new earth fill, as well as a new drainage system: gravel protected by a geotextile.

Loss adjustment – value to be indemnified under the policy

The loss was caused by a defective conception of the earth fill foundation, being under the terms of the Policy attributable to DEFECTIVE DESIGN.

Once the loss occurred during the Construction Period and that the policy had contracted the SC – Consequences of Defective Design, the risk that originated the loss was guaranteed under the Coverage, being the loss to be indemnified under the terms of such SC.

- The costs with the repairing of the "Defective Part", as well as the costs with additions, alterations and improvements of the original design, are excluded.
- The costs with the repairing of damages in the parts of the works that had been proper executed and sustained damages, as the error consequence, are included.

So, being confirmed that the earth fill body was correctly executed (adequate materials and compacting level) the final result corresponds to the one indicated in Table 1.

Table1. Repairing Works and Policy Liability (values in Euros)

Repairing Works		**Policy Liability**			
Item	**Total Cost**	**SC** Debris Removal	**SC** Consequences of Design Errors	**Excluded**	Remarks
Site investigation bore holes and new Design Solution to be adopted	17.800			17.800	Design Alteration
Removal of earth fill body and Hydraulic Underpass	130.000	130.000			Debris removal
Removal of stone material applied to reinforce the foundation	7.000			7.000	Defective Part
Addtional excavation down to 5 m beneath the base level of earth filll and filling with stony material / draining layer	195.000			195.000	Design Alteration
Execution of soil internal gutters and draining masks	36.000			36.000	Design Alteration
Additional expropriation for trenches opening to conduct the water of the draining layer	8.500			8.500	Design Alteration
Re-construction of backfill, Hydraulic Underpass and slope descents	220.000		220.000		Reconstruction of Parts Proper Executed and have sustained Damage
Total	**614.300**	**130.000**	**220.000**	**264.300**	
		350.000		**Loss to be indemnified**	
		Deductible	**80.000**		
		Indemnity	**270.000**	**<> 44% of Total Cost**	

FINAL REMARKS

Further to what was described in this document, the following main aspects should be highlighted:

- An accident in a Construction Contract corresponds, in the majority of situations, to a Failure of the Construction Solution
- A Construction Insurance Policy never covers All the Losses resulting from a loss
- The Risk should be duly evaluated at the design stage and followed up during the construction stage in order to Prevent the Loss, as a Failure Prevention approach

MONTE GORDO'S SLOPE, VILA FRANCA DE XIRA: ANALYSIS STABILIZING SOLUTIONS

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1. Case Study: Monte Gordo's Hill slope, Vila Franca de Xira

This article, which is based on the dissertation with the same name made by the author in partnership with Câmara Municipal de Vila Franca de Xira, arises from the occurrence of an experienced slope instability phenomenon located in Monte Gordo's Hill in Vila Franca de Xira. Given the characteristics of the hill, it was installed a quarry in its upper part of the emerged rock massive exploitation, consisted on Monte Gordo limestone and marl-limestone and stoneware complex characteristic from the Tagus zone. The slope was located approximately in the middle of the hill. Over time, it was being deposited limestone material from randomized blocks forming the rejected quarry, including the slope area. After the quarry inactivation, an illegal landfill material was created from other excavations, with very weak geotechnical characteristics, covering directly the massive rock formation, creating a layer of landfills, and which coincides with the instable slope location. This action led to the origin of a slip surface given the disparate mechanical characteristics. Indeed, the known instability problems began in the early nineties, with the construction of an urbanization at the slope base (Figure 1). At this time, it was also built a property, in the northeaster part of the slope, whose access road was located parallel to that block of buildings, which was also affected. The topographic gap between them is 20 meters, which demonstrates the significant slope inclination. To implement the considered block of buildings, it was necessary to remove a large amount of ground, which was held without any design study or adequate containment structure. At the final of the construction, it was directly applied the local ground landfill at the buildings behind walls in a height of approximately 9 meters, which made a non-negligenciable impulse. Referring to the building's stability project, it was carried out some gaps, which the most important for the study was the fact that the mentioned walls hadn't been correctly designed. Completed the buildings execution, it was occurred several landslides in the slope, because of the existence of the referred slip surface, responsible for the instability phenomenon located overlying the buildings, in which they cause the increment of the movements in the contact between the two substrates with different properties. Given the slope instability occurring, it was supplemented the instrumentation over the entire instable area, and so, to prevent the continuation of this phenomenon, it was materialized a stabilization solution with a Mechanically Stabilized Earth with geogrids, in 2002/2003 along the entire length of the building block. This solution allowed, not only, the replacement of the unstable materials and the regulation of slopes, but also, to eliminate part of the ground that was directly supported by the building's walls, about 3 meters of total height, which caused impulses that it was not properly design. However, it was remained an impulse of 6 meters height directly applied on the back wall. Regarding the reinforced embankment constructive phasing, it was necessary to remove a significant and complex amount of ground from the slope for his materialization, with approximately 20 m high. This excavation was carried out without the use of any containment structures. Over time, the analysis of the instrumentation devices placed allowed to verify a continuous increase of the ground deformations, and the displacement problems maintained over the years. It was concluded that the slope stabilization solution implemented previously, promoted only a superficial slope stabilization, rather than, its overall stabilization, because they did not have a significant effect in terms of the deeper slip surface, and so, the interaction between the slope movements and the buildings wasn't dissociated (Figure 2).

