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Challenges and Improvement Solutions for Tunis' Soft Clay

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ABSTRACT: The design of foundations resting on Tunis soft clay still remains a very challenging issue to address. However some works have been dedicated to the characterization and the study of behavior of this problematic soil on which recent investigations more focused on improvement solutions. This paper first summarizes the geotechnical properties of Tunis soft clay on the basis of data collected from experienced projects and results obtained from laboratory tests. It has been discussed how Tunis soft clay properties can be affected by disturbance? Some correlations are also suggested from recorded data. Besides, the study of behavior of built embankments on Tunis soft clay is analyzed from numerical computations carried out by 2D version of Plaxis software. It is then concluded, that the improvement of Tunis soft clay by classical techniques like geodrains, sand piles and stone columns is necessary to guarantee the short term and long term stability of structures to be built on such weak soil. Illustrated case histories show the efficiency of these improvement techniques as competitive solutions compared to piled foundation that remains the most used especially for building projects in Tunisia but not cost effective.

Keywords: Behavior, Experiments, Improvement, Prediction, Soft clay

1. INTRODUCTION

Soft clays represent a well known category of problematic soils which are generally encountered under the form of deposited layers in coastal areas. Several problems are faced when dealing with the study of soft clays from field investigation, soil characterization, behavior modeling, and stability of geotechnical structures up to ground improvement solutions.

Studying the behavior of soft clays especially requires a thorough determination of their geotechnical parameters. Hence, it is needed to carry out laboratory tests on extracted specimens either from cored samples or remolded samples in laboratory. Recent studies on Tunis soft clays have been conducted in order to address essentially their geotechnical characterization and mechanical behavior.

Tunis soft clay is a problematic soil because of its weak strength characteristics and high compressibility. Then, designing foundation on Tunis soft clay requires a thorough study especially for the short term behavior and for the long term behavior as well.

At the National Engineering School of Tunis, early in the eighties, the first investigation [11] had been dedicated to the analysis of dynamic behavior of undisturbed cored samples of Tunis soft clay subjected to torsional shear. Later, in the nineties, [6] has investigated the geotechnical properties and behavior of scaled models made up of remolded Tunis soft clay reinforced by sand columns. In the last decade special care was accorded to the study of the behavior of Tunis soft clay by performing experimental works and conducting numerical computations of various soil mechanics applications, [23].

This paper, first, introduces in brief the geological aspects of Tunis City. Then, the identification and the classification of Tunis soft clay are briefly described. Second, recorded results from oedometer and triaxial tests conducted on Tunis soft clay samples are presented. Third, numerical results of a high embankment resting on Tunis soft clay are analyzed, and finally the feasibility and the design of ground improvement techniques (vertical drains, stone columns) is illustrated through selected Tunisian case histories.

2. GEOLOGICAL ASPECTS OF TUNIS CITY

It has been described, [21], in details the history of geological geotechnical context of the ancient Tunis City that was built on a depression surrounded by hills. The old town was established in the southern side, and the new residential suburbs are located in the north. The plateau is identified as an important deposit of the recent Quaternary age that includes sandy levels and is overlaid by anthropogenic formations. The recent Quaternary is resting on clayey to sandy-clay layer of ancient Quaternary. This latter represents the rigid stratum of Tunis City. The alluvial sedimentation in the Tunis estuary can be associated with simultaneous intensive subsidence, [16]. The lagoon depressions, nearby lake-Essijoumi and the Lake of Tunis, are generally filled with mud of the recent Quaternary and of clayey sand or silt sand of the riverbeds. Divergent displacement of Medjerda and Méliane riverbeds during the recent Quaternary are the consequence of a synchronous upward block displacement of the Tunis region. Thereafter, the soft deposits were filled up to create the zones of urban extensions with anthropogenic formations of thickness varying between 1 and 8 m.

The analysis of geotechnical and electronic atlas maps and the geological profiles deduced from the geotechnical and geological information system (GGIS), [9], has conducted to the subdivision of Tunis City in four homogeneous zones: I, II, III and IV. Three main formations constitute the soil of Tunis plateau (zone I): a heterogeneous fill layer of variable thickness 1 to 8m, the muddy complex and the sandy-clay complex that is assumed as a rigid stratum.

The muddy complex deposit is linked to the evolution of the Tunis Lake during the recent Quaternary. Analysis of data from the GGIS shows that, the grey sandy mud was deposited, in a continental environment, directly on the rigid stratum which constitutes the deep mud layer. Then, the marine environment favored the deposit of grey and greenish sandy mud, of gray silt mud and of blue-grey mud. Finally, a lagoon environment has caused the deposit of superficial black and grey mud layer, [16]. This contribution only focuses on the geotechnical properties of Tunis soft clay located in zone I of Tunis City soil profile.

2 GEOTECHNICAL PROPERTIES OF TUNIS SOFT CLAY

2.1 Identification and classification tests

A complete grain size distribution (sieve and hydrometer methods) and the Atterberg's limits test have been carried out. Table 1 summarizes the results of identification tests conducted respectively on undisturbed soft clay samples (UCS) and remolded clay samples (RCS). Averaged total unit weight is by 17 kN/m^3 and specific gravity of Tunis Soft clay varies from 2.32 to 2.65. The grain size distribution of tested specimens shows the minimum content of fines (dimension < 0.08 mm) was about 86%, [12]. The recorded Atterberg limits are: liquid limit W_L= 43 to 79 and plasticity index Ip = 50 for undisturbed soft clay specimens. Water content, ω, ranges from 0.5 W_L to 1.8 W_L, the consistency index is mostly lower than 0.5 that indicates Tunis soft clay has a very low to a moderate consistency.

For soft clays, the presence of organic matter confers a higher compressibility compared to that of soft soils. According to recorded results from two geotechnical campaigns carried out for Radès la Goulette bridge project, [15] Tunis soft clay relatively contains a small to a medium proportion of organic matter which varies from 0.8 to 22%. There are two types of organic matters, free and active organic matter, which are mainly composed by humic colloids. Free organic matter plays a role only at the level of soil fabric; by increasing the void ratio and by decreasing the dry density.

