HISTORICAL QUAY WALL RENOVATION IN ANTWERP, BELGIUM

by

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Abstract: The historical quay walls of the river Scheldt are a major landmark of Antwerp. After almost 150 years of continuing instabilities, the right bank quays are stabilized along their entire length. This durable renovation project, being challenging in its own right from a geotechnical point of view, is further complicated by the requirement to preserve the historical bluestone facing as well as the historical position ('quay line') of the quays. Innovative techniques are devised to restore the Scheldt quays to their old glory in a cost efficient way. The works are currently ongoing in several zones, two of which are discussed as case studies in this paper.

Key words: waterway infrastructure, quay walls, renovation, stabilization, historic masonry

1 INTRODUCTION

Among the most characteristic features of the city of Antwerp (Belgium) are its historical Scheldt quays. Stretching about 5.5km along the western edge of the city centre, the quays were constructed at the end of the 19th century. Severe indications of instabilities have been observed as early as during the construction itself. Despite the multitude of efforts to alleviate these instabilities over the last century, they continue until today.

In 2005, 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:

- The stabilization 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);
- The protection of the city against increasing storm surges as a result of climate change. This goal is part of the integrated Sigma Plan (the plan to protect the whole of the tidal Sea Scheldt bassin against storm surges) and requires a new storm surge barrier (+9,25mTAW) to be built on the existing quays.
- A facelift to restore the city's link with the river by incorporating urban development and mobility, creating new public domains and preserving historical monuments. The quay walls are a significant part of the latter.

In this paper, the focus is on the stabilization of the historical quay walls. First, the history and the construction of the quays are treated, along with a description of the stabilization measures taken since their construction. Then, the master plan for the stabilization of the quay walls is explained before focusing on two case studies of stabilization works in the final section of the paper.

2 A BRIEF HISTORY OF THE QUAY WALLS

2.1 A brief history

Over the last centuries, the river banks of the city of Antwerp have been reorganized several times. After a first wave of harbour expansion works initiated by French Emperor Napoleon at the end of the 18th century, the city experienced an industry boom when the toll charges on the Scheldt were lifted by the

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Dutch in 1863. By the end of the 19th century, the course of the river Scheldt was straightened to make the harbour more accessible, more spacious and more efficient. As a result, all right bank quays had to be renewed. The works were carried out in the northern stretch of approx. 3.5km (1877-1884) and the southern stretch of approx. 2km (1897-1903).

Significantly altering the natural course of the river, hundreds of medieval houses, the old ship yards and the Spanish fortress near the city centre had to be demolished to build the new quays. As a result, the quay walls were only partially constructed on the existing banks and mostly in wet conditions in the original river bed, as shown in Figure 1. These circumstances have had an important influence on the design and the construction method of the quay walls.

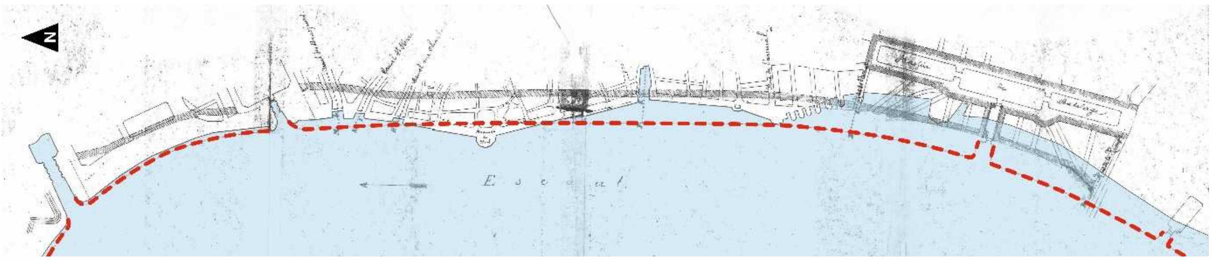

Figure 1: Location of the new quay walls (red line) w.r.t. the original river banks at the end of the 19th century (based on the original construction plans).

2.2 Construction method

The quay walls were designed as massive gravity retaining walls, as shown in Figure 2. The foundations of these gravity walls were conceived as massive concrete bodies, constructed within a pneumatic caisson. The steel caissons are about 25m long, 9m wide and 2,5m to 5m high. The bottom level of the caissons is located between -10mTAW and -15mTAW.

On top of the foundation caisson, temporary steel walls were mounted to allow for the construction of the actual masonry gravity walls in dry conditions. The tidal river Scheldt has mean water levels varying between approx. 0mTAW and +5,5mTAW at resp. low and high tide. Once the low tide line of the river Scheldt was reached, the foundation caissons were filled with concrete. Above the low water line, the quay walls were cladded with quarried Belgian bluestone on the riverside to enhance the durability of the walls. Once the quay wall had reached its final height at approx. +7mTAW, a sand fill was carried out behind the quays to level the surface. In some areas fascine mattresses were used to stabilize the significant underwater filling operations.

