Geotechnical Engineering for a new container terminal in Lomé, Togo, West Africa

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ABSTRACT
A completely new large container terminal in Lomé began in 2014 to handle a large number of containers. The container berth is 1.05 km long with water depth of 16.7 meter. The project includes construction of an area of 55 ha for container stacking and building associated works. The purpose of the container terminal is primarily transshipment of containers between large container vessels from Asia /Europe and smaller container feeder vessels serving other ports in West Africa.
In 2011 a geotechnical investigations program with geotechnical boreholes, laboratory testing and CPT (Cone Penetration Tests) was performed. The geotechnical investigations revealed a tricky deep layer of clay under 15 to 20 m of recently deposited sand due to littoral drift along the coast.
The article will focus on a comparison of the assessment in the design phase of the conditions for foundation of the container berth with actual observations during and after construction.
The article is intended to address the following features:

- *Relative density of sand and need for deep compaction*
- *Vertical settlements*
- *Design of diaphragm quay wall*
- *Monitoring of quay wall and track for quay side cranes*

Keywords: Retaining structures, diaphragm quay wall, deep compaction, monitoring of deformations.

Figure 1 Lomé Container terminal in operation. April 2015.

1. INTRODUCTION

Figure 2 Site view. Port of Lomé with terminal site in background. 2011.

The container berth is 1.05 km long with water depth of 16.7 meter. The project included construction of an area of 55 ha for container stacking and associated building works. The purpose of the container terminal is primarily transshipment of containers between large container vessels from Asia /Europe and smaller container feeder vessels serving other ports in West Africa. The port serves also as a gateway to the landlocked countries of Mali, Niger and Burkina Faso and to the northern areas of Nigeria.

The site is almost entirely located in an area which since the construction of Lomé Port in the 1960'es has developed by progression of the shore line due to accumulation of drifting sand. In the quay alignment roughly 13 m of recently deposited sand was recorded with its present ground level at approximately the high water level of the tide.

The employer for the project was Lomé Container Terminal (LCT) with Cyes_Somague JV as the main contractor and responsible for the detailed design. NIRAS prepared tender documents for an EPC-contract and carried out technical supervision of the construction.

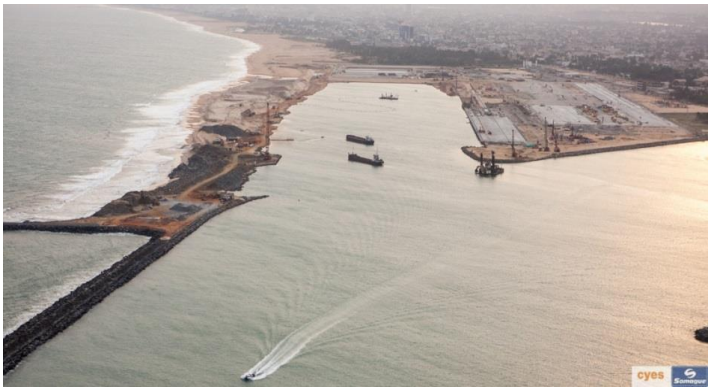

Figure 3 Site with new basin and quay wall (right hand side of basin). Autumn 2013.

2. GEOTECHNICAL INVESTIGATIONS

A preliminary ground investigation program had been undertaken during November 2008 to July 2009 for initial project planning , comprising 7 geotechnical boreholes and 34 dynamic probes.
Before tendering a more extensive investigation was carried out, see Figure 4, including:

- 21 boreholes to depth ranging from 17 m to 51 m.
- 85 CPTU (Cone Penetrations Tests with pore pressure measurements) to depths ranging from 13m to 48 m.
- Laboratory testing for classification and soil strength parameters.

Figure 5 and Figure 6 show the drilling and the CPT rigs that were mobilized for the investigations in 2011. CPT-equipment and counterweight were installed on a sledge towed by a Caterpillar. This vehicle proved convenient for access all over the site.

Later the selected contractor carried out additional investigations as basis for his detailed design.

