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Numerical Modelling of Tunis Soft Clay

M. Klai[1], M. Bouassida[2] and S. Tabchouche[3]
[1,2]*Université de Tunis El Manar – Ecole Nationale d'Ingénieurs de Tunis. LR14ES03-Ingénierie Géotechnique, BP 37 Le Belvédère, 1002, Tunis, Tunisia*
[3]*Laboratoire de Recherche en Génie Civil, Université de Biskra, Biskra, Algeria*
E-mail: Mounir.bouassida@fulbrightmail.org

ABSTRACT: The paper briefly reviews research investigations conducted on Tunis soft clay that is classified as problematic soil. Results obtained from an experimental study carried out on undisturbed Tunis soft clay specimens are presented and interpreted. On the basis of experimental results the paper discusses which constitutive law can describe at best the observed behaviour of Tunis soft clay? The elastoplastic behaviour modelled by the Hardening Soil Model (HSM) is then justified based on numerical simulation of oedometer and triaxial tests carried out on undisturbed soft clay specimens. Adopted parameters of the HSM model are considered to simulate the behaviour of geotechnical structures founded on Tunis soft clay using the finite element code Plaxis V9-2D and the FLAC3D code. The prediction of behaviour of two Tunisian case studies is analyzed: cylindrical oil tank on soft clay reinforced by sand columns and full pile loading. Comparison between predictions by the soft soil model (SSM) and HSM is presented.

KEYWORDS: Behavior, Characterization, Hardening, Numerical, Simulation, Soft clay

1. INTRODUCTION

The soil profile of Tunis City mainly consists of a layer located between 3 and 20 m depth constituted by greyish sandy clay, which is at the origin of the contamination observed on several constructions built on this ground. This soil commonly called the Tunis soft clay (TSC) is very problematic because of the difficulty to extract undisturbed specimens for performing laboratory tests. Besides, performing in situ tests sometimes leads to unrealistic data due to its very low stiffness compared to that of expanded membrane to measure the limit pressure during pressuremeter tests. Bouassida (2006) also reported the difficulty in predicting the undrained cohesion of TSC from in situ vane shear tests due to unreasonable interpretation of these results. In parallel, the use of reconstituted TSC to avoid disturbance of specimens does not reflect the actual behaviour of in situ soil (Klai & Bouassida, 2009).

"Relevant contribution on numerical modeling of TSC was proposed by Tounekti et al (2008). Those authors assessed the validity of soft soil model (SSM) as suitable constitutive law for the remolded Tunis soft clay after comparisons between numerical results (simulation of oedometer and triaxial tests) and measurements during performed tests in laboratory. Numerical prediction of two geotechnical infrastructures have been proposed after the SSM was adopted for TSC."

This paper, first, presents the soil profile and geotechnical properties of Tunis soft clay after collected data from several geotechnical investigations. Then, experimental results obtained from laboratory tests carried out on undisturbed specimens of TSC are presented and interpreted. The simulation of performed triaxial and oedometer tests is conducted by Plaxis software in order to validate the constitutive behaviour law of TSC. Then, the study of behaviour of tank project on reinforced TSC and pile loading test are discussed after numerical computations conducted by the 2D Plaxis software and the FLAC 3D code. Finally, the simulation of full scale load test is undertaken to assess predictions by the HSM and the soft soil model (SSM).

2. GEOTECHNICAL PROPERTIES OF TUNIS SOFT CLAY

2.1 Soil strata

Several soil profiles of Tunis City in the area of Tunis Sport City project and in La Goulette's area are displayed in Figures 1 and 2 respectively. For each investigated site the geotechnical profile is reconstituted from bore holes and pressuremeter tests conducted beyond 40 m of depth. The soil stratigraphy indicates that the layer of the soft soil that is generally overlain by a thin fill layer can extend up to 40 m depth or more.

3. GEOTECHNICAL INVESTIGATIONS: SAMPLING AND LABORATORY TESTS

In the urban area of Tunis City two bore holes namely BH1 and BH2 spaced at 10 m were executed at the "Avenue de la République". Cored specimens namely CS1 and CS2 have been extracted respectively at 7.5 m and 9.5 m depths by a double rotary driller of external diameter 101 mm.

- BH1 soil profile shows an upper fill layer of 7 m thickness overlaying the Tunis soft clay layer of about 18 m thickness. Three undisturbed cored specimens (Specimen 1, specimen 2, and Specimen 3) have been extracted at depths of 7.55 m, 9.85 m and 18.35 m respectively.
- BH2 soil profile shows a similar formation as that observed in BH1. Thickness of the upper fill layer is 2.5 m. Two cored specimens (Specimen 4 and Specimen 5) have been extracted at depths of 3.75 m and 7.75 m respectively.

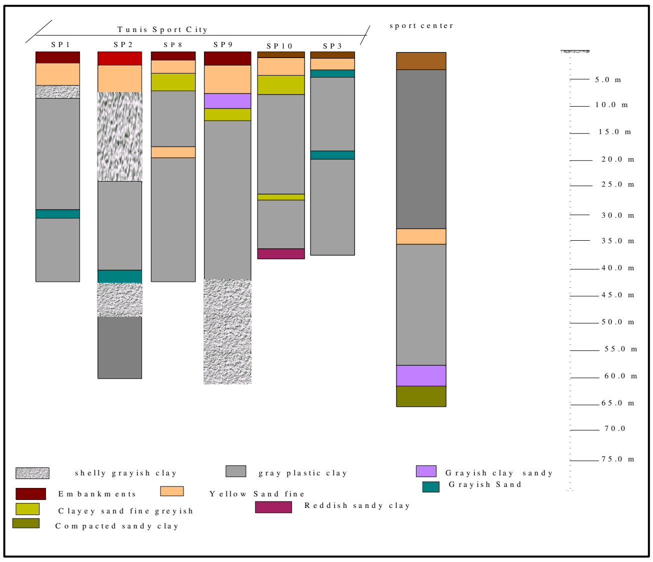

Figure 1 Typical soil profiles at sites of Tunis Sport city and Sport centre A (Klai, 2014).

