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The Development and Rehabilitation of the Hammamlif Coastline: A Geotechnical Study

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Abstract

The coastline of Hammamlif is an area characterized by a flooding problem due to the heavy rain and the wave run up phenomenon. This flooding phenomenon is made worse by both an intense urbanization and the badly designed unitary-type sewerage networks which drain rain water and waste water at the same time. These networks, now silted and overburdened, are not able to carry off water. Hammamlif wave is characterized by a run up in the order of 2.50 m (Doctoral thesis, Abir Baklouti). This value causes a major problem for the low zone of the area. The suggested solutions according to our study are to strengthen the drainage networks and even the construction of a new water purification station and the removal of the existing breakwaters because they are badly designed. Instead, we should apply an artificialy recharged beach and the solution of a submerged geotextile artificial reef to fight against wave action in the area. Applying these major solutions requires a geotechnical study of the Hammamlif area which is the subject matter of this paper. This article aims to present the geotechnical study of the site to come out with specific recommendations on the reserved land to be used as support for future studies and provide various foundation designs for different structures planned in the Hammamlif area.

Keywords: Artificial recharge; Core drilling; Flooding; Geotechnical study; Hammamlif; Pressiometric survey

Abbreviations: SC: Survey Cored; SP: Pressiometric Survey; EI11: Intact sample N°.1 of core drilling N°1; EI21: Intact sample N°2 of core drilling N°1; NG: Natural Ground

Introduction

Hammamlif is a coastal city in the southern suburbs of Tunis, located about ten kilometers from the city center. Attached administratively to the governorate of Ben Arous, it is a district of 38,401 inhabitants according to the 2004 sensus. Its geographical coordinates are 36 ° 44 'North and 10 ° 20' East (Source: Municipality of Hammam Lif). The beach of Hammamlif is 1540 m long and 45 m wide, situated next to the Wadi Meliane that usually pours the drained water into this beach (Source: Coastal protection agency: APAL of Tunis) (Figures 1 and 2). The area of Hammamlif is subject to flood risk due to several factors. To begin with the region witnesses heavy quantities of rain yearly (Average daily flow of rain in 2012 (35923, 8014 m^3/d, National Institute of Meteorology). The sewage network in the area (22 225 m^3/j: capacity of sewage networks to evacuate rainwater) is not able to drain such heavy quantities because they are often clogged and of a unitary sanitation network type draining both of waste water and rain water. In addition, the area of Hammamlif consists of lowlands downstream watershed receiving rainwater from upstream and is subject to the incidence of downstream levels (Sebkha, sea) and even the upstream basins, they are rather insufficient clippers. Another factor is the uncontrolled development of urbanization having rate is around 95% (Source: Coastal protection agency: APAL, Tunis) which constitutes a second aggravating factor, as important as climate change, leading to increased runoff rates which is of the order of 95%. But the most important is the danger coming from the sea when waves exceed the protective dikes height to reach the inhabited area: it is the wave Run up that may reach 2,50 m in height [1]. Wave run-up R is defined as the set of discrete vertical distances of seawater, measured on the foreshore from still water level (SWL). Run-up results from two dynamically different processes: (i) wave setup, a super elevation of mean water level at the shoreline and (ii) vertical fluctuations about that mean [2] (Figure 3).

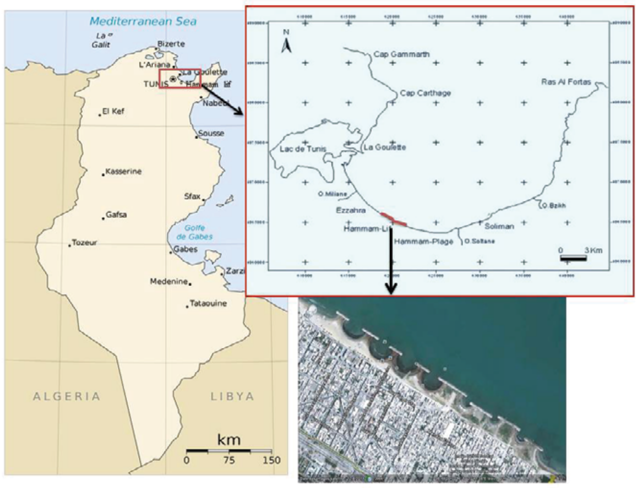

Figure 1: Hammam Lif one of the southern beaches of Tunis the capital city of Tunisia [1].

The sewage networks anomalies in the area of Hammamlif

The clotting and overflow of networks in rainy weather are the result of generalized connection roof runoff and internal courtyards drainage to waste water network. In addition, there is a difficulty of exploitation that causes generalized and aggravated dysfunction in dry weather and

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can be remarkably felt during the rainy periods. Moreover, sewers have become insufficient because of the rapid growth in Hammamlif over the last 20 years or anarchic poses of some sewers: diameter reductions of the pipes or poses against the slope. Finally, the slow flow in the lower parts may explain the areas of important sedimentation, the dry overload and the significant production of H_2S (Figure 4).