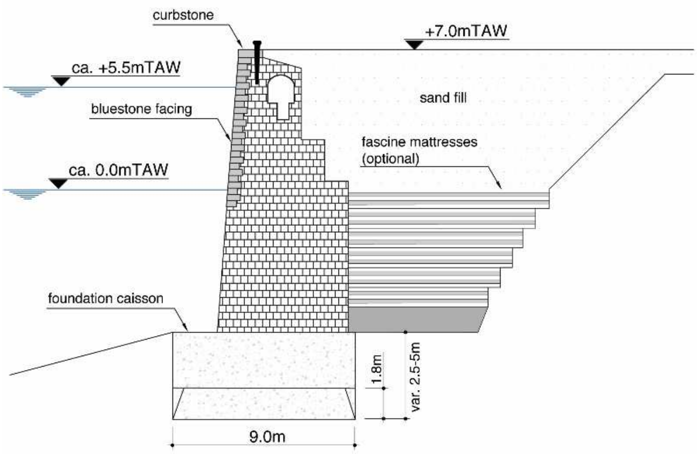

Figure 2: Standard section of the masonry quay walls on top of the foundation caissons, filled with concrete

2.3 Stability issues

As early as during the construction works, the new quay walls started to show signs of instabilities. The resistance against translation proved insufficient and in some areas the quay walls gradually shifted in the direction of the river.

The main cause is attributed to several crucial mistakes in the original design. Not only were the geotechnical soil characteristics underestimated, also and even more importantly, the influence of the ground water levels was disregarded in the design. The foundation levels should thus have been set deeper. Furthermore, it was observed upon completion that the drains behind the quays were mostly malfunctioning.

Although measures were taken instantly during the building phase, at several occasions over the last century severe instabilities have been reported. These instabilities are mostly attributed to high groundwater levels behind the quay walls. The most severe stability issues were encountered in the south, where the new quays had to be constructed in the original river bed (cfr. Figure 1) and where the presence of the tertiary Boom clay layer influences the stability. This significant tertiary clay package is found at shallower subsurface depths in the south than in the north.

As a result, a patchwork of temporary and permanent remedial measures can today be found along the Scheldt quays, some of which are illustrated in Figure 3:

- To increase the sliding resistance, additional counterfort caissons have been placed in front of the quay walls in large stretches of the quays. Sometimes, clay or blast furnace slag embankments were placed in front of the quay walls as a permanent measure as well.
- To alleviate the risk of tilting, additional drains and stress relieving vaults were installed behind the upper parts of the gravity walls. In several areas, the soil behind the quay walls was removed and replaced by lighter materials such as ashes. Exceptionally, the soil behind the quays was permanently excavated, making the quay in this zone useless for any activity.

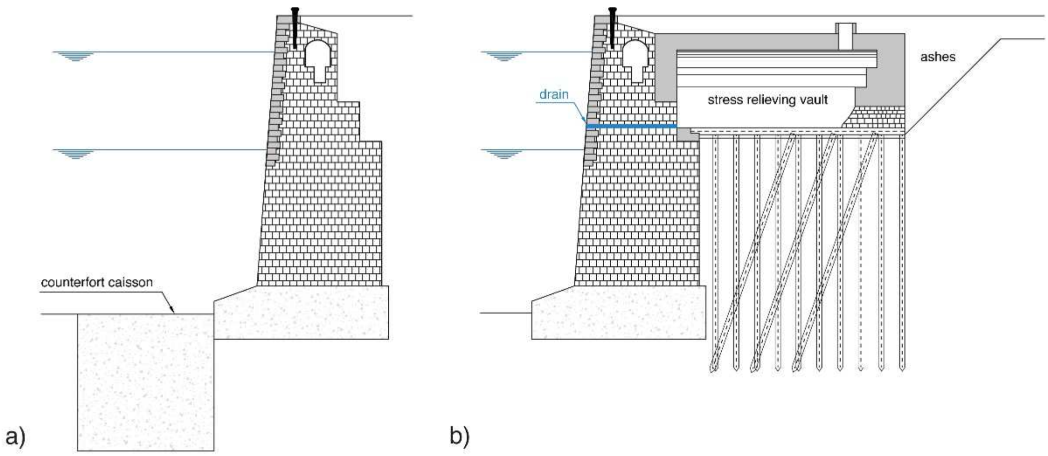

Figure 3: Remedial measures to limit horizontal movements with (a) counterfort caissons in front of the quay wall or (b) stress relieving vaults behind the quay walls

3 A MASTERPLAN FOR THE STABILIZATION OF THE QUAYS

Knowing this history of instabilities, a masterplan for the renovation of the quays was commissioned by DVW. The overall condition and stability of the quays was surveyed and studied along the entire 5,5km length.

First of all, all recorded historical instabilities and stabilization measures were inventoried for each zone. This was accompanied by thorough field inspections, a borehole campaign and diving inspections. A multitude of damage phenomena were found. Some of the most frequent observations are listed in Figure 4.

a) Cracks near the juncture between two caisson walls

b) Damage of the bluestone facing in the tidal zone as a result of weathering

c) Diagonal crack near a corner due to shifting towards the river

d) Permanent excavation behind the quay wall

e) Subsidence on the quay due to movements of the quay wall and heavy loads

f) Damage to the curbstone as a result of collision with moored ships

Figure 4: Inspection of damage phenomena on the Scheldt quay walls

It is clear that not all the recorded damage was due to the instability of the gravity walls, e.g. cases (b) and (f) in Figure 4. Nevertheless, the renovation of the quays is seized as an opportunity to tackle all observed damage phenomena for the future.