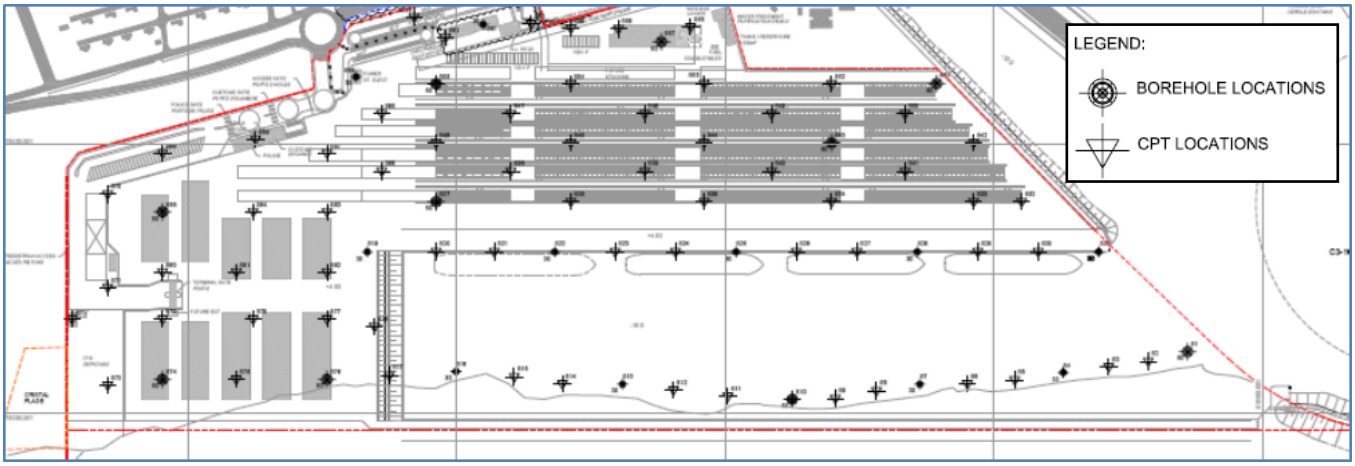

Figure 4 Plan of pre-tender soil investigations 2011.

Figure 5 Geotechnical borehole with an interim dam to the left for protection of sea waves. March 2011.

Figure 6 Cone Penetration Test (CPTU) equipment (20 ton capacity). March 2011

3. SOIL CONDITIONS

A typical cross section in the main quay wall is shown in Figure 7which also includes references to the soil layers A-F. Little variation across the site was detected.

The geotechnical parameters adopted for the layers are summarized in Table 1.

Table 1 Geotechnical parameters.

Layer		γ (kN/m³)	c' (kPa)	φ' (°)	E (MPa)
B	Sands (SP)	19	0	37	40
C_0	Silt and Silty-Clay (SM to SC)	20.2	0	30	10
C_1		20.1	15	22	8
C_3		20.1	45	23	15
D_1	Silty sands (SM)	21	0	35	40
D_2	Silty Sands (SP)	21	0	35	45
E	Sandy organic clay (CH)	18	12	17	8
F	Silty (SP-SM)	20	0	35	35

The dominating sub-ground is sandy, but the sand is intersected by two cohesive strata C and E throughout the site. The soil investigations and associated laboratory tests resulted in some apparently contradictory properties for the two clay layers which are reflected in design parameters that may be on the conservative side.