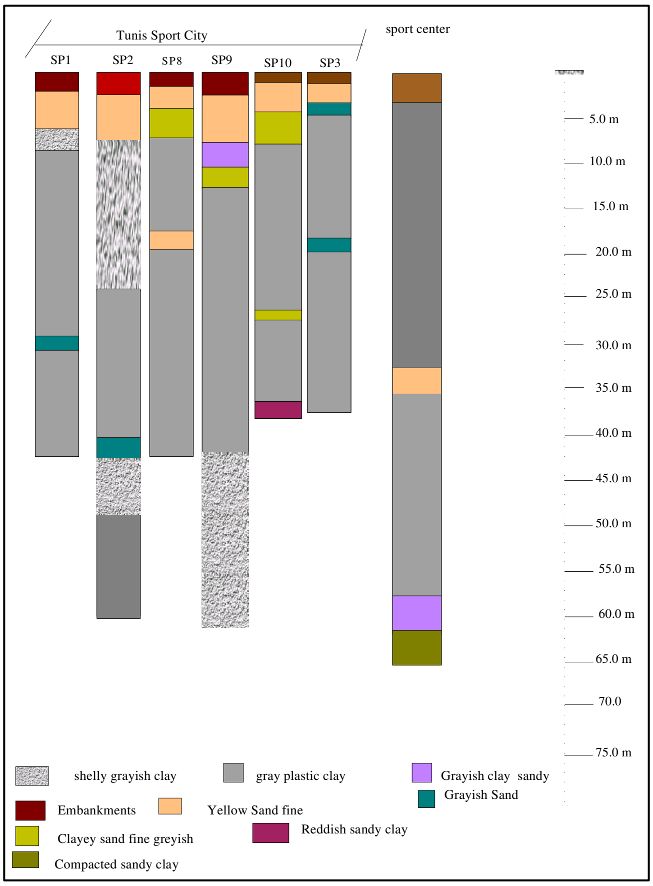

Figure 2 Typical soil profiles of La Goulette site (Klai, 2014)

Undisturbed samples are cored in PVC tubes of 101 mm external diameter, logged in the rotary driller gently penetrated within soft clay layer at displacement rate of about 10 mm/minute. Extracted PVC tubes are then placed in wood boxes and transported from the site to laboratory so that shocks are prevented.

In laboratory, undisturbed soil specimens are extracted by penetrated thin cutting shoe in the direction of in-situ extraction.

Therefore, soft soil specimens are ready for laboratory tests from extracted cutting shoe.

Laboratory tests have been carried out at the soil mechanics laboratory of the Higher Institute of Technological Studies of Rades (Tunis). The soil identification tests included: grain size distribution (sieve and hydrometer), total unit weight, specific gravity, Atterberg limits and content of organic wastes (OM). The second group of tests included oedometer tests (compressibility and consolidation), consolidated undrained (CU) triaxial tests and consolidated drained (CD) triaxial tests.

3.1 Experimental investigation: tests and results

3.1.1 Classification tests

As part of soil identification wet sieve and sedimentation analyses were performed on five undisturbed soft clay specimens. Grain size distributions show the average minimum fines content (grain size < 0.08 mm) is about 87% (Klai, 2014). Table 1 summarizes the identification parameters of the five undisturbed soft clay specimens.

The classification of saturated Tunis soft clay is highly plastic silt with very low consistency. For undisturbed soft clay specimens, which contain wastes of shell, Atterberg's limits values are lower than those obtained for the reconstituted soft clay (Bouassida, 2006).

Several useful properties also help in a better identification of soft clays. Indeed, chemical tests for the determination of content of organic matter and the calcium carbonate respectively provide useful information about the compressibility and strength (Das, 2006).

Table 1 Identification parameters of undisturbed Tunis soft clay

Specimen N°	Specific gravity	Unit weight (kN/m^3)	W_L	I_C	I_P
1	2.62	17.4	46	0.31	19
2	2.50	16.1	50	0.52	5
3	2.53	18.0	51	0.70	9.5
4	2.32	17.6	65	0.40	15
5	2.39	16.9	79	0.65	29

W_L = liquid limit; W_P = plastic limit; I_c = consistency index; I_p = plasticity index.

The percentage of organic content recorded for reconstituted soft clay was about 3.12 %. Undisturbed soft clay has a higher organic content than the reconstituted soft clay which confirms its low compressibility of about 10%.

According to recorded results during the geotechnical investigations carried out within the framework of Radès La Goulette Bridge project (Groupement Nippon Koeï et al, 2001), the Tunis soft clay is characterized by variable percentage of organic content found in the range of 0.8 to 22 %.

3.1.1.1 Oedometer tests

After performing oedometer tests, recorded values of compression index, swelling rate and pre-consolidation stress for the undisturbed soft clay are summarized in Table 2. The values of compression index and swelling rate indicate that the soft clay is highly compressible and with insignificant swelling index.

