Land reclamation and soil improvement works for two deep water ports in Vietnam

Abt Herveⁱ⁾

i) Proposal Director, MENARD Asia, 2-1 & 2-2 Jalan USJ 10/1E, 47 620 Subang Jaya, Selangor DE, MALAYSIA.

ABSTRACT

In the mid to end 2000, Vietnam embarked in the construction of a series of deep water ports along the Cai Mep Thi Vai river, in the province of Vung Tau, close to Ho Chi Minh city. Those ports were partly built on the tidal plain and partly reclaimed from the river. Two such ports Cai Mep International Terminal (CMIT) and GEMALINK, close to the mouth of the river, were showing a clay layer with very poor mechanical property of respectively 36 and 42 m. It was not possible to reclaim the 6 m of sand fill needed to put the container yard above the 100 years flood, without some sort of reinforcement/consolidation of the clay layers; Two problems were encountered, one of stability at the edge of the reclaimed land, the other one of the bearing capacity and long term settlement. The initial solution along the edge envisaged by the client was a block of soil mixing, but both the cost and the time needed were not satisfactory and the EPC-contractor call for an alternative solution. Such alternative, implemented successfully, consisted in increasing the shear capacity by forced consolidation of the clay using Vacuum Consolidation. The case histories of the two projects will be presented.

Keywords: soft clay improvement, embankment stability, vacuum consolidation,

1 INTRODUCTION

The author participated in the definition and follow up of two container port projects in Vung Tau, VIETNAM, the Cai Mep International Terminal (CMIT) and the GEMALINK port, and more particularly the stabilisation / settlement control of the river bank. Both ports were build along and close to the mouth of the Cai Mep / Thi Vai river.

The port consisted in both case of a concrete deck on piles, linked to the hinterland with deck on piles. The container yard was reclaimed from the tidal plain, mainly a soft clay deposit of respectively 36 and 42 m thick. The finish level, planned to be above the 100 years flood, implied to raise the natural ground level by 3 to 6 m along the river bank; under such load, the stability was not ensure and the level of settlement expected was in the range of several meters, not compatible with the smooth operation of the port.

The solution initially considered in both cases was to create a block of soil mixing.

2 PROJECT DESCRIPTION

The two ports were similar in their concept, in their soil conditions and their reception criteria. We will described in detail the conditions of the GEMALINK project, where the soil conditions were the more dramatic.

2.1 Project layout and main features

As shown in Fig. 1, a 800 m deck is linked to the container yard by three bridges, from 100 to 150 m away from the edge of the platform; Along the deck, the river bed at CD -5~-10 was to be dredged to CD -15.5 to allow for 160,000 DWT vessels. The offset of the berth allowed the reduction of dredging quantities, as well as to accommodate a relative gentle slope (No steeper than 1/6, compared to the existing 1/10).

Fig. 1. Project Layout

Out of the 73 ha hinterland, only 33 ha were developed in the first phase as the container yard and general port facilities. In order to increase the bearing capacity and to reduce the post construction settlement, the whole area was treated by vertical drain and surcharge, increasing further the need of

stabilization along the river bank during the application of the temporary surcharge.

2.2 Ground conditions

The soil investigation campaign revealed the presence of a very soft (layer 1) to soft clay (layer 2) of poor mechanical properties to CD -40, followed by a layer of sand in a loose to dense conditions. Granite was found at CD -50 m to -70 m. The properties of the first two layers are given in table 1.

Table 1. Stratigraphy.

Description	Symbol	Unit	Layer 1	Layer 2
Top El		m CD	+0.5	-10
Thickness		M	10.5	30
Wet Density	γ	g/cm ³	1.52	1.59
Void ratio	e_0		2 to 2.6	1.7 to 2
Primary compression index	C_c		1.0	0.8
Secondary Compression index	C_α		0.025	0.021
Consolidation Coefficient	C_v	m ² /year	1.26	1.02
Intrenal Friction Angle ϕ_u (Undrained)		°	0	0
Cohesion (Undrained) C_u		kPa	7.5	15 to 45 ¹
Intrenal Friction Angle ϕ' (Drained)		°	25	23
Cohesion (Undrained) C'		kPa	5	10
SPT blows count	N_{SPT}		0	2 to 7

¹: Increase of cohesion in direct proportion to depth

Layer 1 and 2 are normally consolidated (OCR=1)

2.3 River water levels

The tidal levels recorded at Vung Tau are summarized in Table 2:

Table 2. River Water Levels

Tide Levels	Abbreviation	Level (m CD)
Highest High Water Level	HHWL	+4.43
High Water Level	HWL	+3.97
Mean Sea Level	MSL	+2.67
Low Water Level	LW	+0.58
Chart Datum	CD	+0.00
Lowest Low Water Level	LLWL	-0.47

2.4 Design load and design criteria

A design life of 20 years post hand-over of the port was specified, period after which the finish level of the platform along the river bank should not be lower than CD +5.7 m (and CD + 6.3 m hinterland). Considering a construction period of 12 months for the soil improvement out of the 3 years allowed for the construction, the residual settlement was considered for a period of 22 years post soil improvement. Considering other criteria related to the settlement of other part of the site, the allowable 22 years settlement was determined not to exceed 60 cm.

More than the residual settlement, the defining

criteria was the overall stability. Eurocode 7 (design approach1) was used, with an overall safety factor of 1.35 for short term and long term, partial factor of 1.0 for permanent action, 1.11 for variables actions and 1 on materials. For long term, earthquake load was included ($a_g = 0.044g$), as additional vertical and horizontal force (Eurocode 8).

3 SOLUTION PROPOSED

3.1 Existing solution

The existing solution consisted in a Deep Soil Mixing (DSM) block as illustrated in fig.2, to the full depth of the soft clay and a width of 42 m, completed by a smaller block 30 m wide, but only 16 m high to ensure the stability while the hinterland was treated by vertical drain and preload to a depth of 10 m only.

Fig. 2. Initial solution for slope stability, using DSM

The solution proposed was having a substantial impact on the overall cost of the project and needed the use of equipment (marine means) that was not yet available in Vietnam.

3.2 Description of solution proposed

We considered the effect of the DSM block, which was in fact to force the slip failure down to the layer 3 and propose instead to create such effect by creating a block of vacuum (temporary stage) and an increase of the shear strength of the clay by pre-consolidation (long term) as illustrated in fig. 3.

The treatment of the hinterland was kept unchanged to the initial solution: short vertical drain, albeit we proposed as well to have a transition with the vacuumed area by reducing progressively the length of the vertical drain.

Fig. 3. Alternative solution for slope stability, using vacuum

We introduced as well a stepped increase of the surcharge, both on top of the vacuumed area and the vertical drain with preload of the adjacent block.

3.3 General principle of the Vacuum:

The method consists in using the atmospheric pressure to create a load onto the block of clay to be consolidated.

It starts with installing vertical drain that will both allow to drain out the water of the clay but also to transmit the vacuum to the full block to be treated.

The vertical drains terminate in the upper sand blanket, where a network of horizontal drains is laid. There is no direct connection between the drains, the hydraulic continuity being ensured by the sand blanket. The sand blanket is covered by a membrane (delivered in roll and welded in situ) that is tucked in its periphery into the native clay, thus creating a complete encapsulation of the block. The horizontal drains are connected to water/air pumps that ensure the evacuation of the percolating water while maintaining a vacuum below the membrane.

Vacuum load applies from all sides of the block; there is no deviatoric stress, as compared to a surcharge, vacuum promotes stability of the block as illustrated in fig.4 below:

Fig. 4. Stress Path using a surcharge and vacuum preloading.

4 DESIGN OF THE SOLUTION

4.1 Design principles

In term of settlement, the design principle consists in reducing the initial void ratio to a targeted value that will ensure an acceptable long-term settlement and comprise the following steps:

- Determining the initial void ratio (e_0)
- Determination of the primary and creep settlement during the design lifetime and the corresponding void ratio (e_1)
- Define accordingly the temporary load (vacuum pressure with temporary surcharge) and the necessary consolidation ratio in order to reach a lower void ratio (e_2 , targeted void ratio)

- Define the vertical drain mesh considering the available consolidation period.

There are usually several iterations before the optimized solution is found.

In term of stability, the increase of shear stress is proportional to the degree of consolidation and the applied load.

