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Comparison Between Stone Columns and Vertical Geodrains with Preloading Embankment Techniques

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COMPARISON BETWEEN STONE COLUMNS AND VERTICAL GEODRAINS WITH PRELOADING EMBANKMENT TECHNIQUES

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ABSTRACT

In the framework of "Radès-La Goulette"bridge project (Tunisia), this study focuses on the construction of embankments located in north Lake of Tunis. These embankments with averaged height of about 6 m are founded on highly compressible clayey sand and muddy sand layers. A soil improvement technique is then imposed, to overcome the lack of low bearing capacity and high pronounced settlements. Two solutions of soil improvement have been studied; the first one consists in vertical "Geodrains" drilled until 10 m depth associated with step by step construction of preloading embankment. The second technique is stone columns reinforcement up to 10 m depth. It is focused at estimation of bearing capacity and prediction of settlement of reinforced soil by handling the recent elaborated software programme "Columns". The evolution of consolidation settlement of embankments as a function of time is also considered. The consolidation of improved soil is studied by using the "poroelastic" prediction model and the Barron's theory. A comparison between the two soil improvement techniques from the technical and economical viewpoints is presented. Compared to the "Geodrains" technique, the reinforcement by stone columns including the execution of embankments approximately leads to a gain of eight months and slightly cost reduced.

INTRODUCTION

The study of embankments on compressible soils is one of the delicate problems which has been analysed by a large number of authors. At the present time, in spite of all experience obtained over the last decades, the stability of this kind of constructions still collocates diverse and delicate issues as related to the weak bearing capacity, large settlements due to high deformability and low permeability, and too slowly dissipation of excess pore pressure (consolidation).

Designing embankments on highly compressible soils usually involves soil improvement techniques as useful alternative which allow a reasonable duration of construction especially for big projects in coastal areas which basically includes land reclamation.

The big project "Radès-La-Goulette" bridge which connects the north and south parts of the capital Tunis (fig. 1) comprises four lots. Part of them is the construction of four embankments of access in north lake area. In order to ensure the stability of embankments which final height varies from 5 to 6.5 m, two soil improvement techniques have been studied: prefabricated vertical drains (PVD) with preloading surcharge and stone column reinforcement.

The first technique (PVD) is well controlled and practiced in Tunisia by the local entrepreneurs. In fact, PVD is a very simple technique and it preserves the environment. Contrarily, the second technique (stone column reinforcement) should be carried out by foreign entrepreneurs having long experience on the matter. This technique also requires advanced equipment for installation and to acquire stone material.

The geotechnical behaviour of an embankment on compressible soils incorporating vertical drains or stone columns is analysed during and after the construction period. The first part of this paper is dedicated to analysis of geotechnical data which includes the classification and interpretation of laboratory and in situ tests results.

The second part focuses on the consolidation of highly compressible layers as foundation of high embankments. Prefabricated vertical drains associated with a preloading surcharge are then undertaken.

In the third part, a stone columns reinforcement of compressible layers is suggested and followed by appropriate design by using the software programme "Columns".

Finally, a technical-economical comparison between the two soil improvement alternatives is presented, which makes it possible to decide the adequate solution to build the embankments of access.

In this paper, the French abbreviation NGT means “General levelling of Tunisia”.

Fig.1. Location of the project “Radès-La Goulette” bridge.

GEOTECHNICAL INVESTIGATION OF THE SITE

“Radès-La Goulette “bridge is a big Tunisian project which comprises several parts. A part of them is the construction of embankments with variable height up to +8 m NGT located behind of the abutments of bridges of access; in the north lake’s zone currently presenting a draught of about 1m (the depth of Bed Lake is located between -1 to -0.6 m NGT).

The soil profile, as foundation of embankments, presents successive clayey and sandy deposited under consolidated layers. However, the continuity in horizontal plan of these alternations, along 22 to 25 m depth, is not necessarily proved. Distances between bored holes ranges between 100 m to 200 m.

Geotechnical Model

The geotechnical model is set up based on results obtained from: bored holes, and in situ tests (pressuremeter and SPT data) including the static cone penetration records and pore water pressure (piezocône). The first geotechnical investigation (boreholes, drilled core samples, pressuremeter tests, vane tests, SPT) was mostly conducted along various depths (40 m to 110 m) (NIPPON KOE et al, 2001).

Only the results obtained from borehole (**ard1**), located in the area of North Lake are exploited (fig. 2).

The first geotechnical synthesis displayed a very soft compressible layer I of thickness varying from 8 to 10 m.