Materials and Methods

On-site

During our research study performed in the area of Hammamlif we proceeded as follows:

Two (02) pressiometric polls SP_1 and SP_2 were dug to a depth of 20 m and pressuremeter tests were performed every meter during the digging. After wards, two (02) core drilling SC_1 and SC_2 were performed to a depth of 20 m taking some intact samples during the digging. In order to achieve the pressuremeter and core drilling the following equipment was required on-site:

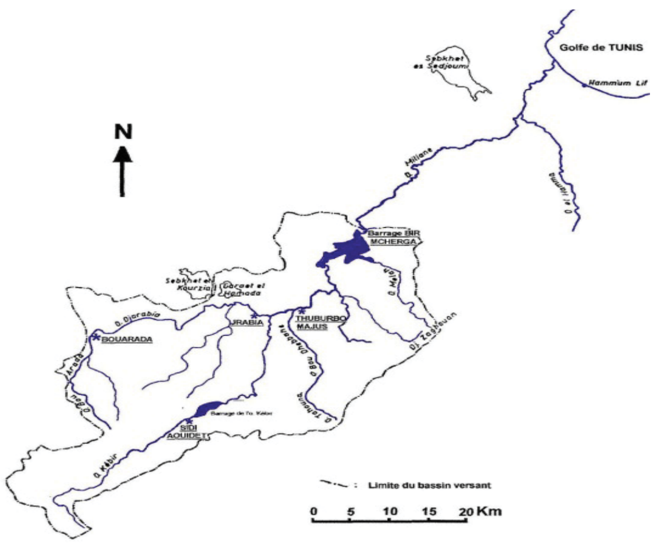

Figure 2: The watershed of the river Meliane that runs through Hammam Lif (Source Wikipédia).

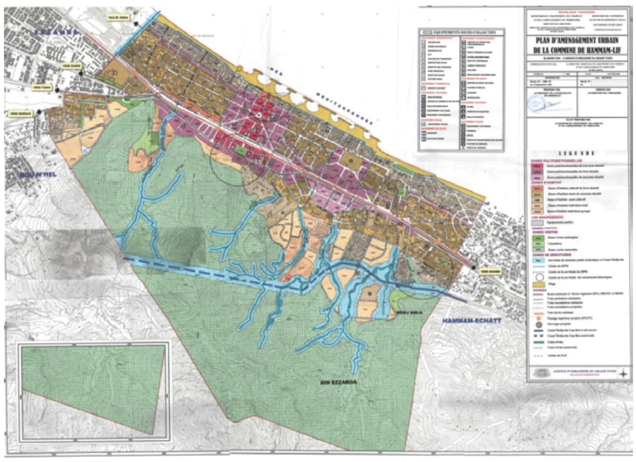

Figure 3: Hammam lif's Urban development plan (Source: Municipality of hammam lif).

a) An hydraulic drill for geotechnical soil investigation and small water wells.(Teredo DC 123) showed in Figure 5.

b) Iveco 4.0 Ton truck.

c) Complete pressuremeters brand Apageo.

d) The drilling tools (single and dual cored).

e) Rods, Pipes and augers.

f) Water tank.

g) Wooden boxes for the classification and preservation of the samples.

In the Laboratory

Some physical and chemical identification tests together with the following mechanical tests were performed:

Water contents (W) by baking (carried out according to the norm NF P 94 – 050)

It defines the ratio in % of water weight (Ww) that the soil contains at Wd dry weight of its elements. The dry elements were obtained by drying the ground for 24 hours in an oven at 105°C. to determine the volumetric weight (γ_h, γ_d). The water contents are determined using method by baking-NF P 94-050.

Particle size analysis by sieving (carried out according to the norm XP P 94 – 041)

The mean sand size distribution is determined using the granular spindle method (André, 1977), It determines the size distribution by weight of the material components and consists of two operations:

a) Sieving of elements of a dimension equal to or greater than 80 mm.

b) Sedimentometry for the elements of a dimension less than 80 mm [3].

Atterberg limits by use of the cup of Casagrande (carried out according to the norm NF P 94-051)

These are geotechnical parameters for the identification of the soil and characterization of its state from its consistency index. By definition, Atterberg limits (liquid limit and plastic limit) are the water content by weight corresponding to particular states of a soil. They are designed to determine the water area where clay soil has a plastic behavior. Their determination requires the use of the cup of Casagrande.

Content of sulfate (SO_4^{2-}):Chemical analysis by spectrophotometric method

The sulphate content is determined by gravimetric dosing,

Figure 4: Maximum pipe filling rate in dry weather (> 100% for HammamLif) [8].

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according to the standard NF EN 196-2"Test Method cement - Part 2: Chemical analysis of cement". The soluble SO_3 is precipitated with barium chloride solution.

Compressibility testing oedometer, (carried out according to the norm XP P 94-090-1)

A soil sample is placed in a rigid cylindrical box with a circular section between two porous discs ensuring its drainage. A piston is used to ensure the sample is under a constant uniform vertical stress for a specified time. We establish compressibility curves (void ratio as a function of the constraint) and consolidation (relative variation of compaction against the time logarithm).

Shear tests rectilinear to the box, (carried out according to the norm NF P 94-071-1)

Casagrande box consists of two half-shells on which pressure is exerted perpendicularly to the junction plane. The compressed sample undergoes a compaction, that is to say it loses a certain proportion of water. One of the two shells being fixed, a lateral pressure is then exerted tending to push the other parallel to their separation. While gradually increasing this pressure, the resistance of the sample is found to increase, reach a maximum and then decrease until rupture occurs. The use of this test is particularly suitable for the study of landslides (Figure 6).

Figure 5: Hydraulic drill for geotechnical soil investigation and small water wells (Teredo DC 123).

Figure 6: Shear apparatus of Thynasondage company (MATEST SHARLAB).

Swelling tests (carried out according to the norm XP P 94-091)

The test is carried out on several test pieces of the same sample taken at the same level. Each test piece is placed in a cell on which a constant vertical axial force is applied. The test consists in applying a different vertical contraint to each test piece and measuring its height variation during immersion.

Results and Discussion

Stratigraphy

Pressiometric Poll SP_1:

a) From 0,00 to 1,00 m: Fine sand

b) From 1,00 to 2,00 m: Brown silty clay

c) From 2,00 to 4,00 m: Fine sand

d) From 4,00 to 20,00 m: Greych sand

Pressiometric Poll SP_2:

a) From 0,00 to 1,00 m: Fine sand

b) From 1,00 to 2,20 m: Brown silty clay

c) From 2,20 to 6,00 m: Fine sand whitish

d) From 6,00 to 20,00 m: Greych sand

Core Drilling SC_1 and SC_2: (Figures 7 and 8).