The organic and carbonates contents of six muddy samples extracted from Tunis City (Table 2) were determined by using the Ann and the calcimetry methods. The organic content (OC) is estimated by:

% OC = % Carbon organic x 1.724. The average factor 1.724 is basically adopted when the organic matter contains an average of carbon of about 58%, [24].

2.2 Oedometer and shear strength characteristics

Oedometer tests have been carried out by increased constant stress level (up to 800 kPa) on undisturbed samples extracted up to depth of 18.5 m and on remolded specimens obtained after initial consolidation under 50 kPa stress level.

Consolidated undrained (CU) triaxial tests with recorded excess pore pressure and unconsolidated undrained (UU) triaxial tests have been performed on soft clay cored samples. Oedometer data (Table 3) indicate that Tunis soft clay is classified as significant to high compressible soil, compressibility index is C_c > 0.4.

Table 1. Identification parameters of Tunis soft clay

Sample	Water content, ω (%)	Specific gravity	Unit weight (kN/m^3)	W_L (%)	W_p (%)	I_C (%)	I_P (%)
UCS1	40	2.62	17.4	46	27	0.31	19
UCS2	52	2.5	16.1	55	50	0.52	5
UCS3	44.3	2.53	18	51	41.5	0.70	9.5
UCS4	65	2.32	17.6	65	50	0.40	15
UCS5	60	2.39	16.9	79	50	0.65	29
RCS1	32	2.65	16.2	68	25		43
RCS2	45	2.65	16.4	65	27		38
RCS3	86	2.65	13	92	41		51
RCS4	--	2.65	18.7	68	25		43
RCS5	40	2.54	15.4	43	26		17
RCS6	93	2.6	14.7	68	35		33

Table 2. Organic content (OC) of Tunis soft clay

N°	Samples	Carbon (%)	OM (%) (fraction < 63μm)
RCS1	Radès-Kheireddine Channel (27.3 to 28.0 m)	0.47	0.81
RCS2	Radès-Kheireddine Channel (30.10 to 30.8 m)	0.70	1.21
RCS3	Radès-Goulette-Liaison North route (5.20 to 8.70 m)	3.98	6.86
RCS4	La Goulette-Kheireddine Channel (18.6 to 19.3m)	0.66	1.14
RCS5	South Lake area (Depth = 4 m)	1.72	2.97
RCS6	La Goulette harbor	1.57	2.71

Besides, from Table 3, the over-consolidation ratio (OCR) defined by: OCR= σ_p'/σ_0', indicates that Tunis soft clay layers which extend at depth of about 25 m are, the more likely, under-consolidated. Specimens numbered 1 to 4; (respectively 5 to 8) were extracted from Mohamed V Avenue in Tunis City (respectively Express Route, Tunis - La Goulette).

However, based on drained (or effective) cohesion deduced from consolidated undrained (CU) triaxial tests in Table 3, it can be concluded that some specimens might be classified as over-consolidated. Further an average value of coefficient of consolidation c_v= 2.5 10^{-8} m^2/s was obtained.

Table 3. Shear strength and oedometer parameters of undisturbed Tunis soft clay

Specimen	Extraction depth (m)	OCR	$C_c/(1+e_0)$	c_v ($10^{-8}m^2/s$)	Drained shear strength		CU shear strength	
					C' (kPa)	φ' (°)	C_{cu}(kPa)	φ_{cu} (°)
1	10.5 – 11.4	0.72	0.19	1.7	16	21	20	9
2	15 – 16	0.29	0.16	3	8	20	10	13
3	20.2 – 21	0.73	0.11	1.21	26	28	36	17
4	25 – 26	0.55	0.18	1.78	8	27	10	14
5	4.4 – 4.9	0.93	0.22	1.9	1	19	8	22
6	7.3 – 7.8	1.09	0.2	1.8	25	11	25	6
7	11.4 – 11.7	1.17	0.25	1.9	12	26	6	6
8	25.4 – 25.9	0.21	0.21	3.2	--	--	--	--

The undrained cohesion c_u represents the short term shear strength to be considered for the analysis of stability of structures built on soft clays. Due to the disturbance that affects extracted Tunis soft clay specimens from bore hole, the determination of c_u from laboratory shear tests is a challenge. Therefore, c_u should preferably be predicted via correlation using data from in situ tests like the pressuremeter and vane shear. When staged construction is involved, because of the inherent primary consolidation the undrained cohesion will be increased, then stability will be analyzed by considering the modified characteristic that should be predicted for each stage of construction. Since the overburden stress plays the role of consolidation stress, the increase of c_u with depth is an interesting parameter to be determined. The increase of c_u in function of consolidation stress is commonly written:

$$tg\,\lambda_{cu} = \frac{\Delta c_u}{\Delta \sigma_c} \quad [1]$$

Δc_u and $\Delta\sigma_c$ respectively denote the increase of undrained cohesion and of consolidation stress.

Using CU direct shear test results in equation [1] the expression of c_u (σ_c) is given by the Coulomb strength envelope equation: $c_u = c_{cu} + \sigma_c \text{ tg } \varphi_{cu}$. while the theoretical prediction of undrained cohesion from CU triaxial tests writes (Bouassida, 2006), [2]:

$$c_u(\sigma_c) = c_{cu}(\cos\varphi_{cu}/1 - \sin\varphi_{cu}) + \sigma_c(\sin\varphi_{cu}/1 - \sin\varphi_{cu}) \quad [2]$$

From the results of CU triaxial shear tests, the undrained cohesion corresponds to the radius of the Mohr's circle at failure. From equations [1] – [2] the theoretical parameter governing the increase in undrained cohesion with depth is:

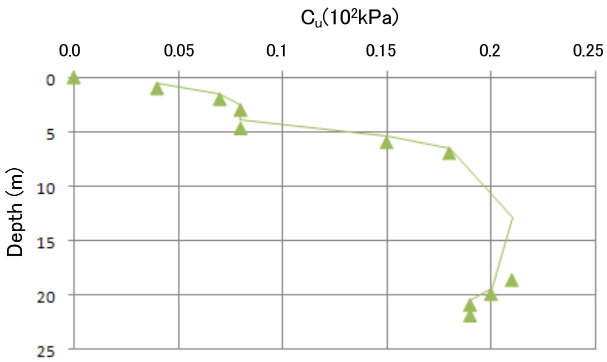
$$tg\,\lambda_{cu} = (\sin\varphi_{cu}/1 - \sin\varphi_{cu}) \quad [3]$$

From Equation [3] the variation of undrained cohesion with depth can be predicted. Assessment of relationship given by equation [2] in terms of consolidated undrained characteristics φ_{cu} and c_{cu} determined from CU triaxial tests has revealed satisfactory, [2]. This result, confirmed by [23], for remolded Tunis soft clay samples, illustrates the benefit in using homogenized remolded specimens having controlled parameters (such as water content, unit weight, etc).