Referring to Table 2 the undisturbed soft clay specimen 1, specimen 2, specimen 3, specimen 4 and specimen 5 extracted at average depth of 8.5 m are classified as under consolidated. Indeed, from Figures 3 & 4 the pre-consolidation stress of tested specimens is lower than the effective vertical stress at extraction depth that varied from 52 kPa to 180 kPa. Compression and swelling indices indicate that undisturbed Tunis soft clay has lower compressibility and swelling than those of reconstituted soft clay (Klai and Bouassida, 2009). Meanwhile recorded values of compression index are in accordance with those reported by (Touiti et al, 2009) from other geotechnical investigations data conducted on Tunis soft clay undisturbed specimens, i.e. $0.4 \leq C_c \leq 0.6$.

Table 2 Oedometer characteristics of undisturbed Tunis soft clay

Specimen N°	C_c	C_s	σ'_p (kPa)
1	0.430	0.057	12
2	0.485	0.056	25
3	0.350	0.057	17
4	0.385	0.057	14
5	0.384	0.057	14

C_c is Compression index; C_s is Swelling index; σ_p' is pre-consolidation pressure

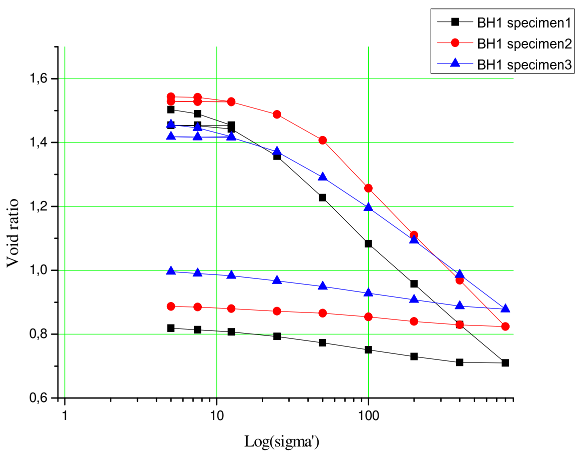

Figure 3 Oedometer curves of three undisturbed specimens.

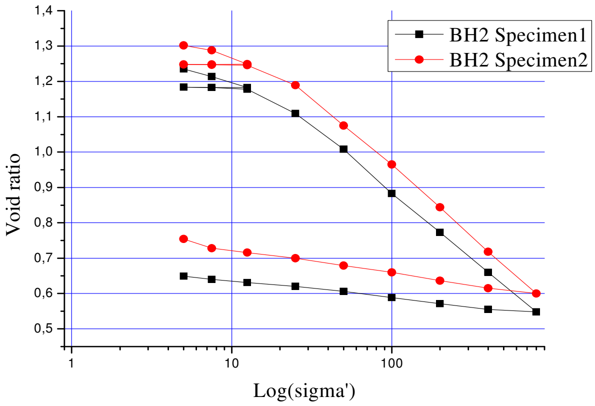

Figure 4 Oedometer curves of two undisturbed specimens

3.1.1.2 CU & CD Triaxial tests

Consolidated undrained (CU) triaxial tests with measurement of excess pore pressure and two series of consolidated drained (CD) triaxial tests have been primarily performed on reconstituted soft clay specimens by Tounekti et al (2008). CU triaxial tests enable the determination of short term shear strength characteristics of undisturbed soft clay specimens. CD triaxial tests are used to estimate the long term shear strength characteristics of compressible clays.

The drained friction angle of tested specimens is found in the range of φ'= 19.2 to 23.7°. The drained cohesion is not very significant since it does not exceed 5 kPa (Table 3). The inherent over-consolidation of tested specimens is more likely attributed to the applied consolidation stress during triaxial test (up to 300 kPa) which largely exceeds the in situ effective overburden stress at depth of extracted specimens (less than 20 m). tg λ_{cu} is the rate of increase in undrained cohesion C_u with depth (or with consolidation stress).

Table 3 Shear strength parameters of undisturbed Tunis soft clay

Specimen N°	C_{cu} (kPa)	C′ (kPa)	φ′ (°)	tgλ_{cu}
1	7.53	5.0	22.7	0.34
2	8.49	4.0	23.7	0.37
3	8.67	5.1	20.8	0.33
4	7.79	3.6	19.2	0.37

C' is drained cohesion; φ' is drained friction angle.

3.1.2 Justification of the Hardening Soil constitutive Model for Tunis Soft Clay

On the basis of obtained results from experimental investigation it is hereby intended to discuss whether the hardening soil model (HSM) can be considered as constitutive law for the TSC as compared with the Modified Cam Clay (MCC) model and the soft soil model (SSM).

The elastoplastic non-linear isotropic strain hardening model, or the well-known HSM, was suggested to reproduce the observed macroscopic phenomena for grounds such as (Schanz and Vermeer, 1998):

- Densification, decrease in the volume owing to the plastic deformations (distortions),
- Increase in stiffness with depth;
- Stress-strain history taking account of pre-consolidation effect;
- Dilatancy (resp. contractancy) increase (resp. decrease) in volume.

Contrarily to other models like the Modified Cam Clay, the amplitude of deformations (distortions) of the ground can be modelled with more precision by incorporating three stiffness parameters which correspond to the rigidity of triaxial loading (E_{50}), the rigidity of triaxial unloading-reloading (E_{ur}), and the œdometer modulus (E_{oed}). Figure 5 displays the stiffness moduli considered by the HSM in the deviatoric stress-axial strain plot as usually recorded during shear triaxial test.