In 2013, given the unaffordable situation of slope instability, affecting human lives and material goods, it was preceded a second stabilization solution that guarantees that all ground pressures acting on the buildings were eliminated through a controlled ground movement, until reach buildings foundation quota, approximately. The face of Monte Gordo´s Limestone rock massive, which was covered by the landfill layer, was exposed and suitably consolidated using covering solutions, with nailing and reinforced shotcrete, and it were materialized slopes and intermediate benches, with an efficient drainage system throughout the area. Given the buildings deterioration state, it was decided to implement a preliminary phase of excavations, called by immediate intervention, associated to an efficient drainage system, consisting of the removal of the first 3 meters of the reinforced soil embankment, to reduce some active impulse. The final solution is currently applied in the slope, with a favourable effect in mitigating the slope instability phenomenon, as it was observed the decay of the displacements in the instrumentation placed in the area, despite the slip surface continued to happen above buildings foundation ground. Finally, in 2017, it was demolished, from the slope, two of the buildings with major damages observed, to definitively end the slopes problems.

2. Geologic and Geotechnical Scenario - Immediate Intervention

With regard to the geotechnical-geological scenario, in addition to have slightly increased the slopes stability, the immediate intervention was important for the definition of the final solution, at the characterization of the area. It was verified the high complexity of the Limestone rock mass configuration, and its uncertainty to a greater depth, since it was not visible. Also, it was observed that, at a more superficial level, the Abadia marls were very plastic and humid, becoming surfaces of fragility in the slope. Furthermore, the area of the slope was crossed by numerous tectonic accidents, particularly a tectonic fault to a great extent, constituting preferred ways of water percolation, accelerating the kinematic movement between the two layers at a higher depth. In addiction, it was inferred that the materials inside the Mechanically Stabilized Earth with geogrids were very humid and decompressed, since the peripheral drain, at its back, operated as a water collector. So, it were distinguished two lithological units, one given by the most recent landfill deposits already described, and another, given by the Jurassic formations. The first one is directly based on the last mentioned. The first stabilization solution was considered as a landfill depot due to its high deterioration state.

3. Stabilization Final Solution

The final solution was followed during its execution, and it is actually materialized in the slope, consisting in a considerable ground movement, in order to eliminate the impulses provoked by the slope instability on the buildings, based on a bench materialized in all its behind length, corresponding to its foundation quota. This solution had the particularity of being related to the Limestone rock mass configuration and, therefore, to the adaptation of the actual geological-geotechnical conditions found in the place, as the work was carried out. It was possible to identify three zones of the solution (Figure 3). The Zone 1, where the previously mentioned slopes were materialized, considering an adaptation to the ground in order to optimize the solution. The Zone 2, where the Limestone rocky massif was undiscovered, and so, the definition of its face treatments were adapted according to its degree of degradation/cracking by the design team, and so, according to the observed in the immediate intervention, it was considered a systematic treatment of the face to be implemented. And the Zone 3, where it was materialized the slopes with the benches mentioned above, which finished directly in the Limestone mass rock face already exposed, and it was considered a bench placed along its entire face, in order to permit periodic inspections, and to link all the other benches. In Zone 2, as would be expected, major modifications occurred, since the Limestone rock massif was not visible, and it was not known its configuration at deeper levels. Therefore, it was considered three zones with different solutions and characterizations in Zone 2 (Figure 3).