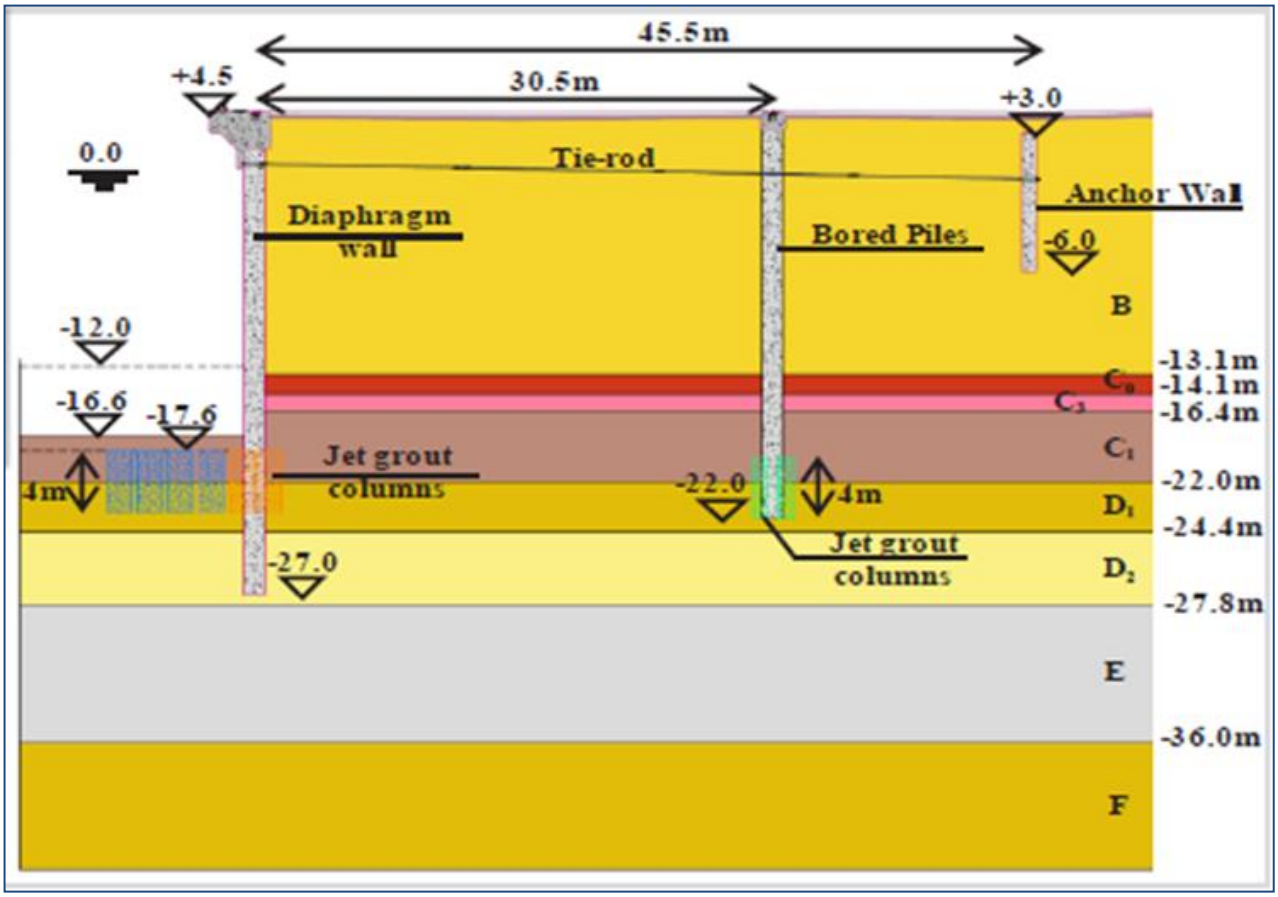

Figure 7 Soil layers and configuration of quay structure.

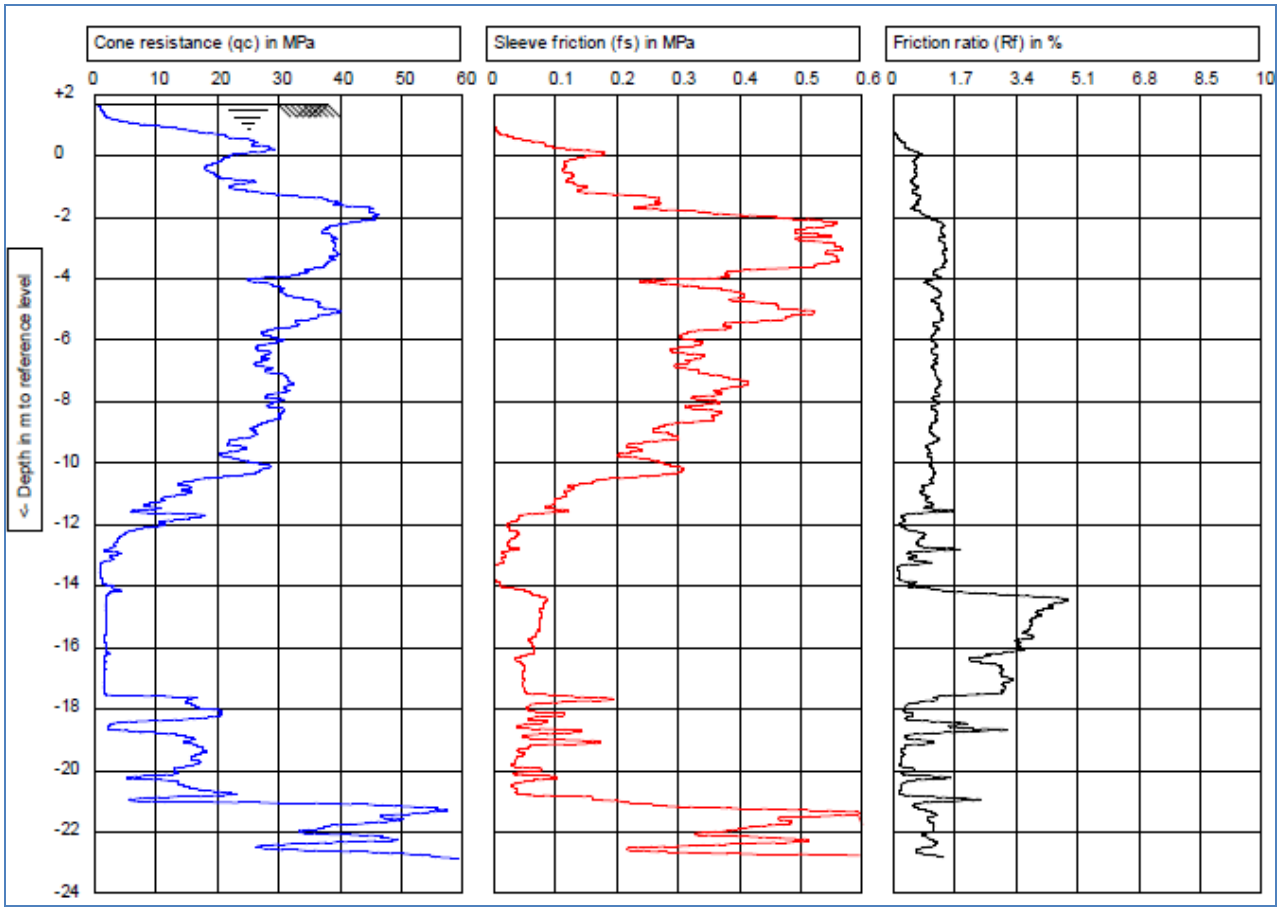

Figure 8 Results from Cone Penetration No. S24 indicating a clay layer at level -14 to -18.

