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Full scale load test on historic quay wall in Antwerp

Essai de chargement sur un mur de quai historique à Anvers

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ABSTRACT: In the city of Antwerp the historic quays near the right bank of the river Scheldt will have a complete make-over. The historic quay wall, which was constructed in the 19th century, has known stability issues since its construction. To avoid further structural quay wall damage and to guarantee the safe use of the quay area in the future, a quay wall stabilisation program is started. Invasive and thus costly stabilisation works are needed over almost the entire length of the 5.5 kilometre long quay wall. If, for certain quay walls, stabilisation works can be avoided, the project cost can be reduced and the important historic values of the quay wall can be preserved. This paper describes how this twofold objective is achieved for two specific quay walls in the surroundings of the Marguerie dock.

RÉSUMÉ: Dans la ville d'Anvers, les quais historiques situés en rive droite de l'Escaut feront l'objet d'une rénovation complète. Construit au XIXe siècle, ce mur de quai historique connait des problèmes de stabilité depuis sa construction. Afin d'éviter des dommages structurels aux quais et de garantir leur utilisation future, un programme de stabilisation du mur de quai a été lancé. Les travaux de stabilisation conséquents et coûteux seront réalisés sur quasiment l'entièreté de la longueur du mur de quai long de 5.5 km. Si, pour certains murs de quai, les travaux de stabilisation peuvent être évités, le coût du projet peut être réduit et les valeurs historiques importantes du mur de quai peuvent être préservées. Cet article décrit comment ce double objectif est atteint avec succès pour deux murs de quai situés dans les environs du « Dock Marguerie ».

Keywords: historic quay wall; full scale load test; masonry gravity wall; monitoring.

1 INTRODUCTION

1.1 Masterplan Scheldt quays

In the city of Antwerp the historic quays at the right bank of the river Scheldt will have a complete make-over. The council of the city of Antwerp and DVW (i.e. De Vlaamse Waterweg nv, the independent agency of the Flemish Government in charge of the management of the Scheldt river banks including the protection against storm surges) have agreed to carry out the most significant renovation works in over a century. Three main goals were set for this long term plan:

1. The stabilisation of the historical quay walls with a quay level of 7mTAW. (TAW i.e. 'Tweede Algemene Waterpassing' is a topographic reference level in Belgium corresponding to the low water tide in Ostend: 0mTAW);

2. The protection of the city of Antwerp against -what is expected- increasing storm surges in the future . This goal is part of the integrated Sigma Plan (the plan to protect the whole

of the tidal Sea Scheldt basin against storm surges) and requires a new storm surge barrier (9,25mTAW) to be built in the area of the existing quays.

3. A facelift of the whole quay area to restore the city's link with the river by incorporating urban development and mobility and creating new public domains.

1.2 Stabilisation program

The historic quay wall, which was constructed in the 19th century, has known stability issues since its construction (Feremans, 2018). To avoid further structural quay wall damage and to guarantee the future use of the quay area, a quay wall stabilisation program is started. This program resulted in costly stabilisation works that are needed over almost the entire length of the 5.5 kilometre long quay wall. During the elaboration of the project, optimizations are studied to reduce the costs. This paper describes how this search resulted in two specific quay walls in the surroundings of the Marguerie dock, where costly and invasive stabilisation works are avoided.

1.3 Project location

The two quay walls are situated near the Marguerie dock and are named Quay Wall C (QWC) and Quay Wall D (QWD), as shown in Figure 1.

Figure 1: Project location

2 EXISTING QUAY WALL

2.1 Quay Wall C (QWC)

No construction drawings of this quay wall are available. To determine the geometry and the current condition of the existing wall, determination tests are executed.

A topographic and a bathymetric survey give the levels of the dock bottom and the quay deck. One core drilling with endoscopic inspection, one hammer drilling and a thorough visual inspection of the quay wall show only esthetical damage. There are no signs of severe structural damage. The drill holes show that the formerly unknown foundation level of the quay wall is situated at around -0.25 mTAW. To gather information on the thickness of the wall, an inspection pit is digged. The depth of this pit had to be limited to 2.8 m (4.2 mTAW) because of a layer of debris behind the quay wall and the encounter of ground water. Behind the quay wall cone penetration tests and boreholes are conducted and two piezometers are installed.