Interpretation of the Cores Drilling

According to the litho-stratigraphic sections of completed Core Drilling, the underlying floor presents a lithological and geotechnical homogeneity between the general polls. The lithological column of the foundation ground is characterized by a yellowish sand layer over which lies a brown silty clay layer that reaches a depth of approximately 2.00 m. This layer forms the roof of a thick sandy sequence to become some type of yellowish mean sand. The lithological column ends with a thick layer of greyish sand which extends to the end of the core drilling under investigation.

Hydrogeology

At the time of pressure meter polls and core drilling (November 2014), underground water line was met at a level of -2,50 m/TN.

Figure 7: Localization of pressuremetric polls and core drilling.

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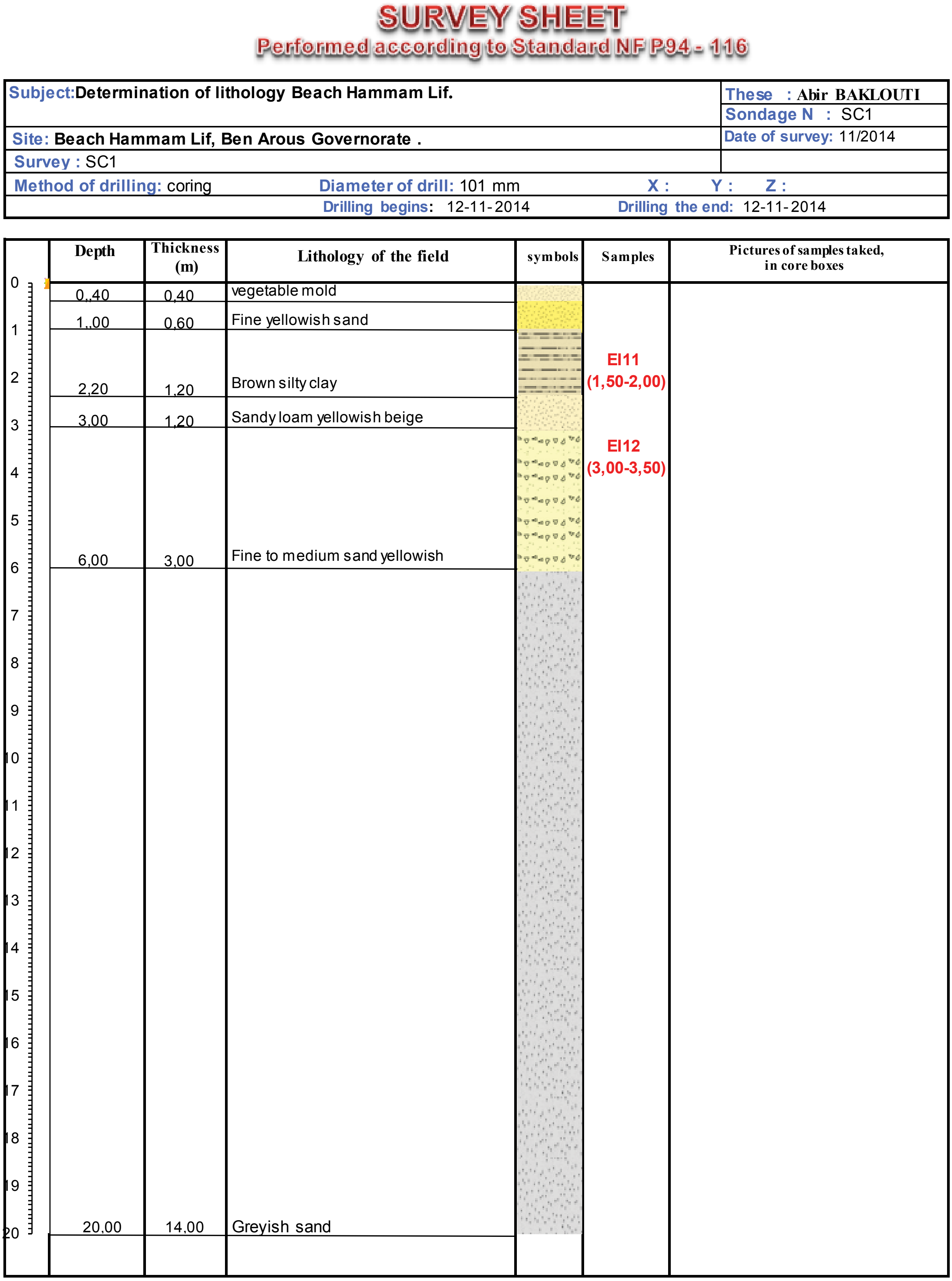

SURVEY SHEET

Performed according to Standard NF P94 - 116

Subject: Determination of lithology Beach Hammam Lif.	These : Abir BAKLOUTI Sondage N : SC1
Site: Beach Hammam Lif, Ben Arous Governorate .	Date of survey: 11/2014
Survey : SC1	
Method of drilling: coring Diameter of drill: 101 mm X : Y : Z :	
Drilling begins: 12-11-2014 Drilling the end: 12-11-2014	

Depth	Thickness (m)	Lithology of the field	symbols	Samples	Pictures of samples taked, in core boxes
0,,40	0,40	vegetable mold			
1,,00	0,60	Fine yellowish sand			
2,20	1,20	Brown silty clay		El11 (1,50-2,00)	
3,00	1,20	Sandy loam yellowish beige		El12 (3,00-3,50)	
6,00	3,00	Fine to medium sand yellowish			
20,00	14,00	Greyish sand			

Figure 8: Core Drilling SC1.