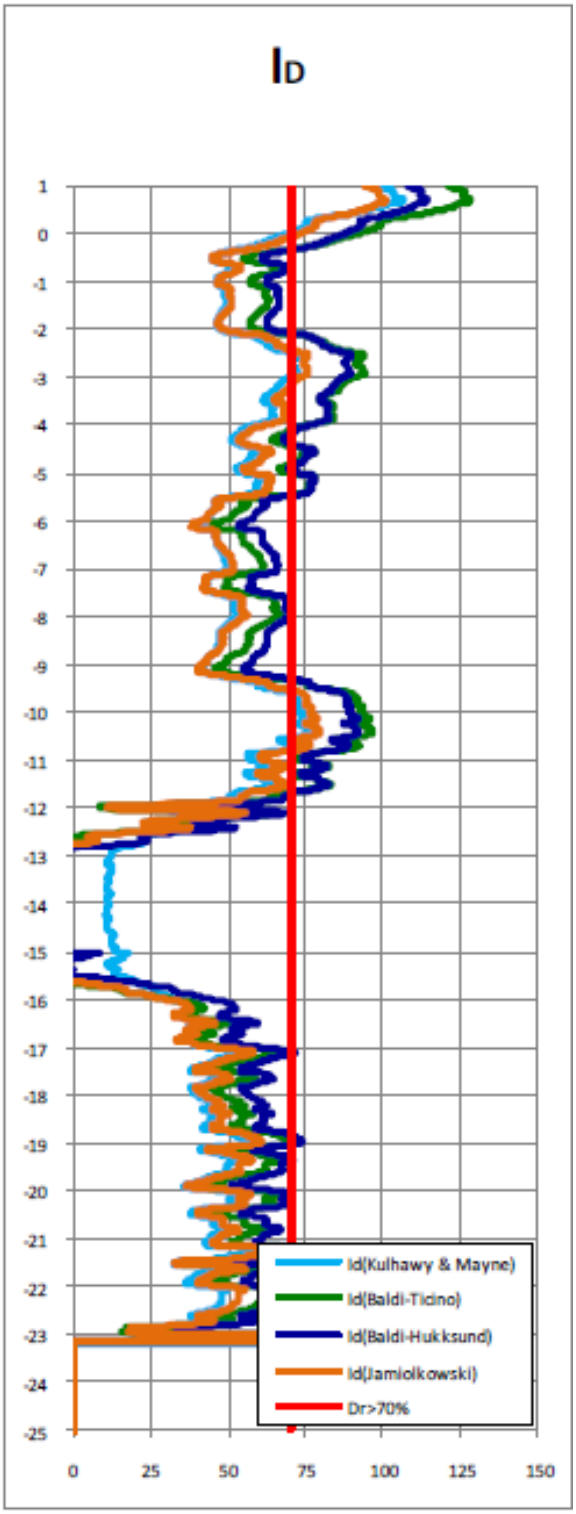

Figure 9 Assessment of relative density I_D, based on CPT-tests.

4. DESIGN AND CONSTRUCTION OF DIAPHRAGM QUAY WALL

The quay structure consists of a 1.2m thick reinforced concrete wall, cast in-situ as a diaphragm wall as shown in Figure 7. The wall is anchored to a separate diaphragm anchor wall by steel tie rods at an average distance of 1.5m between anchors.
A track for quay side container cranes is supported on the quay wall and on separate bored piles on the land side.

The dimensions of the quay structure are constant along the entire length of the main quay. Minor variations in the soil profile are accounted for by local strengthening of critical sections with jet-grouting columns – as shown in Figure 7.

The design of the quay structure was made in accordance with BS6349-2:2010, BS EN 1992-1-1.2004: Euro-code 2 and BS EN 1997-1. 2004: Euro-code 7 and included a large number of load combinations to verify ultimate limit state (ULS) and serviceability limit states (SLS). Several aspects were taken into account, such as water level variation, dredging level variations, STS-crane loads and container live loads on the apron. ULS combinations included limit states GEO and STR.

Typical result schemes are shown in Figures 10 and 11.

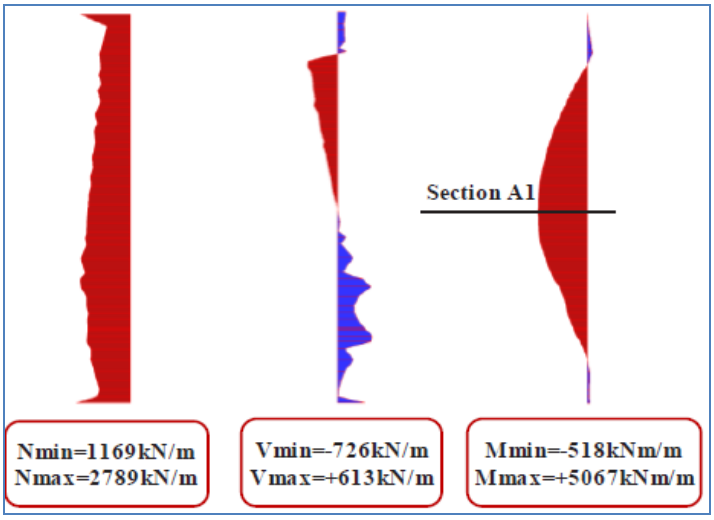

Figure 10 Example of calculated forces in quay wall.