Figure 1. Aerial view of the unsteady slope location area (Nunes, 2015)

Figure 2. Deformations/Rotation of the affected buildings (Nunes, 2015)

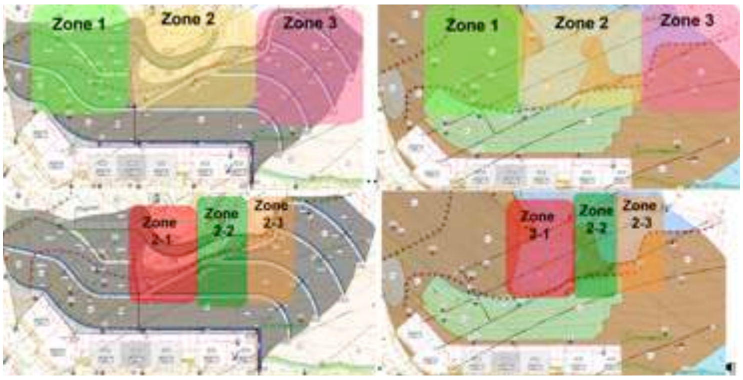

Figure 3. Stabilization Final Solution – Zoning along the slopes area (Nunes, 2015)

In the Zone 2.1, almost in front of the buildings with greater deformations, it was verified the presence of an edification (Figure 4), whose wall settled directly in the Limestone rock massif face. This last one was somewhat degraded, as such, given the proximity of the tectonic fault, it was decided to apply the systematic treatment of the face previously defined, with reinforced shotcrete and nails. At higher depths only reinforced shotcrete was placed, in order to protect against atmospheric agents, since the Limestone rock massif face has improved its mechanical characteristics (Figure 5).


Figure 4. Limestone rock massif face at zone 2.1 (Nunes, 2015)


Figure 5. Limestone rock massif face at zone 2.1 (Nunes, 2015)

In Zone 2.3 area, the Limestone rock mass showed signs of some alteration/cracking, however, the substrate was self-supporting, as it can be observed by the occurrence of another house inside the massif (Figure 6), so it was applied the reinforced shotcrete. The mechanical characteristics of the massif were improved in depth, therefore, the reinforced shotcrete was projected. However, it was placed some nails in some detached blocks (Figure 7).


Figure 6. Limestone rock massif face at zone 2.2 (Nunes, 2015)


Figure 7. Limestone rock massif face at zone 2.2 (Nunes, 2015)

In Zone 2.2, in the course of the excavations works, it was observed an interruption in the alignment of the Limestone rock massif between the two massifs presented in the two zones mentioned above, where a Marl rock mass was found, with lowly mechanical characteristics, already measured in the slope longitudinal direction at the immediate intervention (Figure 8). However, in the slope transverse direction, it was not known its extension (to the interior of the slope), which affected the access to the upper part of the slope. It was decided to keep it, which would cause the Marl rock mass to have unacceptable inclinations for their properties. As such, the stability of the site was again analysed, and it was considered the systematic treatment of the design phase, but with a tighter mesh of nails, to improve slope safety conservatively (Figure 9).

Figure 8. Limestone rock massif face at zone 2.3 (Nunes, 2015)

Figure 9. Limestone rock massif face at zone 2.3 (Nunes, 2015)

Continuing the excavations over the slopes area, the Limestone rock massif was found with a slightly different geometry, reason why, the disposition of the benches were adapted in Zone 3. However, it was materialized the bench accompany the whole massif face, as previously planned. At Figure 10, it could be seen some of the most important stages during the construction work, for instance, the Golden Stone demolition, the execution of the drainage system, and the concreting of the bench at the length of the buildings behind wall. Some aspects of the construction phases and the work evolutions must be highlighted, such as, the multi-fronted excavations for time optimization, the work platforms, always maintaining the workers safety on site, and the adverse atmospheric factors that were felt at that time. Figure 11 shows an overview of the work carried out in the year 2015.

Figure 10. Construction techniques and stages

Figure 11. Overview of the work carried out in the year 2015

Numerical Modelling

The modelling aim was to simulate numerically the mechanical soil behaviour, in relation to the interventions carried out over time in the slope, in order to evaluate the phenomena that triggered the various instabilities that occurred in it. With this modelling, it was possible to evaluate the slope stability in the various intervention phases, by observing the critical slip surface and corresponding safety factor, using two commonly used methods, the finite element method (FEM) with a strain-deformation analysis with the EF program (Plaxis 2D), and the limit equilibrium method (LEM), with the program GeoStudio-Slope/W (2007-versão 7.2.3). Also, it was possible, not only, to verify the buildings interaction with the ground, but also, to see the action of the buildings in the mitigation of slope instabilization phenomenon. In other words, if the buildings contributing to the slope stabilization, or if they increased further the slopes instability.

