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Preloading of harbor's quay walls to improve marine subsoil capacity

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ABSTRACT

Preloading is an improvement technique for compressible soils, and has been applied at DjenDjen port in Jijel province, Algeria, as part of its development and expansion. In addition, this treatment to eliminate the risk of wharf caissons instability. Decent number of research and development works of the preloading process in different countries by several authors have been cited, with the aim of justifying our research and results. The objectives are to understand and apprehend the coastal soil preloading method and its application in terms of the sensitivity of the intervening factors on its achievements, and their effect on the behavior of the soil and the marine structure during and after its implementation. Furthermore, a numerical simulation of the real test of the method of treatment is carried out, by the plaxis 2D code in finite elements, also respecting the actual construction phasing of this structure, in order to compare the calculation results with in-situ measurements to validate the numerical models and to check the stability of the harbor structure. A matrix of consolidation process during pre compression is proposed.

1 Introduction

Marine structures are the most important structures, harder to design because it depends on several factors that must be respected, and quay walls sizing requires hydraulic, structural and geotechnical analysis, which should cover all identified failure modes. The construction of a port, its equipment, the layout of its access, the protection of the shore against the action of the sea constitute a set of complex operations. In fact, their maritime character comes mainly from the site in which they are produced or because they are intended for the reception of ships whose size has become very important. The need to consider the complete life cycle of the harbor structure, from design to decommissioning, when planning and designing marine structures [1, 2]. Specific needs are usually defined from feasibility studies that have had to incorporate different factors such as the economic justification and the physical, social and environmental impacts of the development, these studies - which can be substantial - are often essential to determine the viability and acceptability of the development.

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The geotechnician is interested in soil as it is the main element of the context in which the stability of a structure will be conceived [3]. It was therefore quickly considered to study the mechanisms of rupture to increase their bearing capacity and eliminate settlements and risks of liquefaction [4]. Soil improvement methods are one of the tools available to the engineer to solve the stability problems or deformations he encounters when developing a project. A large number of processes exist [5]; the preloading process was applied at the Djen Djen port, which is the subject of our study, in order to improve the supporting soil that will receive the quay walls (caissons).

2 Quay wall and Failure Analysis

When the subsoil provides good resistance, the quay walls are made in the form of massive structures capable of withstanding the horizontal forces (to the earth caused by the berthing of vessels and to the pelvis, caused by the thrust of the embankment and mooring vessels) and the vertical forces due to their own weight [6]. The caissons are used to provide continuous quay walls or structures in discontinuous support and can perform the supporting role of solid land embankments in the case of container terminals.

Failure is a response to a defined load (the breaking load) for a given design situation. The rupture is therefore characterized by a significant increase in the response generated by a minor increase in shares (actions). All modes of rupture must be considered in the design of structures, although their relevance varies according to the structure, the localization and dimensioning scenarios. It should be noted that these modes of rupture are often closely related; for example, a settlement of the structure can induce major wave overtopping, which can then provoke instability of the inner slope of the structure. Ruptures are usually due; either to the action of the wave, either to geotechnical factors, who are influenced by self weight, hydraulic and seismic actions (figure 1). The effect of the caisson on the stability of the rockfill (rip rap) basement, The erosion of foundations and the great landslides that result, we must carry out checks for each potential failure modes. The main failure modes for quay walls are shown in figure (2).

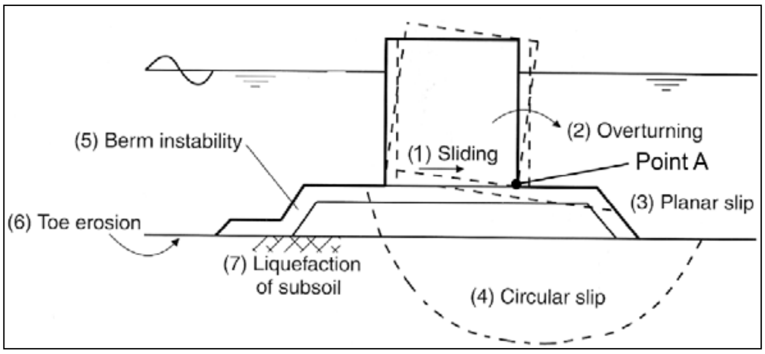

Fig. 1– Failure mechanisms of a caisson before embankment [1].

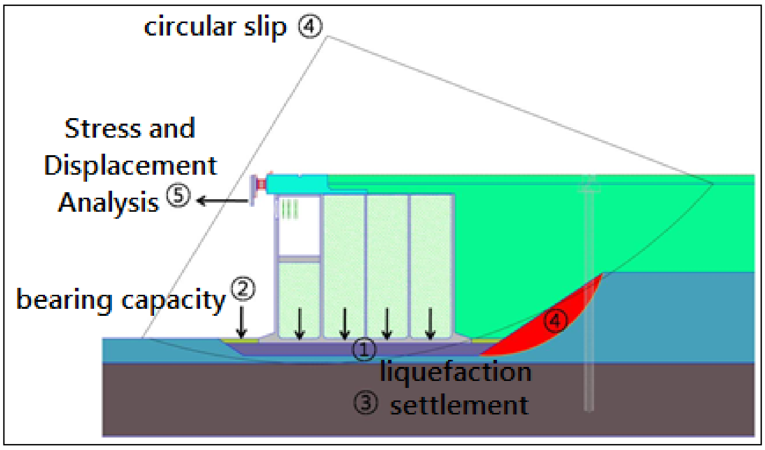

Fig. 2– Failure mechanisms of a quay walls after embankment (for a container terminal) [7].

3 Soils of coastal areas

With respect to the existing natural material an important factor is whether it represents the entire soil of the site. Many of coastal developments overlay problematic grounds such as soft marine deposits, karstic ground, corals, reef limestone, vuggy and weak sedimentary rocks that may affect the performance of coastal developments [8]. Marine sediments consist of mineral, organic and liquid phase. In the geology dictionary [9], sediments are defined as "an assembly consisting of the joining of more or less large particles or precipitated materials having, separately, undergone a certain transport". These particles come from the erosion of rocks and soils, organic activity (accumulation of shells, …) as well as local discharges due to human activity [9, 10]. Sediments are fine particles (clays, silts) to coarse (sand), displaced and transported, in particular through climate actions (wind, tides …) and human [11]. The low bearing capacity and high compressibility of these deposits affects the long term stability of major infrastructure (Johnson 1970) [12]. Therefore, it is imperative to stabilize these soils before commencing construction to prevent unacceptable differential settlement.

Soil structure can be considered as an ideal and suitable place by engineers to have 4 features [13]; 1. Having enough shear strength and bearing capacity of the soil. 2. having low degree of immediate settlement and consolidation caused by the load. 3. Acceptable changes of volume expansion of the soil (for instance, swelling caused by unloading or humidity rise in clay soil) or volume contraction of the soil (caused by decreased humidity) so that the construction effacing won't be conflict. 4. Not having any serious problem in construction place. Therefore, such ground must be assessed in relation to the functionality of the development, expressed through the target performance criteria. For example the expected long term settlement induced by any underlying soft deposit must be considered in the overall settlement calculations.

The type and characteristics of the original ground is of paramount importance in assessing whether the desired functionality of the improvement can be achieved and, if so, estimating the program and budget for the works [14]. The use of soil treatment methods implies knowledge of their respective performances and limits. It is however clear that the other identification factors and the mechanical parameters of the soils are to be taken into account in the precise definition of the treatment of each concrete case [15].

4 Preloading

Pre-loading is a simple solution recommended for very compressible saturated soils with a view to partially accelerating their primary consolidation which is accompanied by a reduction of settlement and as a result of an increase in their undrained cohesion.. When it comes to building on saturated soil with low bearing capacity and (or) relatively compressible, pre-loading is the simplest technique to ensure short-term shear strength improvement [16]. Usually, the aim is to eliminate 100% of primary consolidation settlement and enough secondary settlement such that the residual settlement is within acceptable performance limits. The residual settlement for a given length of time after construction can be estimated as the remaining secondary settlement that occurs during the required time after the eliminated equivalent time of secondary compression has elapsed [17].

