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Geotechnical characterisation and monitoring versus FEM calculation results for a massive quay wall in the harbour of Antwerp, Belgium

Caractérisation et auscultation géotechniques comparées aux résultats de calculs FEM pour un mur de quai massif dans le port d'Anvers, Belgique

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ABSTRACT

For the construction of a container terminal (Deurganckdok) in the harbour of Antwerp in Belgium, an extensive soil investigation was performed : CPT, borings with continuous and discontinuous sampling, laboratory tests (triaxial, but also bender element tests). Out of the results of this investigation program, soil profiles with characteristic values of the soil properties were put forward. Design calculations of the massive quay wall were performed, using as well analytical as finite element methods. During construction a monitoring program, including inclimometers and extensometers was set up.
The paper focuses on the site characterisation and selection of soil properties as well as on the monitoring program. Furthermore predicted deformations and actual performance of the structure are compared.

RÉSUMÉ

Pour la construction d'un terminal pour containers (Deurganckdok) dans le port d'Anvers en Belgique, un programme complet de reconnaissance géotechnique a été établi : essais de pénétration, forages avec échantillonnage continu et discontinu, essais de laboratoire (triaxiaux, mais aussi essais sismiques). A partir des résultats de ces essais, des profils géotechniques reprenant les valeurs caractéristiques des paramètres ont été dressés. Les calculs de dimensionnement du mur de quai massif ont été faits en utilisant des méthodes aux éléments finis et analytiques. Pendant la construction, un programme d'auscultation comprenant des inclinomètres et extensomètres, a été élaboré.
Les déformations calculées et mesurées ont ainsi être pu comparées.

1 INTRODUCTION

In order to deal with the exponential growth in container traffic in the port of Antwerp (Belgium) it was decided in 1998 to build a new tidal container dock, the Deurganckdock, on the left bank of the Scheldt.

The construction of the dock, with about 5 km quay wall length, is being carried out in three phases. Phases 1 and 2 are nearly completed, so that the first part of the container terminal will be operational in 2005.

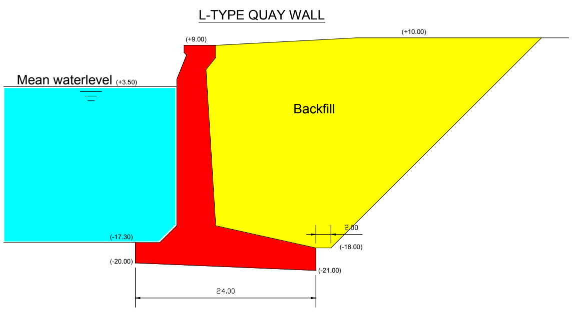

Figure 1: L-shaped quay wall

The quay walls are L shaped reinforced concrete walls of the semi-gravity type (figure 1), with a height of 30m and a width varying from 22.5m to 24.0m. Sheetpile walls at the front and rear end are installed to prevent soil-particle transport.

The walls are built in a deep excavation pit, necessitating an important ground water lowering (26m) combined with recharge wells in order to prevent subsidence of nearby industrial buildings and private dwellings in the village of Doel. Over a length of about 1km additionally a hydraulic cut-off wall had to be constructed, to a depth of about 50m (2m in the Boom clay).

2 GEOTECHNICAL INVESTIGATION

The geotechnical investigation program consisted of

- Electrical and mechanical CPT (103) to a depth of 30 to 50m
- Borings (31) with discontinuous and continuous sampling (hollow auger borings), and undisturbed sampling
- Piezometers (30) for follow up of effects ground water lowering
- Laboratory tests on undisturbed samples (330) for determination soil type, shear strength, compressibility and permeability
- Additional laboratory testing (bender element tests) on undisturbed samples (20) for determination of soil stiffness parameters
- Control testing (proctor tests, density measurements and CPT) of backfill behind the quay wall

3 SOIL PROFILE AND GEOTECHNICAL PARAMETERS

Out of CPT results and borings typical soil profiles along the dock perimeter could be defined. Figure 2 shows the CPT plot and corresponding soil profile for phase 3 of the dock.