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Physical and chemical characteristics of the investigated soil: The physical and chemical characteristics were measured on intact samples of core drilling. The natural water content W measured on an intact sample according to XP P 94-050 standard varies between 20.30% and 28.61%. The wet unit weight achieved on an intact sample according to XP P 94-050 standard varies between 19.77 KN/m^3 and 20.77 kN/m^3. The dry density achieved on an intact sample according to XP P 94-050 standard varies between 16.08 KN/m^3 16,43KN/m^3. The Size analyses achieved on these samples according to NF P 94-041 standard gave the following results:

a) The percentage of fine sand grains (less than 0.42 mm) varies between 84% and 97%.

b) The percentage of fine sand grains (smaller than 0.08 mm) varies between 06% and 84%.

c) The percentage of fine sand grains (less than 0.002 mm) varies between 00% and 45%.

According to these values, the analyzed samples are undoubtedly sequences of clay and sand.

Atterberg limits perfomed according to the XP P 94-051 standard, gave the following results:

a) The liquid limit WL (water content beyond which the soil behaves like a liquid and flows under its own weight) varies between 38.89 and 47.86% and is indeterminate for the sand sample.

b) The PI plasticity index (PI = WL - WP with WP water content defining the threshold between the solid state and the state at which a solid starts deforming) varies between 13.18 and 24.82% and is indeterminate for the sand sample. The sulfate content measured on intact surface of the samples gave a value range between 0.20 and 0.40% and a liquid sample of the order of 1200 mg/l. Tables 1-5 summarize the identification tests and physical analyzes performed on these intact samples.

Mechanical Characteristics of the Investigated Soil

Pressure meter tests

The soil mechanical characteristics were measured on site with pressure meter tests in accordance with NF P 94-110 Standard. The pressure meter type used is MENARD (brand APAGEO with automatic treatment of the results using the computer software XPRESSIO) fitted with 3 cells: one is a measuring cell and the other two are guard cells. The tests were carried out every meter. For each test, we studied the cylindrical expansion according to the lateral soil pressure transmitted to ΔV/V. We plotted the curve $P=f\left(\frac{\Delta V}{V}\right)$ for each test. From each curve, we deduce the following characteristics:

a) Po: initial pressure corresponding to the earth pressure at rest.

b) Pf: pressure flow that marks the boundary of the elastic limit of the soil.

c) Pl: pressure limit is the maximum lateral tensile strength that the floor can withstand.

d) E: pressuremeter modulus is the slope of the elastic portion of the pressure curve - volume.

Table 6 summarizes the values of the effective limit pressure Pl '(Pl = Pl-Po) and the pressuremeter modulus from the results of the tests conducted every meter. Based on these results, we note that:

Polls	Samples	Depth (m)	Initial Water Content (%)	Bulk density (KN/m^3)	Dry density (KN/m^3)
SC_1	EI_{11}	1,50-2,00	20,30	19,77	16,43
	EI_{12}	3,00-3,50	27,53	20,51	16,08
SC_2	EI_{21}	2,50-3,00	28,61	20,77	16,15
	EI_{22}	4,50-5,00	26,54	20,42	16,14

Table 1: Summary of results of laboratory identification tests (Method by baking–NF P 94-050).

Polls	Samples	Depth (m)	% fine <0,42 mm	% fine <0,08 mm	% fine <0,002 mm	Cu	Cc
SC_1	EI_{11}	1,50-2,00	97	84	45	-	-
	EI_{12}	3,00-3,50	84	07	-	2,59	0,90
SC_2	EI_{21}	2,50-3,00	93	68	36	-	-
	EI_{22}	4,50-5,00	85	06	-	2,53	0,88

Table 2: Summary of results of particle size testing.

Polls	Samples	Depth (m)	W_L	I_P	I_C
SC_1	EI_{11}	1,50-2,00	47,86	24,82	1,11
	EI_{12}	3,00-3,50	ind	ind	ind
SC_2	EI_{21}	2,50-3,00	38,89	13,18	0,78
	EI_{22}	4,50-5,00	ind	ind	ind

Table 3: Summary of Atterberg limits tests.

Polls	Samples	Depth (m)	SO_4^{2-} Content (mg/l)
SC_2	EL_{11}	2,50	1200

Table 4: Summary of the results of the sulfate analysis.

Polls	Samples	Depth (m)	% Ca CO_3	Classification
SC_1	EI_{11}	1,50-2,00	16	moderately Limestone
	EI_{12}	3,00-3,50	10	low Limestone
SC_2	EI_{21}	2,50-3,00	23	moderately Limestone
	EI_{22}	4,50-5,00	13	low Limestone

Table 5: Summary of the results of carbonates analysis.

Depth	SP_1		SP2	
(m)	Pl'	E	Pl'	E
	(bar)	(bar)	(bar)	(bar)
1	5	34	3	24
2	6	45	6	34
3	7	54	6	37
4	7	45	6	43
5	6	34	5	24
6	6	45	6	37
7	5	31	6	32
8	6	22	6	34
9	6	42	5	37
10	5	31	6	31
11	6	31	6	31
12	5	31	6	30
13	6	45	7	47
14	5	31	7	55
15	7	49	7	37
16	7	34	8	48
17	8	56	8	49
18	8	63	9	44
19	8	62	9	58
20	9	41	9	54

Table 6 : A summary of the pressiometric results.

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a) Along the pressuremeter survey, the mechanical characteristics are low to medium throughout the explored depth correlatively with consequential facies.

b) The values of E and Pl ' classify the soil in Class I in all the explored depths.

Laboratory tests

The mechanical properties of the soil were measured in the laboratory by a shear test box (according to XP P 94 - 090-1 Norm), the compressibility in oedometer (according to XP P 94-090 -1 Norm) and swelling tests. Tables 7-9 summarize the results of the mechanical tests.

Calculation of Foundations

Structures to be built

The geotechnical survey was designed to determine the appropriate type of foundation for charge descents subsequently provided by the Design Office and for the various civil engineering works (construction and rehabilitation of the area Hammam Lif). The foundation of such a work has to meet two criteria:

a) A general stability.

b) A qualified differential settlement.