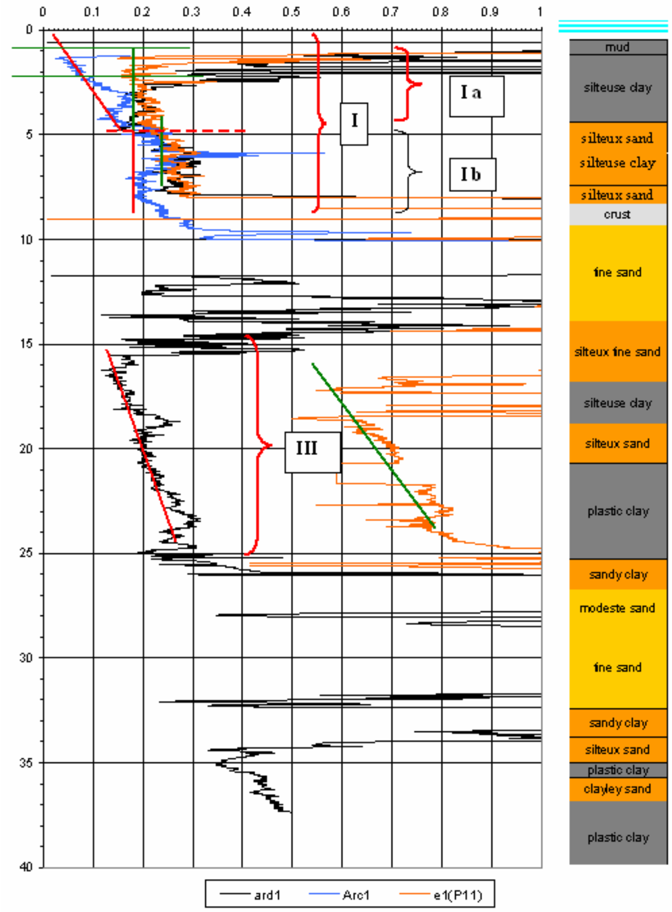

Fig.2. Results of CPT (q_c in MPa) vs depth in meter.

This soft layer can be divided into two sublayers: the first is a highly compressible mud of about 5 to 6.5 m thickness, while the second sublayer consists in compressible sandy clays.
The recorded data static cone test from penetration confirmed the existence sublayer Ib.

It also noticed a moderate difference between the cone penetration resistance and undrained cohesion in the two sublayers Ia and Ib.

For studying the stability and preloading process, a focus on the behaviour of levels I and III has been addressed (fig. 2).
Table 1 groups the significant data related to mechanical characteristics of the soil as foundation of the studied embankments.

Table 1. Mechanical characteristics from in situ tests.

Location	Layer	Tip resistance q_c (MPa)	Undrained cohesion C_u (kPa)
Reference of boreholes	I a	0.025+0.032 z	1.5+2.1 z
	I b	0.2	13
	III, VI	Refusal at 10 m	
Reference of in situ tests	I a	0.19	13
	I b	0.25	17
	III, VI	11.4 z	0.76 z

Results

The sand layer located at 12 m depth has unsignificant plasticity, which indicates its negligible compressibility.

The characteristics of compressibility, measured from oedometric test, for layer I (until 10 m depth) are presented in Table 2.

Table 2. Adopted characteristics of compressibility for soft soil.

Layer	Elevation (m)	γ (kN/m^3)	$C_c/1+e_0$	C_v (E-08 m²/s)	tg ϕ_{cu}
I a	-0.9 – 6.5	16.5	0.15	5	0.158
I b	-6.5 – 9.2	19	0.1	8.8	0.158
II	-9.2 – 18.8	18.5	0.09	-	-
III	-18.8 –26.8	18	0.14	2 to 4	-
IV	-26.8 - 35	19	0.05	-	-
V	-35 - 71	18.8	0.18	5	-

The horizontal coefficient of compressibility C_h is estimated from the vertical coefficient C_v as: $C_h = 5C_v$.

Due to the significant lack of bearing capacity and the high compressibility of soil layers along 10 to 15 m depth, the construction of embankments is definitely compromised. Besides, significant settlements are also predicted in compressible deep layers (levels III & IV). For these reasons making recourse to an improvement solution of soils layers under the embankments, at least along the first 10 m depth, reveals unavoidable.

Such a solution aims, first, the acceleration of consolidation of high compressible layers. In case a reinforcement technique might be envisaged a significant reduction of settlement associated with substantial increase of bearing capacity will be possible. Then, the two alternatives soil improvement techniques are:

- The use of vertical geodrains associated with preloading embankment.
- The soil reinforcement by stone columns (or by sand piles).

Each of the two alternatives has specific advantages. Indeed, by the technique of geodrains, which is characterized by a rapid installation, the consolidation of soft ground is well accelerated. Meanwhile, a staged construction for embankments is necessary. Whereas the stone columns reinforcement alternative has the advantages of significant reduction of long-term settlement and the construction of embankment of access will enhance significant increase of bearing capacity due to mechanical performances of columns material.

STUDY OF THE EMBANKMENTS OF ACCESS

The main difficulties which arise for the construction of embankments of access are:

- Short-term stability of the soft ground as related to bearing capacity verification.
- Long-term settlement of unimproved deep layers (depth greater than 30 m).

Description

The zone of the interchange which comprises the new express route and four embankments of access, cover approximately 16 hectares to be reclaimed in the north lake of Tunis. The final heights (after end of primary consolidation) of these embankments vary from 5 m to 6.5 m.

