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Ground Improvement - Selected Tunisian Case Histories

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Abstract: This paper presents a detailed analysis of three Tunisian ground improvement case histories. The first case addresses the cause of disorders that seriously affected the stability of an oil tank, initially built on a superficial soft clay layer improved by sand piles. Due to the underestimated length of sand piles, the oil tank operations stopped after non-admissible consolidation differential settlement. Retrofit solution using micropiles' reinforcement revealed quite satisfactory to restart the functioning of the oil tank after fifteen years. The second case deals with the reinforcement of compressible silt sand layer by floating stone columns to reduce the long-term differential settlement of a gas storage facility. Recorded measurements during the follow-up of stage construction of the storage facility permitted the assessment of numerical predictions of the settlement of reinforced soil. The third case studies the stability of access ramps of an interchange in Tunis Centre. Built numerical plane strain modelling helped for the prediction of the behaviour of embankment access on improved Tunis soft soil by geodrains. Based on recorded settlements and horizontal displacements, followed the validation of computed consolidation settlement. Adoption of suitable parameters of two constitutive models of the behaviour of Tunis soft clay is discussed.

1. Introduction

The stability of heavy loaded structures, e.g. high embankments, and sensitive infrastructures to differential settlement, for instance gas storage facilities, built on very thick soft deposits is challenging. In addition to the verification of bearing capacity, in short-term condition, embankments construction on saturated clayey soils often requires a special care because of the evolution of long-term (consolidation) settlement in time. When rigid inclusions or pile foundations are not intended for such projects, column-reinforced foundation can be adopted, under the form of floating inclusions in case the stratum layer is very deep.

Stone columns, sand-compaction piles and deep soil mixing are among the most popular techniques enabling the increase in bearing capacity, the settlement reduction and the acceleration of consolidation (Bergado et al, 1996). Mitigation of liquefaction is also another benefit, targeted by using vibro-compaction and stone column techniques (Han, 2015), (Bouassida, 2016). The installation of Rammed Aggregate Pier® (RAP) columns is an alternative mitigation technique that can increase the soil resistance, accounting for its lateral stress increase and for the stiffness increase from soil and RAP composite response (Amoroso et al, 2020).

Reinforcement by floating stone columns or by floating cement soil columns gained more interest as experienced

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in soft clays, Kitazume (2005), Chai & Carter (2011), Safuan et al (2017), etc.

Nowadays, one can note numerous improvement techniques to reinforce soft-highly compressible deposits. In particular, geodrains and vacuum consolidation revealed much more efficient than preloading to accelerate the consolidation settlement of several infrastructure projects (Indraratna et al, 2015; Jebali et al, 2017; Priyanka & Arindam, 2018, etc.).

Further, geogrids reinforcement constitutes a potential technique to improve the bearing capacity of shallow foundations.

Geocells also represents a reliable technique of soil reinforcement; it is recommended that the cells size of geocells should be selected smaller than 0.67 times the width of footing: Gholamhosein et al, 2019 .

Micropiles are equally efficient in many retrofit applications to increase the bearing capacity of shallow and to reduce their settlement: Bouassida et al, 2003.

In the geotechnical Tunisian context, to date, only few ground improvement techniques are of current practice; whilst other improvement techniques, worldwide implemented, yet remain unknown, the vacuum consolidation is an example (Jebali et al, 2017). Bouassida & Hazzar (2008) pointed out the lack of experience of reinforcement by stone columns in Tunisia in comparison to vertical drains. However, regarding research activities several contributions were recognized (Guetif et al, 2007; Bouassida & Hazzar, 2012; Frikha et al, 2015, Jebali et al, 2017, etc).

Main objective of the present paper is to report about some practiced ground improvement techniques in Tunisia with focus on the geotechnical context of Tunis City characterized by the presence of thick soft soil deposits extending up to 50-60 m depth, especially in the area of North and South Tunis Lakes.

In this view, analysis of three selected Tunisian ground improvement case histories aims to capture the learned lessons (for different infrastructures and geotechnical conditions) about either unsafe design, unsuitable foundation solution, or, conversely, successful execution of some ground improvement technique. Hence, due to the lack published papers, the synthesis of the three selected case histories will provide the best highlight to promote much better the practice of ground improvement techniques in Tunisia in forthcoming projects.

2. Insufficient sand piles improvement of Tunis soft soil

2.1. Project overview

Early in the nineties, the National Petroleum Company built an oil cylindrical steel tank of 33 m diameter in the oil products storage area located at Rades suburb of Tunis City. The working vertical load of the tank is equal to 100 kPa.

Geotechnical parameters of soil layers were adopted from an existing geotechnical survey previously carried out for an existing similar oil-tank nearby the studied tank. Table 1 schematizes the soil profile comprising six layers including five compressible silt sand to highly compressible soft clays (Bouassida et al, 2019). Note that all layers of the soil profile in Figure 1 are saturated; the soft silt-clay layer extending from 6 m to 18 m depth is subdivided into three sub-layers. Table 1 summarizes adopted geotechnical parameters of the soil profile under the oil tank with notice that excepting sand layers N° 4 and 5, cohesion and friction angle values in all remaining layers correspond to the short-term shear strength (i.e. undrained parameters). Whilst, cohesion and friction angle values of sand layers correspond to effective (drained) shear strength parameters.