Based on this information, conservative assumptions are made on the cross section of the quay wall, as shown in Figure 2. The wall is constructed with masonry. Since no indication of a pile foundation is present and given the fact that the encountered foundation level is at the top of a medium dense sand layer, a gravity wall is assumed.

2.2 Quay Wall D (QWD)

An old cross section found in the city archive shows a gravity wall constructed of masonry with a height of about 11 meters. To verify this information, the same amount of determination and geotechnical survey tests as for QWC are conducted. No severe structural damage is found, only esthetical damage is identified. The drill holes confirm the foundation level of the old cross section. Based on all available information, conservative assumptions on the geometry of the quay wall are set, as shown in Figure 3. The thickness of the wall is taken from the old cross

section. The occurrence of counterfort walls - found in other archive documents - is neglected as no evidence of their presence in the conducted tests is available.

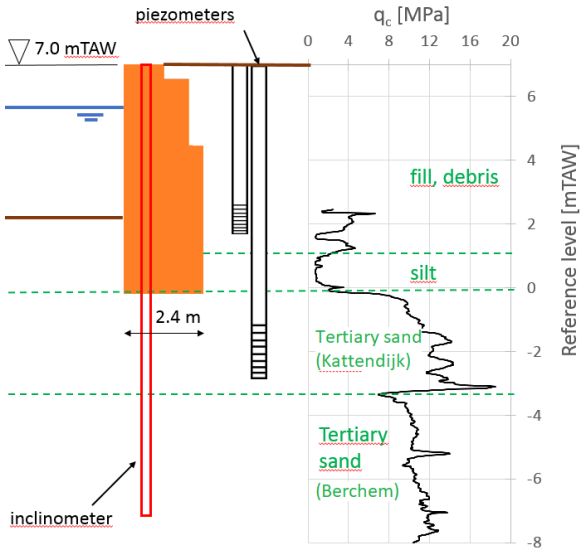

Figure 2: Cross section Quay wall C

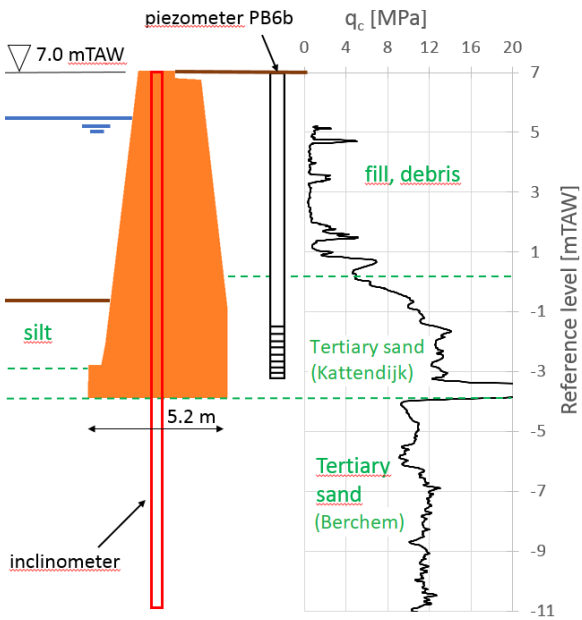

Figure 3: Cross section Quay wall D

2.3 Stability calculations

After determining the geotechnical boundary conditions according to Eurocode 7 (BIN, 2014), finite element calculations of the existing situations for QWC and QWD with the mentioned geometric assumptions are performed. For a situation without surcharge loads, the safety factor of the global stability for both quay walls is below 1.00. This means that the quay wall is unstable under the given assumptions despite the lack of visible structural damage. Probably this is only a theoretical instability because of the conservative assumptions made due to the lack of information on the geometry of the wall and the debris fill. The question also raises if the quay walls stay stable under a surcharge load of 20 kN/m², which corresponds to the future surcharge loads needed in this area.

3 LOAD TESTS

3.1 Design philosophy

Since the stability of the quay walls cannot be proven by the use of calculations, it is decided to conduct full scale load tests. Eurocode 7 (BIN, 2005) allows the verification of limit states by the use of experimental models and load tests.

In the future design situation the following loads have to be withstand by quay walls QWC and QWD: 1) earth pressures; 2) (ground)water pressures; 3) uniform surcharge load of 20 kN/m². Ships are not allowed to moor on both quay walls, so no bollard or mooring loads are applicable. The quay deck level will not be changed by the redevelopment of the public domain. The same holds for the river bottom level at the toe of the quay walls.