Then, calculations were performed to assess the stability of the quays in the current situation, taking into account all remedial measures carried out over time (additional counterfort caissons, relieving vaults, etc.). An extensive geotechnical survey determined the soil characteristics according to present day practice. Subsequently, the stability checks for bearing capacity, shifting & tilting resistance, global sliding etc. were performed in line with the current design codes.

The results of these geotechnical calculations showed that the required safety for the different failure criteria was met in nearly none of the sections. Even after grading the requirements down to a unity check and after calibration of the parameters assuming that a section that had never encountered damage or instabilities in the past should be the reference for equilibrium, the bearing capacity and the resistance to translation proved insufficient in nearly all the sections of the quays.

Based on all data and calculation results collected, a risk chart was developed to indicate the zones with the highest instability risks and hence the highest priority for renovation. In addition, remedial measures were proposed ranging from expensive to cost-effective or, in other words, drastic reconstruction to minor restauration according to the state of the quay walls in each zone.

With the conclusion of the study phase in 2010, the long term masterplan was developed. This masterplan splits the renovation works into subareas with higher/lower risks and priorities. It includes several requirements for future use of the quays. Harbour activities are abandoned as a future use for the main part of the historic quay zone, but mooring of ships at the historic quays is still desired and is taken into account in the calculations as summarized in the following.

- It should be possible to use the quays for mooring of ships, e.g. the design vessels used in the design of d'Herbouvillekaai as specified in Table 1. Therefore, the original bollards installed every 20m along the quays are incorporated in the new design. The required characteristic bollard pull is 1000kN.
- The overload on the quay areas redeveloped as public domain should be at least 20 kN/m².
- In areas where the new storm surge barrier is located close to the quay wall these loads are incorporated in the quay wall renovation design.

		MARITIME NAVIGATION		INLAND NAVIGATION
		passengers	freight	freight (cat. VIb)
LAO	(overall length)	260m	224 m	140m
LPP	(length between perps)	220m	220m	-
B	(beam)	33.1m	32.3m	15m
D	(draught)	7.6m	8m *	3.9m
DWT	(deadweight tonnage)	37 600tons	approx. 30 000tons *	-

* maximum allowable draught restricted from 13.3m to 8m for mooring along the Scheldt quays

Table 1: Design vessels for mooring along the renovated quay wall at d'Herbouvillekaai

4 CASE STUDIES

The stabilization of the quays officially started in 2012. Several subprojects are currently being executed or prepared. In the following, two cases explain the design of the stabilization and several practical issues of execution.

4.1 Case 1: d'Herbouvillekaai

4.1.1 Monitoring & emergency measures

Based on the first conclusions of the stability study discussed in the previous section, the displacements of the quay walls were carefully monitored as none of the quays met all present day safety standards. The dedicated expert panel indicated that several precautionary measures had to be taken in the subarea with the highest risk of instabilities. In this zone along D'Herbouvillekaai in the southern part of the quays, horizontal movements of the quay walls could still be observed, despite all measures that had been taken in the past (counterfort caissons, relieving platforms, etc.). Additionally, this area was at the time still being used for intensive transshipment activities.

As a precautionary measure, the allowable load in the zone of influence of the quay wall was restricted since additional permanent displacements up to 7mm had been measured after the quay had temporarily been used for storage of steel beams. Additionally, an elaborate monitoring campaign was set up. Inclinometers and monitoring wells were installed to follow the evolution of the shifting and tilting movements of the quay walls.

The results of this monitoring campaign were alarming. Despite the restrictions on the allowable loads, the measurements indicated that the shifting movement had worsened. The results also show that the measured displacements are only the result of the moving gravity walls as the soil below the foundation caissons remains stationary.

In 2011, the restrictions were intensified but by September 2012 progressively worsening displacements could still be observed. In one measurement even a tilting of the walls was diagnosed, as shown in Figure 5. As a result, all industrial activities on the quays were prohibited with immediate effect in this area.

Other remedial measures included the installation of a clay berm in front of the gravity walls and eventually even the excavation of soil behind the quays to alleviate the active soil pressures, as shown in Figure 6. Despite these drastic measures the shifting and tilting continued, albeit at a slower rate as shown in the inclinometer results in Figure 5.

4.1.2 Stabilization design

The deteriorating state of the already damaged quay walls numbered down the options for renovation drastically. Any solution considering the reuse of the existing quay walls in the future had to be disregarded for several reasons. First of all, there was no room for experimentation with new techniques in the area. The mere fact that an implicitly uncertain technique would have to be carried out in close vicinity of the already unstable quays was inadvisable. Furthermore, the overall stability in any of the scenarios with reuse would always somehow depend on the structural integrity of gravity walls in their current state. Finally, any solution with a permanent berm in front of the quays for stabilization, similar to the temporary solution in Figure 6, was also not an option as this compromises the requirement for the mooring of ships along the entire length of the quays.