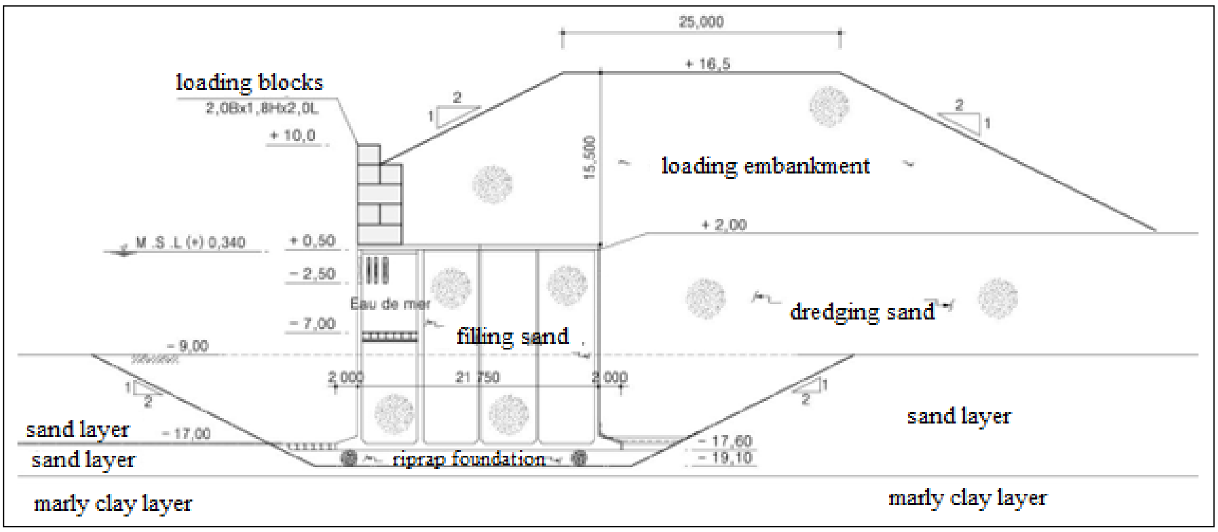

Fig. 3– Cross- section during Preloading of caisson (CTB-12, A4) of new DjenDjen Port Container Terminal [7].

The mechanism of preloading can also be described using a spring analogy (Figure 4). When a surcharge load is applied, the negative pore water pressure in the soil generates. As the applied total stress is constant, the effective stress in the soil increases due to the suction generated. Gradually, the pore pressure decreases (Figure 5) and the spring starts to compress, hence, the soil skeleton gains in effective stress [18]. When the effective stress related to suction pressure increases equi-axially, the corresponding lateral movement is compressive, and therefore, the risk of shear failure can be minimized even with a higher rate of embankment construction. The extent of surcharge fill can be decreased to achieve the same amount of settlement [21].

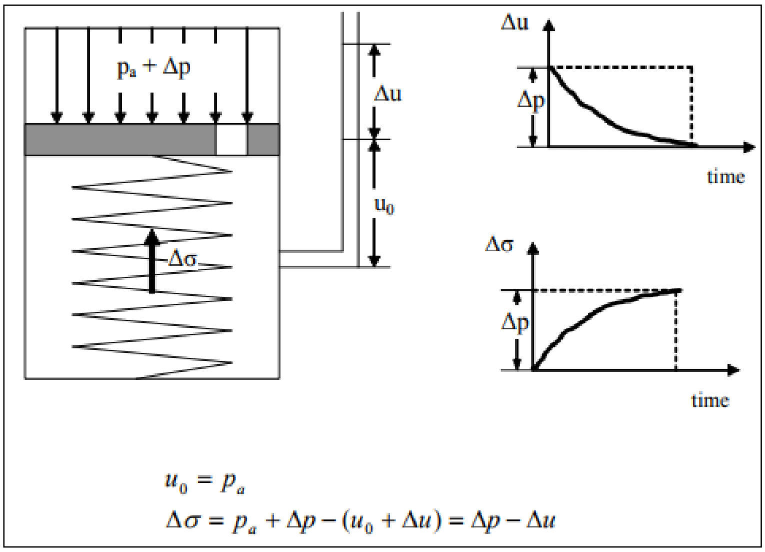

Fig. 4– Spring analogy of surcharge load (After Chu and Yan, 2005) [19].

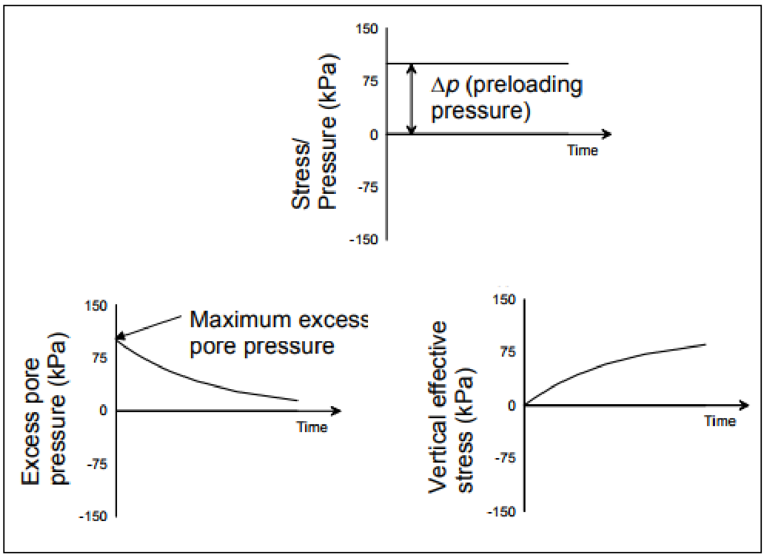

Fig. 5– Consolidation process of conventional loading (after Indraratna et al. 2005c) [20].

4.1 Improved characteristics due to pre-loading

As part of the justification of preloading treatment method, delicate number of research and development began in different countries by several authors, which have generally led to charts to determine the state of the soil or to evaluate the reduction of failures under the structures. These charts are generally very simple to use and are well adapted to the structures. In this part, we will study the potential of pre-loading to increase the capacity of subsoil.

Table (1) [22] summarizes the comparison of engineering properties of the soil prior to and after the implementation of the ground improvement project by pre-compression. The comparison was made in terms of shear strength of the soil (Su) and its compressibility characteristics such as Cc, e_o and Cv. Increased bearing capacity due to preloading can be seen in Table (2) [23]. The increase in the bearing capacity sufficient significant on preloading with a high pile of over 13 cm. The increase in the carrying capacity reaches 101- 242%.

Table 1– Comparison of soil properties prior to and after improvement [22]

Soil Properties	Prior to improvement	After improvement
Su Average	(6 – 30) kPa 10kPa	67kPa
Cc Average	0.2 – 0.6 0.5	0.3 – 0.36
e_0	0.5 – 1.5	1.04
Cv	(6 – 21) m^2/yr	(2.11 – 3.64) m^2/yr

Table 2– Value of bearing capacity due to preloading [23]

Height of embankment (cm)	Bearing capacity, q_u (kPa)	Improvement of q_u (%)
0	2.64	0
1.6	3.13	19
3.3	3.35	27
6.5	3.69	40
13.1	5.29	101
26.1	9.01	242

In order to show the effect of preloading on the soil response, Figure (6) presents a comparison of the evolution of the liquefaction ratio ($r_u = \Delta Pw / \sigma'_{vo}$) as a function of time and depth for the same incoming earthquake in a case with and without preloading [24]. It should be noted that in case the preload has been used, a reduction in the value r_u is found. It is well known that a co-seismic settlement appears with the appearance of liquefaction.