Following soil sequence could be distinguished.

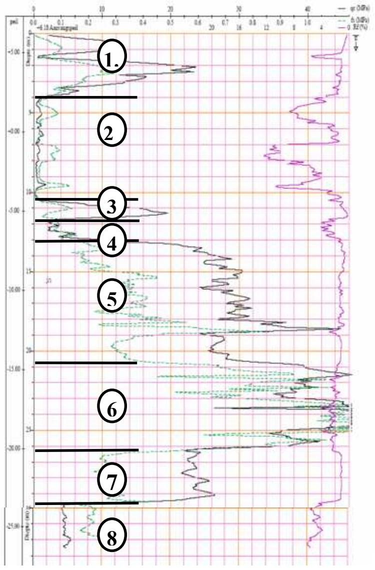

Figure 2: typical CPT plot
1. Hydraulic sand fill
2. Holocene deposits with clay and peat layers
3. Quaternary sand layer
4. Tertiary sand layer sequence (Kruisschans)
5. Tertiary sand layer sequence (Oorderen)
6. Tertiary sand layer sequence (Kattendijk)
7. Tertiary sand layer sequence (Antwerpen and Edegem)
8. Tertiary Boom clay

Following features had a significant effect on design of the quay wall.
- Shear strength parameters of the back fill behind the quay wall
- Top level of the Boom clay (with respect to the foundation level of the quay wall)
- Shear strength parameters of the tertiary sand layers and the Boom clay
- Soil stiffness parameters of the back-fill, tertiary sand layers and the Boom clay

In design shear strength parameters of φ'=32.5°, c'=0 were adopted for back fill sand. Throughout construction quality of back-filling was controlled by means of CPT, proctor tests and density testing.
Top level of the Boom clay was derived from CPT results and borehole logs. Where significant fluctuations of the top level were observed inter-distance of CPT-tests was reduced and a closer mesh of investigation points was adopted.
For determination of characteristic values of shear strength parameters, following procedure was followed:
- Results of triaxial tests (CU) were plotted in s'-t diagram
- The relevant stress range for the actual loading condition was defined, from results of FEM-calculations
- Results of the triaxial tests were considered as a local sampling (all considered tests performed at the location of the project)
- Taking into account that for this quay wall structure a large volume of soil is involved in possible failure mechanisms, it was assumed that a characteristic value close to the mean value could be adopted (compensation from weaker zones to stronger zones is possible). According to EC7 a value is calculated having a chance of 95% that the "real" mean value is higher than the guessed value. The statistical receipt for calculating the 95% confidence level of the real mean out of triaxial test results is given by Bauduin and Van Alboom et al.

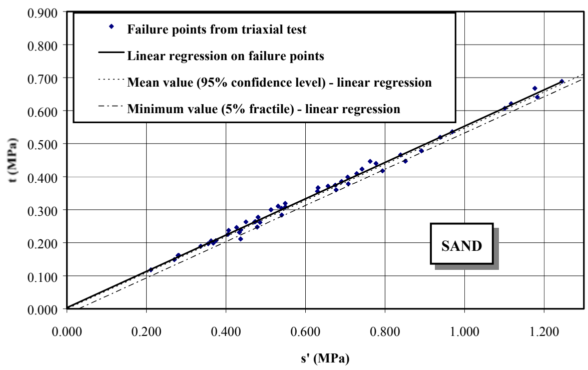

Figure 3 :s'-t diagram with 95 % confidence level for tertiary sand layer

Figure 3 shows the graphical representation of the triaxial test results for the tertiary sand of the formations of Antwerp and Edegem: the full line represents the linear regression and the dotted line the 95% confidence interval.