It was considered a two-dimensional calculation model, however, given the existence of a strongly three-dimensional instability mechanism that involves the entire slope, this approximation has a marked simplification. In order to represent the history of the interventions made over time in the slope, it was considered the events that contributed most to its instability, and so, some considerations were taken in modelling for the element's simulation, mainly in the representation of the buildings soil interaction, as rigid bodies, and the construction of the Mechanically Stabilized Earth with geogrids. As part of the stability analysis, the safety factor (FS) was obtained using the FEM calculation procedure Phi/C Reduction, where the resistance parameters Φ' e c' are successively reduced until soil rupture occurs, and in LEM, it was considered the slice method of Morgenstern-Price, that satisfies both force and moment equilibrium, with a half sine function, and the circular shape of the slip surface by the Grid and Radius analysis type. As regards to the geotechnical zones, and considering the geological and geotechnical recognition of the affected area by the intervening entities, it was considered the geological-geotechnical section most conditioning along the whole unstable area, since it was where the ground presented greater displacements, and includes the property that experienced bigger damages, compared to the other buildings, in relation to the instability phenomenon. In Table 1 and Table 2 the characteristic parameters for each layer and model considered are presented.

Table 1. Soil Parameters in *Plaxis 2D* (Hardening Soil Model) (Nunes, 2015)

Parameters	**Geotechnical Zones**			
	$\mathbf{At_1}$	$\mathbf{At'_1}$	**G**	**M**
	Waste landfill and rejected limestone blocks from the quarry		Stoneware-marl	Marl-limestone
N_{SPT}	12		39	123
γ_{unsat} (kN/m^3)	18		21	21
γ_{sat} (kN/m^3)	20		23	23
Type of material	Drained			
$c'(kN/m^2)$	10		0	200
$\Phi'(°)$	32		39	25
$\Psi(°)$	0	0	0	0
$E_{50}^{ref}(kN/m^2)$	5500	15000	130000	200000
$E_{oed}^{ref}(kN/m^2)$	5500	15000	130000	200000
$E_{ur}^{ref}(kN/m^2)$	16500	45000	390000	600000
m(-)	0,5			

Table 2. Limestone massif limestone parameters in *Plaxis 2D* (Linear elastic Model) (Nunes, 2015)

Parameters	**CMG - Monte Gordo limestones**
γ_{unsat} (kNm^3)	25
γ_{sat} (kNm^3)	26
Type of material	Drained
$E^{ref}(kNm^2)$	250000
ν'(-)	0.15

To represent the Mohr-Coulomb failure criteria of the geotechnical zones, it was considering the soil strength parameters presented in Table 1, and for the limestone rock massive it was considered 25 $\mathrm{kN/m^3}$ for γ, 107 $\mathrm{kN/m^2}$, and 46° for Φ'. Due to the geometry complexity of the problem, in Slope/W program, associated with the LEM methodology, it was necessary to represent the building as a soil mass with properties such that there was not possible the existence of a slip surface inside of the property, as regarded in FEM. The Mechanically Stabilized Earth with geogrids was modelled as in FEM. The slope security verification to global ultimate limit state, as a result of actions undertaken, was carried out by considering the philosophy present in (Eurocódigo 7, 2010), considering the AC1-Comb.2. In Figure 12 it is presented the FS of the various phases obtained by the LEM and the FEM.

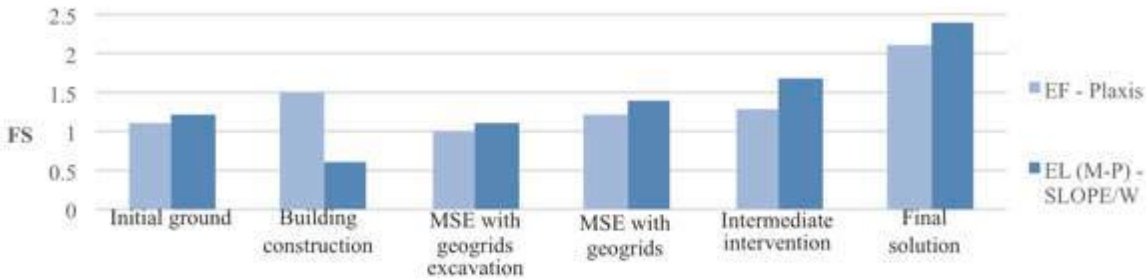

Figure 12. Modelling safety factors (Nunes, 2015)