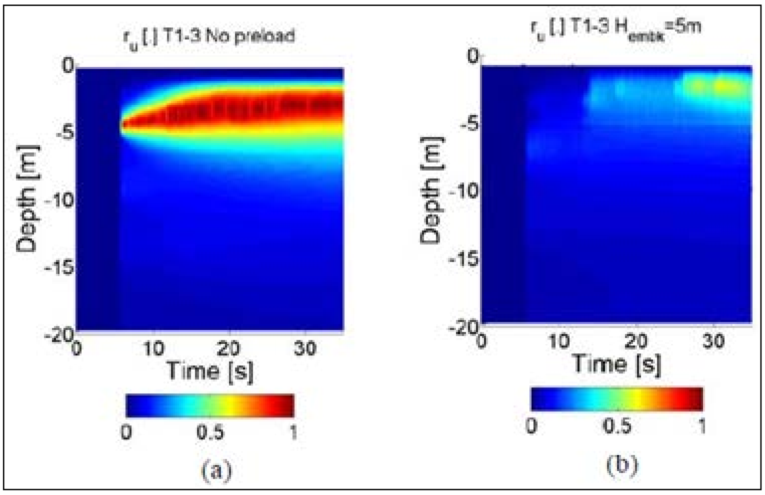

Fig. 6 – Comparison of the evolution of the liquefaction ratio in time and depth for a case a) without and b) with preloading [24]

In order to improve subsurface soft saturated clayey layers under the oil storage tanks in the Mahshahr project [25], the preloading method is assessed. The preloading approach can be applied to enhance consolidation settlement rate, by embankment. Besides, soil settlement induced by oil tanks has been compared before and after preloading based on modified soil parameters (figure 7). The results of the settlements have been plotted, the embankment construction was continued for 108 days, and during the 53 days (waiting time), the soil under the embankment reached 95% of its consolidation. At this stage, the final settlement of the soil 122.2 cm is reached, and the interstitial water pressure has reached its minimum value, which is a sign of the end of the soil consolidation period [25]. At this stage, the soil is slightly inflated and, by removing the embankments, the amount of final settlement has been reduced, the soil settlement has reached 102 cm, indicating that the soils flows from the removal passage of embankment. After building the tanks for 30 days and its placement on the soil, the soil concentration reached about 9 cm after reaching the final consolidation time.

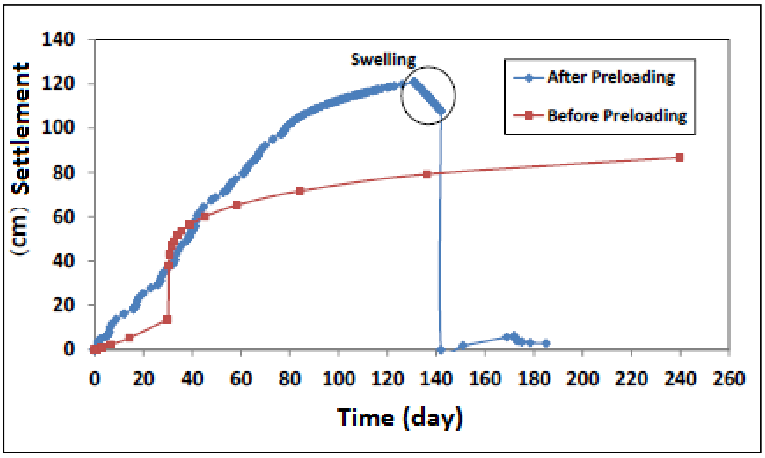

Fig. 7– Graphs of ground settlements before and after preloading (EM-2B in software Plaxis 3D)[25].

5 Case of the new container terminal of DjenDjen port, Jijel province in Algeria 2017

This study aims to verify the stability and strength of the foundations of the caissons and to determine the effect of the preloading method on the ground improvement of the foundations for the North, East and West quay walls (wharf), of Djen-Djen port, in Jijel province, Algeria, during the works of the new container terminal.

5.1 Analysis of the characteristics of the distribution of the soil layers: Container Terminal's wharf caissons area:

The layers are divided into a layer of sand and a layer of marly clay. The sand layer varies from loose to dense; it is saturated, consists of silty sand, sand containing gravel, clayey sand containing a small amount of gravel, etc., with a thickness of ZH (-) 5.2 ~ 16.5m; a layer of gravel is inserted into the layer at BH-3. The marly clay layer varies from very solid to compact appears at ZH (-) 23,16 ~ 27,16m.

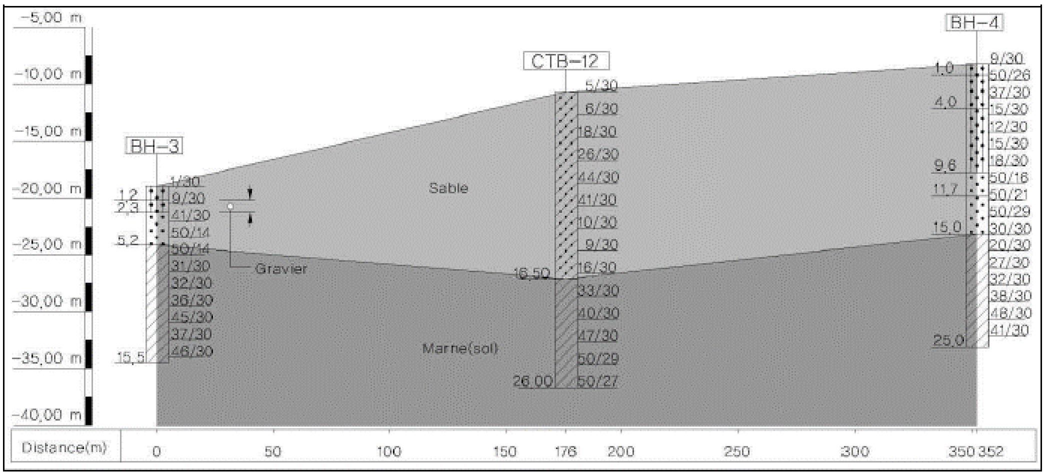

Fig. 8– Current situation of the distribution of the geological layers (CTB-12, A4).

5.2 Analysis of the physical-mechanical and consolidation characteristics:

For the layer of marly clay the water content is distributed in a field of 12.31 ~ 30.00% (average 18.26%), the liquidity limit is distributed in a field of 33.0 ~ 53.0% (mean 41.33%) and the plasticity index is distributed in a field of 14.0 ~ 30.0% (average 19.60%) (figure 9). The content of the fine particles in the marly clay layer being 77,77 ~ 99,17% and its value N being of 21/30 ~ 50/18 (43/30 with the average), we can see that the soil has undrained soil behaviors, however we cannot judge the weakness of the ground.

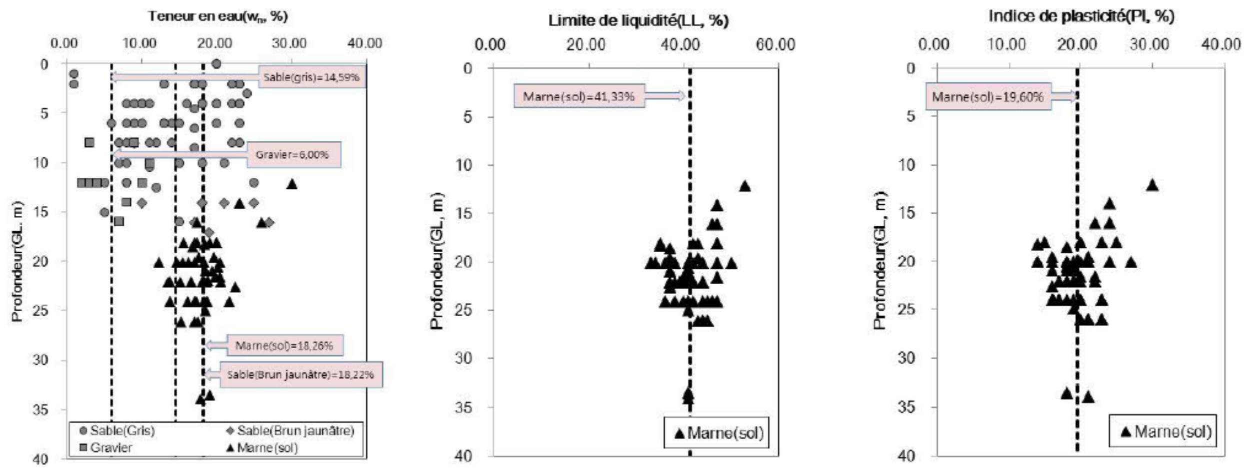

Fig. 9– Water content, Liquidity limit, Plasticity index in zone-A of the port.