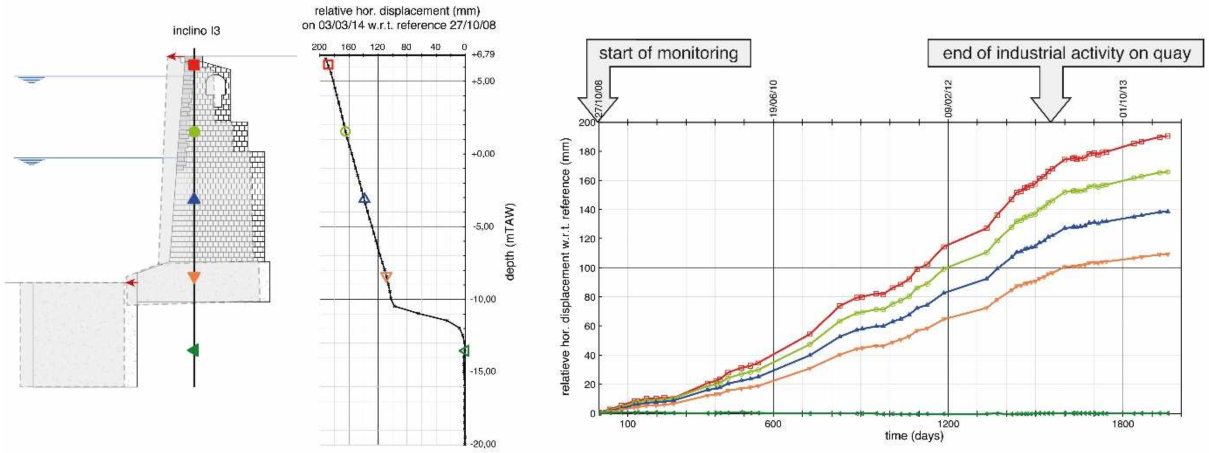

Figure 5: Relative horizontal displacements as a function of depth and time from inclinometer measurements inside the quay wall at D'Herbouvillekaai

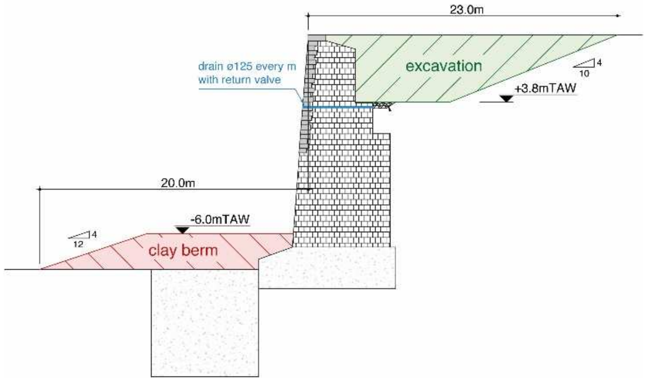

Figure 6: Drastic remedial measures along D'Herbouvillekaai in 2012: clay berm and drained excavation

Another important influence factor is the risk of encountering problems or instabilities during construction. Solutions where the entire present quay wall is retained and encapsulated inside a new retaining structure could involve execution problems, e.g. when counterfort caissons are damaged. Furthermore, an encapsulated wall would damage the historical 'line' of the quays.

In the end, the only viable and durable solution was the radical reconstruction of the quay walls, at the exact historical position, as shown in Figure 7. The existing gravity walls are entirely demolished (including the pneumatic foundation caissons) and replaced by a tube combined retaining wall with 37.5m long steel tubes. Two rows of anchors are attached to an anchor wall located 38m inland. This sheet piling wall is used as retaining wall during the construction phase. The river bed near the toe of the combined wall is protected with a rocky debris fill. Any original counterfort caissons present in the river bed are left at their current location. Removing them would influence the stability of the new retaining walls in a negative way.

4.1.3 Preservation of the historical bluestone facing

The original look of the historical Belgian bluestone facing and curbstones will be preserved by integrating the original bluestone in 7.5m high capping beams on top of the combined wall. To achieve this, the bluestone is carefully removed during the demolition of the gravity walls.