Based on predicted settlements, under centre line of each embankment of access, by the odeometric and pressurmeter methods, the height of preloading embankments was deduced. Because of too low short-term mechanical characteristics of the foundation of embankments, a staged construction is scheduled. Such a procedure will make possible the increase of short-term shear strength of soft layers as consequence of part of the primary consolidation.

As potential soil improvement techniques achievable in the context of "Radès La Goulette" bridge project, the design will be proceeded, first, for the prefabricated vertical drains (PVD) associated to preloading embankments and, second, for the stone columns reinforcement.

Stability of embankments

The slopes of embankments of access are projected as 3 m for horizontal and 1 m for elevation. The platform is located at + 1 m above the NGT level.

The fill material used has an angle of internal friction of 30°, consequently tg 30° = 0.57 > 1/3. Then, a safe stability of slope embankments is guaranteed. The in situ unit weight embankment's compacted material is about of 19 kN/m^3.

Staged construction of embankments

The stages of construction of embankments have been scheduled as follows:

- Reclamation of the total area by a generalized fill at +1 m NGT.
- Arise the thickness of embankments of access, at + 3 m to + 5 m NGT: in zone of connection with the express route.

- The thickness of embankments of access behind the abutments, along 20 m length, is arisen at +8.0 m NGT.
-

The allowable bearing capacity complying with the initial height of the embankment to build is: H_r = 2m. In turn, the construction of an embankment with height exceeding 2m requires a soil improvement solution.

The consolidation of the sandy mud layer by the technique of preloading revealed insufficient. Indeed, for the 2 m initial height of preloading, the time of primary consolidation of the mud layer is about of 58 years, which is quite inadequate with the duration of embankments construction.

A first adequate solution consists in associating with the initial preloading a network of prefabricated vertical drains in order to accelerate the consolidation of the mud layer. However, using the stone columns reinforcement technique, an increase of the bearing capacity, and significant settlement reduction of reinforced soil will be provided, adding to the acceleration of consolidation enhanced by the drained nature of columns material.

IMPROVEMENT BY PVD WITH PRELOADING EMBANKMENT

The prefabricated vertical drain (PVD) with preloading method was considered as the most feasible treatment for the project based on the depth of treatment, cost, allocated time for preloading and other considerations (fig. 3). The objective of using vertical geodrains with preloading technique is to accelerate the rate of consolidation and to minimize the remaining settlement of the treated area under the final (dead and live) loadings. Preloading increases bearing capacity and reduces the compressibility of weak ground by forcing soft soils to consolidate (Van Impe, 1989). Soil improvement works is carried out in such a way that a specified degree of primary consolidation is designed to be attained during the desired time by improving the soil drainage system.

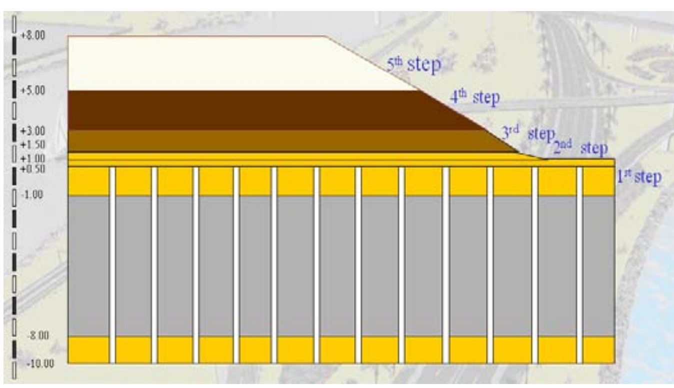

Fig.3. Construction sequences of preloading embankment on improved soil by PVD.

Preloading

Preloading refers to the process of edification of a temporary embankment prior to the placement of final permanent construction. If the temporary applied load exceeds the final loading, the amount in excess is referred to be as surcharge load. Since the preloading is rapidly applied, the resulting settlement of soft mud deposit is divided into immediate and primary consolidation components. This latter generally predominates because of the negligible immediate settlement compared to that of primary consolidation.

The preloading steps are designed based on the gain of undrained cohesion which results from the accelerated consolidation of high compressible layers. The increase of undrained cohesion ΔCu, due to a prefixed degree of consolidation $U(\%)$, will serve to design the next preloading step; then:

$$\frac{\Delta Cu}{\gamma \; H \; tg\Phi_{CU}} = U(\%) \tag{1}$$

$tg\Phi_{CU}$ = the rate of increase of undrained cohesion under the effect of the consolidation. After available data related to the soft ground of Tunis, we have $tg\Phi_{CU} = 0.158$ (Bouassida, 2006).

Table 3 presents the increase of undrained cohesion occasioned in the sandy mud as a result of the primary consolidation which occurs at the end of each preloading step.

Table 3. Increase of undrained cohesion of sandy mud layer resulting from staged preloading.

Elevation (m) NGT	Total height of embankment H(m)	$\Delta C_U\,(kPa)$	$U_r\,(\%)$
-1	0	0	0
0.5	1.5	0	0
1	2	0	0
3	4	0	0
5	4.5	5.5	53
5	5	6.5	54
5	5.5	6	45
5	6	6	40
8	9	11	67

The primary consolidation settlement in centre line of embankment of sandy mud layer assumed as normally consolidated is predicted after one dimension Terzaghi's theory:

$$s_\infty = \left(\frac{C_c H_0}{1+e_0}\right)\log\left(\frac{\sigma_0^{'} + \Delta\sigma_v}{\sigma_0^{'}}\right) \quad (2)$$

$\Delta\sigma_v$ = excess of vertical stress.