Based on the mechanical characteristics encountered, it is possible to provide shallow foundations to establish this structure**.**

Soil working rate

Based on the results of pressuremeter tests: The eligible work rate foundation soils is calculated from the Menard pressuremeter tests according to Fascicle 62 Titles V - DTU 13.12. as follows in equation (1)

$$qa = \gamma \, x \, D + k / F \left(P_l - P_o\right)\left(t / m^2\right) \quad \textbf{(1)}$$

With:

γ: Wet Density taken equal to 1.8 t/m³.

D: Recessed depth of the soleplates.

K: a coefficient that depends on the soil type, the type of the soil (isolated or filante) and of the equivalent Recessed height calculated as follows in equation (2):

$$he = \frac{1}{(Pl'e)} \int Pl'(z)dz \quad (2)$$

$$Avec \; Pl^*_{l\acute{e}} = \sqrt[n]{P'1xPl'2xPl'3x........xPl'n}$$

Pl'1, Pl'2 ....Pl'n are effective pressure limits measured at 1.5 B.

Pl *: is the equivalent net pressure limit calculated as the average value of the existing net pressure limit to a depth of 1.5 B located under the soil. This coefficient varies from 0.8 to 1.1 for a soil of category I.

Based on the results of the mentioned tests, Table 10 gives the values of the soil working rate of an isolated soleplate depending on the recess depth and dimensions of the sole plate. The recommended safe working levels of the subgrade is taken equal to the limited value of 15 t/m² (1.50 kg/cm²) for isolated Recessed soils from - 2.00 m/TN, subject validation by the calculations of settlements (Table 11).

Recommendation about the Swelling

The swelling pressure, measured at Recess under the base of the foundation, gave a relatively low value of the order of 0.50 bars for the clay layer, on the surface, with an average water content about 20%.

Polls	Samples	Depth (m)	Linear Shear test UU		linear shear test CD	
			C'(KPa)	ϕ'(°)	C'(KPa)	ϕ'(°)
SC_1	EI_{11}	1,50-2,00	38,22	8,60	-	-
	EI_{12}	3,00-3,50	-	-	1,62	32,66
SC_2	EI_{21}	2,50-3,00	10,20	31,40	-	-
	EI_{22}	4,50-5,00	-	-	1,88	33,18

Table 7: Summary of the mechanical tests results (Shear test in box).

Polls	Samples	Depth (m)	e_0	C_c	$C_c/1+e_0$	σ'_c (bars)
SC_1	EI_{11}	1,50-2,00	-	0,123	-	1,650
	EI_{12}	3,00-3,50	0,650	0,168	0,102	0,990
SC_2	EI_{21}	2,50-3,00	-	0,131	-	1,800
	EI_{22}	4,50-5,00	0,645	0,163	0,099	0,840

Table 8: Summary of laboratory mechanical tests results (Consolidation oedometer test).

Polls	Samples	Depth (m)	σg (Bar)
SC_1	EI_{11}	1,50-2,00	0,55
	EI_{12}	3,00-3,50	0,00
SC_2	EI_{21}	2,50-3,00	0,00
	EI_{22}	4,50-5,00	0,00

Table 9: Summary of the laboratory mechanical tests results (Swelling test oedometer).

Depth (m)	H (cm) B × L=1 × 1 m	ΔH (cm) B × L=2 × 2 m	ΔH (cm) B × L=3 × 3 m	ΔH (cm) B × L=4 × 4 m
D=2,00 m	<1	1,20	1,50	2,00

Table 10: Summary of compaction values.

Depth (m)	q_a (t/m²) B × L=1 × 1 m	q_a (t/m²) B × L=2 × 2 m	q_a (t/m²) B × L=3 × 3 m	q_a (t/m²) B × L=4 × 4 m
D=2,00 m	15	15	15	15

Table 11: Summary of tillage rates values (isolated base plate).

Settlements

The settlement under a foundation, caused by the charges and overcharges are due to two totally different phenomena:

a) A consolidation process due to the impact of the spherical component of the stress tensor. The increase of the average pressure causes a reduction in the material volume according to the value of the volumetric compression module.

b) A redevelopment phenomenon of solid particles caused by the deviatoric part of the stress tensor. The resulting angular deformations lead to displacements with no change in the materials volume.

The settlement under a soil subjected to a concentrated vertical load can be calculated from the rules of the Securities Issue 62 V - DTU 13.12 based on the results of pressure meter tests as follows in equation (3):

$$\Delta H = 1{,}33.p.R_o.(\lambda_2.R/R_o)^{\alpha}/3E_a + \alpha.p.\lambda_3.R/4{,}5E_b \quad (cm) \quad (3)$$

With:

$R_{0:}$ reference radius Ro = 30 cm

α: rheological coefficient which depends on the nature of the soil

p :stress applied to the soil from the soles.

λ_2 etλ_3 coefficients of respectively 1.12 and 1.1 forms equal for square footings.

E_a: pressiometric module - R (R = B/2) of the seat of the sole.

E_b: equivalent pressuremeter modulus for soil layer - R - 16 R

The medium-term settlements are calculated on the order of 2 cm for most of the overloaded soils, subject to a floor constraint of the order of 15 t/m² (1.50 kg/cm²) to fix the soleplates at= - 2.00 m/TN (Table 10).

Aggressivity of the ground seat

Sulfates react with the aluminate cement giving expansive compounds, gypsum and ettringite $CaSO_4$, $2H_2O$ $3CaO$, Al_2,$3CaSO_4$,$3H_2O$, and causing a base exchange reaction

Ca^{++} Mg^{++}

This leads to the partial dissolution of the calcium cement constituents: $Mg(OH)_2$

The sulfate content measured on a solid revamped sample gave a maximum content of about 0.40% and to a liquid sample of the order of 1200 mg/l, which proves that the soil is rated moderately aggressive, with an aggressiveness degree of A_2 and a level of protection 2. So the adaptation of the composition and implementation to environmental conditions (cement content, cement type, E/C, cure, adjuvanted) are ensured (Table 12).