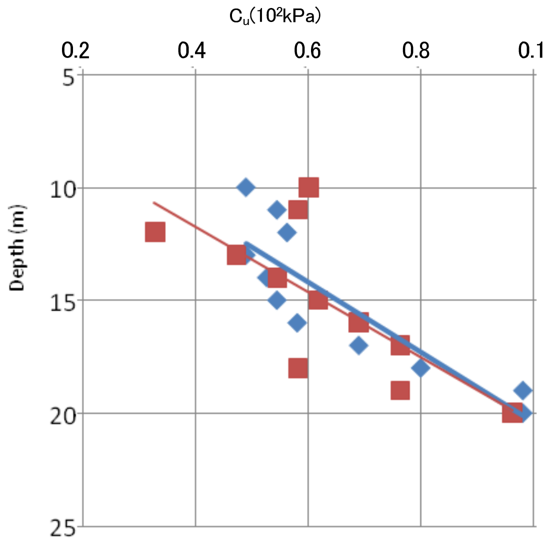
Estimating the increase in c_u with depth is not easy from the records of in-situ vane test which usually show a significant dispersion of results. As an example, using records from pressuremeter tests performed for Rades La Goulette bridge project (Tunisia), the prediction of undrained cohesion with depth, from the correlation given by equation [4], shows that tg λ_{cu} = 0.154 for the top soil layer up to a depth of 6 m, and tg λ_{cu} = 0.07 for layer between 6 m to 20 m depth. However, from Table 3, when the φ_{cu} values recorded at depth less than 20 m are substituted in equation [3] the predicted tg λ_{cu} ranges from 0.12 to 0.2, this result is not in good agreement with tg λ_{cu} = 0.3 that is deduced from results obtained for remolded Tunis soft clay samples, [12]. Hence, it appears the need to adjust between results obtained from tests conducted on remolded samples and other parameters obtained from laboratory or in situ tests. Figures 1a, 1b and 1c show the evolution of c_u in function of depth using recorded data for three sites i.e. La Goulette, Radès and Tunis City.

In Figures 1a and 1b the value of tg λ_{cu} is deduced from the undrained cohesion estimated by the correlation suggested by [1]:

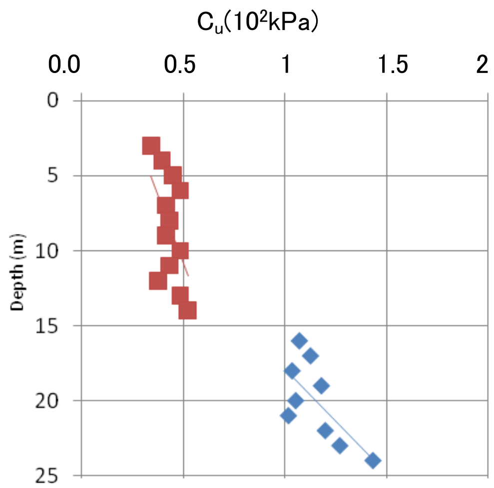
$$c_u = p_l^*/5.5 \quad [4]$$



(a) La Goulette



(b) Rades



(c) Tunis City

Fig. 1: Variation of undrained cohesion versus depth

p_0 is the initial horizontal total stress; and $p_l^* = p_l - p_0$ is the net limit pressure.

The two parameters are recorded at given depth where the pressuremeter test was performed.

It has been recently suggested, [3], the use of analytical elastoplastic determination of undrained that simply writes $c_u = p_l^*/5.4$ and has revealed in good agreement with the correlation given by equation [4].

3 CORRELATIONS BETWEEN PARAMETERS OF SOFT CLAY

Several correlations can be adopted to determine the geotechnical parameters of soft soils from laboratory tests results in order to undertake the design of geotechnical projects and other related structures. Indeed, in soil mechanics, correlations represent a practical tool for the estimation of given soil parameters from other parameters than can be easily determined from current tests. As example since the determination of Atterberg limits is very common and cheap as well, oedometer parameters of clayey soils can be estimated either from liquid limit or plasticity index. Several correlations were suggested and were assessed for a given type of soil (normally consolidated clays, soft soils, etc.). [20], first, have attempted to correlate between geotechnical parameters of Tunis soft clay. Equations [5] – [9] represent some correlation proposed for clayey soils to estimate oedometer or shear strength characteristics from identification parameters (C_c, e_0, ω, I_p or W_L and σ_p').
The procedures mostly referred to in practice now are those developed by, [19]. The set of experimental data given in Tables 1, 2 and 3 for undisturbed Tunis soft clay, is considered for the validation of well known correlations between geotechnical parameters (e_0, ω, W_L, I_p, C_c, c_v, σ_p' and φ') which are:

$$I_p = 102.3(C_c/(1+e_0))+13.34 \qquad [5]$$

$$W_L = 1.39\ \omega + 55.29 \qquad [6]$$

$$W_L = -7.391\ \gamma_d + 156.6 \qquad [7]$$

$$\varphi' = -0.271\ I_p + 22.66 \qquad [8]$$

$$c_v = 0.163\ \sigma'_p - 0.108 \qquad [9]$$

Drawn lines if Figures 2a, 2b, 2c, 2d and 2e correspond respectively to correlations given by equations [5], [6], [7], [8] and [9]. Plotted dots in Figures 2a to 2e represent recorded data from laboratory tests carried out on Tunis soft clay samples. Although it is noted that experimental results are in fair agreement with well known correlations it should be kept in mind such assessment needs more and more data, to be collected from case histories practiced in

Tunis City and its North and South suburbs, in view of a reliable assessment of the suggested correlations.