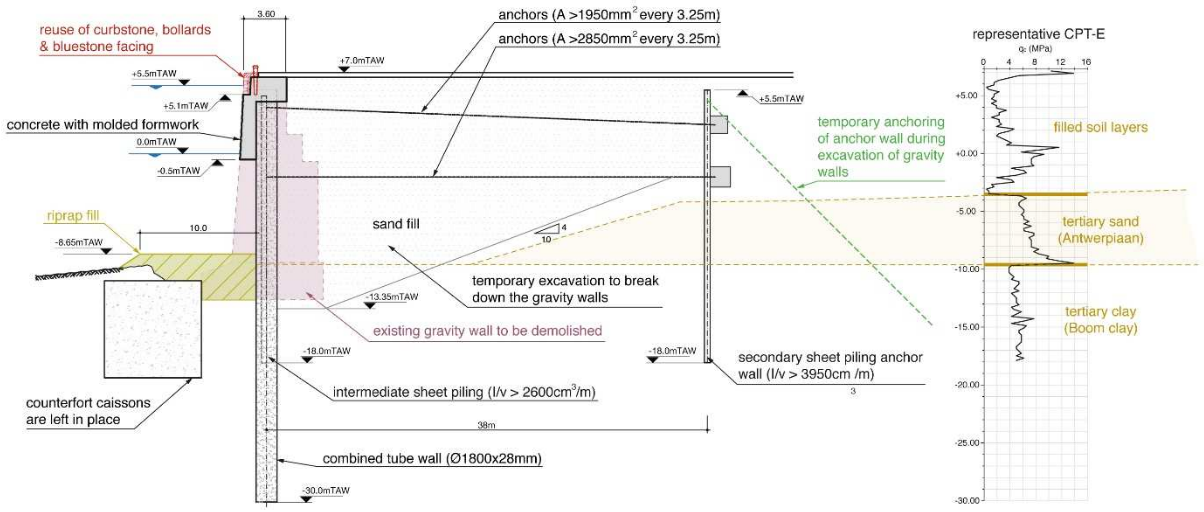

Figure 7: Stabilization design along D'Herbouvillekaai

A detailed photogrammetry showed that it is unfeasible to reuse all bluestone blocks since many stones in the tidal area have weathered and broken over the last century (cfr. Figure 4b). Moreover, a lot of blocks, visually in a good state when still at place, decomposed during or shortly after removing them out of the historical masonry. Because too many stones had become too brittle to reuse, the following innovative solution was introduced to maintain the look of the historical quay walls.

Under the high water line, concrete walls are installed of which the outer formwork is a mold of the original bluestone masonry, see Figure 8. This mold is made on site to preserve the original look of the quays. This approach was approved by cultural heritage given the fact that most of the concrete surface will be covered with mossy and muddy deposition in the tidal zone over time, eventually resembling the current state of the quays. There are several additional advantages:

- In contrast to the bluestone masonry, the concrete is not susceptible to large scale cracking and loss of joint material in the masonry as a result of weathering.
- Only the historical bluestones with the best quality are reused above the high water line.
- The execution is much easier, faster and thus cheaper.

Figure 8: Original bluestone facing (left) and new concrete molded formwork with on top three rows of reused blue stone blocks (right)

4.1.4 Construction phasing and lessons learned

First, the anchor wall and two transverse walls are installed to create a temporary retaining structure behind the original gravity walls. Once the temporary ground anchors in these retaining walls are installed, the construction pit is excavated. Note that the works are carried out in phases in 5 adjacent zones, starting in the middle and moving outwards. The transverse walls can hence be reused in the construction pit of the adjacent zone with temporary ground anchors being installed in the opposite direction. It is important to note that the design is meticulously followed up during execution to avoid interference of the permanent anchors and the temporary transverse ground anchors. Permanent anchors in the vicinity of future temporary ground anchors are protected with steel casings.

In the next phase, the blue stone facing is removed carefully and then the gravity walls are demolished using explosives. Because of the presence of sensitive buildings nearby the construction site, a trial blast campaign is first performed. During these trial blasts, vibrations are measured on nearby buildings. These measurements are used to optimize the blasting program. Thereby, each demolition phase is monitored closely, resulting in a subsequent optimization of the blasting program in the next zone.

The foundation caissons are loaded with a higher dose of dynamite than the upper masonry wall. The dynamite bars are inserted from the top of the wall in semi-vertical boreholes. Before blasting at the boundaries of the demolition zone, vertical joints are drilled in the wall in order to prevent damage of the neighbouring quay wall. The explosives are detonated during a high tide to prevent flyrock.

After the explosions, the masonry is reasonably well fragmented. However, the steel foundation caissons are tougher than expected. Especially in the first zone, the caissons are not very well fragmented after blasting. Additional demolition works are carried out using a pontoon based hydraulic crusher. For the next two zones, a higher dose of dynamite is used and this improved the fragmentation.

Afterwards, the debris is dredged and the steel, concrete and masonry parts are separated. The steel parts are transported to a recycle plant and the concrete and masonry are transported by ship to a crushing plant for reuse outside this project. An aerial view of the project site after demolition of the first zone is shown in Figure 9.