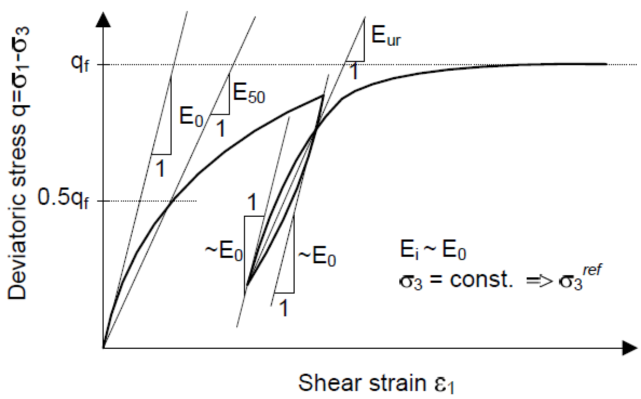

Figure 5 Stiffness moduli adopted by the HSM from typical stress-strain curve recorded in the shear phase of triaxial test

Zimmerman et al (2010) recommended the adoption of the standard HSM for normally consolidated soft clays. Relationships between the parameters of the HSM are as follows:

$$E_{ur}^{ref} = 3E_{50}^{ref} \quad \nu_{ur} = 0.35 \quad P^{ref} = 100kPa \quad K_0^{nc} = 1 - \sin\varphi'$$
$$R_f = 0.9 \quad \sigma_t = 0 \quad m = 1 \quad \psi = 0.$$

The Hardening Soil Model is selected to simulate the behaviour of Tunis soft clay since it is capable to account for the increase in stiffness due to consolidation stress. It is essential for the modelling of foundation that extends to relatively deep soil layers for example underneath an embankment. From recorded experimental data the input parameters of HSM adopted for Tunis soft clay layer are presented in Table 4.

In this paper numerical investigation is carried out to simulate the oedometer and triaxial tests carried out on TSC specimens. Aside from the HSM, the Modified Cam-Clay (MCC) model is also considered to characterize the TSC for the purpose of numerical predictions.

Table 5 presents the geotechnical parameters of the Modified Cam Clay model considered for undisturbed specimens extracted at the Avenue Mohamed V at depths from 3 to 20 m.

Table 4 Hardening soil model parameters of Tunis Soft Clay

Site: Avenue Med V		C' (kPa)	φ' (°)	E_{oed}^{ref} (kPa)	E_{50}^{ref} (kPa)
BH 1	Specimen 1 (7.2 – 7.9 m)	5.0	22.7	1337	1672
	Specimen 2 (9.5 – 10.2 m)	4.0	23.7	1186	1482
	Specimen 3 (18 – 18.7 m)	5.1	20 .8	1643	2054
BH 2	Specimen 1 (3.3 – 4.0 m)	3.6	19.2	1494	1867

Table 5 Parameters of modified Cam Clay Model considered for the Tunis Soft Clay

λ	e_0	κ	M	C' [kPa]	υ	$k_x = k_y$ [m/day]	γ_{sat} [kN/m^3]
0.21	1.55	0.024	0.83	13	0.35	1.73 10^{-6}	17.4

3.2 Simulation of observed behaviour of TSC

The simulation of oedometer and triaxial tests is conducted by using the 2D Plaxis software V9 in axisymmetric condition due to the cylindrical geometry of tested specimens (Figure 6).

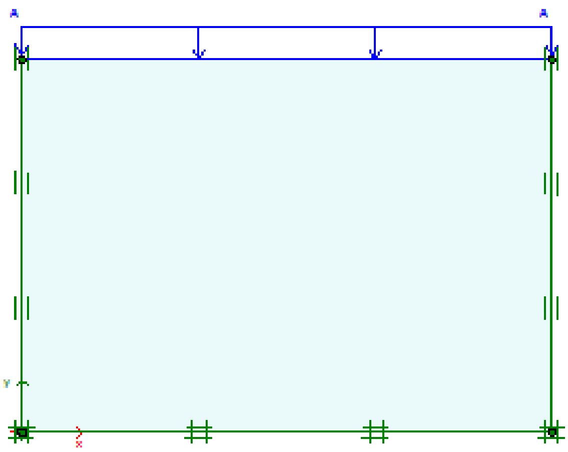

Figure 6 Axisymmetric Plaxis model

3.2.1 Oedometer tests

First, obtained data from 1-D oedometer consolidation test conducted on Tunis soft clay specimens are used to predict the settlement by the Terzaghi's theory of consolidation (Terzaghi, 1941). Second, numerical computations are run by Plaxis software with the assumed HSM and the MCC model input parameters. The settlement predictions obtained by Plaxis software with the HSM input parameters presented in Table 4 are found 7% higher than those calculated by Terzaghi's 1-D theory of consolidation. Due to the difference between the HSM and MCC model the 7% difference in settlement prediction is considered quite negligible. Quarter of the specimen is considered for numerical simulation due to the geometrical and loading symmetries (radius equals 17 mm; Height equals 35 mm).

3.2.1.1 Interpretation of results

From Figures 7, 8, 9, and 10 it is noticed that the numerical prediction by the HSM during the primary consolidation phase is overall in accordance with the observed behaviour on tested specimens.

In turn, significant difference is noticed between experimental data and the numerical predictions obtained by the Modified Cam Clay model that overestimates the prediction of decrease in void ratio.

During the unloading–reloading phase of Figures 7 to 10 (on the right of slope C_s), the numerical prediction by the MCC model slightly underestimates the swelling of specimens, whilst the HSM shows a good agreement with experimental measurements.

The overestimated consolidation using the MCC model is essentially owed to the parameters λ and κ which represent respectively the slopes of the oedometer curve both in consolidation phase and during unloading - reloading of the specimens of Tunis soft clay.