The load test has to simulate the future governing load combination:

1) Most unfavourable earth pressures: The load test takes place on the location with the most unfavourable soil profile.

2) Most unfavourable (ground) water pressures: The tidal river Scheldt has mean water levels varying between approximately 0mTAW and 5.5mTAW at respectively low and high tide. The highest water level differences over the quay wall occur during a monthly spring tide. So the load test is planned during a spring tide. The water level difference over the quay wall is not expected to raise significantly in the future. Piezometer monitoring for a period of one year shows that ground water levels are stable over time

3) Most unfavourable surcharge load: For gravity walls founded in a dense sand layer, the governing failure mechanisms are sliding and/or tilting of the wall rather than bearing resistance or deep sliding plane failure. For these first mechanisms the most unfavourable load situation is one with no load on top of the wall and the area behind the wall fully loaded. The footprint of the loaded area has a width W = H and a length L = 2H with H the height of the retaining wall as shown in Figure 4. The test load is equal to 30 kN/m² (γ_Q=1.5).

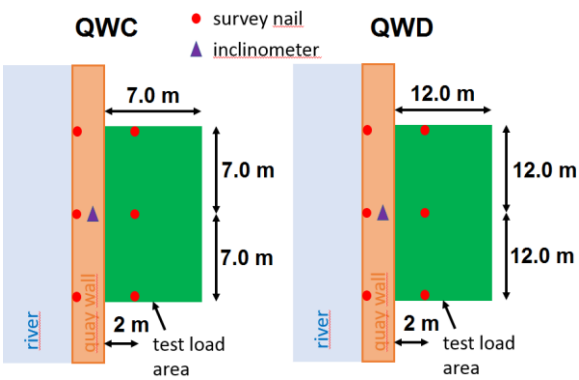

Figure 4: Load test configurations - plan view

3.2 Test program

3.2.1 Load program

Big bags filled with sand are used to realise the test load as can be seen in Figure 5.

Loads are applied according to the load program shown in Figure 6. In the loading phase, an additional load of 10 kN/m² is applied every day.

The maximum test load persists for 7 days. The additional loads are put in place at the moment just before low water so the mobile crane is still staffed in case of necessary emergency unloading.

Figure 5: Load test at QWD.

3.2.2 Monitoring program

The following measurements are performed during the load tests:

1) Deformation measurements (cfr. Figure 4): At each test location, 6 survey nails are fixed and an inclinometer casing is installed through the quay wall. The bottom of the casing is situated 7 meters below the foundation of the quay wall to accomplish a fixed point.

2) Water level measurements: Behind the quay wall piezometers are installed in the different aquifers (cfr. Figure 2 and Figure 3). Water levels of the river Scheldt are measured in a nearby gaging station.

3) A visual inspection of the façade of the quay wall is combined with crack width measurements on existing cracks.

The imposed frequency of measurements during the load tests is given in Table 1.