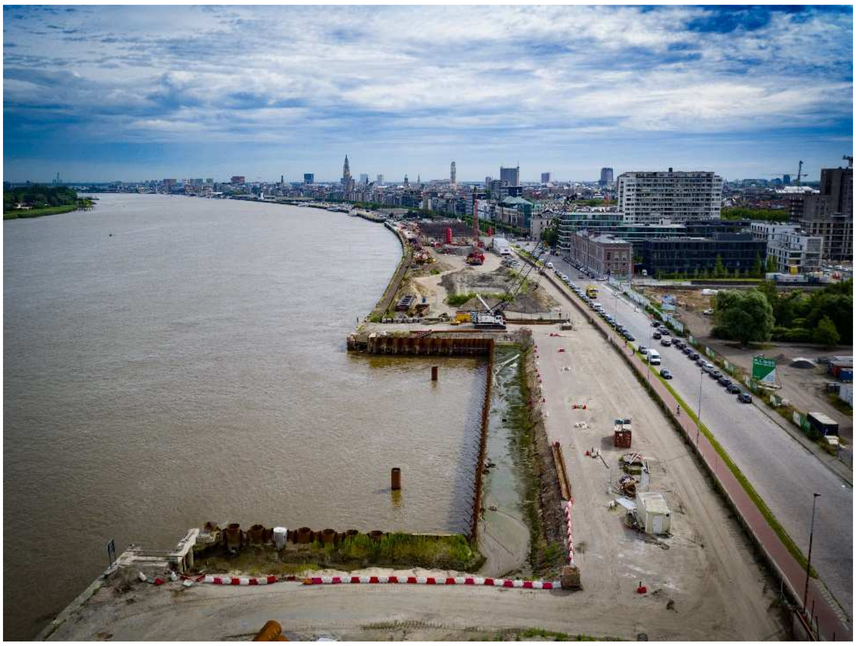

Figure 9: Aerial view of project zone after demolition of quay wall in the first zone (source: Braincube commissioned by DVW)

Then, the new front wall is installed using floating equipment. This combined wall is composed of steel tubes (1800/28) with a length of 37.5 meter and intermediate steel sheet piles (AZ36-700N) with a length of 25.5 meter. The elements are installed by vibration driving and further impact driving with characteristics as shown in Table 2. No other measures are needed to achieve the final depth without generating unacceptable damage or nuisance in the surrounding.

Two double sheet piles of the combined wall are partially omitted to achieve the same water level in the tidal river and in the construction pit at all times. Note that the free standing front wall is not capable of withstanding a large water level difference.

	Vibration driving	Impact driving
Steel tubes: weight of 46.3 tons for 1 tube (front wall)	PVE 300M - total weight including clamps: 45 tons - max. centrifugal force: 6 150 kN - max. frequency: 23 Hz	S-280 (hydraulic hammer) - total weight: 30.5 tons - max. blow energy: 280 kNm - blow rate: 45 bl/min
Sheet piles: weight of 5.1 tons for a double sheet pile (anchor wall and intermediate sheet piles of front wall)	ICE 1412 - total weight including clamps: 13 tons - max. centrifugal force: 2 300 kN - max. frequency: 23 Hz	S-70 (hydraulic hammer) - total weight: 8.3 tons - max. blow energy: 70 kNm - blow rate: 50 bl/min

Table 2: Driving characteristics of sheetpiles and tubes

Subsequently, the rocky debris fill is installed at the toe of the front wall and a sand backfill is placed under water in the construction pit up to a level of -2mTAW. This fill is compacted by vibroflotation. Cone penetration tests are carried out in order to verify the compaction of the fill. When compaction is sufficient the two remaining sheet piles are installed to close the construction pit. Then, the pit is dewatered. On places where no fill is placed after installation of the front wall, as a result of the high silt transport rates of the river Scheldt, a significant siltation is reported. The bottom of the pit is cleaned and profiled.

On the landside of the anchor wall a second construction pit is excavated and dewatered as shown in Figure 10. This pit is used for the construction of the concrete waling beams and for the tensioning of the permanent anchors. These anchors are strand anchors with a double corrosion protection according to NBN EN 1537. First, the lower anchors are installed and tensioned slightly with 40 tons. The construction pit between the anchor wall and the front wall is partially filled. Then, the lower anchors are tensioned to a final lock off load of 200 tons. In the next phase, the upper anchors are installed and tensioned to a lock off load of 140 tons. Afterwards both construction pits are completely filled.

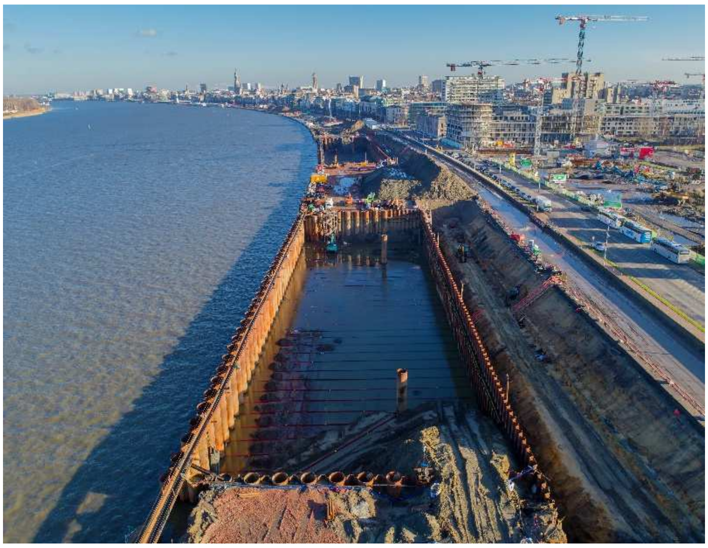

Figure 10: View of the first (left) and second (right) construction pit (source: Braincube commissioned by DVW)