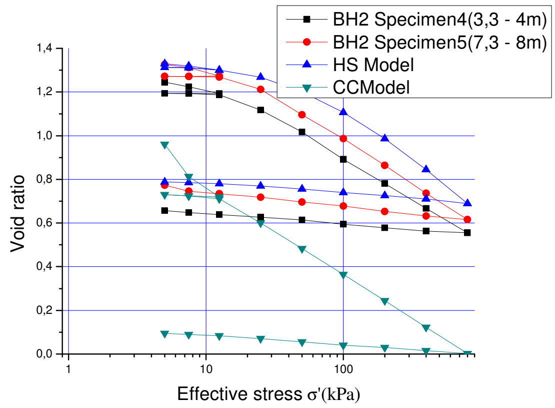

Figure 7 Predicted behaviour of TSC modelled by the HSM and MCC model and experimental measurements from oedometer tests (specimens 4 & 5)

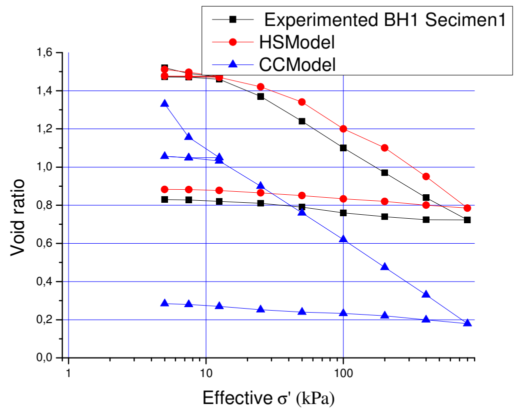

Figure 8 Predicted behaviour of TSC modeled by the HSM and MCC model and experimental measurements from oedometer test (specimen 1)

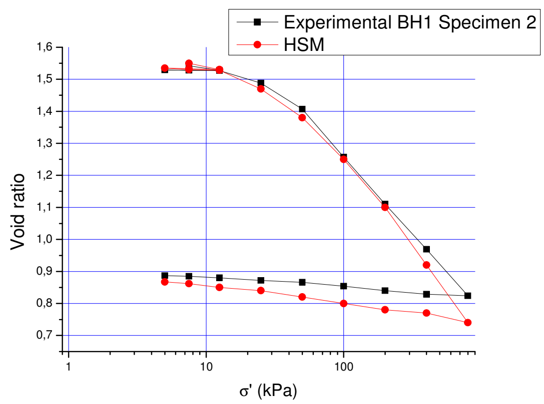

Figure 9 Predicted behaviour of TSC modeled by the HSM and MCC model and experimental measurements from oedometer test (specimen 2)

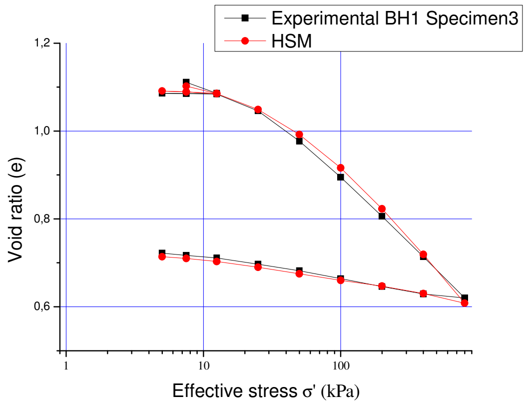

Figure 10 Predictions by the HSM of TSC behaviour compared with data from oedometer test (specimen 3)

3.2.2 Triaxial tests

Figures 11 and 12 show the numerical of deviatoric stress versus axial strain, as predicted by the HSM, for various isotropic consolidation stresses as well as experimental measurements during the shear phase of CU triaxial tests performed on specimens 4 and 5.

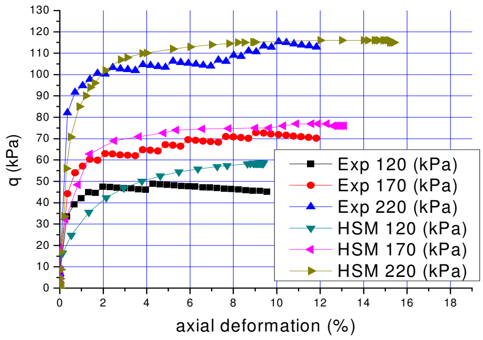

Figure 11 Experimental and numerical results during shear loading of consolidated undrained triaxial test (specimen 4)

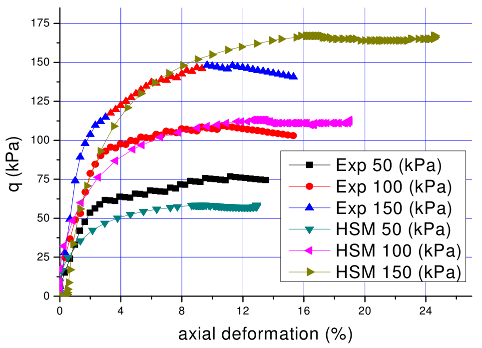

Figure 12 Experimental and numerical results during shear loading of consolidated undrained triaxial test (specimen 5)

The observed behaviour during shear loading is overall in fair agreement with numerical results predicted by the HSM. This leads to the conclusion that the adopted failure parameters (C' and φ') are quite representative of the observed behaviour of undisturbed TSC specimens. Using Plaxis software (version 9.2) the simulation of observed behaviour of those specimens subjected to oedometer and triaxial tests showed that the HSM predictions are in good agreement with measured data rather than predicted results obtained by the Modified Cam-Clay model (Klai, 2014). For this reason the HSM will be considered to model the TSC behaviour for the prediction of two case studies.

4. STUDY OF BEHAVIOUR OF OIL RESERVOIR

Tunisian case study of oil tank at La Goulette is now considered to predict the behaviour of unreinforced and reinforced Tunis soft clay by sand columns both in plane strain and axisymmetric conditions. Data of this case study were considered by Bouassida & Carter (2014) for the determination of optimized design of reinforcement of the TSC by end-bearing sand columns. Diameter of oil reservoir and length of sand columns were 30 m and 10 m respectively.

