Case History

Geotechnical Aspects of Radès La Goulette Bridge Project (Tunisia)

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This paper briefly presents one of the very recent big Tunisian projects that is Radès La Goulette Bridge. This project is accompanied by the construction of the first cable stayed bridge of length 260 m across Tunis channel which connects Tunis City with its north suburb passing first by La Goulette County. The project groups four main lots involving the construction of bridges, highways and reclamation of lake areas. Focus is given on three items. The first one is the in situ and laboratory investigations led for soil characterization. The second item is the description of types of foundations and soil improvement techniques with specific challenges encountered and modern techniques performed during their execution. The third item is related to economical aspects of the geotechnical components which are commented and main conclusions are drawn.

1. Introduction

The owner of project is the Tunisian Ministry of Equipment, Housing and and land Regional Development. The project is managed by the Directorate-General of Highways Department. The project consists in a main bridge construction across Tunis channel for connection between North and South suburbs passing by La Goulette and Rades counties (Figure 1). The project of Rades La Goulette Bridge also includes structures and infrastructures of access toward Tunis City, and North and South suburbs. The project aims at the reduction of traffic intensity within Tunis City centre, keeping in mind that existing infrastructures by means Tunis La Goulette express route, Rades La Goulette ferry connection, the national road N°9 present serious limitations to keep a fluid circulation between La Goulette and Rades suburbs and Tunis City, [10]. The execution of Rades La Goulette bridge project comprises four separated lots: first is the main bridge, second is the south connection of 2.6 km length, third is the interchange in Tunis North Lake, and four is the north connection of 6.5 km length.

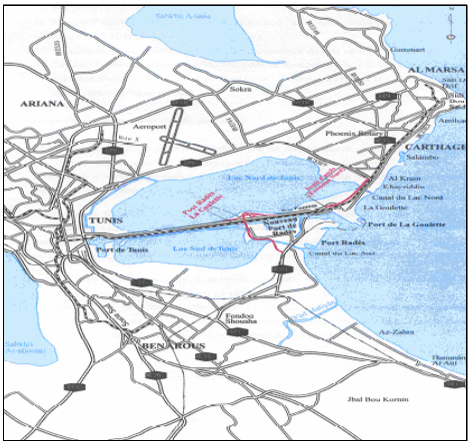

Fig.1. Location of the project "Radès-La Goulette" bridge (Red line)

Lot 1: The main bridge: its structure is characterized by a bridge of total length of 260 m, and a total breadth of 23.5 m. The released navigation gauge of the channel has 20 m in elevation and 70 m in breadth.
The bridge is composed of to ways of circulation each offers two line of 3.5 m, a 2m hard shoulder and a central full ground of 2.5m which shelters the two towers and cables. It is a cable stayed bridge an "extradossed" stay Cable Bridge composed by caissons elements made by pre stressed concrete in three continuous spans of 70m, 120m and 70m respective lengths. The pylons of 20 m in height are founded on 2m diameter deep piled foundation of 75 m length.

Lot 2: The South connection: it comprises four sub lots namely: Planning of rotary junction, the construction of two kilometres express route, two ways each with two lanes, the construction of a pre stressed concrete bridge of 180 m length on Rades channel and the construction of a pre stressed concrete bridge of approach (from south side) having 400m length.

Lot 3: The North connection: It comprises the reclamation of about 20 hectares in North Tunis Lake, the deviation of Tunis La Goulette express route along 2.4 kilometres, the construction of a pre stressed caisson bridge of approach (north side) of 720 m length and the construction of the interchange between the main bridge and the express route Tunis La Goulette. Then, these works needed the draining of North Lake for which environmental challenges should be faced, essentially the contamination of the lake's water by finer of reclaimed zone and the sedimentation of filled material which affects the quality of water. During the reclamation it was observed the formation and migration of mud waves followed by their propagation in the lake, [3]. This problem has been tackled by the installation of geotextile barrier aided by the counter weight flotation technique. Also a reduced intensity of traffic plan was adopted.

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Lot 4: It consists in the construction of 5.5 kilometres new express route two ways each with two lanes, a bridge with 15.5 m length across Khair Eddine channel and a planning of intersection.

In the following the present paper focuses, first, on the programme of soil investigations, second the solutions of foundations with specific in situ conditions, and finally comments in brief are given about the cost and execution of Rades La Goulette Bridge.

2. Soil Investigations

A consistent geotechnical investigation programme had been carried out in two campaigns on the site of the project both on-shore and off-shore locations. The first campaign included 12 boreholes 42m to 117m deep with undisturbed sampling in clay soil, SPT in sand layers, pressuremeter tests, laboratory tests, [8]. The second campaign included additional bore holes, SPT, pressuremeter tests and the conduction of piezocone tests and vane tests, [9]. The consistency of the geotechnical programs is given by table 1.