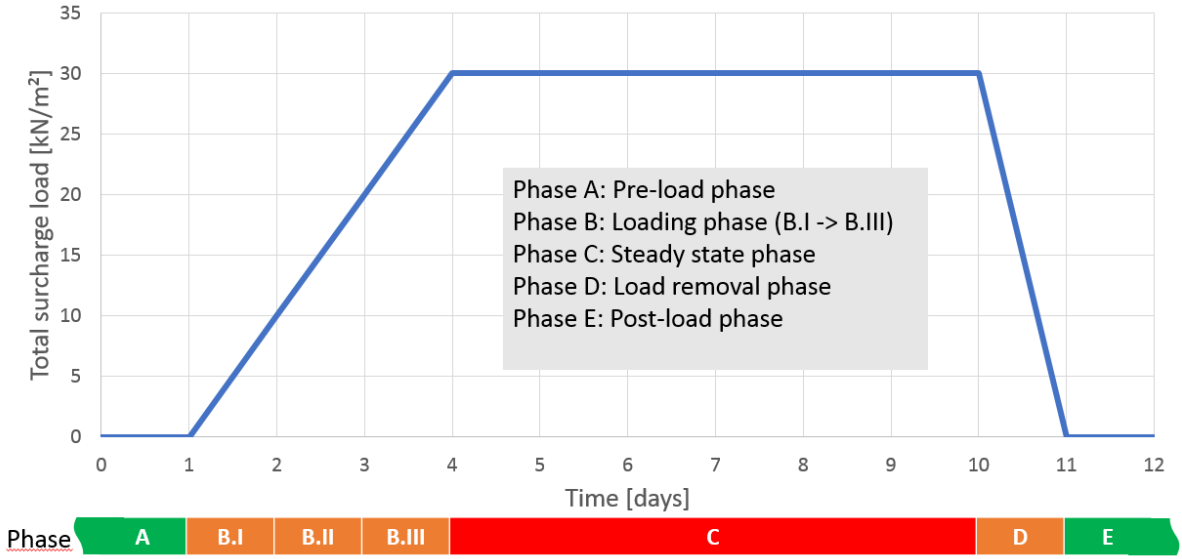

Figure 6: Load program

Table 1: Frequency of measurements during load test

Phase	Deformations	Water levels	Visual inspection
A	During spring tide period for 2 subsequent days: measurement at HW, MW and LW. During neap tide period for 2 subsequent days: measurement at HW, MW and LW.	Continuous (every 15 minutes)	Inspection at LW
B	For every load step: measurement immediately after completion of the load step and then at the subsequent LW, MW, HW and MW.	Continuous (every 15 minutes)	For every load step: inspection at first LW after load addition.
C	Daily measurement at LW.	Continuous (every 15 minutes)	Last day of phase C: inspection at LW.
D	/	Continuous (every 15 minutes)	/
E	After complete removal of the load: measurement at subsequent LW, MW, HW and MW.	Continuous (every 15 minutes)	After complete removal of the load: inspection at LW.

LW: low water level

MW: mean water level: MW = (LW+HW)/2

HW: high water level

3.2.3 Safety measures

In order to avoid a failure of the quay wall during the test, the displacements of the survey nails are carefully observed. The maximum allowed displacement (relative to the normal tidal displacements) is limited to approximately 0.5% of the retaining height. However if during the test 50% of this threshold value is reached, no more additional load will be applied. This leads to the safety criteria in Table 2.

Load addition is also prohibited if during visual inspections large movements or new cracks are observed.

Table 2: Threshold values for displacements of survey nails

	QWC	QWD
Load addition prohibited	10 mm	25 mm
Immediate load removal	20 mm	50 mm

3.3 Test results

The test results for quay wall D are discussed in detail. The results and conclusions for quay wall C are similar.

In both load tests the maximum load of 30 kN/m² is achieved and this load has been held for 7 days as planned.