C_c = index of compression.

$\sigma_0^{'}$ = effective overburden stress at night of compressible layer

Consider data: H_0 = 6.5 m; $\Delta\sigma_v$ = 2×19 = 38 kN/m²; then $s_\infty = 1.0\,m$

The degree of consolidation of foundation layers beneath the embankments of access is approximated by:

$$U(t) = \frac{s_t}{s_\infty} \quad (3)$$

s_∞ and s_t denote respectively the settlements at the end of primary consolidation, and at given time, which corresponds to the degree of primary consolidation $U(t)$.

Characteristics of PVD

A prefabricated vertical drain (PVD) can be defined as any prefabricated material or product consisting of synthetic filter jacket surrounding a plastic core having the following characteristics (Bergado et al, 1996):

- Ability to permit porewater in the soil to seep into the geodrain.
- A tool by which the collected porewater can be transmitted along the length of the geodrain, without any particles migration from the soil to improve during drainage.

The studied case history considers an acceleration of the consolidation by the installation of a grid of PVD descended from a platform levelled +0.5 m NGT, until 10 m depth. The proposed type of PVD is Mebradrain 88 (MD 88) which is of flat type of thickness 0.5 cm and 10 cm width. MD 88 was also experienced in previous soil improvement projects with PVD in Tunisia (reclamation in South Lake of Tunis).

A 0.5 m thickness drainage blanket made up of gravel-sand material will cover the improved soft layer to speed the PVD drained water and will serve as platform for settlement recorders, piezometers.

The geometrical and hydraulical characteristics of PVD are:

- A diameter of the drain:

$$d = \frac{\text{perimeter}}{\pi} = \frac{2\times(10+0.5)}{\pi} = 6.7cm \quad (4)$$

- A capacity of discharge:
$q_w = 5.10^{-5} m^3 s^{-1}$.
- A mass: 96 g/linear meter.

Choice of the drains pattern and preloading schedule

The waiting time between successive stages of preloading has been determined for a given degree of horizontal consolidation calculated by Barron's formula.

$$t = \left[\frac{D^4}{8(D^2-d^2)}\ln\left(\frac{D}{d}\right) - \frac{3D^2-d^2}{32}\right]\frac{\ln\left(\frac{1}{1-U_h}\right)}{C_r} \quad (5)$$

t: time in seconds;
U_h: degree of horizontal consolidation in %;
D = 1.13 L; D: equivalent diameter, L: spacing core to core between drains installed in square pattern.

The waiting time between preloading stages varies from 35 to 70 days for a squared pattern drains spacing of 1.8 m. This corresponds to the agenda planned of the site reclamation, without making recourse to an accelerated consolidation with a tighter platform. The total duration to attain the level + 8 m NGT is 245 days.

However, the fact of adopting a tight grid of 1.2 m spacing, under the most loaded zones, with a transition zone with a grid of 1.5 m spacing, makes it possible to anticipate settlements behind the abutments of bridges access. The total waiting time is 89 days, which corresponds to 63% of the time expected for a pattern where 1.8 m spacing is adopted.

Meanwhile, in the two cases, the elevation of embankment +3m NGT level does not require significant waiting time (less than 15 days).

Tables 4 and 5, and curves illustrated in figure 4 give the predicted aimed waiting time to acquire the improvement of the north Tunis lake area to be reclaimed.

Table 4. Drains installation with spacing 1.8 m.

Elevation (m) /NGT	H_r (m)	Waiting time (days)	Cumulated time (days)
From -1 to +0.5	1.5	0	0
From +0.5 to +1	2	0	0
From +1 to +3	4	0	0
From +3 to +3.5	4.5	50	50
From +3.5 to +4	5	50	100
From +4 to +4.5	5.5	40	140
From +4.5 to +5	6	35	175
From +5 to +8	9	70	245

Table 5. Drains installation of 1.2 m spacing.

Elevation (m) /NGT	H_r (m)	Waiting time (days)	Cumulated time (days)
From -1 to +0.5	1.5	0	0
From +0.5 to +1	2	0	0
From +1 to +3	4	0	0
From +3 to +3.5	4.5	18	18
From +3.5 to +4	5	19	36
From +4 to +4.5	5.5	14	50
From +4.5 to +5	6	12	63
From +5 to +8	9	26	89

Evolution of settlements versus time of preloading

In order to determine the evolution of settlements during the whole staged construction of embankments of access, the variation in time of the degree of consolidation is studied for each soil level (layer).