Numerical computations are conducted by the Plaxis V9.2D finite element code both in plane strain and axisymmetric conditions by adopting the hardening soil model as constitutive law for Tunis soft clay. More else, the finite difference FLAC3D code is also used to predict the behaviour of oil reservoir to be compared with the finite element Plaxis code (Figure 13).

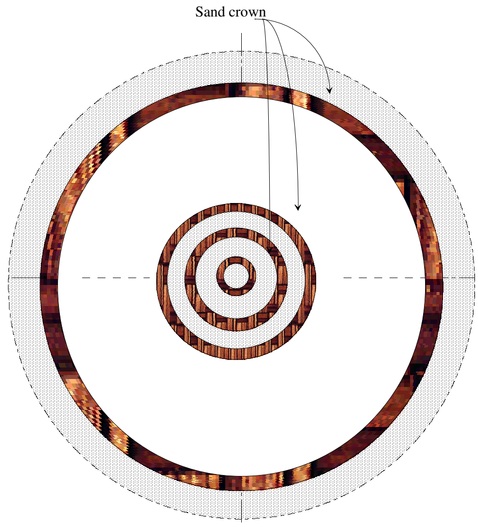

Figure 13 Axisymetric modeling with equivalent sand crowns

Plane strain computation: it is performed after the determination of equivalent width, B, of squared tank that has the same area as the circular tank of radius R = 15 m, then: $B = \sqrt{\pi} R = 26.58 m$ The numerical plane strain modelling of TSC layer reinforced by equivalent sand trenches in Figure 14 is detailed by Klai (2014).

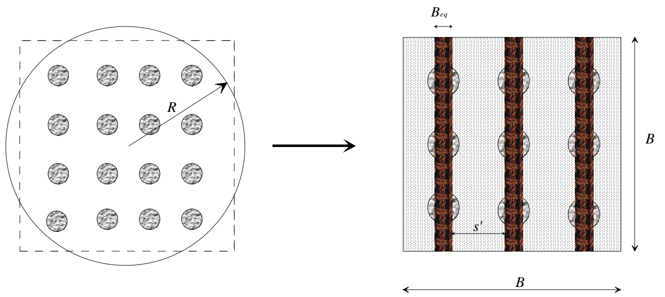

Figure 14 Plane strain modeling modelling with equivalent trenches

4.1 Predictions by the HSM

The variation of the settlement of unreinforced soil at the border of oil tank sketched in Figure 15 shows a maximum consolidation settlement of 4.2 cm. The predicted maximum consolidation settlement is 21.8 cm at the centreline of tank, the consolidation time approaches ten years. It is clearly seen that the long term settlement due to the consolidation of soft clay is slightly larger than the settlement of 20.8 cm predicted by different methods that adopt the linear elastic behaviour for constituents of reinforced soil (Bouassida et al, 2015).

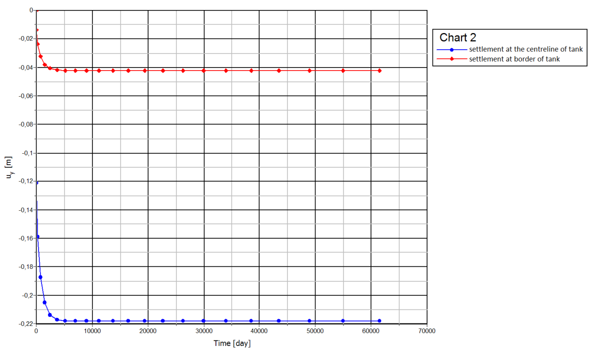

Figure 15 Variation of settlement of oil tank resting on unreinforced soil as predicted by the plane strain condition

The perfect elastoplastic Mohr-Coulomb law has been adopted for the reinforcing sand as constitutive columns' material with geotechnical properties displayed in Table 6 that also includes parameters of TSC considered for the prediction of optimized reinforcement as proposed by the methodology of design detailed in Bouassida and Carter (2014). "Those parameters were adopted after the study presented by Bouassida M. and Bouassida W. (2012) which dealt with the reinforcement by rigid inclusions of retrofit of tank project".

Table 6 Geotechnical properties of sand columns' material

Parameters	H (m)	E (MPa)	ν	φ (°)	C (kPa)	γ (kN/m^3)
Sand columns	10	20	0.33	38	0	18
Tunis soft clay	10	2	0.4	0	24	17

The variation of consolidation settlement, in plane strain condition, is plotted in Figure 16 both at the centreline and at the border of oil tank with predicted values are 7 cm and 3.8 cm respectively.

The quasi constant distribution of settlement at the surface of reinforced ground is partially owed to the presence of blanket drainage layer. Consider the differential settlement equals the difference between predicted settlements at the centreline and border of oil tank. Comparison between Figures 15 and 16 well show that the reinforcement by sand columns greatly reduces the differential settlement from 17.7 to 3.2 cm that plays an important benefit for long term stability of oil tank.

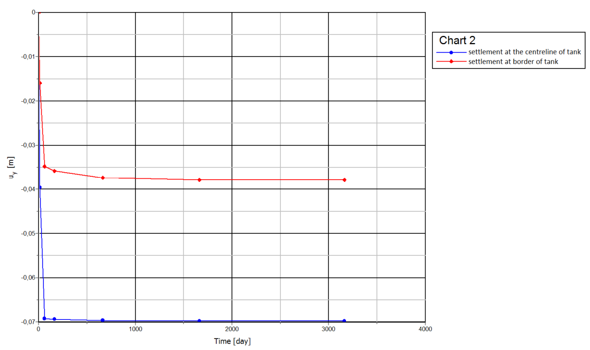

Figure 16 Variation of consolidation settlement of reinforced soil (plane strain condition)

4.2 Predictions of tank behaviour by the HSM (axisymmetric study)

The axisymmetric study is carried out in comparing between the predicted 2D behaviour of unreinforced soil and reinforced soil by sand columns both in plane strain and axisymmetric conditions by implementing numerical computations with Plaxis software. The adopted parameters for column material described by the Mohr-Coulomb model are given in Table 7.