Table 1. Geotechnical investigations carried out for Rades La Goulette bridge project.

Investigation type	Depth (from surface ground)	Number
Bore holes	Between 0 m to 117 m	56
Undisturbed samples	Between 14 m to 105 m	161
Pressuremeter tests	Between 37 m to 115 m	40
Piezocone & CPT tests	Between 14 m to 37 m	20
Vane tests	Between 3 m to 10 m	20

The second geotechnical campaign was much more consistent, it included 1514 lineal meters of pressuremeter borings, 1290 lineal meters of coring, 300 lineal meters piezocone test and 156 lineal meters cone penetration, [9].

Tables 2 & 3 summarize the conducted on-shore and off-shore in situ tests and laboratory tests.

Table 2. In situ geotechnical investigation (depth in meters). CB = Cored borings;
PB = Pressuremeter boring; PZC = Piezocone tests; US = Undisturbed samples; VST = Vane shear test

Survey Lot number	On-shore	Off-shore
Lot 1	- 2 CB (115 m); 16 US & 10 SPT. - 2 PB (115 m) + 3 (26.1 m).	- 2 CB (115 m); 17 US & 3 SPT. - 2 PB (115 m) & 1 PZC (22.7 m)
Lot 2	- 7 CB (10 m) & 9 US - 8 CB (40 m): 24 US; 33 SPT & 9 PB (40 m). - 2 PZC (max 25 m). - 6 SPT (max 28 m). - 11 VST	- 1 CB (40 m): 4 US & 4 SPT + 3 PC (40 m).
Lot 3	- 1 CB (80 m): 6 US & 3 SPT) - 1 VST (6 m). - 3 PB (37 to 40 m) + 1 PZC (37.3 m).	- 9 CB (25 to 40 m); 30 US & 30 SPT - 5 VST (3 to 6 m). - 13 PB (35 to 40 m) - 5 PZC (max 27 m), with pore pressure dissipation.
Lot 4	- 2 CB (34 to 35 m) & 4 US. - 2 PZC (31.7 m). - 1 SPT (29.4 m).	

Table 3. Laboratory investigation, [6].

Lot 1	Lot 2	Lot 3
16 Grain size distribution 4 Triaxial CU+ u tests 3 Triaxial UU + u tests	24 Grain size distribution 11 Oedometer tests 4 Triaxial CU+ u tests 6 Direct shear tests 7 Proctor & CBR tests	33 Grain size distribution 19 Oedometer tests 7 Triaxial CU+ u tests 3 Creep tests

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The geotechnical soil conditions consist in a succession of silty clay and sand layers over the 35 first meters depth. Underneath, the soil is essentially silty clay with sand and shell laminated lenses. A marl stratum is encountered between 103 and 110 m below the ground level. Globally the soil profile can be subdivided into 6 main strata which are described below:
Layer A: Very compressible peat
Layer B: Yellowish fine sand
Layer C: Greyish silty clay
Layer D: Yellowish fine sand
Layer E: Greyish silty clay
Layer F: Marl-greenish and yellow greyish compact clay
Layer G: Compact marl

A soil profile with the geotechnical properties under the main bridge is given by figure 2.

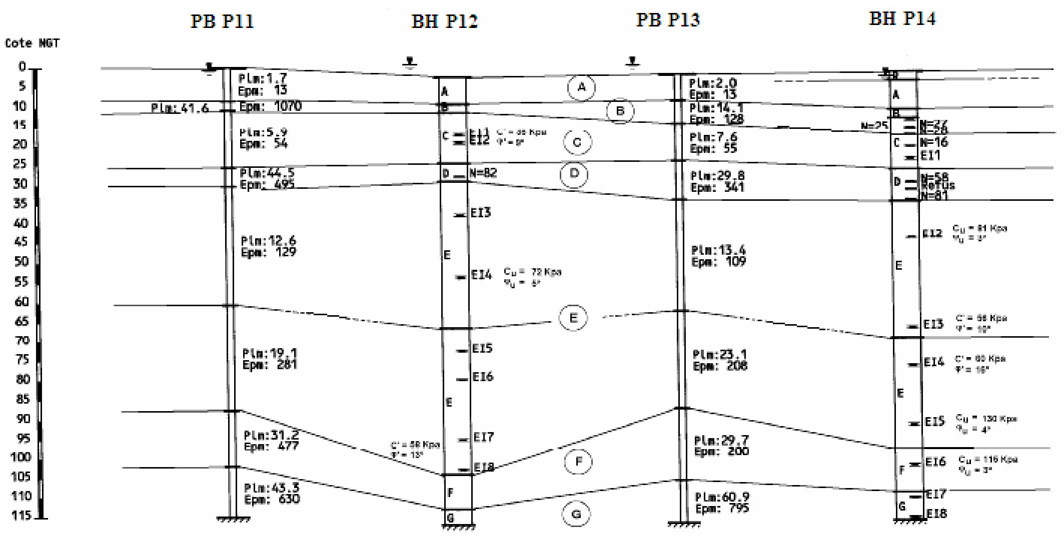

Figure 2. Typical soil profile, [6].