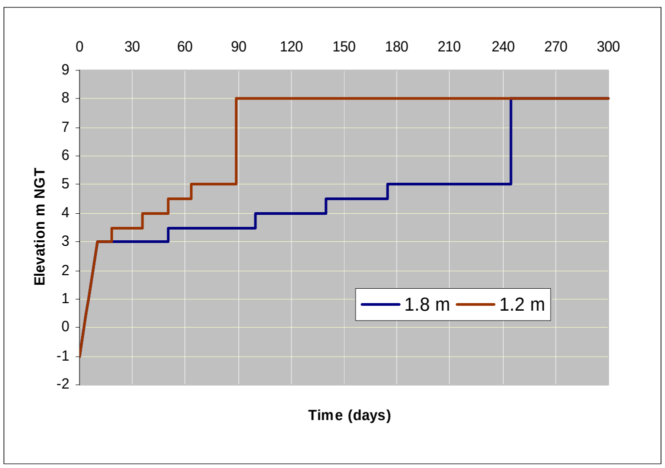

Fig. 4. Stages of embankment's construction vs time.

Figure 5 presents the evolution of settlements of primary consolidation of layers Ia and Ib by taking into account the effect of accelerated consolidation which results from a pattern of squared PVD when the spacing takes 1.2 m and 1.8 m. In fact, figure 5 shows up effectiveness of reduced spacing in the gain of time of consolidation to reach the same magnitude of settlement.

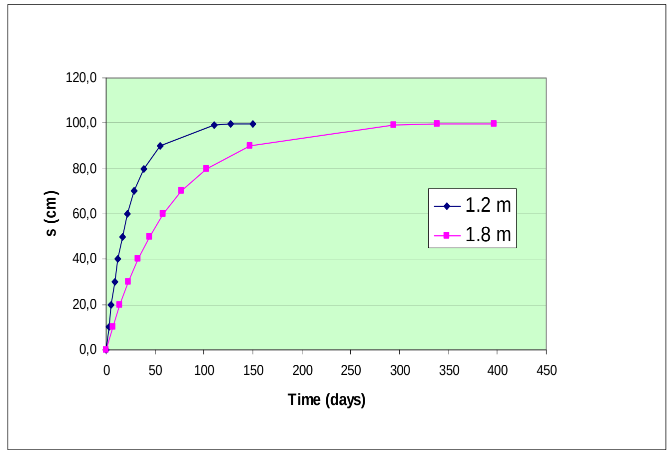

Fig. 5. Evolution of settlements and influence of spacing vs to time of preloading.

It is also worth mentioned to predict the gain of the undrained cohesion of soft layer after improvement by PVD installation. Such a result will serve for studying the stability of the foundation's layer of embankment.

REINFORCEMENT BY STONE COLUMNS

The stone column technique was adopted especially in European countries early in the sixties and became little by little successfully practiced. A stone column is basically a vertical cylindrical "hole" executed in a soft soil layer and filled with compacted stone fragments and gravel having high potential drainage.

This technique can be used to improve soft layers under dams and embankments in order to increase the bearing capacity, to reduce settlements, and to accelerate the consolidation process like vertical drains.

Stone columns are basically installed either by the use of vibro replacement or by use the vibro displacement process. Figure 6 depicts the different stages of a process stone column installation by, the vibro displacement. More detailed descriptions of the equipment and the procedure itself can be found in Moseley & Priebe (1993), Kirsch & Sondermann (2003), Debats (2006).

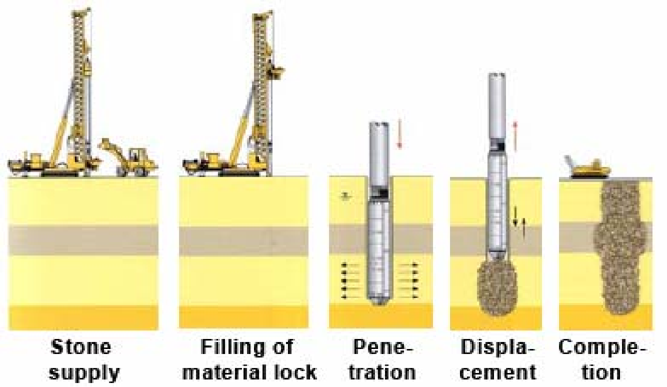

Fig.6. Dry bottom feed vibro displacement method.

Usually the columns are placed in a regular pattern, squared or triangular, improving the weak layers below the embankment.

As basic parameter for the design of column reinforcement is the improvement area ratio defined by:

$$\eta = \frac{A_c}{A} \qquad (6)$$

A : Area of foundation.
A_c : Total cross section of columns located under the loading foundation.

Properties of column material

Stone columns are usually installed using deep vibratory compaction equipment (vibro probe). Columns material is generally acquired from a quarry, i.e. selected crushed gravel having a prescribed grain size. Drained characteristics of stone columns material (Costet and Sanglérat, 1983) are grouped in Table 6.

Table 6. Characteristics of stone columns material.

γ [kN/m^3]	C' [kPa]	E' [MPa]	φ' [°]	ν'
20	0	10	40	0.25

For the present case history, stone columns are designed with final diameter of 1m to be installed pre-bored holes along 10 m depth using a vibro displacement method (fig. 6).