The Proposed Solutions

Sewage networks modernization and reinforcement

The main sanitation development in the region of Ben Arous are first to continue the development efforts of primary rainwater networks (recalibration in downtown and extension to the suburbs). Moreover, the waste water treatment capacity either of the existing stations needs to be reinforced or a more radical solution would be the creation of new waste water treatment stations. Added to this the authorities might think of the creation of a new treatment pole in the west of the city involving the routing and transfer of waste water treatment to the new pole for better results.

To guarantee the efficiency of such suggestions the collection and pretreatment of industrial water networks need to be reconsidered and reorganized. Therefore, the identified suggestions would involve: the reinforcement and recalibration of rainwater collection networks in the city, and even the construction of new networks and the establishment of new additional connections. Thus, it is clear that an extension of the existing old infrastructure (collection and treatment) has become a necessity to increase the treatment capacity and protect the region against the flooding threat (Figure 9).

Aggressiveness level	A_2	remarks
Level of Protection	2	
Minimum cement content (Kg/m³)	$550/\sqrt[5]{D}$ D=maximum diameter of aggregate	According to the maximum dimension of aggregate in mm
E/C	≤0,55	Take into account the absorption by the aggregates
particle size	Particle size meets the compositional rules of the concretes	
Workability	Consistency compatible with good implementation and leading to maximum compactness of concrete. Vibration, possible use of adjuvant	No further addition of water
Cover to reinforcement	≥30 mm	
additional protection	unnecessary	
Choice of cement	HRS	

Table 12: General Recommandations.

The artificial recharge

Hammamlif is an area characterized by low topography. The lower area is usually threatened by the decline of the beach profile due mainly to the effect of the rise of the sea level.

Hammamlif beach profile slips backward by 1 m/year in average since 1950 (Figures 10 and 11).

1: average position of the shoreline (period 2004-2005); 2: average position of the shoreline (period 1883-1886).

Facing these problems together with the flooding threat, an artificial beach nourishment of Hammamlif has become urgent in order to build a new nice beach.

The geotextile artificial submerged reef

Calculation of the coefficient of transmission across the structure: We should have a good knowledge on the swell of the sea waves and their significant heights. Indeed the positioning of the reef has to be adequately chosen. If it is chosen too close to the dyke, water would not have enough time to withdraw after the wave breaking and harmful "bagging" phenomenon would then be produced. Too far from the dike, the deshoaling effects would tend to amplify the wave transmitted to the back of the reef (Figures 12).

Kt = 0.170

The method consists in considering a reef implanted at h = 10 m deep, a hundred meters off the sea wall, with a varied wide side slope (its slope rating side is set to 2/1), the draft (water withdrawal), and its berm length (Figure 13). The characteristics of a successful artificial reef for this study in the Hammamlif area are summarized in the Table 13:

The trapezoidal shape is not necessarily the most appropriate if the objective is to dissipate a maximum energy of incidental waves. However, the benefit of such a shape is simplicity and ease to implement, and cost effective compared to a more complex solution, especially the goal here is to sufficiently reduce the significant height of the waves to prevent them from overtopping the dike [4].

Conclusion

The geotechnical investigation for the rehabilitation and development of the coastal area of Hammamlif to fight against the flooding phenomenon was designed to identify the lithological and sedimentary parameters of the different layers of the underlying soils and their geotechnical parameters As far as the soil is concerned and according to the litho-stratigraphic sections of the completed polls, the underlying floor presents a lithological and geotechnical homogeneity among the general polls, The lithological column of the foundation soils is characterized by a yellowish sand layer just over a brown silty clay layer that reaches a depth of approximately 2.00 m and forms the roof of a thick sandy loam sequence and a yellowish type of fine sand. This lithological column ends with a thick layer of greyish sand which extends to the end of the investigation zones. At the time of execution of the pressuremeter polls and core drillin [5-7], the underground line of water was met at a level of -2,50 m/TN. As for the foundations our recommendations can be enumerated as follows. First, the qualifying

Citation: Baklouti A, Bejaoui M, Medhioub S, Yaich C (2016) The Development and Rehabilitation of the Hammamlif Coastline: A Geotechnical Study. J Civil Environ Eng 6: 206. doi:10.4172/2165-784X.1000206