From the study of (Md. Wasif Zaman and al. 2016/2017, In Bangladesh) [26, 27], various correlations that will help us determine the consolidation and index properties are suggested. It verifies that there is strong correlation between; compression index (Cc) vs. liquid limit (LL), compression index (Cc) vs. water content (W, %), and compression index (Cc) vs. plasticity index (PI). But shows moderate relation between compression index (Cc) vs. in situ void ratio (e_o), and swelling index (Cs /or Cg) vs. plasticity index (PI).

The engineering parameters that are of importance and how they affect a surcharge-preloading scheme need to be understood for achieving a good and effective design [17]. Results from several index tests obtained for a given site can be used to assess the variation in the properties of the soil mass [28], are aiming to provide a conservative correlation between the effective peak angle of shearing resistance and plasticity index (PI). While the drained angle of shearing resistance Φ'_{oc} is more naturally linked to soil mineralogy composition, as expressed partly by the (PI) value, the apparent effective cohesion c'_{oc} is more naturally linked to the soil structure and dilative tendencies [28].

5.2.1 Standard consolidation test or oedometer test (OED)

Natural soft soil deposits typically display low undrained shear strength and stiffness, high compressibility, low permeability and weak structure as a result of complex physico-chemical interactions that take place during soil deposition [29]. A key aspect for the selection of representative soil parameters is to consider the particular stress path imposed by the loads (or preloading) [30]. Knowledge of the consolidation properties of a soil is important in geotechnical design, particularly as they relate to settlement of structures. The standard consolidation test is based on Terzaghi's one-dimensional consolidation theory (1923) and it has been practiced to determine the consolidation characteristics (compression index Cc, swelling index Cg (or Cs), pre consolidation Load Pc, etc) of the marly clay layer; by using these consolidation parameters, it is possible to determine the compaction, compaction speed, etc., of the uni-dimensionally compressed whole layer when it is loaded.

In general, the settlement caused by the construction of embankments on soft soils is controlled by: (1) the overconsolidation ratio (OCR or YSR), (2) the coefficients of consolidation (C_v and C_h), (3) the compressibility index (Cc), (4) creep effects (e.g., Ca) and embankment geometry [29]. While this may often be overlooked, the rigorous selection of soil parameters requires a deep understanding of soil behaviour and proper knowledge of in situ and laboratory testing techniques [31]. Parameters such as OCR, Cv, Cc and Ca play a key role on settlement and pore water pressure predictions.

OCR and Cc control the maximum settlement, whereas Cv and Ca control the dissipation of excess pore water pressure and the settlement rate. As expected, only C_v has a major influence on the predicted pore water pressure [29].

Table 3: Results of the standard consolidation test of three points of this study.

N°	Caisson Type	Depth (m)	Initial Void ratio, e_0	Precompression Load, Pc (kPa)	Compression Index, Cc	Swelling Index, Cg	OCR
CTB-11	A14	19.0~20.0	0.602	245	0.1167	0.0482	1.488
CTB-12	A4	18.0~19.0	0.576	301	0.1534	0.0501	2.027
CTB-13	B3	21.0~22.0	0.688	344	0.15	0.0437	1.850

Based on the results of the consolidation tests (figure 10) on the port area (A), the preconsolidation load is 182 ~ 344 kPa (mean 277.17kPa), the compression index of 0.0067 ~ 0 , 1534 (mean 0.1050) and the swelling index of 0.002 ~ 0.050 (mean 0.0338). The over-consolidation ratio is 0.747 ~ 2.027 (1.428 average). According to Clemence & Finbarr (1980) [32] the soil is considered normally consolidated when the over-consolidation ratio (OCR) is 0.8 ~ 1.5. It is therefore estimated that the marly clay layer is in a normally consolidated state.

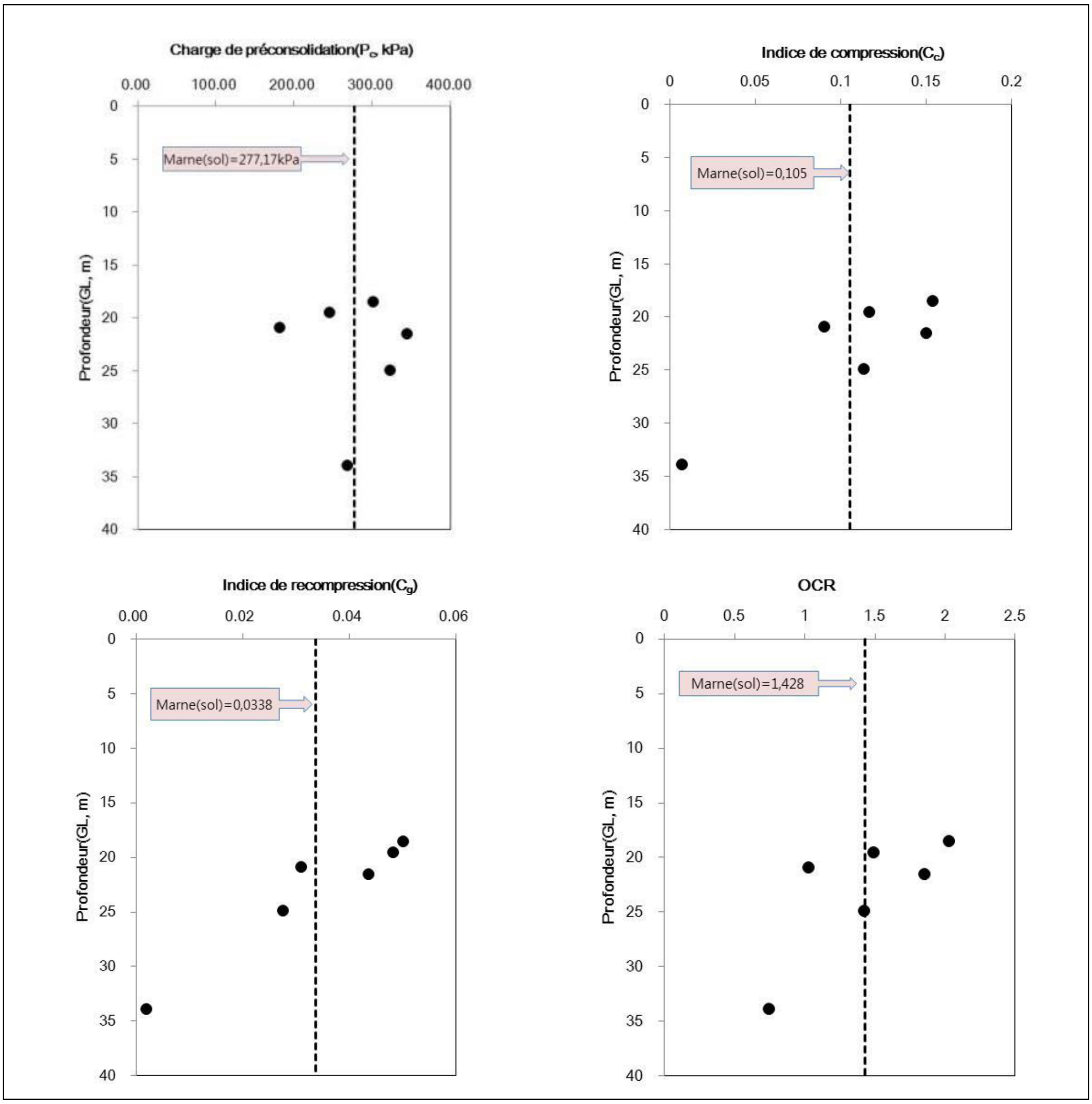

Fig. 10– Pre-consolidation load (Pc), compression index (Cc), swilling index (Cg) and OCR in the zone A of the port

5.3 Two-dimensional numerical analysis and in-situ measurements

The stability study for this project is based on the assumption that the soils near the quay walls are a continuity to make possible the realization on model of the surface of transmission, and to carry out the analysis of elastoplasticity respecting the rate of plane deformation. The modeling was carried out with ordinary properties (Applied model: Mohr - Coulomb model, linear elastic model for reinforced concrete, and Allowable margin for differential settlement: 1/300 (Bjerrum, 1963 [33]: functional defect of the structure) (Figure 11, Table 4). Mesh analysis is applied uniformly across all sections to be studied, and in order to accurately assess the changes caused by the stresses and deformations in the vicinity of the caissons, analysis points have been defined at key locations The extent of the analysis and the boundary conditions refer to the theory of elasticity, were determined by expanding the field of study until there was no further change due to stresses and deformations during dredging and pre-loading. It is necessary to define beforehand the level of settlement (20.0 cm) and the bearing capacity (526.0 kN / m²) of the soil applied, for the soil of the site are those provided by the geotechnical campaign (soil's in-situ tests and survey). Furthermore, the safety factor with respect to the applied failure is 1.0.