Bearing capacity

The bearing capacity of a supported foundation is the vertical stress which causes the yield of underlying soil of foundation.

For embankments of access, the bearing capacity verification has been designed by using the too recent elaborated software "Columns" (Bouassida et al, 2007) as detailed below.

1. The minimum improved area ratio η_{min} is predicted based on the limit analysis approach (Bouassida, 2007). The angle of internal friction of the soil is φ = 16°, then the minimum improvement area ratio is η_{min} = 16.7 %.
2. The prediction of ultimate bearing capacity (q_{ult}) refers to the case of purely cohesive soils reinforced by cohesive frictional columns material. Two methods of design, are involved, namely, the yield design approach (lower bound) which takes account of improvement area ratio and the recommendations of French Standard "NFP 11-212, (2004)" which do not take account, of improvement area ratio. Then the allowable bearing capacity is deduced based on a given global safety factor which depends on the method of prediction (Table 7).

Table 7. Comparing between predictions of allowable bearing capacities.

Methods	Global safety factor	q_{all} [kPa]
Yield design (lower bound)	1	114
NFP 11 -212	2	177

3. The verification of the allowable bearing capacity with respect to the embankment load led to a minimum improvement area ratio: $\eta \geq 16.7$ %. The evolution of lower bound ultimate bearing capacity as a function of improvement area ratio is illustrated in Figure. 7.

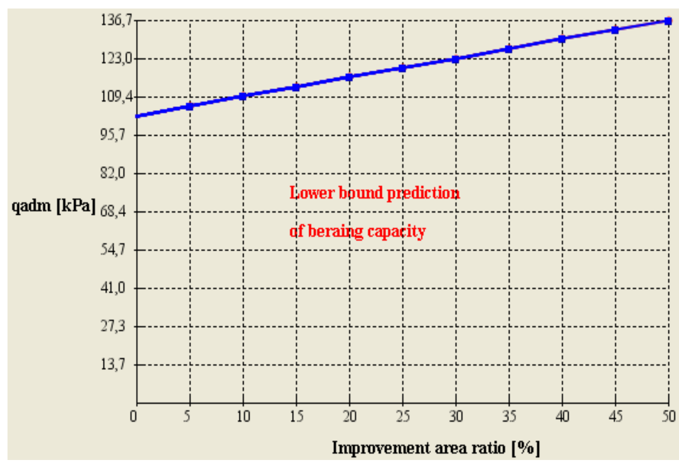

Fig.7. Evolution of the ultimate bearing capacity versus improvement area ratio (output of software "Columns").

Settlement predictions

Presently, available methods for settlement prediction can be classified either as simple methods which use the one dimension linear elastic model assumptions or as sophisticated methods using numerical codes which consider linear elastic and/or elasto-plastic behaviour 2D or 3D model.
For this project, the prediction of settlement is carried out by using the software "columns" (Bouassida et al, 2007), in which the linear elastic behaviour is adopted by several methods of design for constituents of reinforced soil.

Settlement before reinforcement is about 2.1 m in the center line of embankment having 6 m height. Meanwhile, the admissible settlement should not exceed 30 cm.

The settlement, in center line of embankment, generated by the load of final height of embankment (H_r = 6 m), is estimated, assuming the linear elastic behaviour, by the variational approach and French recommendations NFP 11-212 (Table 8).

For each method of design, the settlement complying with admissible bearing capacity is estimated, first, by considering the minimum improvement area ratio and, second, by the optimized improvement area ratio which complies with allowable settlement.

Table. 8. Comparison between predicted settlements by two methods.

Methods	Variational approach	NFP 11-212
η = 16.7 %.	45.5 cm	39.5 cm
η = 31.5 %.	30 cm	28 cm

According to the height of embankment, or conversely the applied load, it is possible with "columns" software to predict the variation of settlement by several methods all assuming columns of end-bearing type (fig .8). The most conservative prediction is given by Chow's method which uses the unit cell model and assumes zero horizontal displacement in each point of soil reinforced. While the variational method uses the group of columns model and takes account of lateral confinement in 3D reinforced soil (Bouassida et al, 2003).

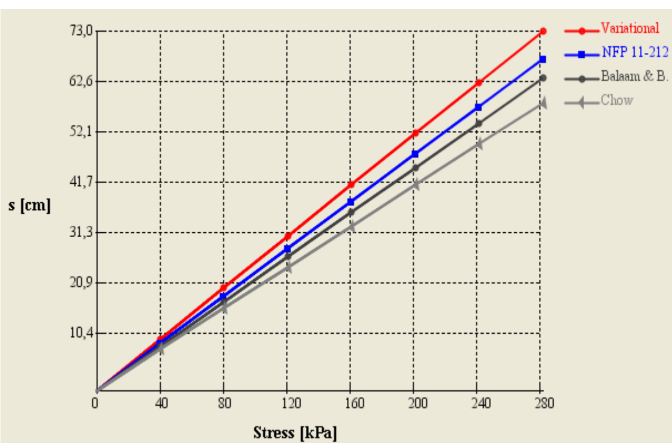

Fig.8. Settlement of reinforced soil versus applied load.