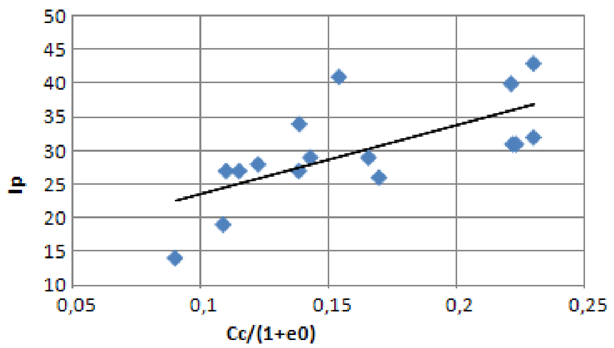


Fig. 2a: Variation of plasticity index vs normalized compression index

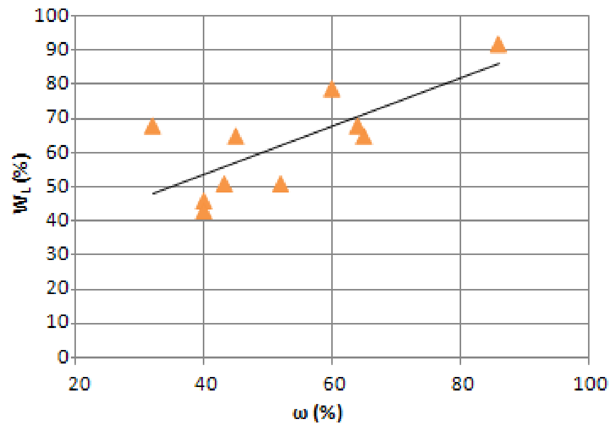


Fig. 2a: Variation of liquid limit vs water content

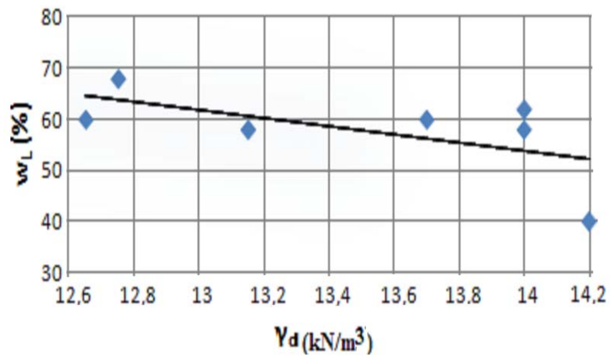


Fig. 2c: Variation of liquid limit vs dry unit weight (kN/m^3)

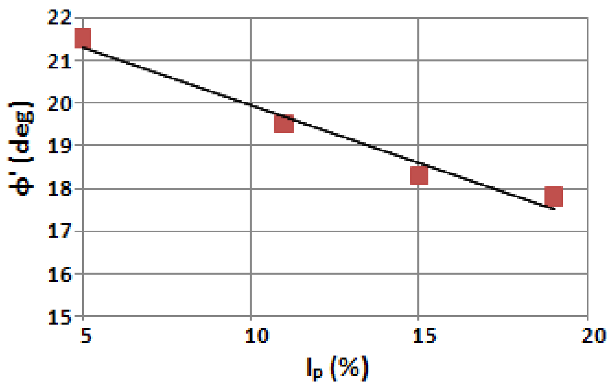


Fig. 2d: Variation of drained friction angle vs plasticity index

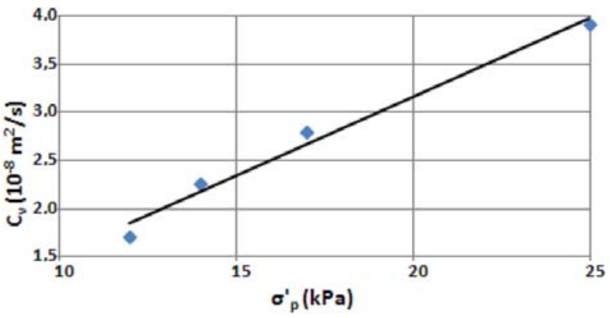


Fig. 2e: Variation of coefficient of consolidation vs preconsolidation pressure

4 STAGED CONSTRUCTION OF EMBANKMENT ON SOFT SOIL

Staged construction of embankments can be adopted in place of any soil improvement technique (e.g. by preloading or under permanent structure). Such a procedure requires that the applied load at each stage of construction should comply with the allowable bearing capacity of soft soil. Under the allowable surcharge (or height) of the embankment a partial primary consolidation is taken place and, then, it results the increase in undrained cohesion. Nevertheless, in case of high embankments the acceleration of consolidation by sand drains or prefabricated drains is usually required to end the primary consolidation in a reasonable time (less than a year), [13].

The use of finite elements codes is helpful for the prediction and analysis of the behavior of embankment built on Tunis soft clay, in particular when the primary consolidation is taken into account.

Figure 3 shows a plan strain model of an embankment resting on thick soft clay layer overlaid by thin clayey soil. The Mohr-Coulomb constitutive law was adopted for all materials with appropriate geotechnical parameters.

The synthesis of numerical results deals with the evolution of settlement under the centre line of embankment and vertical and horizontal displacements at the toe of embankment.

The process of primary consolidation is directly linked to the evolution of excess pore pressure which maximum values are predicted immediately after the loading of embankment was applied. Excess pore pressure progressively decreases in time by water drainage, in parallel effective stresses are increased until a total dissipation of excess pore pressure that marks the end of primary consolidation, [23].