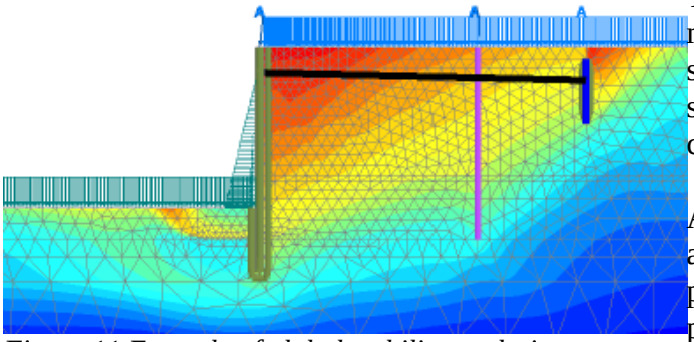

Figure 11 Example of global stability analysis (Plaxis 2D).

The wall was constructed in sections of 3 or 6 m width. Construction included excavation, stabilization of the trench with a bentonite slurry, installation of reinforcement cage and concreting.

All bentonite was recirculated in the works after thorough treatment and testing of properties. Raising the 30 m long, prefabricated reinforcement cages from horizontal to vertical proved difficult and required application of some skill from the contractor's side.

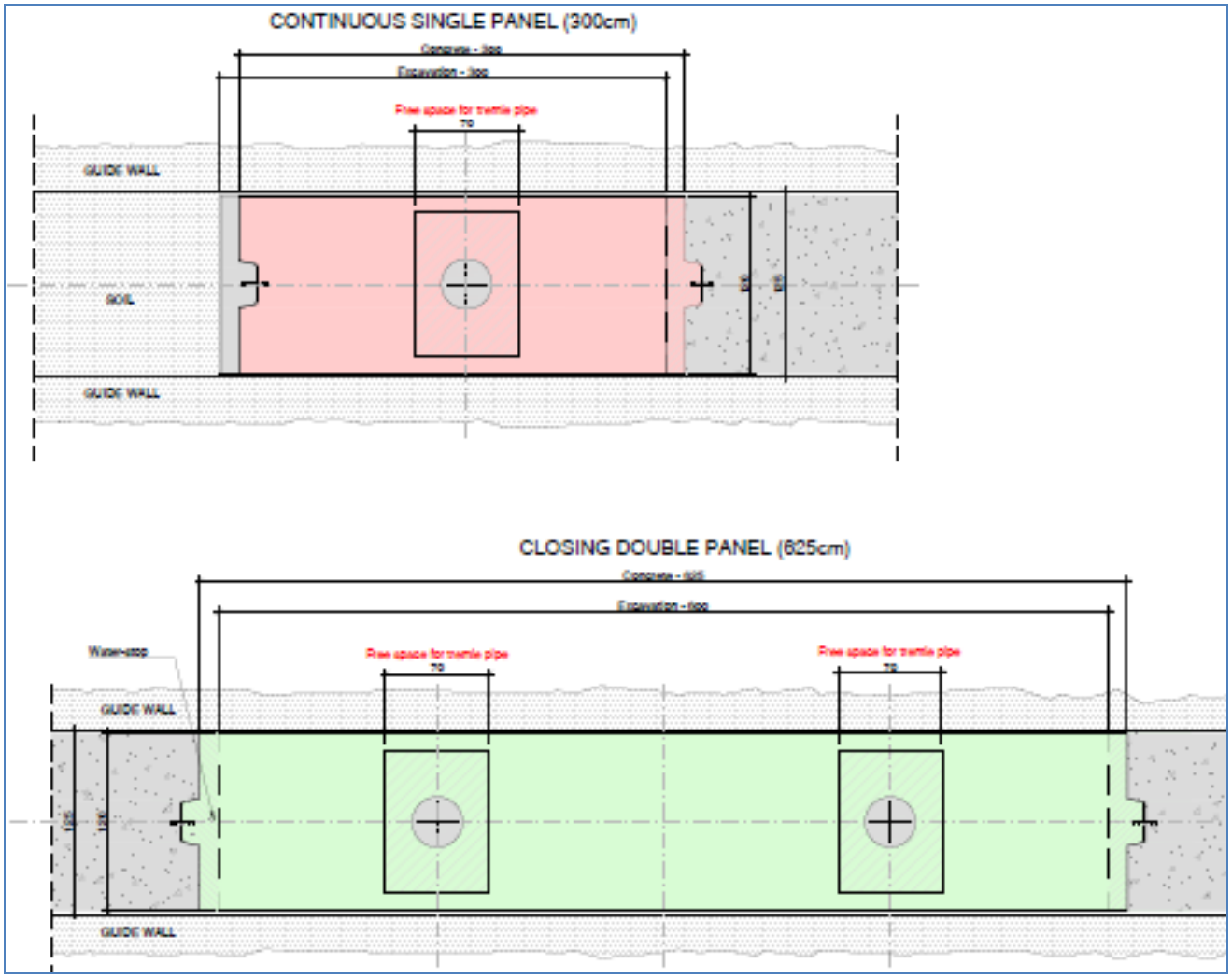

Figure 12 Typical panel sections in diaphragm wall with watertight joints between panels.