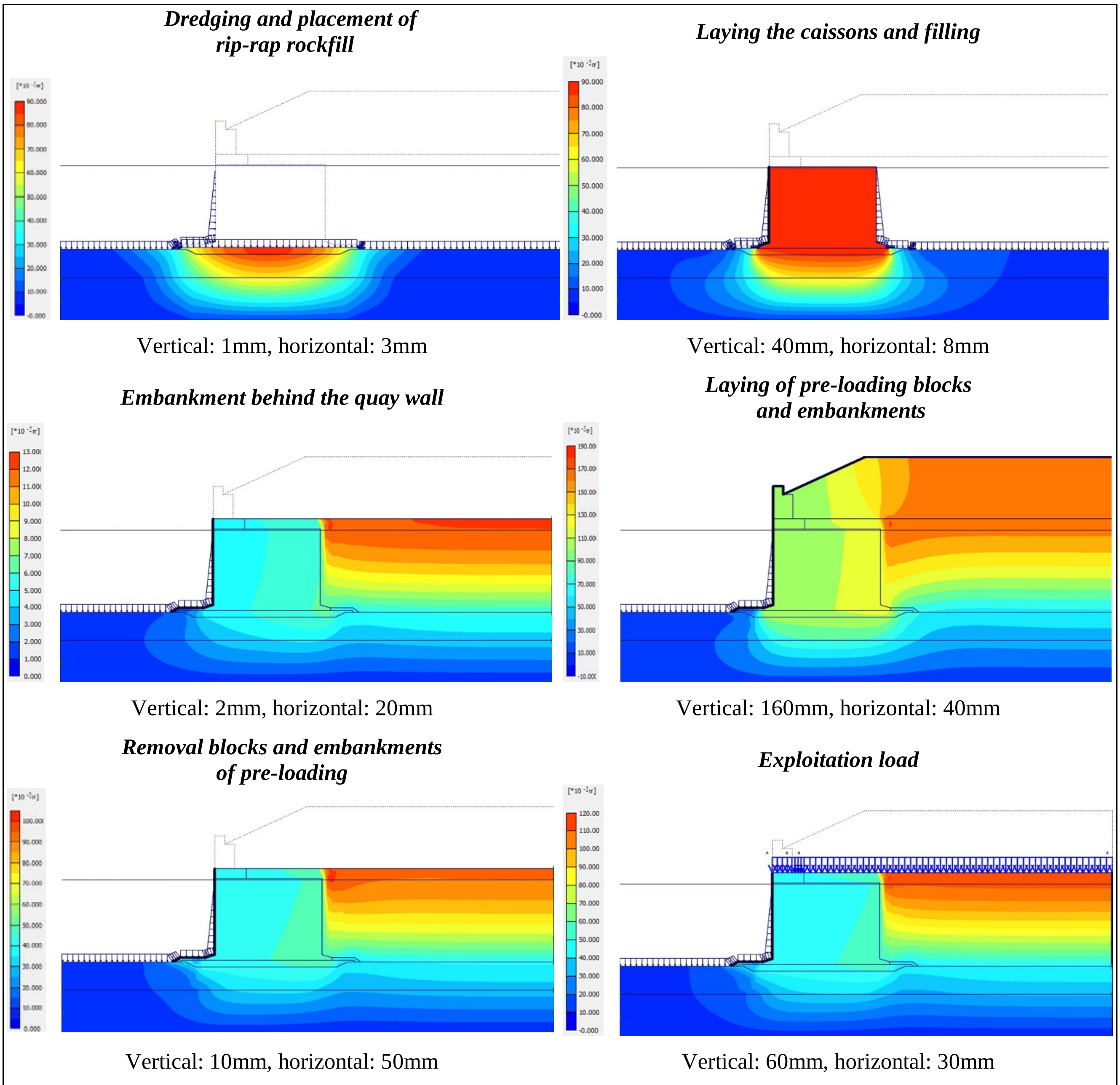

Fig. 11– Results of marine subsoil displacement by stage of quay wall works (in software Plaxis 2D)[7]

We started the operation of pre-loading by the blocks and embankments, but unfortunately the suction dredge was disrupted, which forces us to replace the pre-loading embankment with concrete blocks as illustrated in the Figure (18) of Appendix (A). The settlement expected during construction is 0.15 to 0.16m. A monthly settlement check of the caisson above our actual treated soil; carried out until 25/05/2017 by the BCS of the project; found an average of 14.80 cm of settlement Figure (17 b); illustrated in Table (4). This difference in displacement is due to the effect of the soil treatment (Preloading), giving an increase in bearing capacity and an improvement of the compactness (density) of the soil which becomes denser and which has a great effect on the settlement and the deformation of the soil. Since the removal of the preload blocks to the construction of the crown beam and its accessories, we have not noticed any settlement or geotechnical problems encountered, which gives the high reliability of this marine subsoil treatment method. This gives great credibility to our two-dimensional modeling.

Table 4: Evolution of caisson settlement type "A14" (West Quay) until 25 /05/ 2017, "B3" (EAST Quay) until 16 /06/ 2016, and type "A4" (North Quay) until 14 /02/ 2017.

Caisson points		*Level before filling (m)*	*Level after filling (m)*	*Level after preloading (m)*	*Level after embankment (m)*	*Blocks preloading delay (days)*	*Settlement after filling (cm)*	*Settlement blocs preloading (cm)*	*Settlement embankment (cm)*	*Final settlement (cm)*
1	*A14*	*0,639*	*0,547*	*0,512*	*0,511*	*4*	*9,2*	*3,5*	*0,1*	*12,8*
	B3	*0,64*	*0,578*	*0,541*	*-*	*11*	*6,2*	*3,7*	*-*	*9,9*
	A4	*0,66*	*0,55*	*0,52*	*0,488*	*15*	*11*	*3*	*3,2*	*17,2*
2	*A14*	*0,646*	*0,568*	*0,505*	*0,503*	*4*	*7,8*	*6,3*	*0,2*	*14,3*
	B3	*0,69*	*0,636*	*0,596*	*-*	*11*	*5,4*	*4*	*-*	*9,4*
	A4	*0,632*	*0,54*	*0,538*	*0,504*	*15*	*9,2*	*0,2*	*3,4*	*12,8*
3	*A14*	*0,623*	*0,561*	*0,489*	*0,489*	*4*	*6,2*	*7,2*	*0*	*13,4*
	B3	*0,68*	*0,632*	*0,583*	*-*	*11*	*4,8*	*4,9*	*-*	*9,7*
	A4	*0,661*	*0,54*	*0,53*	*0,498*	*15*	*12,1*	*1*	*3,2*	*16,3*
4	*A14*	*0,626*	*0,549*	*0,507*	*0,505*	*4*	*7,7*	*4,2*	*0,2*	*12,1*
	B3	*0,62*	*0,537*	*0,491*	*-*	*11*	*8,3*	*4,6*	*-*	*12,9*
	A4	*0,65*	*0,55*	*0,538*	*0,518*	*15*	*10*	*1,2*	*2*	*13,2*
Means settlement	*A14*						*7,725*	*5,30*	*0,17*	*13,15*
	B3						*6,18*	*5,73*	*-*	*10,48*
	A4						*10,575*	*3.93*	*1.35*	*14,88*
Maximum settlement	*A14*						*9,2*	*7,20*	*0,20*	*14,30*
	B3						*8,30*	*4,90*	*-*	*12,90*
	A4						*12,1*	*3,00*	*3,40*	*17,20*

On the basis of the AMBRASEY law and the Algerian anti-seismic standards, the examination was carried out in sections susceptible to liquefaction (sand above the layer of marly clay) in order to know if the results answer the safety factor of reference of 1.25. The results obtained according to the criteria mentioned above confirm the liquefaction potential as a function of the depth given the weight of the quay wall, the zone where the sand layer remains corresponds to the safety factor of reference (1,25) (figure 12).