3. Specific geotechnical problems

The high compressibility of the top peat layer (layer A) and its very low shear strength did not allow considering a shallow foundation system for the cable stayed bridge and its approaching bridges. Moreover, the approaching embankments could not be raised more than 1 m above the ground level without soil treatment, [2]. Excessive settlements are expected under the approaching embankments, such settlements are essentially due to the compressibility of top peat layer and that of the deeper clay layers.

It was then decided to adopt bored piles 2 m in diameter and 75 m deep as foundations for the cable stayed bridge and driven piles embedded in the sand layer located between 25 and 30 m (Layer D) below the soil surface for the approaching bridges. For the approaching embankments, an improvement soil solution using the preloading technique associated with vertical geodrains is used to increase the bearing capacity of the soil support and to reduce the settlements to an allowable limit. Each of these solutions will be detailed below.

3.1 Foundations of the cable stayed bridge

The deck of the cable stayed bridge is supported by two central pylons at the ends of the central span and two piers at the ends of the deck. Each of the two central pylons of the cable stayed bridge is founded on 9 bored piles 2m in diameter and 75 m deep. Each of the two piers is founded on 12 bored piles 1.5 m in diameter and 60 m deep. The allowable piles capacities are calculated using the rules of the "*Fascicule 62 Titre V*" a French standard for pile load capacity determination, [4].

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In order to confirm the theoretical load capacity of a single pile, two pile tests each using a different technique had been carried out. In the first test, the pile subjected to the test is 1.5 m in diameter and 60 m long. An Osterberg cell had been inserted into the reinforcement cage at the bottom third of the pile separating the pile into two segments and cast within the pile. The loads are applied by the Osterberg cell simultaneously to the lower and the upper segments of the pile pushing down the lower segment and up the upper segment so the two segments of the pile are tested against each other (figure 3). The pile is equipped with strain gauges and LVDT transducer for deformation and displacement measurements at specific locations along the pile. From strain gauges records with those of the pressure applied to the Osterberg cell and the pile head settlements, the equivalent load applied on the pile head is calculated and the corresponding settlement-load curve is drawn. Figure 4 gives the pile head settlement versus the equivalent applied load.

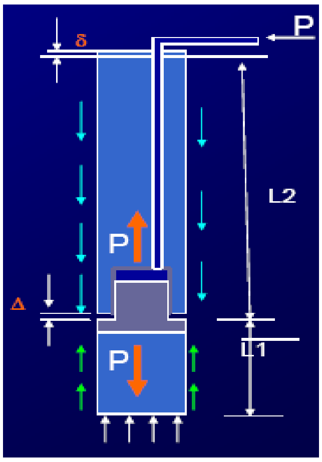

Figure 3. Osterberg pile test system

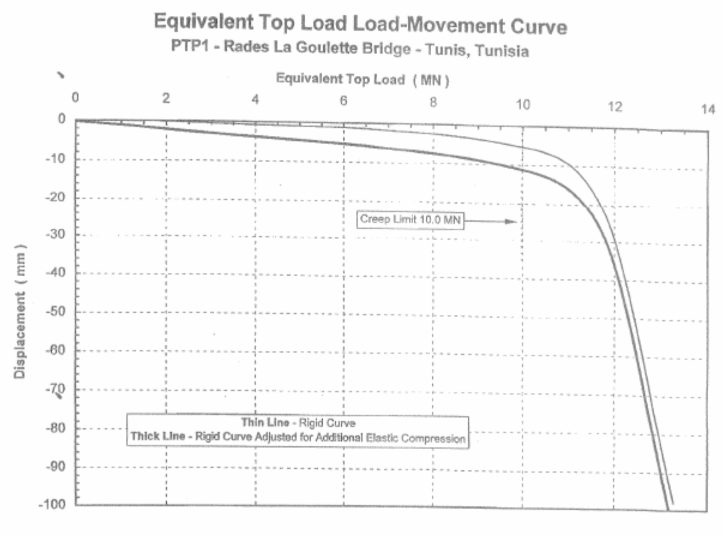

Figure 4. Equivalent Top load-movement curve (Osterberg test)

The second test pile is of a conventional type in which the load is applied on the pile head using jacks and reaction beam fixed to reaction piles (figure 5). The tested pile was 1 m in diameter and 75 m long. The settlement-load curve obtained from this test was found to be in good agreement with the predicted curve (figure 6) given by FOXTA software using a beam on elastic foundations model. Based on the theoretical load calculations and the piles load tests, the allowable load capacity is set to 8250 kN for a single pile under the central pylons and ??? kN for the piles under the piers, [11], [5].