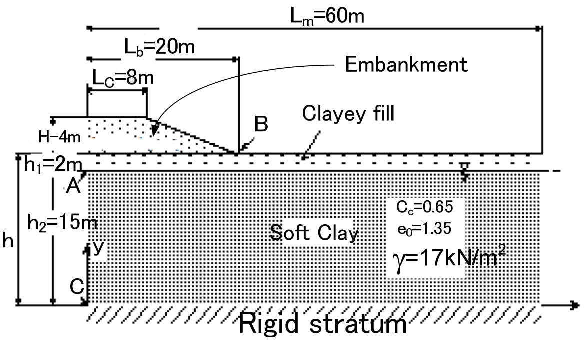


Fig. 3. Model of embankment on soft clay layer

5.1 Geometry and boundary conditions

Recommendations suggested by [14] were adopted, for ratios H/h and L_m/L_b, to carry out the numerical plan strain analysis of the behavior of embankment built on Tunis soft clay. Plaxis 2D code was used with a generated finite element mesh composed by 232 fifteen nodes triangular elements. As boundary conditions, the horizontal displacement is zero on lateral borders and zero components of displacement at stratum depth (Figure 4). At rest K_0 state was used to model the initial state of stresses. Embankment and clayey fill layer were assumed as unsaturated material.

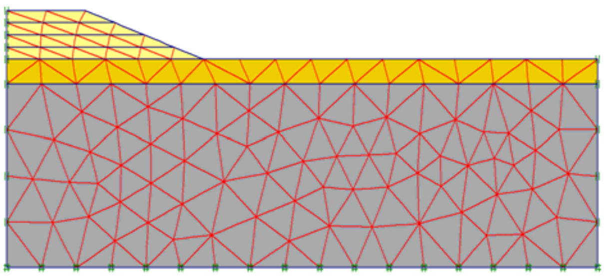


Fig. 4. Finite elements mesh and boundary conditions.

Computation stages: The simulation of embankment construction was scheduled in four computation stages, during each the primary consolidation of soft clay is programmed for a period of 50 days. Whereas for the final loading (full height of embankment) the primary consolidation is simulated until total dissipation of excess pore pressure. The allowable bearing capacity of soft clay layer must be verified at each stage of the construction of embankment.

5.2 Interpretation of numerical results

The maximum of consolidation settlement occurs at point A that coincides with the centerline of embankment where the induced vertical stress due to the applied load embankment load is at maximum as well (Figures 3 & 5). Contrarily the horizontal and vertical displacements are negligible at the vicinity of borders y = 0 and X = L_m in accordance with prescribed boundary conditions. This also confirms the convenient values considered for ratios H_r/h and L_m/L_b of the numerical plan strain model.

The evolution of settlement at point A is shown in figure 6. For a height of embankment of 4 m the final consolidation settlement, of about 0.84 m, is expected to end after 24 years. This prediction well illustrates how problematic is the stability of constructions built on Tunis soft clay due to its high compressibility that is associated to the long duration of primary consolidation.

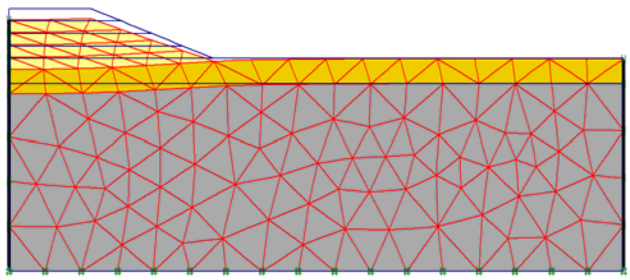


Fig. 5. Deformed mesh of embankment project at final stage of construction.

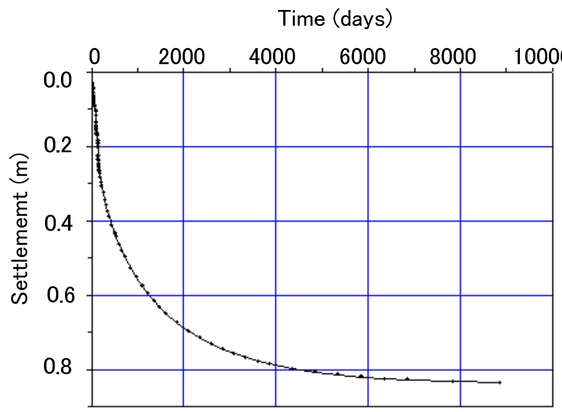


Fig. 6. Settlement evolution at point A (centre line of embankment).

5.3 Settlement and horizontal displacement at toe of embankment

At the end of embankment construction the short term deformation of soft clay is characterized by an upward movement at point B (Figure 3). This magnitude of vertical displacement is about 10% of total primary consolidation settlement that stabilizes after about twenty five years (Figure 7). At mid depth of soft clay layer the maximum horizontal displacement of 0.16 m prevails, it is due to the assumed zero horizontal displacement condition adopted along the centre line of embankment and stratum level. Figure 8 illustrates the evolution of horizontal displacement at point B in function of the stages of embankment loading. After each increase of loading increment it is predicted an immediate increase of upward movement at point B that is followed by a progressive settlement of consolidation, [23].

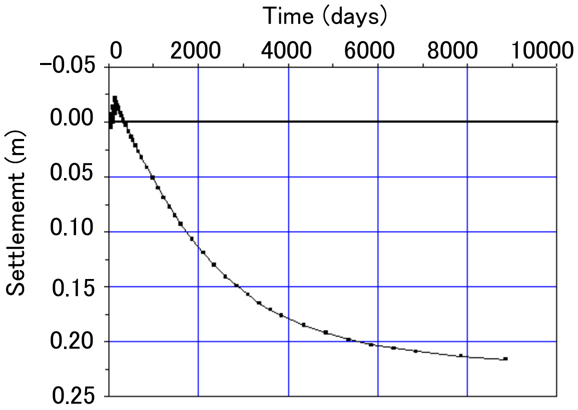


Fig. 7. Evolution of settlement at point B: toe of slope embankment

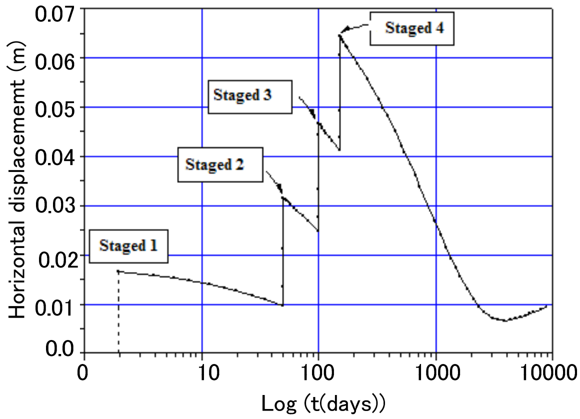


Fig. 8. Horizontal displacement at point B toe of slope embankment

Thus, it is necessary to proceed for the reduction and acceleration of consolidation settlement. As potential soil improvement techniques the stone columns reinforcement and prefabricated vertical drains (PVD) associated with preloading embankments are suggested. The feasibility and design of these two soil improvement techniques are discussed below.