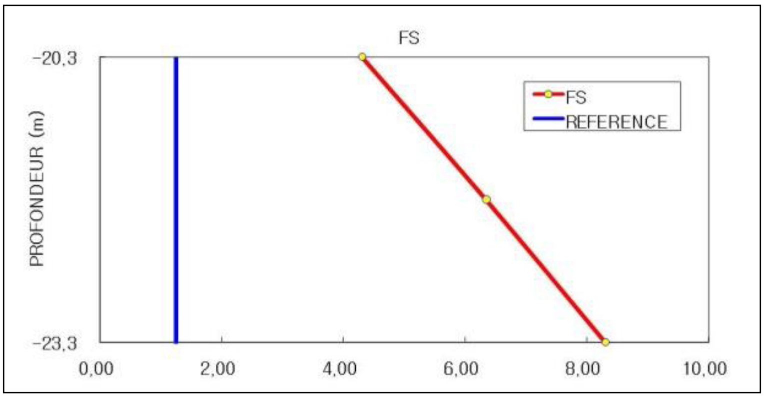

Fig. 12– Safety Factor to liquefaction of the sand layer above the marly clay layer according to the depth.

6 Results and discussion

The filling caused consolidation of the sand layer and the rockfill (rip-rap) foundation, which explains the abrupt settlement at the beginning of pre-loading. we can see a consolidation behavior in a similar way, which explains why the increase of the four (04) graphs of the settlements is almost the same (figure 13), only there is a small difference in settlement and time values between the four graphs until stabilization, caused by the variation in the soil index properties. The same pre-consolidation load was actually used, but each zone needs a pre-consolidation load and a defined time and each zone has different characteristics (index properties), that is why we obtain a difference in time for the stabilization of settlement. If we compare the actual settlement curves (A4) and the numerical modeling (plaxis) of the same area (CTB-12), we can observe a similar behavior, but with two differences; the first is the amount of pre-loading settlement because the software cannot really simulate at one hundred percent consolidation phenomenon without making uncertainties between real and digital data, the second is the abrupt settlement (6 cm) caused by the container terminal operating load which is not yet applied in our real case; which gives us a forecast of future settlement of outstanding quay wall operation, and we can be considered acceptable.

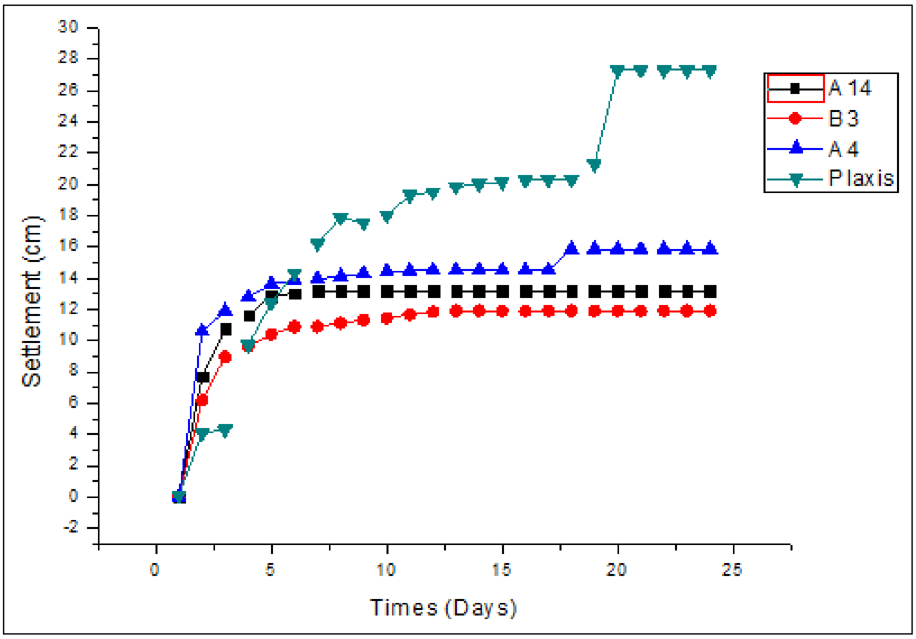

Fig. 13– Comparison of settlement curves of two-dimensional modeling and in-situ measurements of three caissons as a function of time during pre-loading

The graph (B3, CTB-13) give a settlement amount of 11.91 cm with a swelling index of Cg = 4.37%, the graph (A14, CTB-11) give 13.195 cm with a swelling index of Cg = 4.82%, and the graph (A4, CTB-12) give 15.855 cm with a swelling index Cg = 5.01% of pre-loading in order to stabilize its settlement. Therefore it can be concluded that the amount of settlement depends on the swelling index (Figure 14, Table 3), and that the amount of settlement (Su) is proportional to the swelling index (Cg/ or Cs).

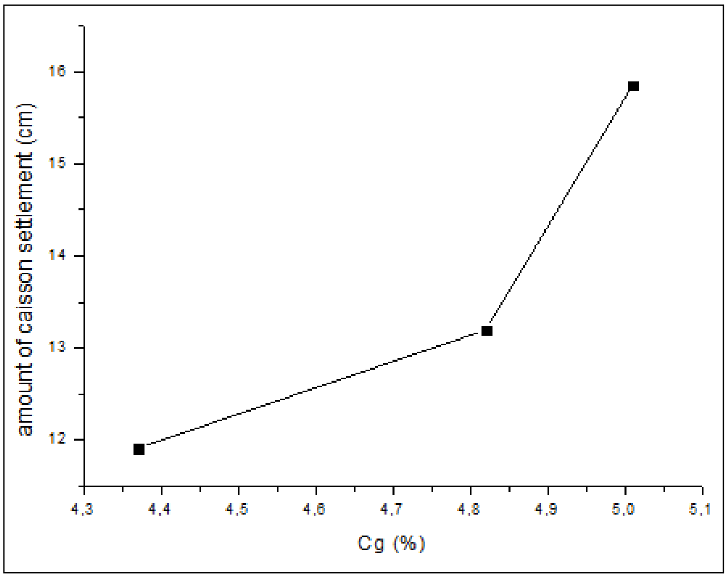

Fig. 14– Amount of settlement (Su) Vs swelling index (Cg/ or Cs)

The graph (B3, CTB-13) takes 11 days (with a compressibility index Cc = 15%, and over consolidation ratio OCR = 1.850) of pre-loading in order to stabilize its settlement, the graph (A14, CTB-11) give 4 days (with a compressibility index Cc = 11.67%, and over consolidation ratio OCR = 1.488) of pre-loading in order to stabilize its settlement, the graph (A4, CTB-12) give 15 days (with a compressibility index Cc = 15.34%, and over consolidation ratio OCR = 2.027) of preloading in order to stabilize its settlement. When we compare the time required to stabilize the settlement, we can OCR and Cc. Therefore it can be concluded that the times of settlement depends on the compressibility index (Cc) and over consolidation ratio (OCR) (Figure 15, Table 3), and that the times of settlement (T) is proportional to the compressibility index (Cc) and over consolidation ratio (OCR), and at the same time that the two latter are proportional.