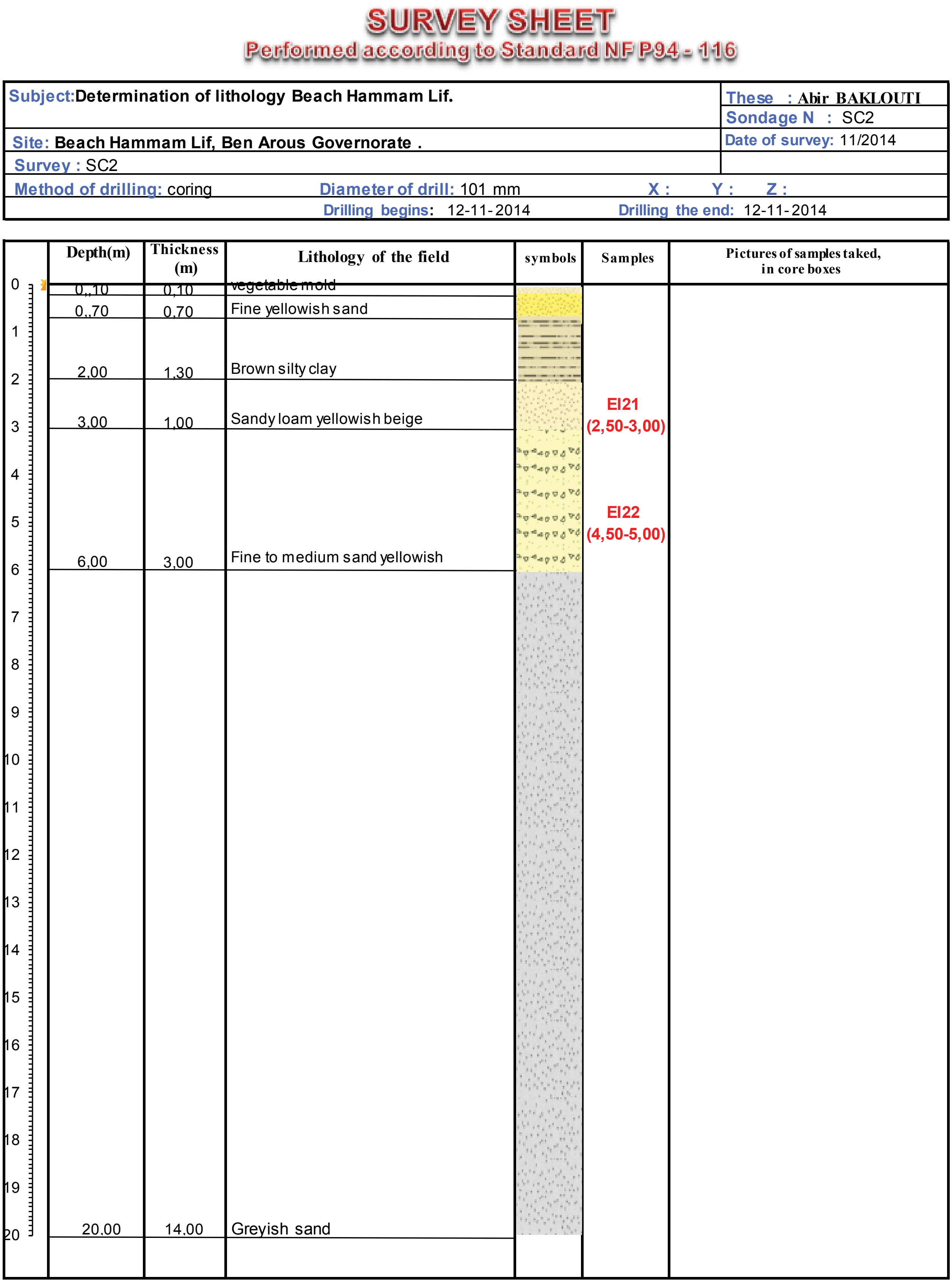

Figure 9 : Core Drilling SC2.

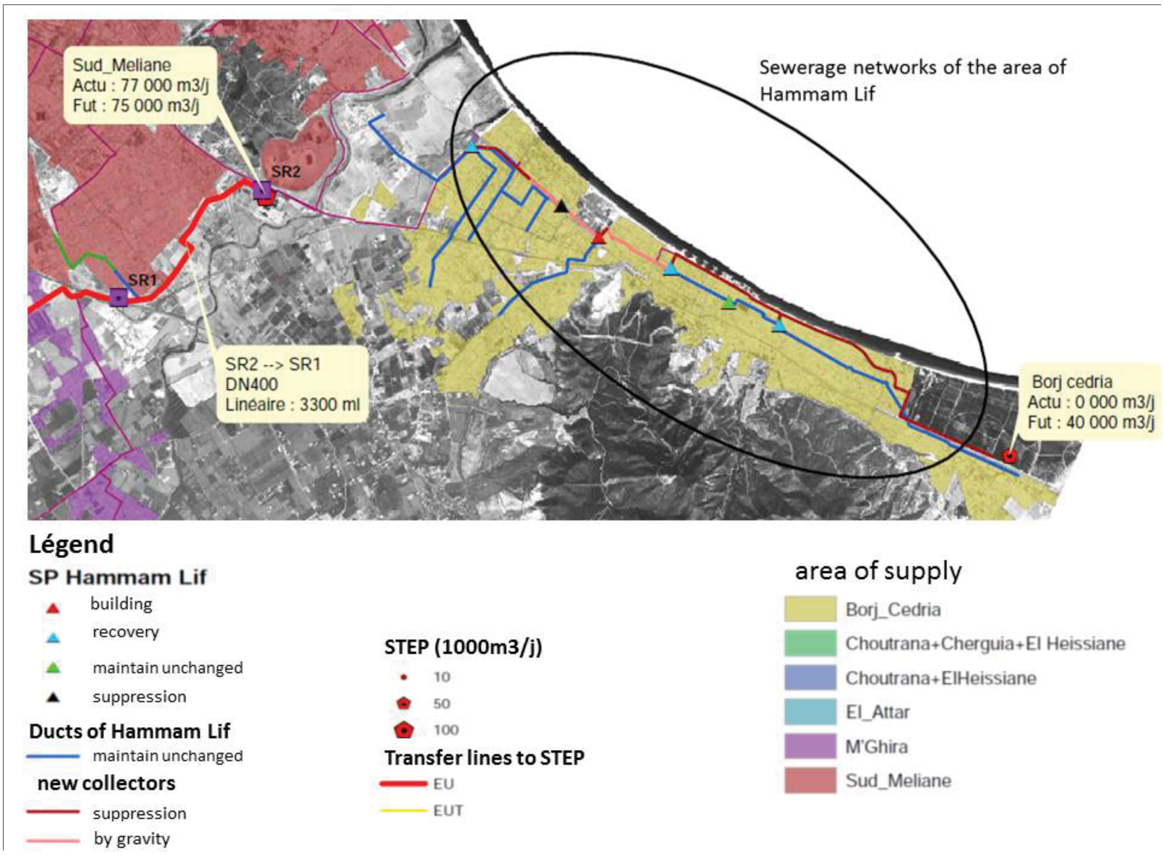

Figure 10: Reverse sewage networks direction towards the projected « Borj Cedria » station [8].