Apart from this statistical approach engineering judgement was essential in selecting geotechnical parameters to be used in design calculations. Experience gained from other comparable studies and data from literature formed a.o. the input for an engineering judgement based parameter selection.

Table 1 shows, for the soil layers involved in equilibrium of the wall, shear strength and soil stiffness parameters.

Table 1: geotechnical parameters

Soil layer	φ'	c' (kPa)	G_o (Mpa)	G(Mpa)
Back-fill (sand)	32.5°	0	20	20
Tertiary sands: -Kattendijk -Antwerpen /Edegem	 36° 33.5°	 0 0	 150 135	 75 60
Tertiary Boom clay	23°	24	60	30

For the selection of soil stiffness parameters following data were available:
- G_0 moduli derived from bender element tests
- G_0 moduli derived from CPT correlations
- Comparable experience from a nearby quay wall construction (Verrebroekdock)

From bender element tests following mean values of G_0 could be derived for the considered stress range:
- G_0= 158 MPa for Kattendijk sequence
- G_0= 175 MPa for Antwerp/Edegem sequence
- G_0= 119 MPa for Boom clay

Special attention was paid to possible unloading effects on the Boom clay. For this purpose CPT's and borings were performed in a nearby dock that was excavated down to the Boom clay, and has been flooded for about 10 years. There was no significant reduction in cone resistance but results of bender element tests on samples taken at shallow depth in the upper Boom clay showed a clear decrease in soil stiffness. G_0 –values dropped to 85 to 121 MPa in the considered stress range, and to 68 to 73Mpa for lower stress ranges.

Table 2: derived G_o-values

Soil layer	Bender element test	Robertson and Campanella correlation
Kattendijk sequence	158 MPa	170 Mpa
Antwerp/Edegem sequence	175 MPa	135 MPa
Boom clay	119 Mpa (68 to 121 Mpa)*	80 Mpa (Maine)

*for unloaded Boom Clay

Use of the CPT correlation chart proposed by Robertson and Campanella resulted in a second set of G_0 -values for the sand layers. The correlation formula for clay (Maine 1993) was used for determination of stiffness parameter of the Boom clay.

These derived values were matched with soil stiffness parameters used for the design of a nearby quay wall construction. An other input were back calculation results from a monitored retaining wall structure in comparable tertiary sand layers.

For design actually 2 sets of G-values were selected:

- G_0 for calculation stages where small deformations were expected.
- A G-value, corresponding to approximately half the maximum value, representative for calculation at greater deformations (exploitation phase)

For the back fill layer relative movement of this soil body with respect to the L-shaped wall was expected to be very small in all stages of construction. For this reason only the G_o-value was taken into account.

4 DESIGN CALCULATIONS

Following limit states were considered in design:

1. Bearing resistance failure
2. Failure by sliding
3. Failure by topping
4. Loss of overall stability

Both analytical and finite element calculations were performed. For analytical methods the global safety factor concept was adopted, for FEM calculations (φ',c'-reduction scheme) the partial safety factor concept (EC7) was used.

The construction was also checked for excessive deformations through FEM calculations. The results of this deformation analysis are now matched against monitoring results.

5 MONITORING PROGRAM

To evaluate the behaviour of the quay wall during construction and in service, an extensive monitoring program was set up.

Instrumentation of the 5 km long quay wall will, in the end, consist of 18 inclinometers and 5 borehole rod extensometers with 4 anchor points each.

Inclinometers were installed to allow observation of the horizontal movements of the concrete quay wall structure, and the underlying soil layers. During the excavation operation, a borehole with PVC-tubing diameter 200 mm reaching into the Boom clay layer was realised. This PVC-tube was extended with a steel tube diameter 219 mm connected to the reinforcement cage of the concrete. In this way the inclinometer casing could be installed right after completion of the first concreting phase, and before the back-filling started. The inclinometers, reaching to about –41.00 TAW, are supposed to have a fixed reference point at that level. Successive measurements with the inclinometer probe, give an idea of the horizontal movements of the whole soil structure entity in time.