Figure 5. Pile load test

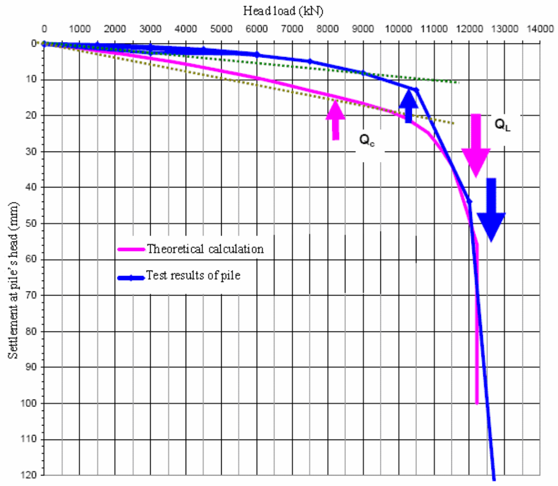

Figure 6. Pile head load-settlement

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The borings for the central pylons piles had been executed using a 2 m diameter bucket for the first 70 m and continued to the final depth using reversal circulation technique. The pile spacing was set to 3 pile diameters. Each pile had been equipped with 6 steel pipes for later controlling of the pile shaft integrity by the transparency method also called sonic coring method (figure 7).

3.2 Foundations of the approaching bridges

The approaching bridges were founded on driven precast concrete piles with 0.45m x0,45m square section (figure 8) embedded in the yellow fine sand layer located between 25 and 30 m below the ground surface. Each pile is composed of two assembled segments each 15 m long. The allowable load capacity based on the rules of "*Fascicule 62 Titre V*" and 4 pile load tests was set to 800 kN for a single pile.

Figure 7. Pipes set up for shaft control

Figure 8. Driven piles at the location of approaching bridge pier

3.3 The approaching embankments

There are 4 embankments 4.5 m high to approach approaching bridges. The very low shear strength of the top peat layer makes impossible the edification of these approaching embankments without soil improvement. Furthermore, the compressibility of the underlying clay layers leads to excessive settlement and negative skin friction along the piles under the abutments of adjacent bridges. Two problems are then to be solved: the first is to have a sufficient bearing capacity to support the four approaching embankments; the second is to reduce the settlement induced in the deep layers to an allowable amount.

For the purpose of increasing the bearing capacity of the top peat layer, a preloading solution using vertical geosynthetic drains is used. The preloading consists in a 7 m high embankment edified according to a specified schedule based on the increase of shear strength of the top peat layer as soil consolidates under the preloading, [1]. The chosen geosynthetic drains are band type with 10 cm width and 5 mm thickness. The drains should be pushed down to the sand layer located at 10 m below the ground surface. The spacing between drains was set to 1.2 m near the bridges abutments and 1.8 m elsewhere. A sand blanket 1m thick had been spread over the preloaded area prior to drains installation, [2].

Slope stability analysis indicated that this sand blanket should extend over 12 m beyond the preloaded area boundaries to guarantee the stability of the embankment regarding slope failure during the first stage of the construction where the height of embankment should exceed 3 m, [7].

During the construction of the sand blanket small mud waves (figure 9) had been observed in front of the blanket as it progressed, a geotextile cover had been then lain on the sea floor which was found very effective in preventing mud waves formation (figure 10).

Prior to the embankments construction, the preloaded zone had been monitored for surface and deep settlements, pore pressure and lateral displacements measurements. The preloading embankments reached their final height (about 7 m) after one year from the beginning of the construction. 100 to 150 days after the end of the construction date, all records showed an asymptotic evolution. The maximum measured settlement was 140 cm and the measured pore water pressures correspond to the initial conditions with zero excess water pore pressure. The estimated corresponding degree of consolidation of the top peat layer was found to be about 90%. Figures 11 and 12 show up the evolution of settlement and the excess pore water as a function of time and the preloading embankment height at abutments A and B locations.