Tank execution was preceded by the improvement using sand columns of 0.6 m diameter over the first six meters depth, i.e. the highly compressible sandy silt layer. This technique is well-known in Tunisia as “sand piles” which execution starts by the penetration of a metallic casing inducing a lateral expansion of the soft soil. Then, the casing is filled by sand without being compacted; a step-by-step withdrawl of the casing along with added sand forms the sand pile that is mainly viewed for vertical drainage rather than a reinforcing inclusion (Bouassida & Klai, 2012).

Table 1: Geotechnical parameters of soil layers

Layer n°	Thickness (m)	Cohesion C (kPa)	Young modulus E (MPa)	Poisson’s ratio ν(-)	Friction angle φ (°)	Total unit weight, γ (kN/m3)
1	6.0	10	2.5	0.33	0	17.5
2a	3.0	15	2.0	0.45	0	17.0
2b	6.0	30	3.0	0.40	10	18.0
2c	3.0	35	5.0	0.35	12	18.5
3	5.0	40	7.0	0.30	13	19.0
4	5.0	0	15.0	0.25	37	18.5
5	7.0	5	10.0	0.33	32	19.0
6	10.0	50	9.0	0.30	15	20.0

Improvement of the tank foundation comprised the installation of 481 sand piles of length 6m in a non-regular pattern with an improvement area ratio equals 11%. This solution essentially aimed at accelerating the consolidation settlement in the upper compressible silt-sandy layer, rather than reinforcement of this crossed layer followed by an increase in soil bearing capacity (Bouassida, 2016). Hence, the sand piles overall played the role of vertical drains by accelerating the settlement of improved compressible upper layer, as a consequence of induced upward water drainge due to the dissipation of excess pore pressure. The overlaying blanket layer, first, contributed to the evacuation of drained water at the ground surface. Second role of this blanket layer, of thickness 1.2 m, was to render the settlement of tank uniform and, therefore, to minimize the risk of differential settlements due to unequal excess vertical stresses generated by the tank load at the ground surface.

Meanwhile, it is obvious that the executed sand piles improvement could not accelerate the induced long-term settlement extended at least up to 25 m depth where the tank of 33 m diameter induced non-negligible excess of vertical stress.

Using Terzaghi's method via Equation (1), the calculation of consolidation settlement (i.e. the oedometer method) of unreinforced compressible layers, all assumed normally consolidated, up to 23 m depth is detailed in Table 2.

$$s = H \frac{C_c}{1+e_0} log\left(1 + \frac{\Delta\sigma'}{\sigma'_{v0}}\right) \quad (1)$$

From the predicted long-term settlements in Table 2, it should be emphasized that the long-term settlement of layers 2a, 2b and 2c is approximately 60 cm, over a thickness of 12 m. After Bouassida & Hazzar (2008),

Table 2. Prediction of long-term settlement of unreinforced clay layers.

Layer n°	Thickness (m)	Compression index, Cc	Initial void ratio, e_0	Effective vertical stress σ'_{v0} (kPa)	Excess of vertical stress, $\Delta\sigma$ (kPa)	Settlement (cm)
2a	3.0	0.600	1.00	55.5	95.0	25.8
2b	6.0	0.572	1.88	90.0	80.0	26.0
2c	3.0	0.365	1.33	126.8	65.0	8.5
3	5.0	0.145	0.67	162.0	52.5	5.3

s = consolidation settlement of soil layer of thickness H; c_c = compression index; e_0 = initial void ratio; $\Delta\sigma$ = excess of vertical stress due to the tank load and σ'_{v0} = effective overburden stress.

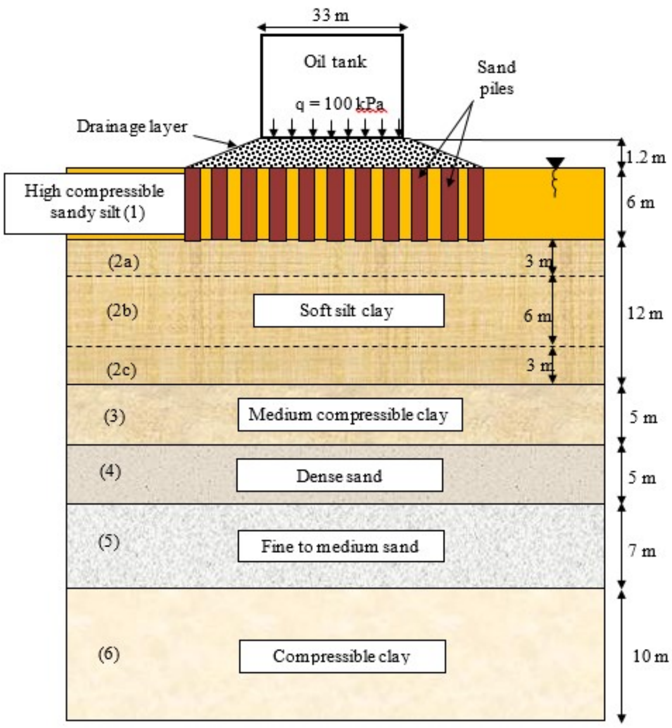

Fig 1. Adopted soil profile of oil tank foundation (Bouassida et al, 2019)

adopting the coefficient of vertical consolidation of 10-8 m^2/s for clay layers 2a, 2b and 2c the estimated degree of consolidation, after twenty years, is about 35%. Therefore, the consolidation settlement of the unimproved layers is still in progress under the tank load. Thus, the improvement by sand piles, of 6m length, only reduced and accelerated the settlement of the upper high compressible sandy silt layer.