Table 7 Parameters of the Mohr-Coulomb model for column material

γ_c [kN/m^3]	e	E' [kPa]	G [kPa]	ν_c	E_{oed} [kPa]	φ_c [°]	$K_x = k_y$ [m/day]
18	0.5	20,000	7519	0.33	29,630	38.0	10

The variation of consolidation settlement, versus time, at the surface of unreinforced soil is sketched in Figure 17 that shows the primary consolidation is expected to end within 12.5 years. The maximum settlement at the centreline of tank equals 27 cm that exceeds the predicted settlement in plane strain condition. The same remark holds for the predicted settlements at border of tank that are 8.3 cm and 7 cm in axisymmetric and plane strain conditions respectively. However, when adopting the consolidation procedure it was found that the end of primary consolidation settlement remains unchanged (12.5 years) both for the plane strain and axisymmetric analyses.

The variation of consolidation settlement at the surface of reinforced soil by sand columns (axisymmetric study) is plotted in Figure 18. From this figure it is noticed that the settlement at the centreline of tank of 9.4 cm is greater than the predicted value of 7 cm in plane strain condition (Figure 15). The same remark holds for the predicted settlement at the border of oil tank: 4.5 and 3.8 cm in axisymmetric and plane strain conditions respectively.

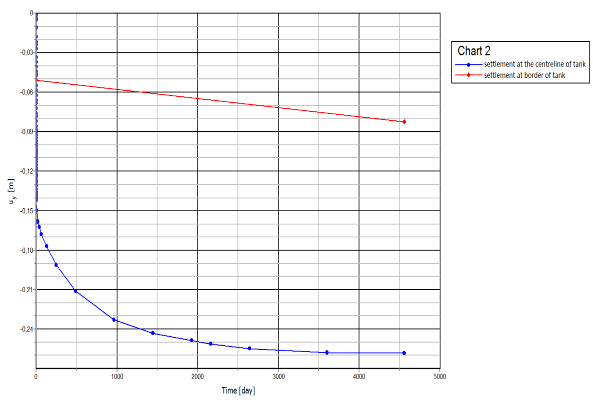

Figure 17 Variation of consolidation settlement of unreinforced soil (axisymmetric condition)

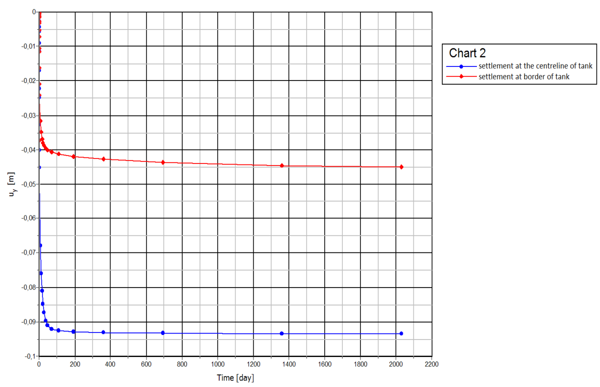

Figure 18 Variation of consolidation settlement at the surface of reinforced soil

Table 8 summarizes the predicted consolidation settlements at the centrelines of the La Goulette oil tank both in plane strain and axisymmetric studies performed by the Plaxis V2.9 and 3D model generated by the FLAC 3D code. It is noticed that the predicted settlements of unreinforced soil are quite similar by the Plaxis plane strain modelling and the FLAC3D model. In turn, the Plaxis axisymmetric study leads to quite overestimated prediction of consolidation settlement of unreinforced soil. In case of reinforced soil it is found that the plane strain condition slightly underestimates the settlement prediction compared to that obtained by Plaxis software in axisymmetric condition and the one obtained by FLAC3D model. It is, then, concluded that comparable results of the behaviour of reinforced soil are obtained by using any computational tool (Plaxis or FLAC) dependent-less of the adopted modelling either 2D (plane strain and axisymmetric conditions). Those findings are explained with more details by (Bouassida et al, 2015).

Table 8 Settlement predictions at centreline of the La Goulette oil tank by Plaxis V2.9 and FLAC 3D codes

Numerical tool	Option computation	Unreinforced soil (cm)	Reinforced soil (cm)
Plaxis V9.2D	Plane strain	21.8	7.0
Plaxis V9.2D	Axisymmetric	24.8	9.4
FLAC 3D	3D model	20.4	9.88

4.3 Simulation of loading test on single pile: Case history Radès La Goulette's bridge

This case study is investigated in view of comparison between predicted behaviour of TSC modelled the HSM and the soft soil model (SSM) previously considered by Tounekti et al (2008) for the study of behaviour of engineering structures founded on TSC.

According to the geotechnical report of the Radès La Goulette bridge project (Nippon-Koei, 1991) the soil profile and geotechnical parameters considered for the soil-pile modelling are summarized in Table 9. Detailed description of the soil profile is given in Kanoun & Bouassida (2008). Parameters of hardening soil model listed in Table 9 have been determined, following the method of determination detailed in Plaxis user's guide, from the results of geotechnical investigation presented in the report by Groupement Nippon-Koei et al, (2001).