With reference to the FS obtained by the FEM, it could be seen that the value determined by the LEM is higher than the measured by FEM, as inferred in (Aryal, 2006), with a dissimilarity between 1% to 6%, except in buildings construction phase, as it had significantly simplifications, and the final solution. Considering Figure 13, it was concluded that the LEM allows to evaluate with a high degree of conservatism the proximity of the collapse and the geometry of the critical slip surface for most of the current cases, however, by including a rigid-plastic constitutive law, it doesn't allow the assessment of the generated displacements compatibility in the ground. This limitation is overcome in the FEM, which allows a greater results consistency, even when the geometry of the problems is quite adverse, as in the case study, although it presents significant convergence problems to obtain the solution.

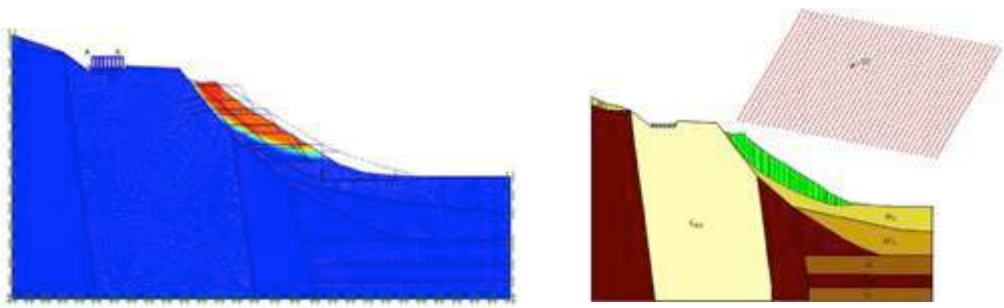

Figure 13. Critical slip surface in initial ground stage modelling: MEF (Left) and MEL (Right) (Nunes, 2015)

Particularly for the case study, considering the initial ground, represented in Figure 13, it was verified that its stability is quite precarious. For the buildings implementation stage, since numerous approximations were taken in LEM, the FEM result was considered more consistent, as the FS is superior to the unit, and has an order of magnitude higher than that corresponding to the previous step. At Figure 14, the SR develops only behind buildings, so it was confirmed that, although in the first instance its construction originated the instability problems, it was verified that the buildings acted positively in the mitigation of the slope instability process later. Also, the buildings rotation observed in the local meets the mentioned, since it is contrary to all the applied actions at its behind wall.

The FS value for first stabilization solution compared to the buildings construction phase was smaller, thus it demonstrates the inefficiency of the solution, and, as can be seen from the concentration of plastic points in Figure 15, the action of the Mechanically Stabilized Earth with geogrids weight increased the slippage at the interface between layers with rigid disparities. Furthermore, it was still applied an impulse of 6 meters at buildings behind wall, and they presented the previously mentioned deformed in the place, so the buildings continued to have a beneficial effect on the slope stability. The solution that is currently in the slope promotes its stabilization conditions, since the FS is well above the limit considered acceptable for stability, although the interaction between these and the soil was not yet dissociated due to the SR that developed to a greater depth and, therefore, there were still observed displacements in the instrumentation.

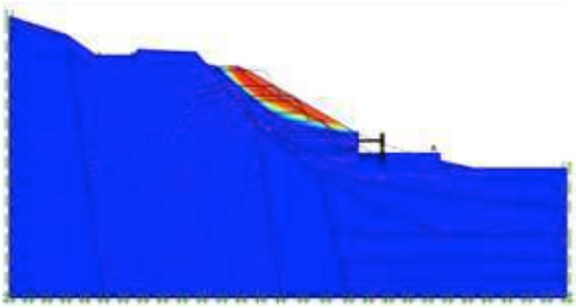

Figure 14. Critical slip surface from Plaxis 2D for initial ground phase (Nunes, 2015)

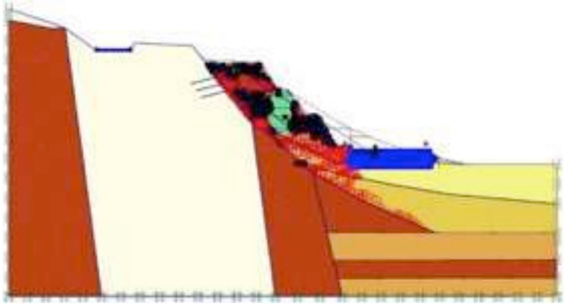

Figure 15. Critical slip surface from Slope/w for initial ground phase (Nunes, 2015)