Few years after the commencement of tank operations, on-site observations showed that its cylindrical shell suffered severe buckling deformation. After fifteen years, the visually-observed settlement due to the primary consolidation of compressible layers attained 20 cm. This was followed by the decision in stopping the tank operations. Retrofit solution should be foreseen for repairing the oil tank. Two retrofit solutions were suggested.

First proposal is a reinforcement by micropiles (MP) of length reaching the top side of the dense sand layer, located between 23 and 28 m depth (Figure 1). Second retrofit option suggested the reinforcement by inclined rigid inclusions (IRI) embedded in the dense sand layer.

2.2. Reinforcement using micropiles

The dismounting of the entire tank's shell is required for the installation, in a concentric polygonal mesh, of seven micropiles, (**Figure 2**). Sixty four (64) micropiles, of 30 cm diameter and 25 m length, only reacting by the shaft resistance were installed. To ensure a uniform distribution of loads throughout the tank area, the micropiles' heads are embedded in concentric reinforced concrete beams. Due to the decreased induced vertical stress in horizontal distance (from the centre to the tank border) of applied tank pressure, equal number of micropiles are located on the polygonal perimeter. Although the reinforcement by micropiles is a non-cost effective retrofit solution, it warrants the long-term tank stability without risk of non-admissible residual settlement.

2.3. Reinforcement by inclined rigid inclusions (IRI)

The installation of inclined rigid inclusions (IRI) embedded within the sand layer at 23 m depth can be designed to avoid the entire dismounting of the tank and to proceed for repairing only the affected areas by buckling.The skin resistance generated between the soil and the IRI balances a given proportion of the total weight of the oil tank structure. Therefore, one can estimate the total allowable skin resistance developed by the IRI and, then, to deduce the required number of those inclusions to the adopted proportion of tank load. Consider this latter estimated as 67%, the remaining 33% of tank load will be balanced by the consolidated first layer which degree of consolidation increased by approximately 35% during twenty years of tank operations. Since the IRI should be covered at the top by a reinforced concrete raft, an enhanced load concentration is afforded; therefore, the number of IRI is lesser than that of the vertical micropiles.

Bouassida & Bouassida (2013) performed an axisymmetric numerical model, using Plaxis' 2D software, for studying

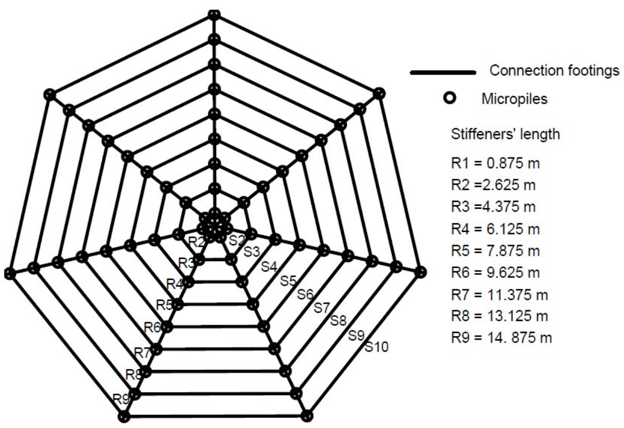

Fig 2. Layout of reinforcing micropiles embedded in reinforced concrete connection beams (Bouassida & Mejri, 2011).

the effect of installing the IRI embedded in the sand layer, on the evolution of consolidation settlement within the soft silt clay layers (2a, 2b and 2c, Figure 1). The reinforcement using inclined rigid inclusions is then less expensive than that of micropiles connected by reinforced concrete beams. The owner of project favoured the micropiles' reinforcement; however, it requires the dismounting of the entire shell of the oil tank, which also is time consuming.

3. LGP storage facility on reinforced soil by floating stone columns

3.1. Project overview

The second case history addresses a storage facility, located at Ghannouche (South East of Tunisia), comprises two bullets of butane and five bullets of propane protected in mounded banks. **Figure 3** schematizes the cross section of the completely integrated embankment. Geotechnical properties of soil layers are obtained from measured CPT values during the soil investigation and laboratory tests results conducted for the project. **Table 3** summarizes the properties of crossed soil layers as identified from oedometer tests and tip resistance estimated from CPT results.