From this Table it is noted that soft clay and clay layers are modelled the HSM whilst other soil layers are modelled by the Mohr-Coulomb constitutive law. Further, for consolidation analysis, isotropic permeability is assumed: $k_x = k_y = 8.64E\text{-}4$ m/s. The coefficient of at rest pressure $K_{0NC} = 0.58$ and Poisson's ratio is $\nu_{ur} = 0.15$.

Table 10 presents the adopted parameters of SSM of three soft clay and clay layers for numerical computations in view of comparison with predictions of pile response. Those parameters were adopted from the geotechnical report by "Groupement Nippon Koeï et al (2001).

Table 9 Geotechnical parameters of HSM for soft clays layers and Mohr Coulomb model for sandy layer

Soil	Model	Type	ν	E (MPa)	c' (kPa)	φ'	E_{50}^{ref} [MPa]	E_{oed}^{ref} [MPa]
Soft clay I	HSM	Undrained	0.49	-	0.1	20	65.2	9.0
Sand II a	MC	Drained	0.30	15.3	0.1	30	-	-
Sand II b	MC	Drained	0.30	7.7	5	20	-	-
Clay III	HSM	Undrained	0.49	-	0.1	25	60.9	8.1
Sand IVa	MC	Drained	0.30	65.5	0,1	32	-	-
Sand IVb	MC	Drained	0.30	20	10	26	-	-
Clay V	HSM	Undrained	0.49	-	0,1	25	65.2	9.0
Concrete	LE	-	-	1 E4	-	-	-	-

Table 10 Geotechnical properties of soft clay and clay layers described by the soft soil model

Soil	γ (kN/m³)	c' (kPa)	φ'	$Cc/(1+e_0)$	λ*	κ*
Soft clay I	16.5	0.1	20	0.15	65.2	9.0
Clay III	19.3	0.1	25	0.14	60.9	8.1
Clay V	19.1	0.1	25	0.15	65.2	9.0

The loaded pile of diameter 1 m is made up of reinforced concrete. It is assumed the installation of this pile does not affect the geotechnical properties of the seven penetrated layers. To assess the predicted allowable pile capacity a monitored full scale loading test was performed as modelled in axisymmetric condition by Figure 19.

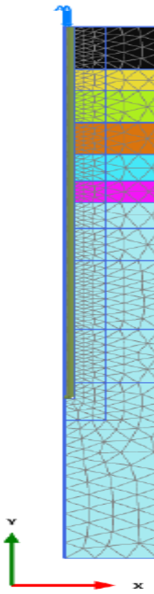

Figure 19 Generated mesh by Plaxis software of axisymmetric soil–pile model.

Interface element was used for the modeling of contact between Tunis soft clay and pile. The interface element has been characterised by coefficient of friction equals 0.67 to carry out coupled analysis.

Figure 20 shows the results of the load-settlement curve from measurements recorded during the loading pile test and the numerical predictions obtained by Plaxis software both for the hardening soil and soft soil models adopted for the soft clay and clay layers respectively.

In Figure 20 the prediction by the soft soil model is almost identical to measurements of the loading test up to a service load of the order of 10.500 kN. In turn the predicted soil-pile behaviour using the HSM quite underestimates the settlement up to the service load. This finding leads to conclude that the SSM is more suitable than the HSM for the prediction of the service pile load. Meanwhile Figure 18 illustrates that beyond the service load of soil-pile model numerical predictions performed in axisymmetric condition by Plaxis software were not capable to simulate the recorded load-settlement curve up to the ultimate pile load.

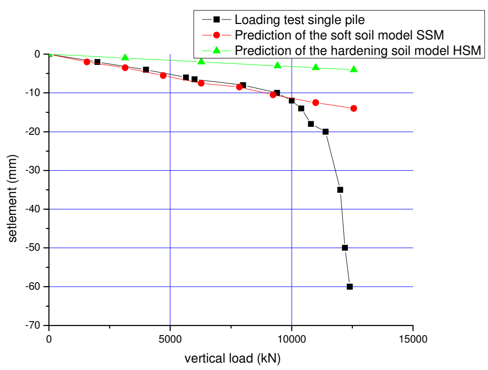

Figure 20 Load – settlement curve from full scale loading test on single pile of 1m diameter (SSM and HSM Plaxis 9.2) compared with that of the loading test of pile

5. CONCLUSION

This paper discussed the behaviour of Tunis soft clay as observed during an experimental investigation carried out on undisturbed specimens. Then, the simulation of performed oedometer and CU triaxial tests, using Plaxis 2D software has been considered. Two constitutive behaviour laws were tested to model the Tunis Soft Clay: the hardening soil and soft soil models (HSM and SSM). Comparisons between numerical results from the simulated oedometer and triaxial tests favoured the adoption of the HSM for TSC in order to predict the behaviour of structures founded on typical soil profile comprising the soft clay layer.

An oil tank example served to highlight the benefit of sand columns reinforcement in both reducing and accelerating the consolidation settlement compared the predicted behaviour of unreinforced TSC. Note that FLAC3D predictions have confirmed the settlement predictions both of unreinforced soil and reinforced foundation by sand columns given by the 2D Plaxis model.

The simulation of full scale pile load test showed up that the SSM reveals much more realistic than the HSM to describe the load-settlement TSC. Indeed the SSM enabled to accurately simulate the observed behaviour of loaded pile in the range of serviceability load.

When predicting the behavior of engineering structures the induced stress paths within soft clay are different from those induced during laboratory tests. This leads to conclude: although the prediction of observed behaviour by certain model (i.e. HSM) is meaningful the prediction of engineering structures founded on such soil is not automatically the best when compared to that predicted by another similar constitutive law i.e. SSM. As such the predictions by each constitutive law shall need a parametric study to show its limitations when performing numerical simulations.

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