Figure 13 to Figure 15 show some views related to the construction of the quay wall.

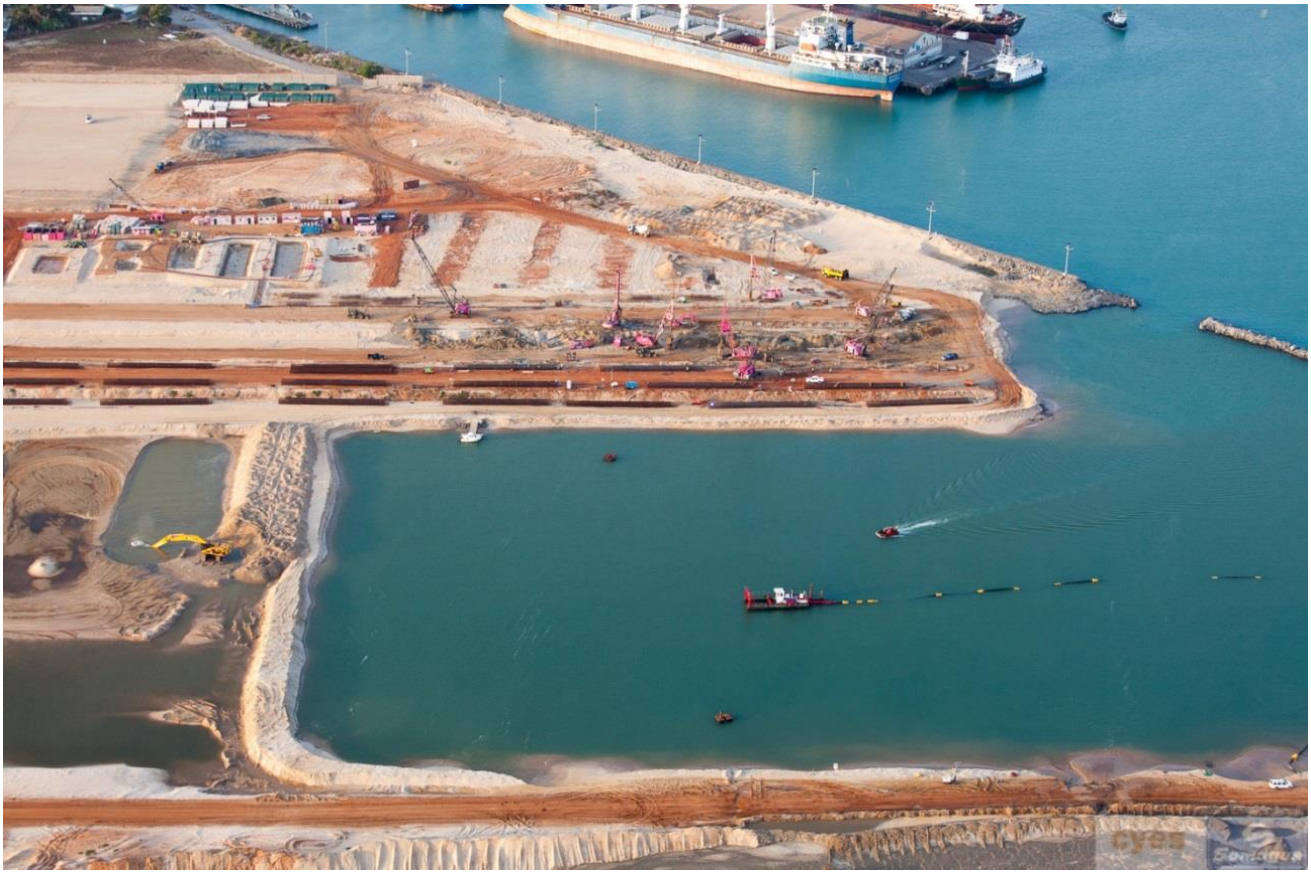

Figure 13 East part of the new basin with reinforcement cages ready to be installed. November 2013.

Figure 14 Machinery for the diaphragm wall and reinforcement cages to be installed. November 2013.

Figure 15 The reinforcement cages (app. 30 m long) for the diaphragm wall were lowered into the trench. November 2013.

The analysis carried out during detailed design included extensive Plaxis-analysis of the performance of the quay structure in terms of displacements of quay wall and rail beams during the different construction phases and after completion when the structure is exposed to traffic loads. According to BS 6349, for quasi-permanent load combinations, the maximum horizontal displacement accepted for a vertical wall is H/300 (where H is the total height of the wall).