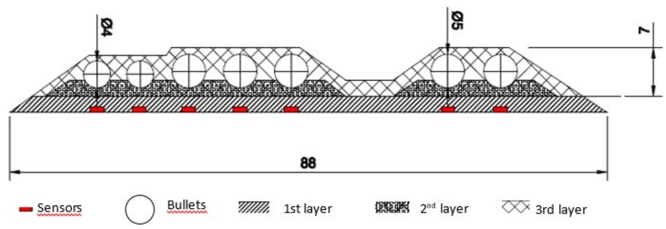

Fig 3. Cross section of completely integrated embankment (dimensions in meter*)*

Table 3. Geotechnical parameters from soil investigation

Layer	Thickness (m)	Tip resistance, CPT (MPa)	Total unit weight, γ (kN/m3)	Initial void ratio, e0	Compression index
Fine sand	2.5	12.0	--	--	--
Medium sand with mud & clay inclusions	6.5	4.0	--	--	--
Clay and sand partially cemented	5.5	30.0	19.0	0.65	0.100
Alternating bedding of fine sand and clay	7.5	30.0	17.0	1.26	0.395
Mostly hard clay and sand	--	--	19.0	0.65	0.100

Reinforcement by stone columns is suitable to reduce the unallowable settlement as predicted under the applied embankment load equals 120 kPa. The stability is required for an allowable residual settlement equals 4 cm, over 15 years in post construction of the storage facility (El Ghabi et al, 2010).

Hence, significant reduction of settlement associated to the prescribed margin of security has led to the installation of floating stone columns of 11 m length, embedded in medium sand layer. Stone columns of 0.9 m diameter were installed in a triangular pattern with an improvement area ratio = 16%.

3.2 Numerical behaviour of the LGP storage facility

Numerical simulation of the embankment behaviour was carried out by Plaxis 2D software. Built plane strain model comprised a 7 m embankment height with an upper crest width equals 56 m and lower base of width equals 88 m. Foundation of this embankment is described by the soil profile detailed in **Table 3**. After project data, 46 stone columns were installed along the horizontal direction, with an axis to axis spacing of 1.9 m, and 30 stone columns were installed, along the perpendicular direction, with an axis to axis spacing equals 2.2 m, over 64 m length.

Simulation of the behaviour of reinforced ground in plane strain condition, assumed the modelling of the group of stone columns by a group of equivalent trenches of equivalent thickness as calculated by El Ghabi et al, (2010). Forty-one trenches of stone material are considered in the built numerical model with thickness = 0.3 m; length = 11 m and a spacing between edges of trenches = 1.9 m. Elastic perfect plastic Mohr-Coulomb constitutive model describes the behaviour of soil layers.

Table 4 presents the adopted geotechnical parameters of soil layers, embankment material and reinforcing stone material. The numerical simulation of embankment behaviour with staged construction comprised four phases (Bouassida, 2016).

Figure 4 shows the contours of vertical displacement with maximum value equals 8 cm at the upper crest of embankment facility. Whilst, at the surface of reinforced soil, predicted quasi-uniform settlement equals 6 cm over the width of upper crest beneath the embankment with an applied load of 120 kPa.

Table 4. Adopted soil parameters for numerical modelling

	Thickness (m)	E (MPa)	C (kPa)	φ (°)	γsat(kN/m3)
Backfill material	6.0	10.0	1	30	20.0
Fine sand	2.5	30.0	5	30	19.0
Soft silty clay	6.5	5.7	2	24	18.0
Firm clay	5.5	60.0	2	24	19.0
Silty clay	7.5	12.0	15	10	18.0
Stiff clay	6.0	80.0	2	24	20.0
Stone material	11.0	60.0	0	40	20.0

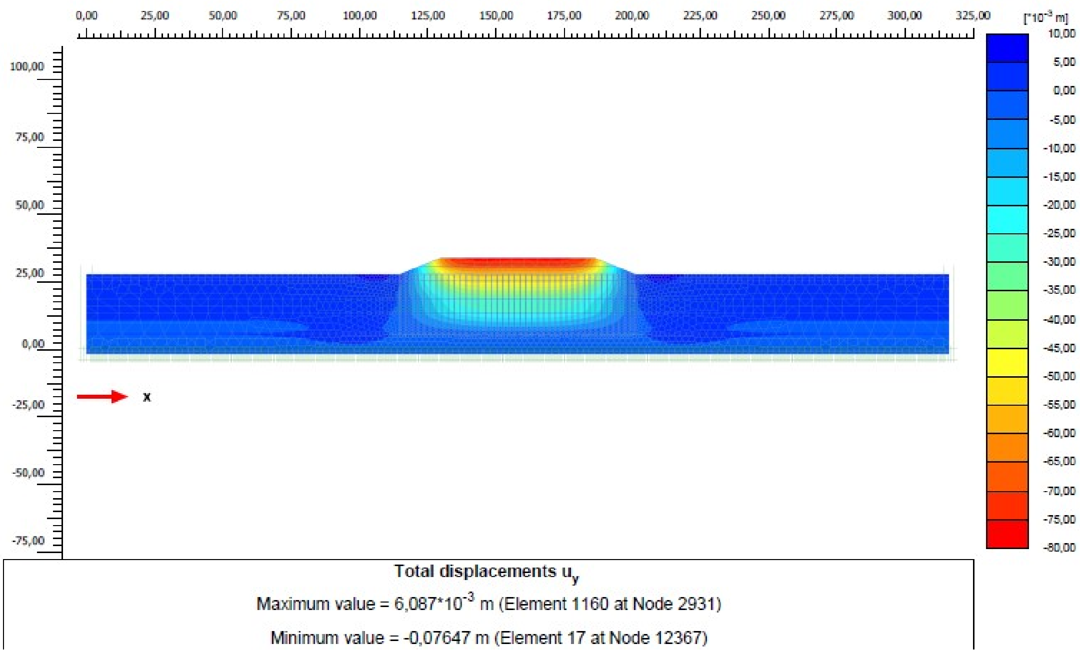

Fig 4. Contours of vertical displacement of embankment on reinforced soil by floating stone columns (Bouassida, 2016).