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Figure 9. Mud waves

Figure 10. Installed geosynthetic drains and marks for settlement measurement

The exceeding fill materials had been then removed to give to the embankments their final profiles and the pile driving of the approaching bridges abutments were proceed.

Theoretically, after this preloading, no residual settlement will occur in the peat top layer. However, the underlying soil is expected to undergo more settlement due to the excess of the stresses induced by the approaching embankments. In order to prevent such a settlement to occur, especially in the vicinities of the abutments, a solution of reducing the net excess of stress in the deep soil layers is used by lightening the embankment, [7]. It was proved that the net excess vertical stress induced by the approaching embankment on the top of the clay layer located between 12 to 15 m below the ground surface (layer C) should not exceed 50 kPa. For this purpose, expanded clay had been used as a fill material to substitute the existing embankment over a zone extending 20 m behind the abutments. The bulk unit weight of the used expanded clay is 3 kN/m^3. At locations beyond this zone under the approaching embankments, the settlement is judged not harmful.

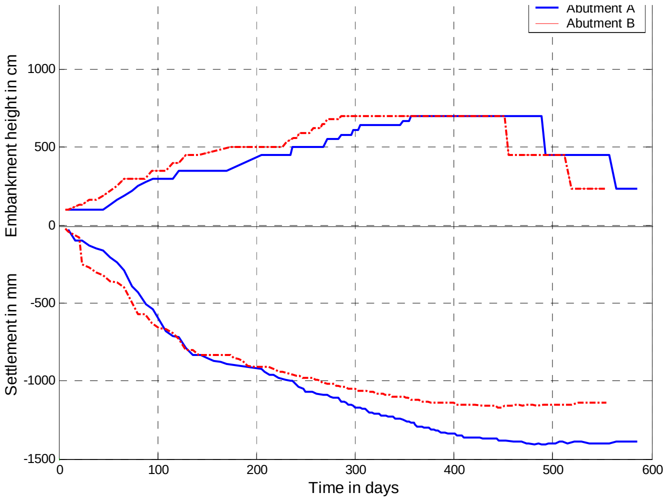

Figure 11. Embankment height and settlement evolution

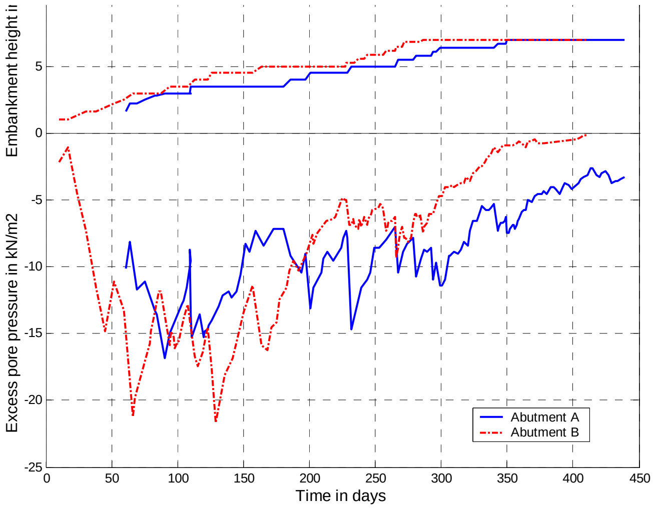

Figure 12. Embankment height and pore water pressure evolution

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4. Cost of geotechnical investigations

The total cost of geotechnical investigations and executed foundations represents by 30% to 50% of the total cost of the project which amounts by 141 Million Tunisian Dinars (The equivalent of 120 US dollars). Six Tunisian contractors and four foreign contractors participated in the construction of four lots which started by 2003 and are expected to be completed by February 2009.

5. Conclusions

The Tunisian Rades La Goulette bridge project, with first construction of cable stayed bridge, represents a well marked symbol with specific modern design. As main challenges it is worth mentioned: the construction of a cable stayed bridge of 260 m length using the technique of balanced cantilever method (by 65.000 of concrete cubic meters and 8.000 tons of steel), the installation of very deep Bored piles of 2m diameter (cumulated length is about 2550 lm) , reclamation of total area of about 25 hectares on 10 m depth of very soft soil conditions (more than a million of cubic meters filling), The installation of more then 10000 driven piles, the incorporation of the project in sensitive and specific ecosystem and difficult access to the site (installation of about 630000 m of prefabricated vertical drains).

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Acknowledgements

Authors gratefully acknowledge Mister Ghazi Cherif engineer from the Directorate-General of Highways Department of Tunisian Ministry of Equipment, Housing and land Regional Development. As Head of Rades La Goulette bridge project Mr G. Cherif provided most of the referenced documents and made the review of the manuscript paper.