6 AN OVERVIEW ON THE PRACTICE OF GROUND IMPROVEMENT IN TUNISIA

In Tunis City, around the North Lake, and coming soon for the South Lake, high buildings and interchanges are usually projected. Obviously, because shallow foundations do not suit for this kind of projects, two foundation's solutions are possible: either deep bored piles (of length between 40 m to 65 m) or soil improvement by using the preloading technique associated with vertical drains.

Classical soil improvement which only involves a preloading had been practiced in the early seventies where an embankment of 11 m height had been erected to consolidate the foundation of the Mazagran Bridge. Later on, in 1978, preloading associated to vertical sand drains had been practiced to consolidate the foundation of a sewerage channel. Since then, preloading using vertical drains, especially geodrain type, became a common practice in Tunisia to consolidate shallow week soil layers up to 18 m depth, [4]. Table 4 summarizes the data of main case histories where sand drains and geodrains have been practiced in Tunisia.

Table 4. Vertical drains projects in Tunisia

Project (year)	Type of drain	Length of drain (m)	Installa tion	Total lineal (km)
Int. La Charguia (1997)	Sand	34	Substit ution	5.536
Reclama-tion of South Lake (1999)	Geo Drain MB8 8	10 to 12	Displac ement	250
Rades La Goulette Bridge (2005)	Geo drain	12 to 16	Displac ement	57.81
Formerly Abu Nawas Int. (2006)	Geo drain	18	Displac ement	100

Int. denotes “interchange”.

Nevertheless, due to the well gained experience in bored piles, however with high cost, columnar reinforcement technique remains to date very unattended in the Tunisian context. Especially the well known “stone columns” reinforcement is not practiced by Tunisian contractors. Nevertheless, such reinforcement in Tunisia was executed in very few circumstances by foreigner contractors, [7].

As example the vibro compaction technique has been practiced three times for dams’ projects in view of preventing the liquefaction of loose sand layers in Northern West of Tunisia, [8].

Reinforced earth using geotextiles had been used to stabilize a natural slope. As one of the major project recently executed, the Radès La Goulette Bridge's project permitted the use of several soil improvement techniques essentially aimed at challenging problems posed by the Tunisian soft clay layer (French named “vase”) that covers the first 22 to 25 m depth around Tunis city and the suburbs of Radès and La Goulette, [10].

7 ILLUSTRATIVE TUNISIAN CASE HISTORIES OF GROUND IMPROVEMENT

7.1 Sand drains at La Charguia interchange

The main part of the project was to construct two approach embankments of heights respectively 3.5 m and 5.5 m, the breadth at upper crest of embankment is 24 m. Geotechnical conditions indicate a soil profile composed by six compressible clayey layers up to 52 m depth where a stratum level is encountered. The water table level was at 2m depth.

Regardless the verification of bearing capacity, a too much significant long term settlement is predicted on the 45 m thickness of compressible layers. Note that the evolution of consolidation settlement was predicted over some hundred years! The agreed solution was to install non displacement vertical drains by substituting the compressible soil with clean sand obeying in some filter condition. The length of sand drains of 34 m has been decided to accelerate the consolidation settlement along 70% of total thickness of compressible layers. The design of sand drains of 30 cm diameter only considered the prediction of spacing on the basis of Barron's theory so that the vertical consolidation was actually negligible as compared to that occurring in the horizontal direction. Soil improvement consisted in the installation of sand drains in triangular pattern of spacing of 3 m. It was necessary to install an upper drainage blanket of 50 cm thickness to evacuate the collected water from vertical drains as result of the consolidation of compressible layers.

The requirement of the project was to ensure a minimal global degree of consolidation of 90% in six months, while the in situ observations from recorded settlement have shown that only 80% of primary consolidation took place during eight months. From the synthesis of results it has been concluded that sand drains did not function successfully as expected. This discordance between predictions and in situ records was argued by the use of an overestimated coefficient of horizontal consolidation. Spacing between vertical drains was determined from the overestimated coefficient of horizontal consolidation c_h its self being predicted from an overestimated equivalent coefficient of vertical consolidation c_{ve} of the five compressible clay layers. Accordingly the design data were: c_{ve}= 2.8 10^{-8} m^2/s, $c_h = 5\ c_{ve} = 1.4\ 10^{-7}\ m^2/s$.

The learned lesson from this case history is the lack of geotechnical parameters cannot help to conclude with suitable predictions i.e. in accordance with the observed in situ records. Therefore, it was suitable in performing much more oedometer tests on soil samples extracted from the compressible layers in view of a better determination of their vertical coefficient of consolidation.

7.2 Prefabricated vertical drains (PVD): embankments of approach in Tunis North Lake

The construction of Radès La Goulette bridge had included four main lots among which the north connection that consisted in the reclamation of 20,000 hectares in the Tunis North Lake. Part of this lot is a pre stressed caisson bridge of approach and the interchange connecting between the express route of La Goulette and the main bridge that is lot 1 of this Mega project, [10]. 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. These reasons have revealed that the recourse to an improvement solution of soil layers under the embankments, along the first 10 m depth, was compulsory [6].