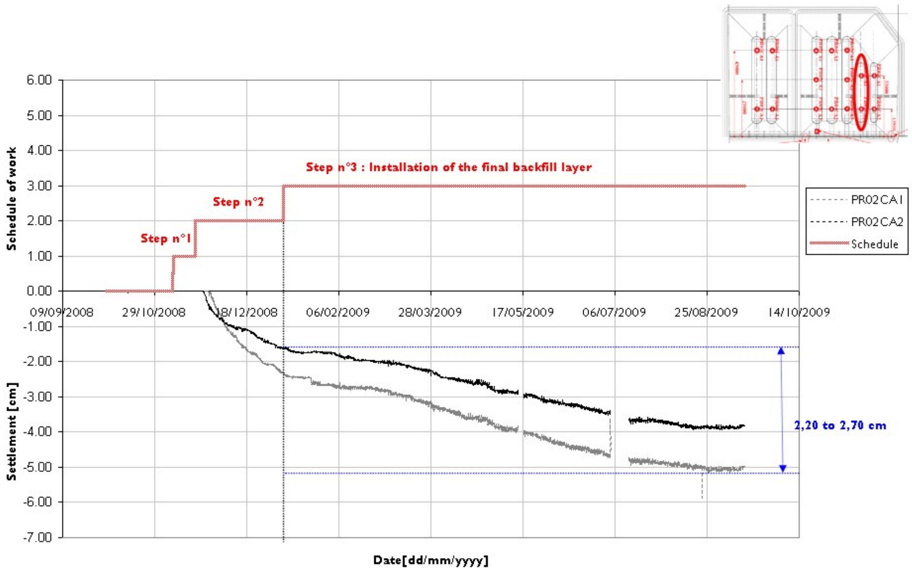

Fig 5. Evolution of recorded settlement vs. time at location PR02 (Bouassida, 2016)

Predicted consolidation settlement equals 6.5 cm should occur in four years. Follow up of the behaviour of storage facility built on reinforced ground by floating stone columns was performed by means of data acquisition unit that connects the pressure sensors, to record the evolution of settlements, located at the surface of reinforced ground (Bouassida, 2016).

An axisymetric numerical simulation with a composite cell model, in oedometer conditions, was also conducted (Ellouze et al, 2017). The settlement of reinforced soil is 4 cm for a long term analysis of fifteen years after the end of installation of backfill layer.

Figure 5 displays the evolution of recorded settlement in function of time at profile PR02. Recorded settlement, after the installation of the first backfill layer, varied between 1.0 and 1.7 cm and, then, became stabilized after a month. Upon the completion of the embankment construction, the magnitude of measured settlements was lower than 3.0 cm. Based on this, the floating stone columns reinforcement experienced at Gannouche's site fulfilled the requirement of an admissible consolidation settlement lower than 4 cm.

It is, therefore, concluded that plane strain predictions are in a better match with recorded in-situ settlements. This result sounds obvious because the axisymmetric model reduced the reinforced soil to a unit cell in oedometer conditions.

3.3 Observed behaviour of the storage facility

Detailed description of the acquisition unit comprising the installation of settlement gauges and recorded settlements was reported by El Ghabi et al (2010). Measured settlements induced by the acquisition unit data occurred since the first step of embankment construction, and, then, eight months after the edification of final backfill layer, the settlement evolution became stabilized.

The third phase of numerical staged construction included a consolidation analysis corresponding to the final height of embankment. This phase simulates the long-term behaviour after fifteen years of the construction of storage facility. Along with the progress of embankment stage construction, the settlement significantly increases in different locations and, then, it becomes almost onstant within the allowable limit of settlement that is 4 cm.This long-term settlement corresponds to the induced deformation within unreinforced sub-layers (Bouassida & Hazzar, 2015). From Figure 5, the recorded settlement evolution shows the benefit of stone columns in accelerating the consolidation of soft silty clay layer illustrated by a stabilized settlement eight months in post edification of the final backfill layer.

The study of the second case history well demonstrated the usefulness of floating stone columns reinforcement as no residual consolidation settlement, occurred in the unreinforced sub-layers. Hence, one concludes that the design of foundation of bullets of butane and propane integrated into an embankment on compressible layers reinforced by floating stone columns was successful. Indeed, this design permitted to comply with the allowable settlement of the foundation over fifteen years as predicted by the numerical computations. Those predictions revealed in acceptable agreement with the measured settlement that remained under 4 cm over 15 years.