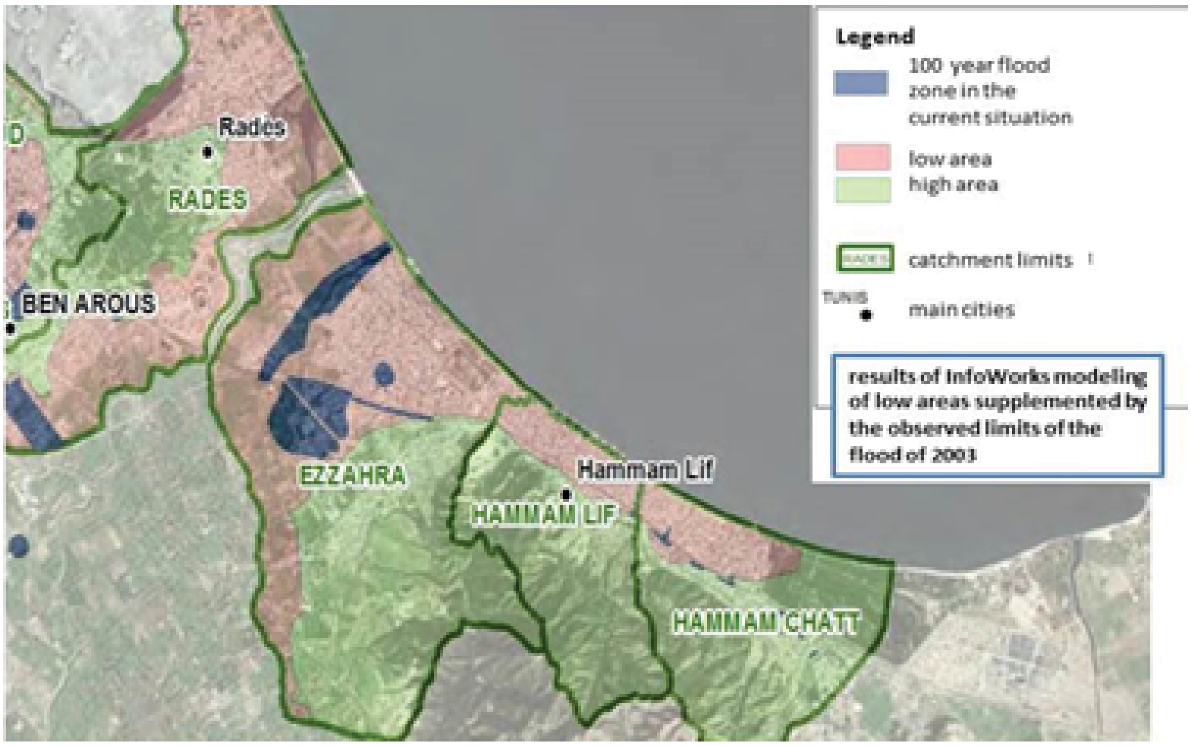

Figure 11: The low coastal area of Hammam lif [6].

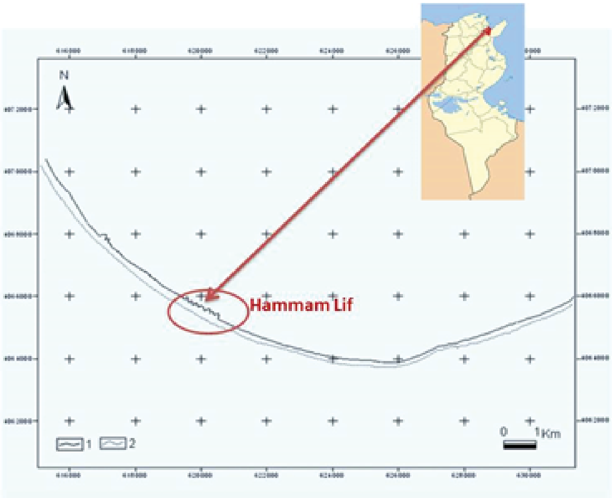

Figure 12: Diachronic evolution of the coastline [7].

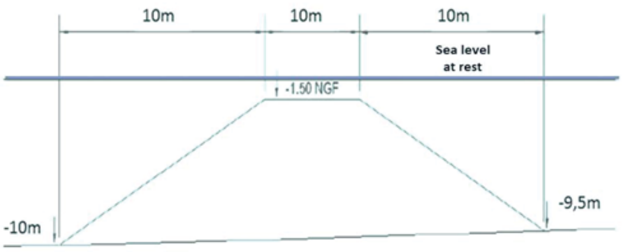

Figure 13: Sectional view of the proposed reef in the sea of HammamLif of a trapezoidal shape.

Berm width	Slope	Reef width	Sectional area	Total volume for a 200 m long reef	Transmission coefficient Kt
10 m	2/1	30 m	160 m^2	32 000 m^3	0,170

Table 13: Dimensions selected for the estimated reef in the sea of Hammam Lif.

work recommended rate of the foundation soils is taken equal to the limited value of 15 t/m² (1.50 kg/cm²) for the installation of isolated base plates from - 2.00 m/TN with a slowdown in the medium term of about 2 cm [8]. Second, the swelling pressure measured at recess in the base of the foundation has given a zero value, so the stability of the structure is ensured. Third, the sulphate content measured on an intact solid sample surface to give a maximum level of about 0.40% and about 800 mg/l on a liquid sample, which proves that the soil is classified moderately aggressive with A_2 as a degree of aggressivity and 2 as a level of protection. So the adaptation to the environmental conditions and composition (cement content, cement type, E/C, cure, adjuvanted) is guaranteed (Table 12).

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