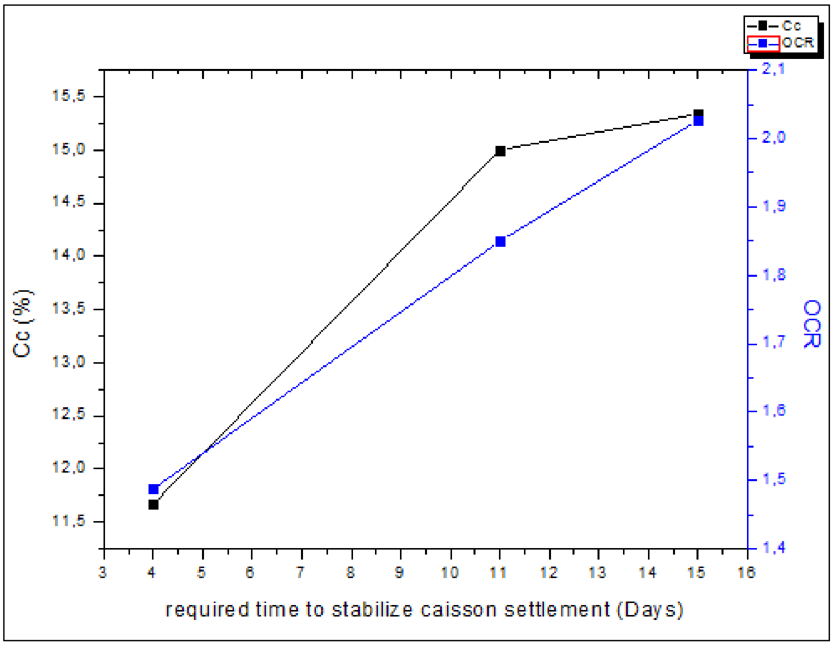

Fig. 15– Required time to stabilize caisson settlement Vs. OCR Vs. Cc

As illustrated in Figure (16) according to our study; the consolidation phenomenon has been summarized in a consolidation cycle which consists of three basic components, pre compression stress (Pc), times (T) and settlement (Su), that they are related to each other by three main index properties, over consolidation ratio (OCR), compression index (Cc) and swelling index (Cg).

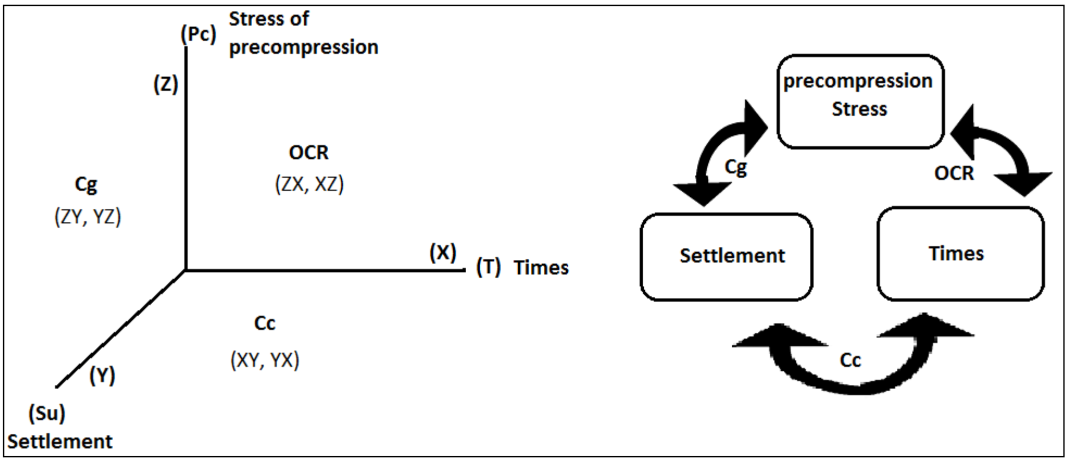

Fig. 16– The soil consolidation cycle and its representation on the Cartesian coordinate system according to our hypothesis

If we take all the previous data and since we proved that indexes and components are linearly dependent we can easily obtain a **consolidation matrix**, which clearly illustrates that three basic components are the diagonal and the other three main index properties compose the rest of the matrix:

$$\begin{pmatrix} XX & XY & XZ \\ YX & YY & YZ \\ ZX & ZY & ZZ \end{pmatrix} = \begin{pmatrix} T & Cc & OCR \\ Cc & Su & Cg \\ OCR & Cg & Pc \end{pmatrix} \text{ Consolidation Matrix}$$

7 Conclusion

Marine and coastal structures may break during construction or even in service, particularly on muddy and soft soils characterized by low bearing capacity, or excessive generalized, localized or differential settlements. The final situation of the structure is not necessarily the most critical, and particular attention must be paid to the identification and description of all critical situations that may arise during construction work.

The pre-loading technique consists of densifying and increasing the compactness of the soil in order to improve their Physical -mechanical characteristics by applying the vertical compressive load in place. The effectiveness of this method of soil treatment was demonstrated by the results of the available laboratory tests and in-situ monitoring inspections which verified the settlement of the soil support before and after the completion of the treatment. In addition, this treatment to minimize the risk of liquefaction and quay wall instability, in addition to the reasonable cost advantage compared to the importance of the project, thus no negative effects have been reported on the environment. In conclusion, preloading gives very satisfactory results in terms of marine soil improvement.

In this paper, from our study, a consolidation matrix was proposed during pre-compression. This matrix gives us a new method to facilitate the calculations of the parameters involved in the consolidation, which gives a great credibility to our hypothesis.

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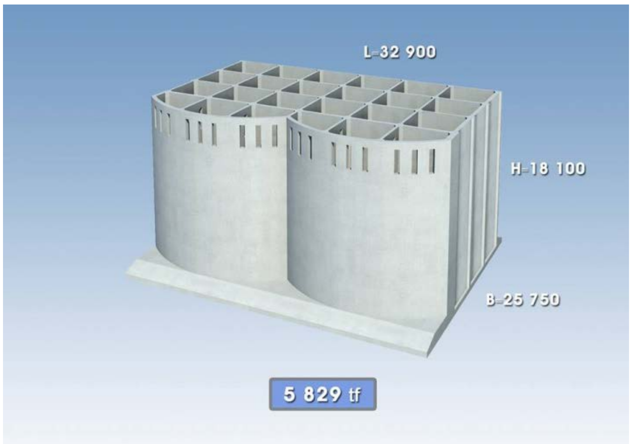
Appendix A.

Table 5: **The blocks number to put for each type of caissons.**

Caisson	Pre-loading load (kN)	Number of blocks (U)	Dimension of blocks (m^3)	Volumetric mass of blocks concrete (kN/m^3)
Type-A	146,966.40	378	4.5Lx2.0Bx1.8H	24
Type-B	100,310.40	258	4.5Lx2.0Bx1.8H	24



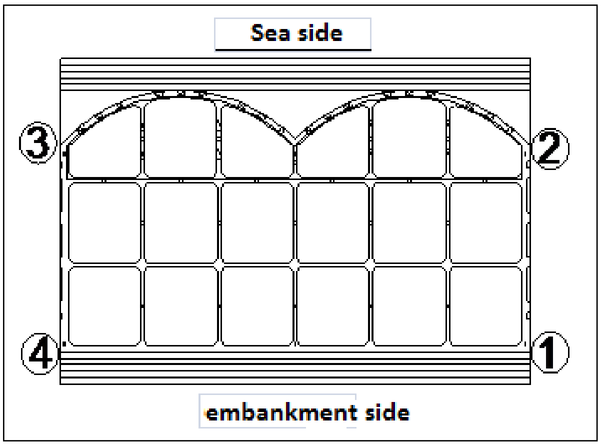


Fig. 17– (a) Caisson (type A) concerned by the study [34], (b) the four points above caisson (type B) for settlement monitoring

Table 6: The values and properties of the Soil- Quay wall profile (CTB-12, A4) simulated in this study

Description	Sand	Marly Clay	Foundation rip-rap	Carapace	Caisson and Preload Blocks	Sand for filling and preloading
Model	M.C Undrained	M.C Undrained	M.C	M.C	L.E	M.C
γ_{SAT} (kN /m ^{3})	18	20	19	21	23	21
ϕ' (degré)	32	15	42	40	-	38
C' (KN /m^2)	10	243,8	0	0	-	0
E (KN /m^2)	6,4×10^3	6,45×10^4	5,0×10^4	5,0×10^4	2,74×10^7	4,0×10^4
υ	0,33	0,488	0.2	0.2	0.167	0.35
WL (%)	-	38.9	-	-	-	-
WP (%)	-	20.2	-	-	-	-
IP (%)	-	18.7	-	-	-	-
G_d (KN /m^2)	4,96×10^4	2,26×10^5	-	-	-	-
E_d (KN /m^2)	1,48 ×10^5	6,71×10^5	-	-	-	-
K_d (KN /m^2)	4,16 ×10^6	7,81×10^6	-	-	-	-