Such a solution aims, first, to 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 of soil improvement techniques were, [6]:

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

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

The geotechnical model is set up based on results recorded from bored holes and in situ tests (pressuremeter and SPT) data including those recorded from the static cone penetration and pore pressure (piezocone test). The first geotechnical investigation (boreholes, drilled core samples, pressuremeter tests, vane tests, SPT) was mostly conducted over various depths (40 m to 110 m) [15]. The results obtained from boreholes located in the area of North Lake were exploited. The preliminary geotechnical synthesis has displayed a very soft compressible layer of thickness varying from 8 to 10 m.

This soft layer is divided into two sub layers: the first is a highly compressible mud of about 5 to 6.5 m thickness, while the second sub layer consists in compressible sandy clays. It was noticed a moderate difference between the cone penetration resistance and undrained cohesion in the two sub layers. Focus on the behavior of these two levels has been addressed for a thorough study of the stability and preloading embankment designed for the geodrains' solution. The soil profile and geotechnical parameters of each horizon were presented in [10]. The installation of a grid of PVD descended from a platform levelled +0.5 m NGT, up to 10 m depth. The proposed type of PVD is Mebradrain (MD) 88 which is of flat type of thickness 0.5 cm and 10 cm width. The MD 88 material was experienced in prior prefabricated vertical drains projects as the reclamation of South Lake of Tunis.

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

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

7.3 Stone columns reinforcement: oil tank at Zarzis

An oil storage tank was built at Zarzis terminal on reclaimed area at the South East Tunisian Coast [18]. The applied load by the tank is approximated as quasi-uniform vertical stress of 120 kPa, which exceeds the allowable bearing capacity of initial soil. Therefore, it was necessary to increase the allowable bearing capacity at least to 120 kPa, and to reduce the tank's settlement to the allowable limit of 6 cm which slightly exceeds the ratio 1/1,000 of diameter tank usually required for oil tanks projects.
Reinforcement by end-bearing stone columns was agreed to ensure the overall stability of tank. The reinforcement was executed along an average depth of 7 m with a nominal diameter of 1.2 m of columns installed in equilateral triangular grid. Figure 9 summarizes the geotechnical properties of initial soil and column material and shows stone columns installed in circular area of radius greater than the tank radius + 4 m. This case history has shown a successful use of stone columns as for the recent project reported in [7].

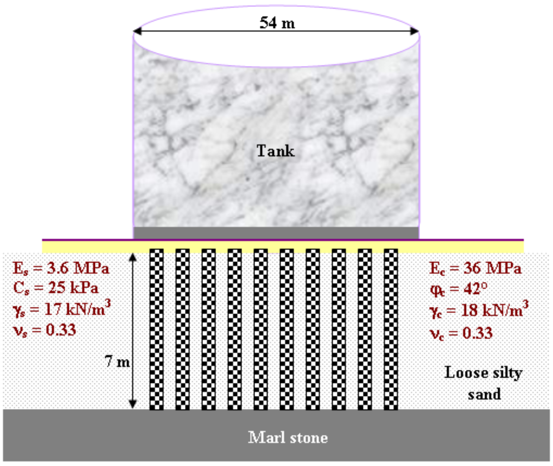


Fig. 9. Oil tank on improved soil by vibro compacted stone columns

7.4 Reinforcement by sand piles

The use of sand piles in Tunisia offers an intermediate alternative between vertical drains (sand and geo) of diameter less than 30 cm and the well known reinforcement by stone columns. Sand piles are installed by some Tunisian contractors under the form of driven metallic tubes of 40 cm diameter with a capped tip. Installation of these tubes is made by a vibrated penetration in laterally expanded soft clay layer up to prefixed treatment depth of 6m to 8m. Figure 10 explains the formation of sand piles by passes of 50 cm as result of the withdrawal of metallic tube with recoverable tip. The installed network of sand piles, as for reinforcement by stone columns, is overlaid by a draining blanket of 40 cm thickness made up of the sand piles' material to allow a more uniform settlement of the loaded structure and to enhance the horizontal drainage of collected water from the sand piles that behave like vertical drains of material obeying to the filter condition [13].

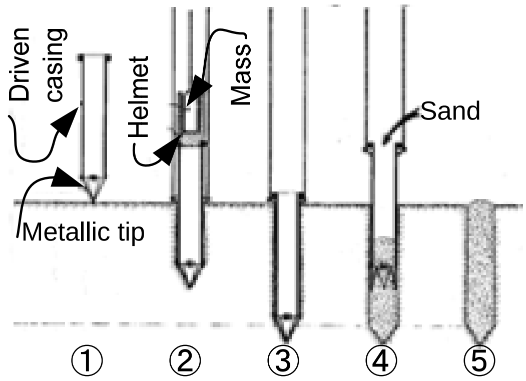


Fig.10. Installation of sand piles, [13]

The reinforcement by sand piles whenever designed without specific care may lead to unhappy circumstances. In fact for a project tank of 31 m diameter built in Radès La Goulette area the installation of sand piles of 6 m length has led to unallowable settlement of consolidation that seriously affected the cylindrical structure of the tank. After fifteen years, the commission of tank has been stopped in order to suggest a soil improvement technique to neutralize the settlement's evolution. This unsuccessful design consisted in underestimating the depth along which a significant settlement is expected to occur (minimum of 25 m). Elsewhere the improvement area ratio, that defines the quantity of reinforcing sand to be incorporated within compressible soil, was also underestimated.

8 CONCLUSION

This paper reported on the geological context of Tunis City and the geotechnical properties of Tunis soft clay that is a problematic soil characterized by a weak shear strength and high compressibility. Identification parameters of Tunis soft clay, usually determined from laboratory tests, can be correlated between each others. But shear strength characteristics, due to the disturbance that occurs during the extraction of soil specimens, should be predicted via correlation through recorded data from pressuremeter, vane shear and other in situ tests.
Although several projects of various infrastructures have been built around the North and South Tunis' Lakes few lessons were learned about the characterization of Tunis soft clay. In this way the present paper can be considered as a first attempt to suggest meaningful correlation between the currently used geotechnical parameters. However, as advice, it is should be collected much more synthesized data about Tunis soft clay in view of its better characterization.
The numerical study of behavior of an embankment built on Tunis soft clay has highlighted the need to practice soil improvement techniques like vertical drains, and stone columns to overcome the lack of bearing capacity and/or significant settlement of consolidation that normally develops over dozen of years.
The discussion of selected Tunisian case histories well

illustrated the feasibility and the efficiency of usual soil improvement techniques, like geodrains associated to preloading, in Tunis soft clay when well controlled soil parameters are used for the design.