Conclusions

The analysis carried out on the instability phenomenon has demonstrated the causes and effects experienced in the slope and, consequently, in the buildings located at its base in an assertive way. It was concluded that the inadequate buildings location on the slope, despite being at the origin of the instability problems were, in the long term, likely to promote the ground stability. According to good practice, the use of a building as a stabilizing element should obviously not be the main objective of any edification. It should also be pointed out that the redundancy of the performance of a construction, due to the non-feasibility of others previously carried out, confirms the importance of a reliable analysis of the situation, especially stability evaluations, in order to cover all possible consequences, and promote a greater reserve of human and material goods, as well as the assertive choice between the various solutions. In cases of a more complex nature, like this, with very adverse socio-economic implications, and sensitive structures, the importance of implementing an instrumentation and observation plan, with a geological prospecting over time, will have an added value to anticipate the events, which took place in the slope and timely interpret the soil behaviour, in favour of the affected structures safety, whose instability phenomenon consequences can be prolonged indefinitely. The definition of these plans should be an investment, with proactive and advantageous results, and not as an additional cost. Although the case study is not an exception to the rule, the random deposition of materials from other places, instigating the formation of illegal landfills on land that is not intended for this purpose, should have assertive control by the various entities, despite their difficulty, promoting good ethical conducts. It is also highlighted the complete used ground classification by the various municipalities, through the definition of Territorial Planning Plans, in order to avoid construction in areas less favourable to occupation, such as the present case.

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SOLUTIONS FOR THE TREATMENT OF THE FOUNDATION SOFT CLAYS OF CARREGADO HYGHWAY INTERCHANGE, LOGISTIC PLATFORM OF NORTH LISBON AND RESPECTIVE CONNECTIONS TO HIGHWAY A1 AND NATIONAL ROAD EN1

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The interchange between highways A10 and A1, the new Logistic Platform of North Lisbon (PLLN) and the respective accesses to highway A1 and national road EN1 are located 8 km north of Vila Franca de Xira, near the Carregado village. These infrastructures are located in the right bank of the alluvial basin of Lower Tagus Valley which hosts a sedimentary record relating sea level rise and fluvial sediment supply, making this a unique site along the European Atlantic margin (Figure 1).

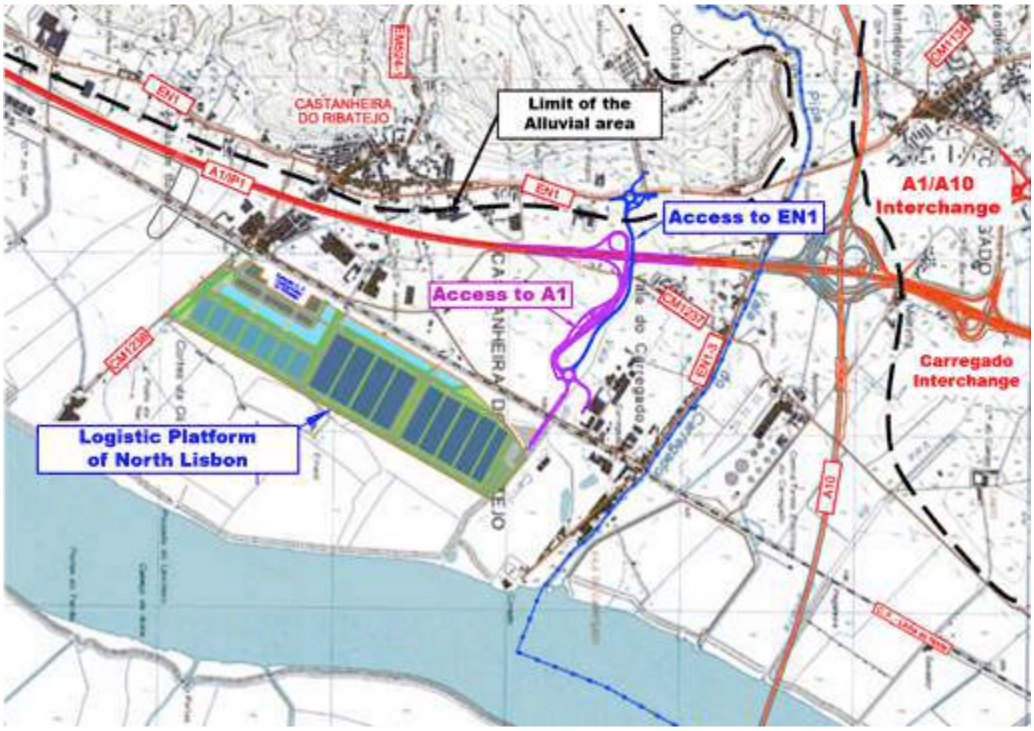
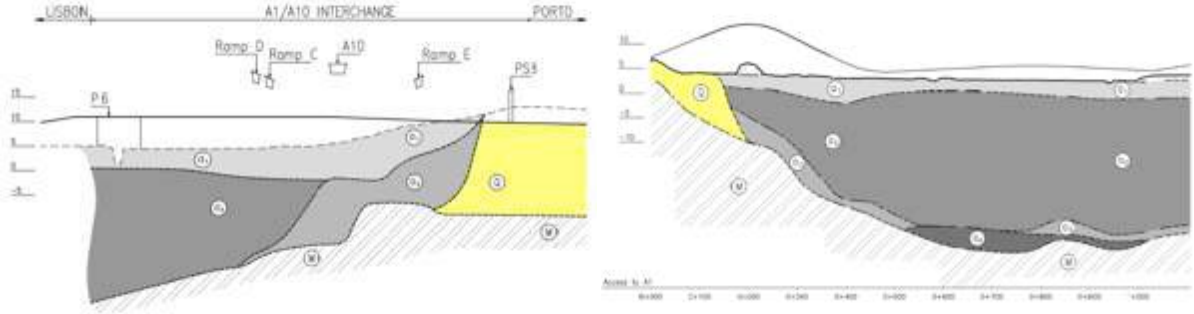