4.2 Consolidation calculation

The primary settlement depends on the initial stress and the additional one applied. For normally consolidated clay :

$$S_c = \frac{H \cdot C_c}{1 + e_0} \cdot \log\left(1 + \frac{\Delta P}{P'_0}\right) \quad (1)$$

Where: S_c is the primary settlement
 H is the thickness of the clay
 P'_0 is the initial effective stress
 ΔP is the additional imposed load

The secondary settlement is the additional settlement that can be observed at the end of the primary consolidation. It is independent from the load applied and only time dependent.

$$S_{sc} = \frac{c_\alpha}{1 + e_0} \cdot H \cdot \log\left(\frac{t}{t_p}\right) \quad (2)$$

Where: S_{sc} is the secondary settlement
 H is the thickness of the clay
 t is the time at which the secondary settlement is calculated
 t_p is the start time of secondary settlement

The speed of the consolidation depends on the length the water has to travel (drainage length) and the permeability of the clay. The global consolidation is divided into a vertical and horizontal terms, as per Carillo theory and Hansbo.

$$1 - U_{\%} = (1 - U_v) \cdot (1 - U_r) \quad (3)$$

U_v and U_r are the vertical and horizontal consolidation ratio and are calculated as per (4) and (5).

$$U_v = \left(\frac{1}{1 + \frac{1}{2T_v^3}} \right)^{1/6} \quad (4-1)$$

$$\text{With: } T_v = \frac{C_v \cdot t}{H_d^2} \quad (4-2)$$

And for the horizontal consolidation:

$$U_r = 1 - e^{\left(\frac{-8T_h}{F(n,r)} \right)} \quad (5-1)$$

$$\text{With: } T_h = \frac{C_h \cdot t}{D^2} \quad (5-2)$$

$$F_n = \frac{n^2}{n^2 - 1} \cdot \ln(n) - \frac{3n^2 - 1}{4n^2} \quad (5-3)$$

$$Fr = \left(\frac{k_h}{k_s} - 1 \right) \cdot \ln\left(\frac{d_s}{d_m}\right) \quad (5-4)$$

Where:

- t is the time at which the consolidation ratio is calculated
- H_d is the vertical drainage length
- n = D/d_w
- D is the effect area diameter of a single drain (D=1.05 L, for a square mesh of spacing L)
- d_w is the equivalent diameter of a single drain
- d_m is the equivalent diameter of the mandrel
- d_s is the diameter of the smear zone, area disturbed by the mandrel (d_s/d_m = 2.5)

4.3 Stability calculation

The stability was calculated with the finite element software Talren using Bishop's method. The increase of undrained shear strength was obtained at each stage using (6):

$$\Delta C_u = U_{\%} \cdot \lambda C_u \cdot \Delta P' \quad (6)$$

4.3 Adopted sequence

A rectangular mesh of 0.9 m was retained for the vertical drain and the sequence described in table 3 was adopted for the vacuum area:

Table 3. Sequence of Works

Step	Time (days)	Description
Step 1	235 to 285	50 days of reclamation without installation PVD
Step 2	285 to 345	Installation of PVD and Vacuum system
Step 3	345 to 370	Building up the vacuum pressure from 35 to 65 kPa
Step 4	370 to 400	Building the sand filling height up to C.D. +7m
Step 5	400 to 430	Building the sand filling height up to C.D. + 9m
Step 6	430 to 525	Building the sand filling height up to C.D. + 11.5m
Step 7	525 to 1080	Stop Vacuum system and wait for consolidation till Hand-over
Step 8	1080 to 8380	Calculation in 23 year (long-term)

The stability was checked at the beginning of each step, taking into consideration the gain of shear strength from the preceding step.

5 IMPLEMENTATION

5.1 Actual sequence

The proposed sequence was carried out, albeit

the installation of the surcharge on the hinterland part was not as quick as expected (difficulty to source sand timely).

5.2 Actual Settlement

As shown in Fig. 5, the forced settlement was in excess of 5 m, in accordance with the estimate of 5.0 to 5.5 m. The control of actual consolidation ratio was carried out using Asaoka method (using the settlement plates and extensometers) and the decrease of the pore water pressure (using the pore pressure cell), thus giving confidence in the results before the vacuum pumping is stopped.

Fig. 5. Settlement function of time and additional surcharge.

5.3 Shear strength gain

The gain in shear strength was verified after treatment and stopping of the vacuum, using CPTu and vane shear tests; the gain was higher than expected, as shown in fig. 6.

Fig. 6. Post-treatment shear strength.

6 CONCLUSIONS

Vacuum consolidation is quite often considered only as a replacement to a conventional surcharge. However, considering the inherent stability that it provides during its implementation and the

acceleration of the consolidation, thus the gain in shear strength for a clay or a silt, other application may be considered.

The method has been proposed in lieu of a soil mixing block for the stabilization of a river bank in two port projects in Vietnam, namely the CMIT and the Gemalink container terminals.

Wherever there is a problem of stability at the edge of a reclamation, which is often the case, the solution should be given a look, as much as other more conventional solution such as the Deep Soil mixing or the installation of stone columns.

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