The apparent normalized Young modulus of the reinforced ground is represented in Figure 9 with other modulus estimated by linear elastic method: variational method (Bouassida et al, 2003), (Balaam & Booker, 1981), (Chow, 1996) and (NFP 11-212, 2004), as a function of the improvement area ratio.

E_a and E_s denote respectively the apparent modulus of reinforced soil and Young modulus of initial soil.

It is observed a quasi linear relationship for all methods of prediction.

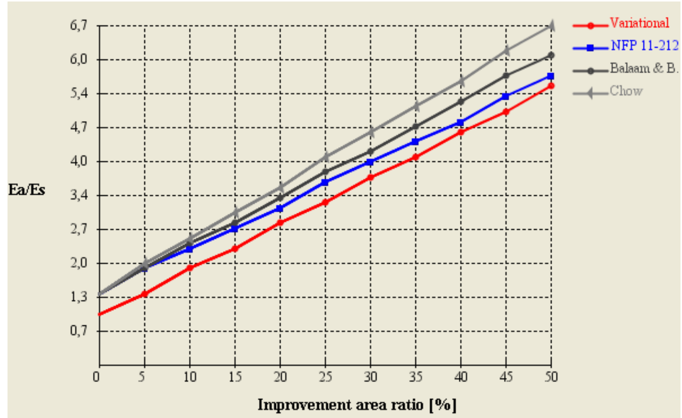

Fig. 9. Variation of Normalized Young modulus of reinforced soil versus improvement area ratio (η).

Design of stone columns network

The stone columns network has been designed with specific parameters grouped in table 9.

Table. 9. Designed stone columns network.

Length (m)	Substitution factor (%)	Columns diameter (m)	Spacing (m)	Pattern
10	31.5	1.0	1.7	Triangular

Consolidation

Stone columns also behave as vertical drains and because of the drained property of their constitutive material which accelerates the process of consolidation.
In order to predict the evolution of settlement versus time, performing the poroelastic approach (Guetif and Bouassida, 2005) programmed in "columns" software, the evolution of settlement of columnar reinforced soil is predicted as a function of the history of loading.
Horizontal permeability is the needed parameter for carrying the poroelastic approach:

$$k_h = \frac{C_h \cdot \gamma_w}{E_{oed}} \quad (7)$$

Odeometric modulus is:

$$: E^{oed} = E_s \frac{(1-\nu_s)}{(1-2\nu_s)(1+\nu_s)} \qquad (8)$$

ν_s : Poisson's ratio.

γ_w : Unit weight of water.

NA: $k_h = 1.67\ 10^{-9}\ m/s$.

Prediction by the poroelastic approach illustrated in Figure 10 shows the evolution of settlement versus time, and indicates that the final consolidation settlement is expected in 97 days.

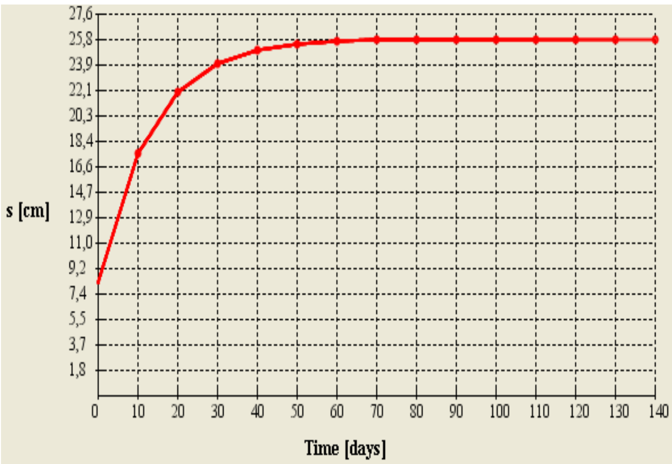

Fig. 10. Settlement evolution of reinforced ground versus time.

The evolution of consolidation settlement is greatly influenced by the value of substitution factor as shown in Figure 11. Meanwhile for a wide margin of the substitution factor, currently practiced for stone columns technique, the end of primary consolidation of reinforced soil in average takes 150 days. Note that for low values of improvement area ratio (less than 10%) predicted settlement by the poroelastic approach is not realistic.

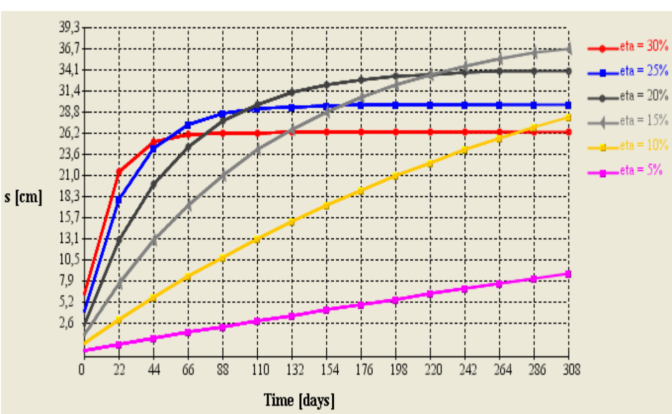

Fig. 11. Variations of the settlement of reinforced soil vs time, for various η.