The vertical movement of the structure is measured by means of borehole extensometers. These could only be installed after the back-fill was more or less completed and the top of the concrete quay wall was attainable for a boring machine. The lowest anchor at level –45.00 TAW is chosen as reference point. The top anchor provides the total movement (settlement) of the top of the quay wall. The two intermediate anchors allow to measure the settlement of the sand layer below the footing of the quay wall.

6 MONITORED VERSUS PREDICTED BEHAVIOUR

The monitored and predicted behaviour are compared for following construction stages:

1. Back-filling of quay wall at mid height
2. Back-filling of quay wall at full height
3. Dredging and flooding of dock

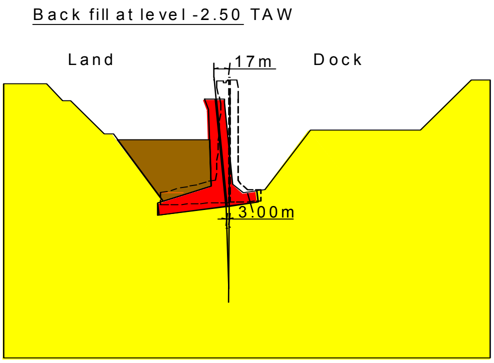

Figure 4: stage 1

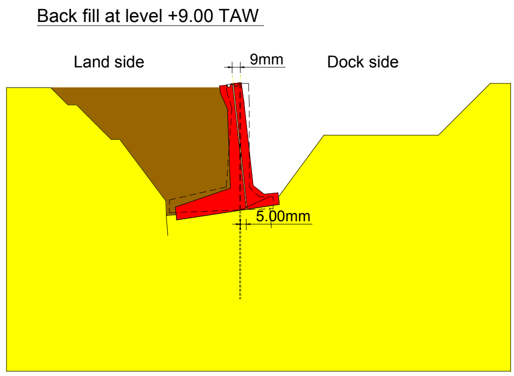

Figure 5: stage 2

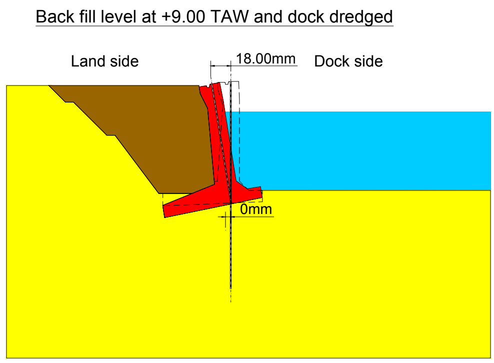

Figure 6: stage 3

Figures 4 to 6 show for this three stages the evolution of measured movements. In phase 1 (figure 4) we discern a backward tilting of the quay wall, due to the load of the back-filling on the toe-element of the wall. As back-filling proceeds (figure 5) the ground pressure on the vertical element of the wall becomes predominant: the quay wall tends to move forward. The last stage (figure 6) when dredging is proceeding and the dock flooded, shows again a backward tilting. One should expect here a further forward movement and tilting due to the unloading effect of the dredging. Somehow the water-pressure on the dock side of the wall seems to be more effective. Last measurements however show a slight forward tilting.

When design values (table 1) are introduced in FEM serviceability limits state calculation, calculated movements and deformations do not match well with monitoring results (figure 7).