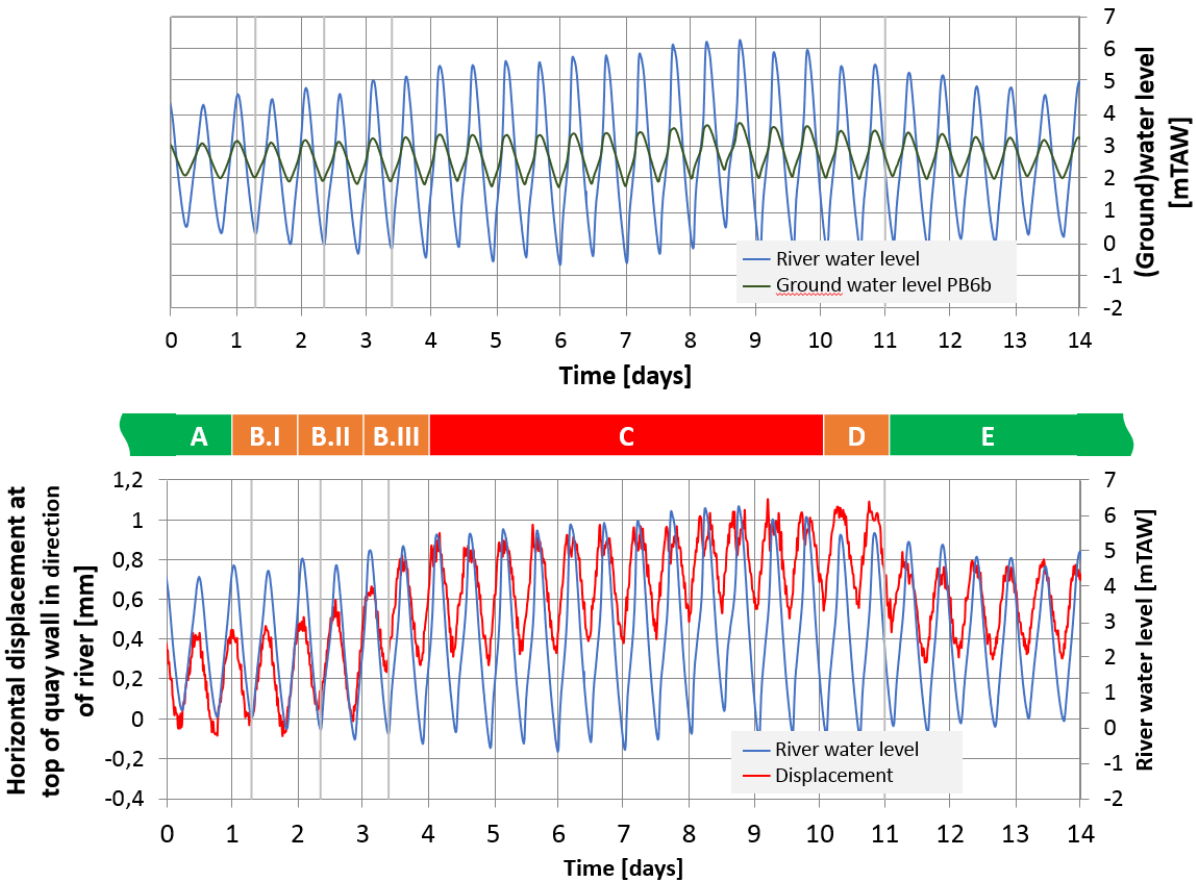

Figure 7: Measurements Quay Wall D

3.3.1 Water levels

The measured water levels are shown in Figure 7. The maximum load is present during a spring tide period. The tidal influence in the ground water level is clearly visible.

3.3.2 Deformations

The measured deformations are small. The survey nails show displacements that are lower than the measuring accuracy of the topographic measuring system, i.e. lower than 1.0 mm.

The In-Place Inclinometer (Model 6150C MEMS, Geokon inc.) has an accuracy of 0.05mm/m, so the overall accuracy at the top of the quay wall is in the order of 0.9 mm. The results in Figure 7 clearly show the influence of the tides. Every tide, the top of the quay wall moves about 0.5 to 0.6 mm in the horizontal direction normal to the quay wall. At a high water level, the top of the wall shifts towards the river which seems contra intuitive: on other gravity walls in tidal environment, shifting towards the river is observed during low tide, for example (Turner, 2013). The tidal movements of a gravity wall are caused by two mechanisms: 1) The weight of the water at a high water level compresses the subsoil in front of the quay wall so the wall tilts towards the river; 2) A negative water level difference over the gravity walls at a low water situation causes a resulting force and hence movement towards the river. For quay wall QWD (and also QWC) the influence of the first mechanism appears higher than the second one.

As can be seen, the load increase during phase B causes an additional horizontal displacement of about 0.6 mm towards the river. At the end of phase C, the movements tend to stabilise, however a prolongation of phase C should be desirable to make this effect more visible. Contractual restrictions made this prolongation not feasible.

After phase D, when the load is removed, the top of the wall shifts back about 0.3 mm landwards. Two weeks after phase D, the irreversible horizontal displacement of the top of the wall remains 0.2 mm relative to the pre-load test situation.

Although the measured displacements are small, the effects of the tides and the load test are clearly visible. The displacements never exceeded the threshold values in Table 2.

3.3.3 Visual inspections

The visual inspections show no signs of additional damage to the quay wall. Not one existing surface crack did grow larger during or after the load test.

4 CONCLUSIONS

Since the stability of the quay walls cannot be proven by the use of calculations, full scale load tests are conducted. Both quay walls endured the load test with success so they can be safely released for a surcharge load of 20 kN/m². If no load test was applied, costly stabilisation measures - as for most of the other quay walls on the right river bank in Antwerp - would be needed to guarantee a safe future use of the quays. Now only limited surface restoration works are required.

The full scale load test approach could be a valuable alternative to a classic design by calculations approach in cases where conservative design assumptions are needed due to lack of information on the foundations of historic constructions.

5 ACKNOWLEDGEMENTS

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