Fig. 18– Photos of the quay wall

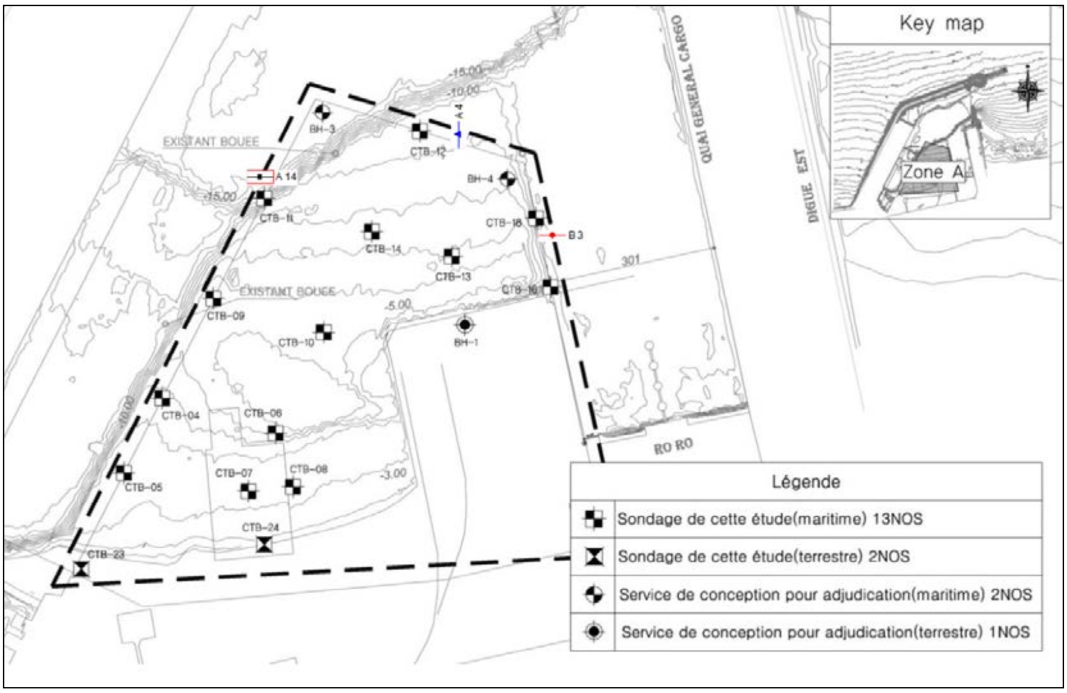


Fig. 19– Location surveys.

A.1. The three points oedometer test (OED) of this study:

The standard consolidation test is a consolidation test which gives a consolidation load with a load increase rate of 1 on a specimen ø 60mm, height 20mm; the test remains at each loading step for 24 hours before proceeding to the next loading step.

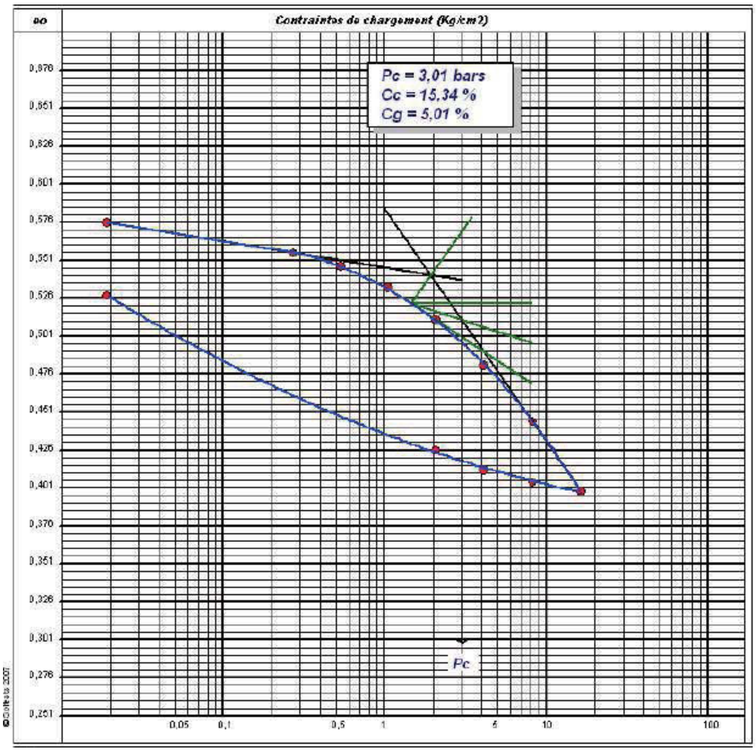


Fig. 20– (a) OED of CTB-12

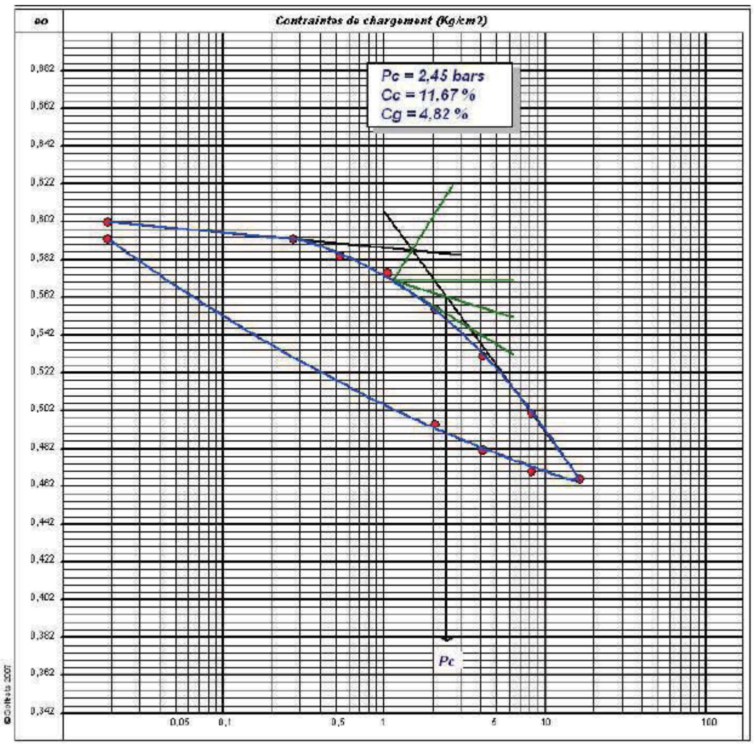


Fig. 20– (b) OED of CTB-11

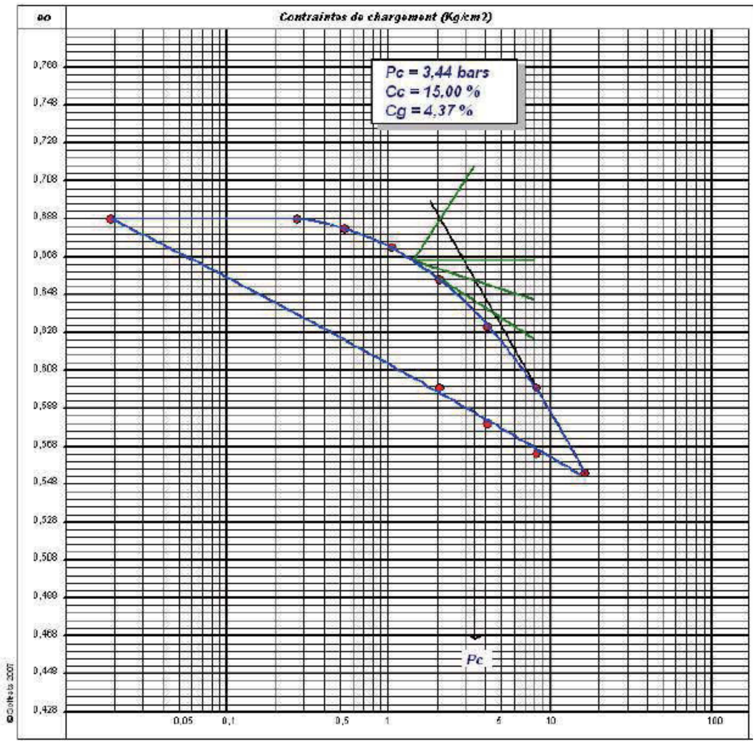


Fig. 20– (c) OED of CTB-13.

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