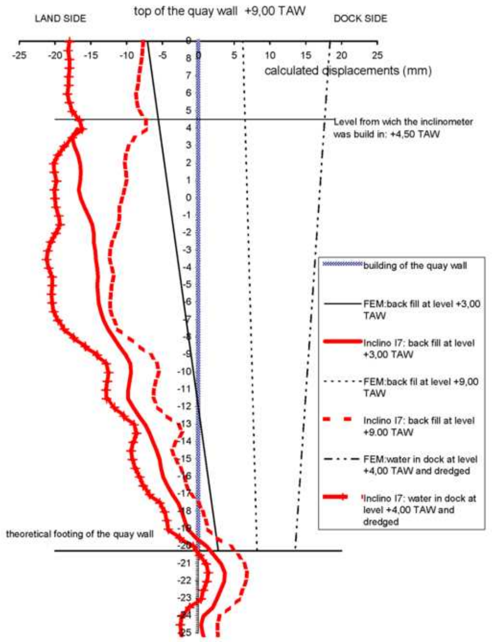

Figure 7: monitored versus calculated deformations of quay wall (design calculations)

In order to calibrate calculated against measured deformations design parameters were reevaluated.

It shows that the quality of back-fill was clearly better than anticipated (this was confirmed by CPT results) which resulted in an adjustment of the back-fill parameters.

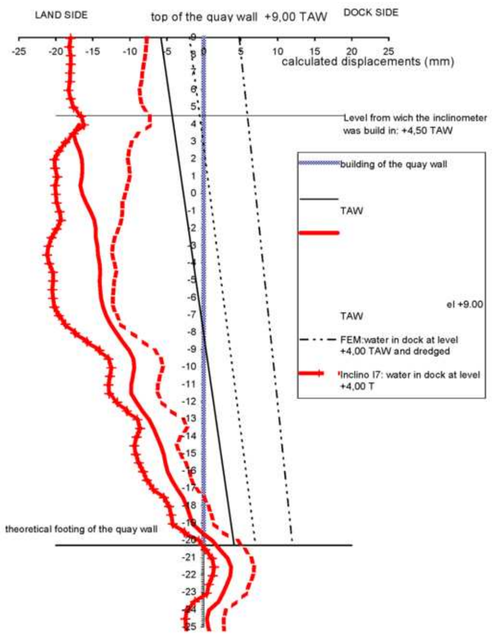

Figure 8: monitored versus calculated deformations of quay wall (adapted calculations)

From the above mentioned Robertson-Campanella correlation chart, the value of the soil stiffness was readjusted (75 to 125 MPa instead of 20 Mpa). This resulted in a better agreement of calculated and monitored deformations.(figure 8)

Nevertheless, the inverse movement in the calculated "dredging stage" remained.

In an attempt to simulate the measured movements, back-calculations with a sensitivity-analysis of following parameters were performed:

- shear strength and density of back-fill
- roughness of the interface (CR-values) along the vertical element of the quay wall.
- introduction of an interface along the slopes of the excavation pit.

Non of these calculations however showed a backward movement for the dredging stage.

Results of the deformation analysis are also matched against measured vertical deformations (extensometer readings), as shown in table 3. Minus sign indicates settlement.

Table 3: vertical deformations

Reference point	Monitored value	Calculated value
Top quay wall	-2.0mm	-2.1
Toe quay wall	-1.3m	-2.1
Top Boom clay	-0.4mm	-1.8

Here there seems to be a rather good agreement between measured and calculated deformations, taking into account the relatively small movements that were observed.

7 CONCLUSION

Out of an extensive soil investigation program shear strength and soil stiffness parameters were derived. As well statistic procedures as comparable experience were used within a global engineering judgement based framework.

As planning and construction of the different phases of the dock proceeded, more geotechnical data came available. Out of this growing population of test results more accurate (and less conservative) values could be derived for geotechnical parameters.

In addition to limit state design, specific attention was paid to deformation analysis of the quay wall structure. The results of these calculations were matched against monitored movement of the soil-structure entity.

It showed that in order to get a better compatibility, geotechnical parameters of the back-fill had to be readjusted. More rigid behavior of the back-fill material was confirmed by control CPT-results. The movement involved in the dredging and flooding phase could however not accurately be modelled. A more detailed sensitivity analysis couldn't either give a better match.

Most recent inclinometer measurements show however a trend that is in agreement with modeling results. This needs however further confirmation and study.

It should be stressed that global deformation performance is yet in full accordance with performance criteria.

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