Displacements, horizontal and vertical, were calculated for different control points in the quay structure (top of quay wall, crane beams, anchor wall etc.) and for different stages of construction and operation. Typical results from the analysis are shown in Figure 19 where each color represents a defined control point.

A maximum horizontal displacement of 61mm was calculated, meeting the requirement (< H/300 ≈ 100mm). The relative displacements of crane beams were also calculated to be manageable with regard to the selected rail system.

Figure 16 Vibrator for the deep compaction. May 2013

Figure 17 Local recess of the ground surface during the deep compaction. May 2013.

5. RELATIVE DENSITY OF SAND AND NEED FOR DEEP COMPACTION

The work specification included deep vibro-compaction (see Figure 16) of all naturally deposited sand (layer B in Figure 7) to reduce settlements in the future stacking areas.
The contractor's compaction methodology and procedures were initially refined and optimized in a few test sections based on CPT-testing before and after compaction. From the tests it became clear that in large parts of the construction area the effect of deep compaction would be rather limited. Some densification was achieved in the lower part of the sand layer. But penetration of the upper zone resulted in apparent loosening of naturally dense sand as indicatively shown in Figure 17.
The tests resulted in a significant reduction of deep compaction works.

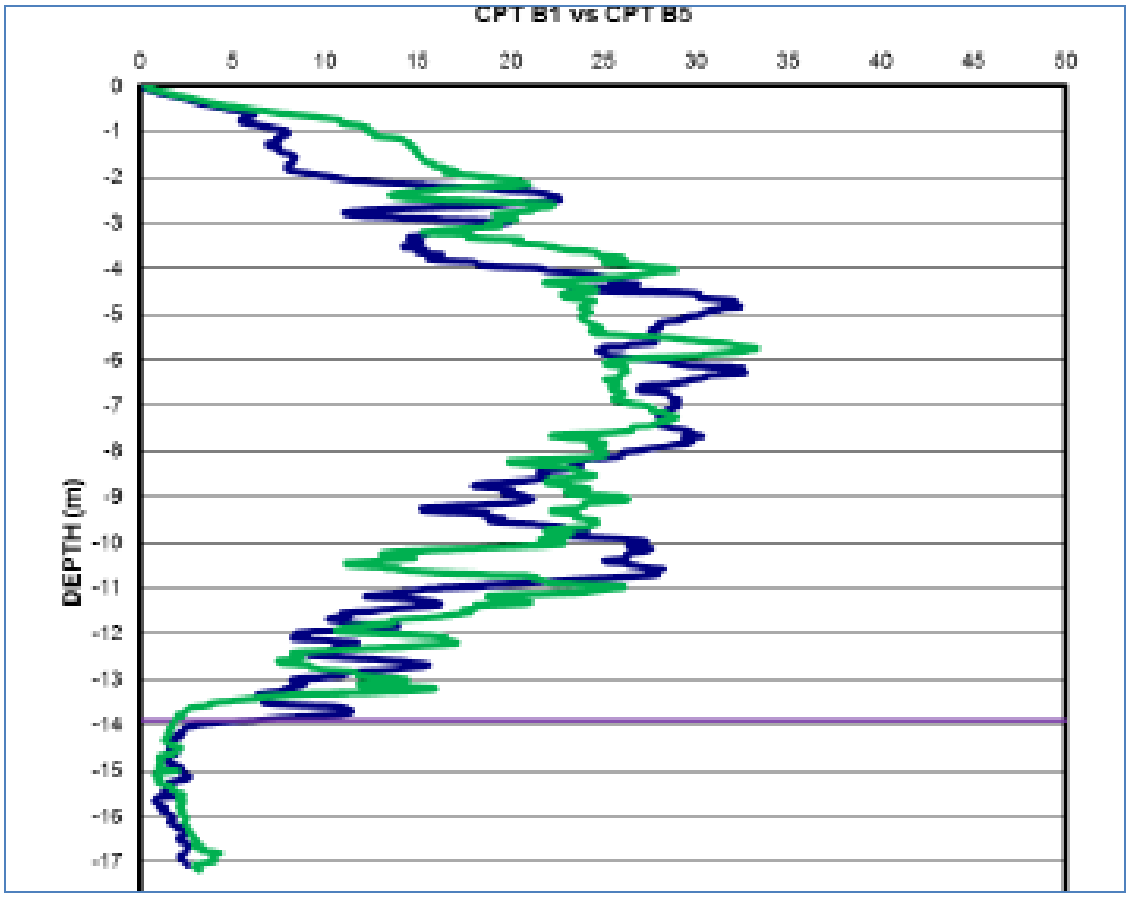

Figure 18 CPT-testing results before and after compaction (Blue color: Before compaction. Green color: After compaction).