Figure 1. Localization plan of Carregado Interchange A1/A10, PLLN and accesses to A1 and EN1

The alluvial basin is characterized by the presence of highly compressible and weak alluvial deposits, overlaying Miocene formations. The unfavourable geotechnical characteristics of the sites, where the alluvial soft clay sediments reach a maximum thickness of around 40 m, imposed important constrains for the conception and design of the works, namely embankments height limitations and a set of special foundation solutions to guarantee the adequate behaviour of the embankments.

The type of solutions defined for the foundation treatment included: i) preloading embankment, with or without vertical strip drains and with or without lightweight expanded clay aggregate in the definitive embankment; ii) load transfer platforms with geogrids or concrete slabs on concrete precast driven piles or bored piles; iii) preloading embankment with stone columns.

As shown in the longitudinal profiles of A1 highway in the interchange (Figure 2a) and the access from A1 to PLLN (Figure 2b), the alluvial formations are mainly constituted by soft clays (unit a_2), by silty-clay formations (units a_1 and a_3) and sandy formations (a_4). Unit a_1 is consolidated silty-clay alluvia with a maximum thickness of about 5 m. Unit a_2 is composed of high compressibility and low strength soft clays, with some lenticular levels of silty-clay sands, presenting in these zones a maximum thickness of about 30 m. Unit a_3 is a silty-clay alluvia, in general with small thickness, but in some places can reach about 15 m. Unit a_4 is a sandy gravel alluvia soil with thickness reaching a maximum of 10 m.



a) b)

Figure 2. Geological longitudinal profile (V scale 10x H scale): a) A1 in the interchange zone; b) PLLN access from A1

The soft clays present the following geotechnical characteristics:

- %<0,074 mm - 62 to 90%; %<2 μm - 30 to 67; LL - 60 to 82%; IP - 32 to 48%; w_n - 44 to 80%; S_r - 91 to 100%
- Saturated bulk weight - 15,4 to 17,3 kN/m^3; Dry bulk weight - 8,6 to 12 kN/m^3
- c_u - from 0 to 4 m: 15 to 20 kPa - lower: between c_u = 7,14+1,96 h and c_u = 11,34+2,13 h (vane test and DMT)
- C_c - 0,55 to 1,00 (most frequently lower than 0,70); $C_c/1+e_0$ - 0,21to 0,32 (most frequently lower than 0,25)
- C_v – 1,5 x10^{-8} to 4,6x10^{-8} m^2/s (oedometric test); k - 1,0 x10^{-8} to 3,2 x 10^{-10} m/s (oedometric test)
- C_h – 5,9 x10^{-8} to 10,0 x10^{-8} m^2/s (dissipation tests CPT_u)

In the A1/A10 interchange (Figure 3), due to the high relative levels of the new A10 and the connecting branches to A1, it was decided to accept only the construction of embankments with a maximum height of 4 m and, consequently, to build a significant number of viaducts with piled foundations. The embankments foundations were consolidated using preloading. In the zones of widening of the old existing embankments of A1, a solution of load transfer platforms with geogrids supported on concrete precast driven piles was adopted, to avoid inducing settlements in the existing highway. In Figure 3 the location of the embankments and of the load transfer platforms are represented. The solution of load transfer platform is presented in Figure 4.