Finally, the large capping beam is constructed at the top of the front wall by means of prefabricated concrete elements with a weight of 90 tons each and a height of 6 meter as shown in Figure 11. These elements are mounted on the combined wall and the joints between the element and the steel wall are sealed. In the transverse walls of each element, interconnection holes are foreseen every 1.0 meter in vertical direction. These holes are supplied with a fine metallic grid which is water permeable and which holds fresh concrete. An interconnection between the river and the chamber formed by the element and the steel wall is essential to secure the stability of a mounted element in the tidal environment. This will prevent the element from displacing and floating. Reinforcement is placed in the chamber and consequently concrete is poured in steps of 1.0 meter. Before each pouring phase previous concrete surfaces and rebar are cleaned. The bollards are integrated in the concrete capping beam.

Figure 11: Prefabricated concrete facing elements of the front wall with interconnection holes, ready for placement (left) and elements mounted on the combined wall (right)

Finally, the historical bluestone masonry and curbstone are placed above the high water line. The bluestone blocks are recycled from the old gravity wall. Experience shows that only 30 to 40 percent of the original stones is fit for reuse after dismantling the old wall. Due to the historical port activities on the project site, the original curbstones have disappeared or are heavily damaged, which makes reuse impossible. New Belgian bluestone curbstones are applied here.

A temporary surface finishing is foreseen on the quay areas until the final works on the new storm surge barrier (+9,25mTAW) commence.

4.2 Case 2: De Gerlachekaai

4.2.1 Test phase

The quay walls at De Gerlachekaai are in a better condition than the quay walls at D'Herbouvillekaai. Hence, measures with a minor impact on the original quay walls were at first considered for stabilization.

In a first design effort, a stabilization technique with transverse grout walls below the current gravity walls was considered as shown in Figure 12. Five parallel and equidistant shear panels were foreseen per foundation caisson to extend the foundation depth. Each shear panel consisted of multiple cylindrical VHP jet-grout columns with a large diameter. The jet-grout columns were constructed after drilling through the existing masonry walls and foundation caissons from the quay platform. They were armoured with rebar steel upon completion. Before the jet-grouting started, all voids and cracks intersecting with the drill holes were gravitationally injected with a cement grout. Additionally, ground anchors had to be installed to limit the horizontal movements of the quay walls. An integrated concrete capping beam with bluestone façade was constructed at the top of the gravity wall. The joints between adjacent caissons under the low water line were repaired.

While this remedial technique is much less invasive, it is also more risky. The jet-grouting had to be performed at very large depth, in a very stiff clay layer, underneath the historical structure. As a result, the quality of the grouting operation was difficult to control. It was therefore decided to perform the stabilization in a test section first to assess the efficiency of the proposed design.

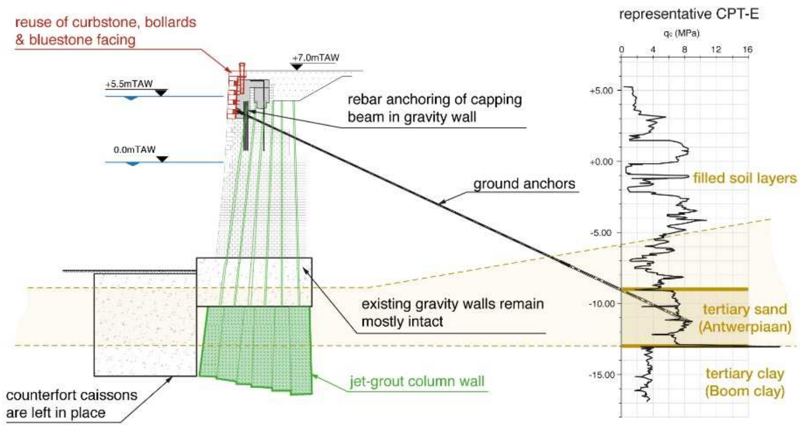

Figure 12: Stabilization design with jet-grouting in test phase

Already during the first jet-grouting operations, problems were encountered. The spoil return flow at the top of the drill hole was not continuous. The quality of the large diameter grout columns was investigated with core drill holes. Many clay inclusions were found along the entire depth of the grout columns, as shown in Figure 13. The formation of these inclusions was attributed to the properties of the tertiary Boom clay. Due to the high stiffness and plasticity of this pre- and overconsolidated clay layer, it seemed impossible with large diameter jet-grouting to cut loose clay particles that are small enough to travel through the ring gap in the drill hole. As a result, clay chunks block the ring gap in the drill hole and pressure is built up, within seconds, in the subsoil. The spoil is then directed to alternative low resistance paths, e.g. via the bottom of the caissons, and the quality of the grout columns is drastically reduced. In addition, during jet-grouting operations, the movement of the quay wall was monitored continuously. Even after successfully executing the first series of grout walls, unacceptable movement of the historical quay wall was monitored during jet-grouting operations.