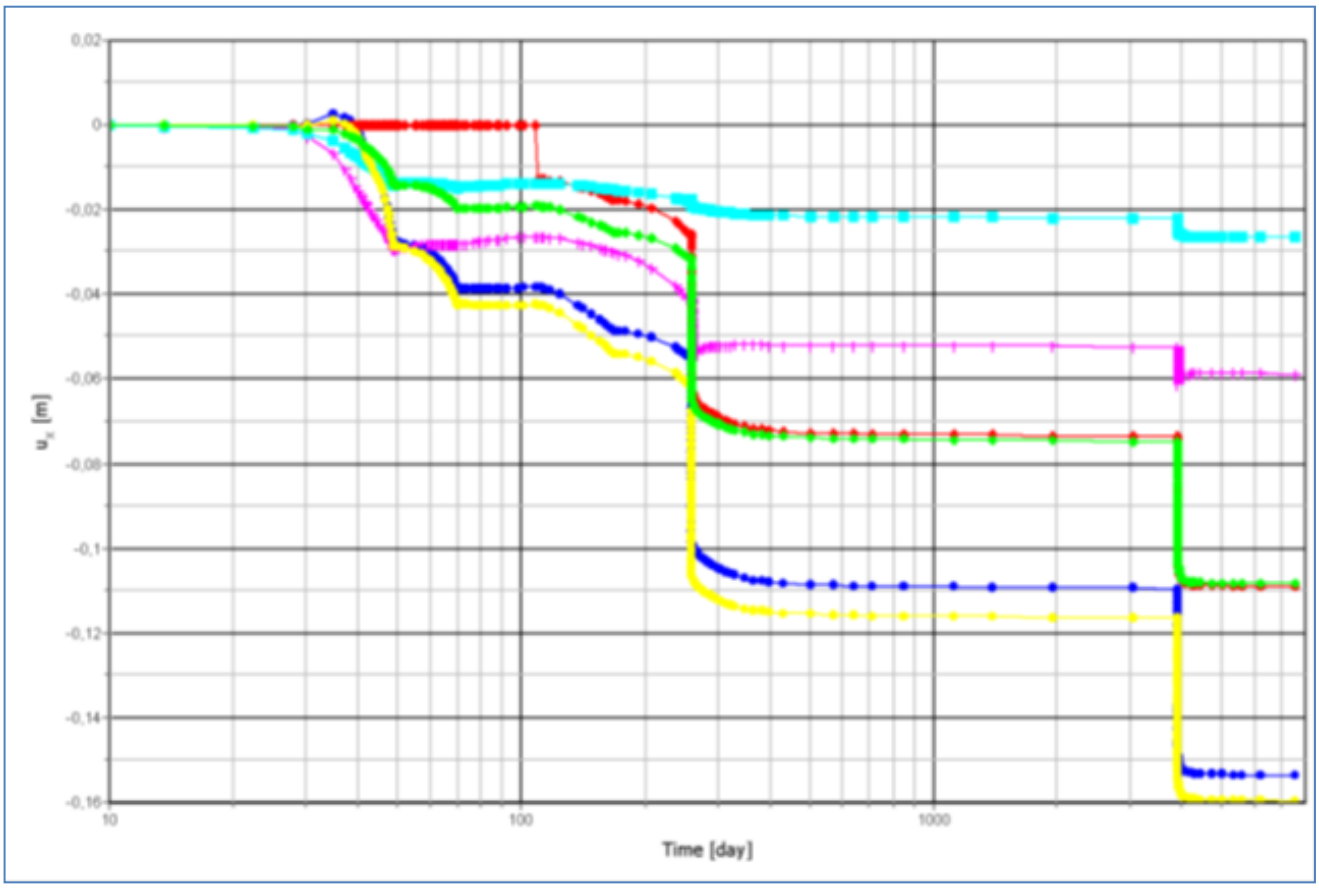

Figure 19 Calculated horizontal displacements with time for different control points.

6. RECORDED VERTICAL SETTLEMENTS

Monitoring included systematic measurement of settlements. The recorded settlements were surprisingly small although the vertical soil stresses increased due to the weight of 2 to 2.5m of sand fill and pavement from raising the general level of the terminal. Settlements of 10 to 15 mm recorded and developed almost instantly.

7. MONITORING OF QUAY WALL AND TRACK FOR QUAY SIDE CRANES

The specified monitoring program also included observation of the movement of quay structures during construction and after exposure to operational loads from quay side container cranes and cargo. The program included horizontal and vertical displacements and inclinometer records.

Until this date the recorded displacements, both vertically and horizontally, have proven significantly smaller than the predicted displacements as described in section 4.

The Plaxis calculations predicted for the front top corner of the quay structure overall horizontal displacements up to 140 mm. The recorded displacements are less than 30 percent of this figure.
The lateral displacements of the seaside crane rail are less than 50 percent of the calculated displacement resulting from crane loads.

8. CONCLUDING REMARKS

One of the major challenges of this type of project is the pre-assessment of the soils to be verified by detailed monitoring of the performance of the structures . The assessment depends on the specification of the geological-geotechnical campaign prior to construction and from the interpretation of the results.

An experience from this project is that in spite of the efforts by several experts in the investigation and design phases, subsequent monitoring of a structure may still show results that deviate quite significantly from the predicted performances.

9. REFERENCES

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10. ACKNOWLEDGEMENTS

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Figure 20 Machinery for deep vibration compaction. Distance between the points is app. 3 m. July 2013.