COMPARISON BETWEEN SOIL IMPROVEMENT TECHNIQUES

In this study, for materials to be acquired and installation techniques, the current costs in Tunisia are applied.

<u>Soil improvement by PVD associated with embankment of preloading</u>

The construction of embankments of access is carried out by using a selected light weight material (expanded clay of unit weight = 7 kN/m^3) along 10 m length behind the abutments, and the volume of filled material is of about 2500 m^3. Table 10 summaries the improvement technique by PVD associated with preloading embankment. Note, that the time of execution is about sixteen months.

Table 10. Cost of execution of soil improvement by PVD.

Volume of embankment of preloading	48310 m^3
Volume of material to acquire	17843 m^3
Volume of weight light material	2467 m^3
Linear meter of PVD	57810 lm.
Cost (TND)	1,472,090

<u>Stone column reinforcement technique</u>

The predicted settlement at end of construction of embankments on columnar reinforced soil is about of 40 cm. Then, consumption added column material is required for the definitive embankments and consequent cost follows. Table 11 indicates the cost of execution of the reinforcement technique by stone columns.

Table 11. Cost of soil reinforcement by stone columns.

Volume of embankment of access	17493 m^3
Linear meter of columns	3690 m
Cost (TND)	1,226,955

<u>Economical comparison between the two alternatives</u>

The estimated cost for the alternative "Soil improvement by PVD with preloading embankment" is about 1,472,090 TND which includes the cost of preloading and unloading embankments, and the cost of geodrains. The cost of installation of linear meter of MD 88 geodrains is 2.5 TND. The estimate of the second alternative "Reinforcement by stone columns" technique is about 1,226,955 TND. The cost of installation of a stone column linear meter is by 80 TND.

From Tables 10 & 11, it is clear that the column reinforcement technique provides a reduction of about 16.6% on the cost of the foundation under embankments of access. The time of execution is of about eight months which provides a substantial gain of eight months compared to PVD installation. Because PVD improvement includes preloading and unloading steps during embankments it takes a longer time of construction than that estimated for stone columns reinforcement.

Multi criteria analysis

Table12 recapitulates the specifications of each improvement technique envisaged during the execution of project "Radès La Goulette Bridge" for the foundations of the exchanger in north Lake of Tunis.

Table 12. Multi criteria analysis of studied improvement techniques.

Techniques	Improvement with PVD	Stone column reinforcement
Qualification of local entrepreneurs	Very good	less
Duration of execution	Long (16 months)	Short (8 months)
Environnemental impact	unsignificant	unsignificant
Cost	normal	Less important
Comments	Well controlled	Little use

The multi criteria analysis highlights the economical interest (cost and time of execution) shown by the stone column technique which appears more advantageous than PVD with preloading embankment.

Despite the advantages in favor of stone columns reinforcement technique, the owner of “Radès La Goulette” bridge project decided the execution of PVD as improvement solution. Such a choice is justified based on a much better qualification of Tunisian entrepreneurs for PVD installation and, in parallel, few practice of stone columns installation.

GEOTECHNICAL SURVEY

The geotechnical survey in reclaimed north lake area has been instrumented by installed piezometers and settlements plate readings located along the cross section of embankments of access.

Figures 12, 13, 14, 15, 16 and 17 illustrate successive operations of PVD’s installation and location of in situ record instruments.

Fig. 12. Preparing PVD installation.

Fig. 13. Starting PVD installation.

Fig. 14. PVD fixed to mandrel.

Fig. 15. Installed settlement plate.

Fig. 16. Protected settlement recorder.

Fig. 17. Locations of settlement recorders in PVD improved soil.

Recorded in situ measurements are still continuing (until end of 2007). The first results inform consolidation settlement is not completely stabilized under embankments of access which end of construction was in March 2007. For this comparison between predicted evolution and observed settlements did not yet start.

CONCLUSION

Because of the mediocre characteristics of the Tunis subsoil along the first twenty meters, a soil improvement solution has been decided to make possible the construction of embankments of access in the north lake area of Tunis as part of the big project "Radès-La Goulette" bridge.

Two solutions of soil improvement have been studied; the first one consists in vertical "Geodrains" drilled until 10 m depth associated with step by step construction of preloading embankment, the second solution is stone column reinforcement up to 10 m depth.

- Improvement by PVD:

It revealed, when associated with preloading embankments, as convenient solution to reach a high degree of primary consolidation. Consequently, major part of settlement will be released during the period for construction of embankments of access. Effectiveness of PVD soil improvement largely depends of adopted spacing between drains.

- Stone columns Reinforcement:

The gain in time of execution and subsequent economical cost are in favour of this reinforcement technique which guarants significant increase of bearing capacity, decrease in consolidation settlement, adding to accelerated consolidation

A multicreteria analysis comparing between the two improvements techniques highlighted the stone columns as more advantageous essentially the economical viewpoint. In turn, the PVD technique, being more experienced in similar previous project of reclamation by Tunisian entrepreneurs, was finally decided for execution.

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