4. Access ramp Foundation on improved Tunis soft clay by geodrains

4.1 Project overview

The studied area is the Republic Avenue that extends over a distance of two kilometres. This Avenue connects between the North and the South sides of Tunis City by the highway A1. After collected data from several geotechnical investigations, Mezni & Bouassida (2019a) conducted the characterization of the soil profile. Suggested correlation for soil parameters were adopted to simulate the behaviour of access ramps of interchange “Cyrus Le Grand” located at the Republic Avenue of Tunis City. Validation of predicted settlements aimed to assess, in particular, the adopted constitutive model and related Tunis soft clay parameters.

4.2 Geotechnical profile

Figure 6a shows the geotechnical profile under Cyrus Le Grand interchange that comprises, from the ground surface, a fill layer (N° 3) of 4.0 m thickness followed by a soft greyish clay layer (N° 4) of 11m thickness. Then, a black clay layer (N° 5) of 4m thickness and sandy clay layers (N° 6) of 40m thickness up to the top level of rigid stratum layer located at 60 m depth (Mezni & Bouassida, 2019b). This soil profile is overlaid by by a blanket layer of 0.5 m thickness on which the preload emabankment of 3.1 m height is built.

Foundation of the piers and abutments of the main bridge of this interchange comprises a group of piles, embedded in the stratum layer. In turn, the access ramps of approach embankments were built on an improved soft soil by geodrains to accelerate the consolidation settlement of Tunis soft clay. Mebradrain geodrains of 18 m length, installed in square mesh with an axis-to-axis spacing of 1.1m, permitted to accelerate the consolidation of high compressible soft soil upper layers with a compression index equals 0.38 to 0.42. Construction of preloading embankment of 3.1 m total height comprised two phases.

The first phase simulates the preloading during 24 days to reach an embankment height equals 2m. The second phase refers to a preloading scheduled for 95 days to reach a total embankment of height equals 3.1 m (Mezni & Bouassida, 2019b).

A drainage sand mattress of 0.5 m thickness preceded the embankment construction at the surface of improved soil to facilitate the water evacuation from the geodrains (**Figure 6a**). This upper sand layer can also contribute in a better load transfer avoiding differential settlement. **Table** 5 presents the characterisation of each soil layer, including the preload embankment, sketched in **Figure 6a**.

The stability of the preload embankment shown in Figure 6a requires, first, verification of the admissible bearing capacity and, second, the settlement. Using soil parameters given in Table 5, by assuming the embankment as a strip footing with zero embedment, from Terzaghi’s bearing capacity equation it is easy to check that uniform embankment load equals 62 kPa is admissible. Meanwhile, the prediction from Eq (1) of the consolidation settlement at the axis embankment leads to a non-admissible value nearly equals 0,5 m. 90% of this settlement is expected to develop in 300 years! Therefore, it was concluded to accelerate the long-term settlement by using the technique of geodrains, of length 18 m, associated with a preload embankment.

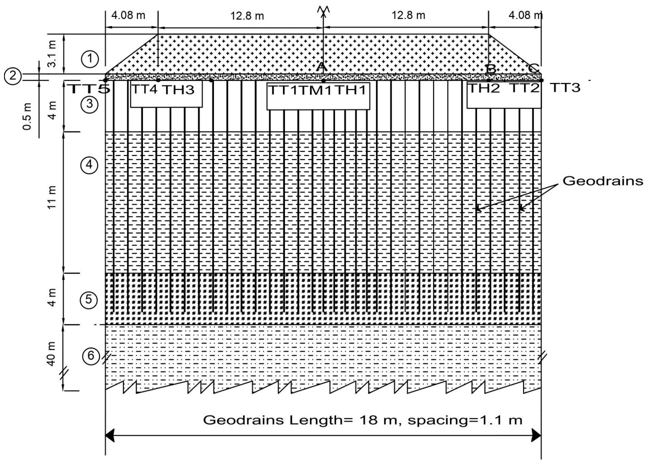

Fig 6. Installed settlement recorders under access embankment

Table 5. Geotechnical parameters of soil layers and embankment material

Soil layer [n°]	Thickness (m)	Young Modulus [MPa]	Cohesion [kPa]	Friction angle [°]	Total unit weight [kN/ m3]
Embankment [1]	3.1	10.0	1	30	20.0
Drainage blanket [2]	0.5	30.0	5	30	19.0
Fill [3]	4.0	5.7	2	24	18.0
Soft greyish clay [4]	11.0	6.0	2	24	19.0
Black clay [5]	4.0	12	15	10	18.0
Sandy clays [6]	40.0	8	2	24	20.0

4.3 Follow up of accelerated consolidation settlement

Three types of settlement gauges were installed under the access ramp (Mezni & Bouassida, 2019b). **Table 6** summarizes the location of all installed settlement recorders as follow up instruments under the access embankment.

The settlement recorders were installed at 5 m in front of the abutment of the main bridge and at 10m behind it. In the front of this abutment, rod settlements TT1, TT2 and TT3, TT4 and TT5, were installed at the embankment axis, on the right side and on the left side, respectively. Behind the abutment C1, rod settlement TT6 was installed at the embankment axis, TT7 and TT8 on the right side, TT9 and TT10 on its left side.