9 REFERENCES

[1]. Amar S, & Jézequel JF, "Essais en place et en laboratoire sur sols cohérents", Bull liaison Ponts et Chaussées n°58, 1972, pp 97-108. Paris.

[2]. Bouassida M, "Modeling the behavior of soft clays and new contributions for soil improvement solutions", Keynote Lecture. Proc. 2nd Int. Conf. on Problematic Soils. December 3-5th. Petaming Jaya, Salengor, Malaysia. Editors Bujang, Pinto & Jefferson, 2006, pp.1-12.

[3]. Bouassida M, & Frikha, W, "Extreme pressure due to the expansion of a cylindrical or spherical cavity in limitless medium: Applications in soil mechanics. Acta Geotechnica", 2, 2007, pp. 87-96.

[4]. Bouassida M, Guetif Z, Lefi M.S, & Chouikha A, "Comparaison colonnes ballastées-géodrains avec pré chargement", dans le cadre du projet " Pont Radès-La Goulette". *L'ingénieur tunisien*, n°27, 2003-2004, pp. 20-30.

[5]. Bouassida M, & Hazzar L, "Comparison between Stone Columns and Vertical Geodrains with Preloading Embankment Techniques", paper 7.18a. CD Rom of, 6th Int. Conf. On Case Histories in Geotechnical Engineering. Arlington VA, August (USA), 2008, pp. 11-18.

[6]. Bouassida M, "Etude expérimentale du renforcement de la vase de Tunis par colonnes de sable - Application pour la validation de la résistance en compression théorique d'une cellule composite confinée", Revue Française de Géotechnique, n° 75, 1996, pp 3-12.

[7]. El Ghabi B, Hermges T, Lambert S, Ellouze S, Bouassida M, & Mejri A, "Comparison between predicted settlement of a group of stone columns foundation and in- situ records", *2nd Int. Conf. on Geotech. Eng.* October, 2010, pp. 25-27, Tunisia: 255-264.

[8]. El Ouni, M.R., Bouassida M. & Das M.B., "Vibrocompaction improvement of Tunisian liquefiable sands" Proc. 17th ICSMGE'09. Vol 3, 2009, pp. 2366-2369. Editors Hamza, Shahien & El-Mosallamy. Alexandria 5th – 9th, Egypt.

[9]. Kaâniche A, Inoubli MH & Zarguouni F, " Développement d'un système d'informations géologiques et géotechniques et réalisation d'un atlas géotechnique électronique " , Bull Eng Geol environ 58(4), 2000, pp. 321- 335.

[10]. Kanoun F, & Bouassida M, "Geotechnical aspects of Rades La Goulette project (Tunisia)", ISSMGE Bulletin of News, 3rd Volume September, 2008, pp. 6–12.

[11]. Kanoun F, "Propriétés dynamiques de la vase de Tunis", Thèse de Doctorat de 3ème cycle, Univ. P. Sabatier, Toulouse, 1981.

[12]. Klai, M, & Bouassida, M. "Comparison between behaviour of undisturbed and reconstituted Tunis soft clay", Proc. 2nd Int. Conf.New Develop. In SMGE. Edit. Atalar et al. 28-30 May NEU Nicosia, 2009, pp. 414 – 419.

[13]. Magnan J.P, "Théorie et pratique des drains verticaux", Edition Technique et Documentation – Lavoisier, Paris, 1981, 335 pages.

[14]. Mestat P, "MOMIS : Une base de données sur la modélisation numérique des remblais sur sols compressibles et sur la confrontation calculs - Mesures in situ, Bulletin des laboratoires des ponts et chaussées -232", mai-juin - réf. 4376, 2001, pp. 43-58.

[15]. Nippon Koei, Pci, Scet – Tunisie & Studi Ingénierie, "Etude d'exécution et supervision de construction du pont Radès-la Goulette", Phase 1. Rapport de la campagne géotechnique, 2001.

[16]. Piementa J, "Le cycle Pliocène-actuel dans les bassins parliques de Tunis", Mémoires Soc Géol France 38, 1(85), 1959, pp. 1- 176

[17]. Keller-France, "Réservoirs de GPL à Gabès ", Rapport de visite de chantier (In French), 2009.

[18]. Solétanche-Bachy, "Bac de stockage du terminal pétrolier de Zarzis", Internal report (French), 1980.

[19]. Terzaghi K. & Peck BB, "Soil Mechanics in Engineering Practice", 2nd Edition John Wiley & Sons. Inc., New York, 1967.

[20]. Touiti L, Boussetta, S, & Bouassida M, "Contribution à l'identification et à la classification de la vase de Tunis. Comptes rendus n°1 14ème Congrès Régional d'Afrique de Mécanique des Sols et de géotechnique, Les sols de l'Afrique", Yaoundé, 26-28 novembre. Eds Bouassida et al, 2007, pp. 177-184.

[21]. Touiti L, Bouassida M, & Van Impe W, "Discussion on Tunis soft soil sensitivity", Geotech Geol Eng, 27, 2009, pp. 631- 643.

[22]. Tounekti, F, Bouassida, M, Klai M, & Marzouki, I, "Etude expérimentale en vue d'un modèle de comportement pour la vase de Tunis. Revue Française de Géotechnique", Paris, 2008, pp. 114, 25 – 36.

[23]. Tounekti F, Bouassida M, & Mroueh H, "Numerical study of the behaviour of projects founded on Tunis soft clay", Proc. Int. Conf. Geotech. Eng. "Innovative Geotechnical Engineering", March 24-26th Hammamet (Tunisia), Editors Bouassida et al, 2008, pp. 353-360.

[24]. Vidaly JF, "Relation entre les propriétés physico-chimiques et les caractéristiques mécaniques des sols compressibles", Rapport de recherche LCPC No 65, Paris, 1977.

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