Figure 3. Plan and aerial view of A1/A10 interchange

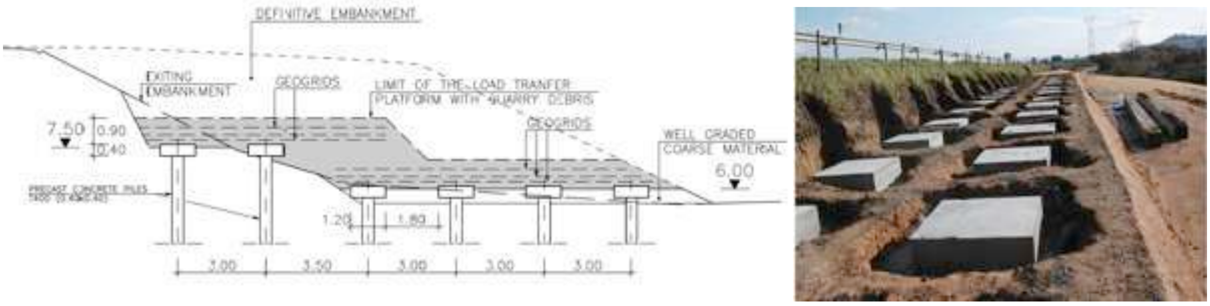


Figure 4. Existing old fills widening foundation solution. Cross section (left) and driven pile caps (right).

In the PLLN the adopted solutions for the industrial building platforms was preloading and stone columns associated with small thickness preload embankments. The solution of preloading consisted of putting over the general fill of the logistic platform, with thickness of 2.0 to 2.5 m, a 3.5 m thick preload embankment. The general fill of PLLN was built using quarry debris resulting from the exploitation of limestone rocks, in Alenquer quarries, located at a distance of about 13 km. In the PLLN embankments construction was envisaged to use $2x10^6$ m^3 and in the access embankments about $0{,}1x10^6$ m^3. In Figure 5 can be observed the general embankment of debris and a clay preload fill over the general fill.

Figure 5. PLLN general platform with query debris and preloading embankment, with clayed soils

In the connections of A1 and EN1 to PLLN (Figure 6) the solutions adopted consisted mainly of preloading fills, with or without vertical wick drains and with or without lightweight expanded clay aggregate in the definitive embankment, and load transfer platforms with geogrids supported on concrete precast driven piles or with concrete slabs supported on bored piles. In Figure 7 it is presented the combined solution with wick drains, preloading and lightweight aggregates in the definitive embankment. In Figure 8 it can be seen the wick drains installation, the preload embankment execution and the lightweight aggregates embankment execution.

Figure 6. PLLN road accesses plan view

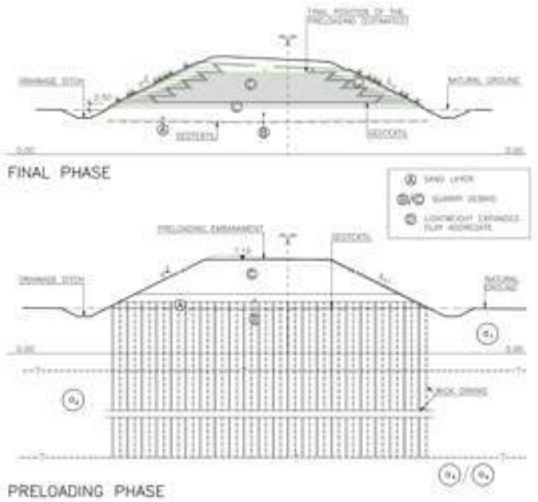


Figure 7. Cross section of preloading with wick drains and lightweight fill in the final phase

b) Preloading construction

a) Wick drains installation

c) Definitive lightweight fill

Figure 8. Execution of wick drains, preloading embankment and lightweight embankment

This presentation focuses on the description of the most relevant geological and geotechnical conditions related with the soft clay deposits and on the main aspects related with the design and construction of the special foundation treatments, which were considered necessary in order to allow the construction of the envisaged structures.

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