Hydraulic settlement TH1 and multipoint settlement TM1 were installed at the embankment axis; TH2 and TH3 hydraulic settlements were installed on the right and on the left side, respectively. **Figure 6** shows the locations of some settlement gauges along a vertical cross section of the access embankment, i.e. perpendicular to the traffic direction..

Follow-up of the settlement took a period of three months. Unfortunately, after this short period there were no recorded settlements. **Figures 7, 8 and 9** show the evolution of recorded settlements, at the axis and the two extremities of instrumented preload embankment. At the axis of embankment, the recorded settlements are higher than those measured at the embankment crest and toe (**Figures 7 and 8**).

Further, Figure 7 shows that recorded settlement variations by TT2 and TT4 are quite similar. As for TT3 and TT5 rod settlement recorders quasi-identical settlement evolution is noted. Such a behavior is attributed to the symmetrical locations of those settlement recorders with respect to the longitudinal axis of preload embankment.

Table 6. Locations of the installed settlement recorders under access embankment

Type *Location*	Rod settlement	Hydraulic settlement	Multipoints settlement
Axis	TT1, TT6	TH1	TM1, TM5
Crest	TT2, TT7, TT4, TT9	TH2, TH3	--
Toe	TT3, TT8, TT5, TT10	--	--

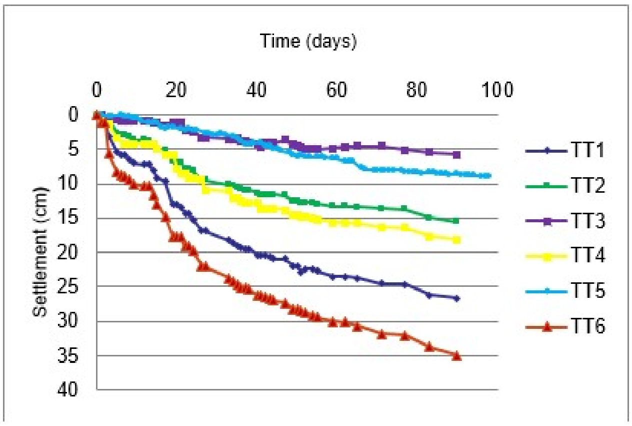

Fig 7. Recorded settlement under access embankment

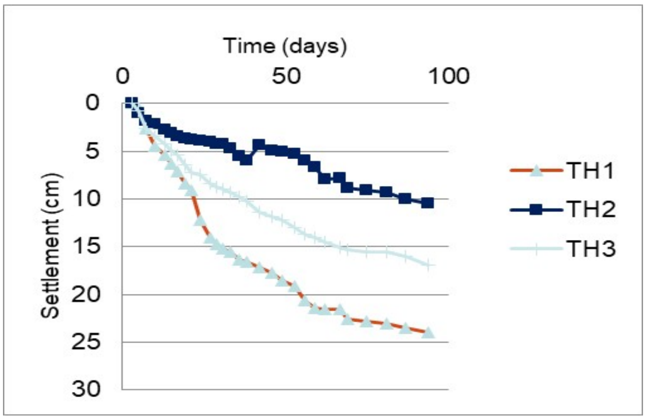

Fig 8: Evolution of recorded settlement by hydraulic settlement gauges

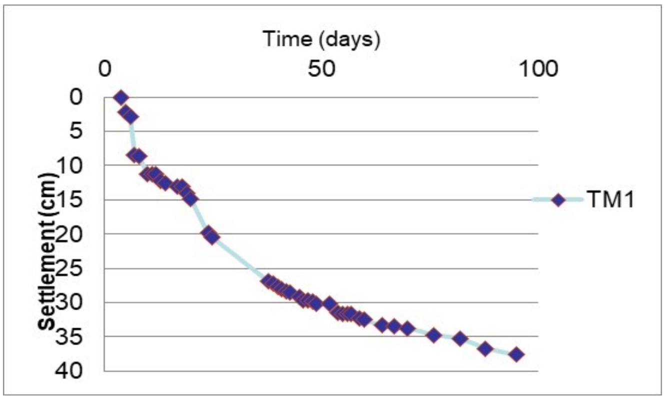

Fig 9. Observed settlement at the axis, left and right sides of embankment

The multipoint settlement (TM), along the embankment axis, recorded a value of 37.6 cm after 95 days (Figure 10). However, by the hydraulic settlement the recorded value was limited to 24 cm after 94 days. Consideration of recorded values by the multipoint settlement gauge calls for caution. Indeed, on-site observation indicated that the vertical displacement of the probe of the multipoint settlement was prevented due to the lateral deformation of the tube (MEHAT, 2007). Recorded values by the rod settlement were in-between 26.7 cm and 35 cm after 90 days over a distance of 15m between TT1 and TT6 recorders. The rod settlement recorded values were in between the provided measurements by the hydraulic settlement and the multipoint settlement.

Mezni & Bouassida (2019b) carried out a numerical simulation, using Plaxis software, to predict the evolution of settlement under the ramp of access embankment. Built plane strain model served for the validation of the predicted behaviour of the ramp of access embankment when compared to the observed settlement evolution. The soft soil model (SSM) was considered to describe the behaviour of compressible layers. **Table 7** summarizes the geotechnical parameters of the soft soil model, including the compression index C_c, the swelling index C_s, initial void ratio, isotropic permeability and long-term shear strength characteristics, adopted for the soft clay and greyish clay layers. Detailed description of the geotechnical parameters and method of determination can be found in Klai et al, 2015.

Numerical predictions of settlements under the ramp access embankment in different locations led to quasi similar by the SSM. It was, then, concluded, for the studied case history, the SSM revealed suitable to describe the behaviour of Tunis soft clay (Mezni & Bouassida, 2019b). Indeed, the comparison between predicted settlements by the SSM and the recorded values during the follow-up of embankment were in acceptable agreement in particular when consider the recorded settlement values by the multipoint settlement (**Figure 10**). It was also checked that the installation of geodrains provided a good acceleration of consolidation settlement in comparison to the predicted settlement of unimproved soil.

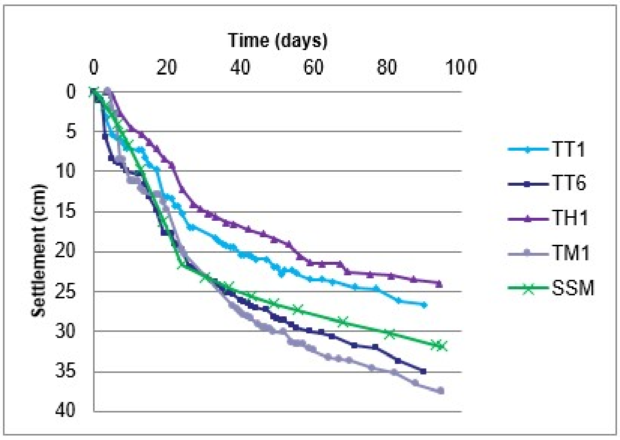

Fig 10. Evolution of predicted settlements by the SSM and recorded data

Table 7. Geotechnical parameters of the softening soil model (SSM) adopted for Tunis soft clay

Parameters	Soft greyish clay	Black clay	Soft greyish clay	Black clay
γ_h (kN/m^3)	17.0	18.0	17.0	18.0
γ_{sat} (kN/m^3)	19.0	20.0	19.0	20.0
$k_x = k_y$ (m/day)	1.5 10^{-4}	1.0 10^{-4}	1.5 10^{-4}	1.0 10^{-4}
C_c	0.420	0.380	0.420	0.380
C_s	0.056	0.057	0.056	0.057
e_0	1.20	1.04	1.20	1.04
C'	-	-	6	8
φ'(°)	-	-	20	21
ψ(°)	-	-	0	0

Ψ = angle of dilatancy

Meanwhile, from Figures 7, 9 and 10, the maximum recorded accelerated settlement approximates 35 cm. Adopting this latter and compared to the total estimated consolidation settlement of 50 cm, one deduces that a global degree of consolidation of 70% is achieved in three months. Then, one concludes that the completed consolidation by the geodrains is expected to end from six to nine months after the edification of preload embankment.

5. Conclusion

This paper addressed three Tunisian ground improvement case histories. Each case history is associated to a specific ground improvement technique which design complies with the long-term stability of an oil tank, embankment storage facility and access embankment for interchange project, respectively. From those case histories, it follows the learned lessons and recommendations hereafter summarized.

After the studied first case history, Insufficient design of improvement using sand piles, for the oil tank project, resulted from the lack of data to consider from a specific geotechnical survey and an unsuitable design of improvement characterized by short sand piles of length 6m. This improvement technique revealed unsuccessful due to non-admissible consolidation settlement that affected the serviceability of the oil tank, as ceased after 15 years. Hence, successful retrofit technique reinforcement using micropiles of length 25 m was necessary to transfer the load tank to deeper soil layers thereby by passing the consolidating layers.

Second case history discussed the reinforcement by floating stone columns of compressible layers at Ghannouche site. Stage construction of the storage facility, comprising two bullets of butane and five bullets of propane protected in mounded banks, was simulated by Plaxis software in four phases. Using an equivalent 2D modelling of reinforced ground by floating trenches of length 11 m, the prediction of behaviour of the storage facility showed that the prescribed residual settlement, occurring after the end of stage construction, did not exceed 3.5 cm as observed from recorded settlements during the follow up of storage facility. This prediction fulfilled the required value of residual settlement equals to 4 cm over fifteen years.

Third case history addressed the acceleration of consolidation of Tunis soft clay by geodrains for the embankment access of Cyrus Le Grand interchange. The efficacy of geodrains, in accelerating the consolidation of soft soil up to 18 m depth, is proven from the follow-up of settlement recorded by three types of settlement recorders. Those in-situ data permitted to assess numerical predictions of the settlement after implementation of a plane strain modelling, in which the soft soil model suitably describes the behaviour of Tunis soft clay. The two modelling led to comparable predictions of the accelerated settlement evolution.

Nomenclature

LGP Liquefied Petroleum Gas
SSM soft soil model

Disclosures

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