Figure 13: Core drill holes of the jet-grout columns at De Gerlachekaai with clearly visible clay inclusions

About 20 different sets of jet-grouting parameters were tested, but none of them gave satisfying results. In the end, it was decided that the performance of the grouting technique was too uncertain in the Boom clay and this stabilization technique was completely abandoned.

4.2.2 Stabilization design

A more expensive but more controllable design was developed for this area of the quays. In the optimized design, which also preserves a big part of the historical wall, a diaphragm wall is installed behind the quay walls, as shown in Figure 14. The diaphragm wall is embedded in the Boom clay at a depth of -23mTAW. The horizontal stability of the diaphragm wall is ensured with 2 rows of ground anchors every 1.8m. An additional ground anchor is installed every 20m at the location of a bollard.

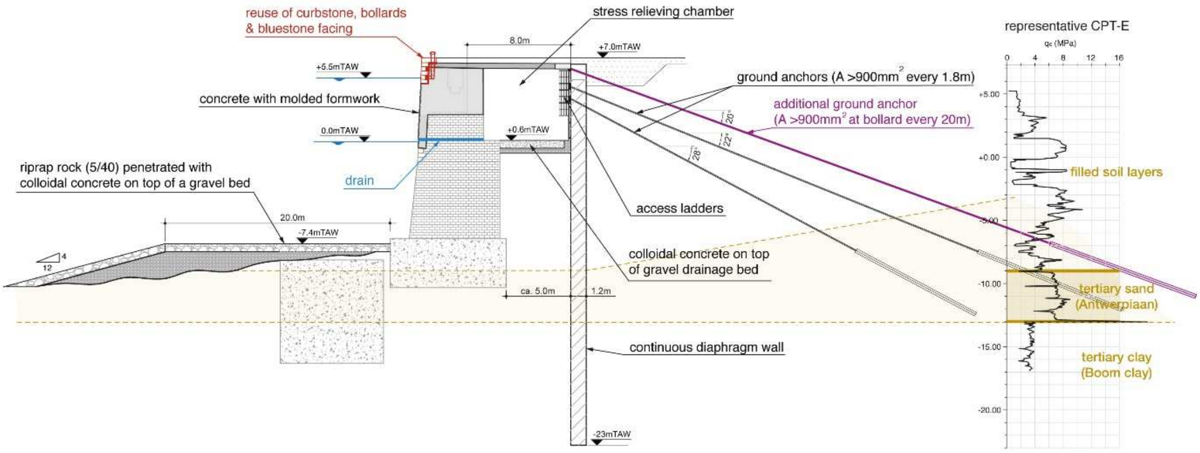

Figure 14: Stabilization design for De Gerlachekaai

Between the original gravity wall and the diaphragm wall, stress relieving chambers are additionally installed to reduce the loads on the original gravity wall. Four drainage tubes per caisson are drilled through the original masonry wall to allow the stress relieving chambers to follow the tide in the river Scheldt.

Similarly as in the design of D'Herbouvillekaai, the upper part of the masonry quay walls is replaced by a concrete structure to enhance the durability of the quays in the tidal zone for the future. Again, the outer formwork is a mold of the original bluestone masonry. Above the high water line, the original bluestones as well as the bollards are reused together with new Belgian bluestone curbstones.

At the river side, a small elevation of the original river bed and a new bed protection, which should avoid erosion, are foreseen. Inside the stress relieving chamber, the floor is conceived as a 0.6m thick layer of colloidal concrete on top of a gravel drainage bed.

4.2.3 Execution phasing and lessons learned

Because the changes to the original gravity walls are less drastic in this area, the phasing of the works is less complicated in comparison with D'Herbouvillekaai. In the first phase, the new bed protection is installed. A rip rap layer with a variable thickness and a width of 30 meters is placed at the toe of and parallel to the quay wall. Then this layer is covered with immersed asphalt mattresses to obtain a durable bed protection as shown in Figure 15.

Figure 15: Immersion of the asphalt mattresses in front of the quay wall

Afterwards, the new diaphragm wall is constructed behind the quay wall. After partial excavation behind the existing quay wall, the ground anchors are installed. The excavation continues until the bottom level of the relieving chamber and the chamber floor is constructed. In the next phase, the chamber front wall and the chamber roof plate are constructed as shown in Figure 16. At the locations of the bollards, additional ground anchors are placed.

Figure 16: View of the chamber front wall placed on the existing quay wall (left) and view inside the stress relieving chamber (right)

Now, holes can be drilled at the bottom of the relieving chamber, connecting the river Scheldt. At the rhythm of the tides, twice a day, the chamber fills with water from the river, relieving the old historical wall from water pressure. The first chamber went operational in 2015. Because of the high silt transportation rate in the river, monitoring of the sedimentation load in the chamber is necessary. Through the two years of measurement, we notice a sedimentation of silt against the diaphragm wall diminishing in a funnel shape to the drilled holes at riverside. After 2 years, the sedimentation seems to find an equilibrium. However, further monitoring is required for the next years to implement a maintenance program if necessary.

Finally, the top of the original quay wall is demolished and the new concrete capping beam is installed. The original bluestone masonry and the bollards are installed as well. The old curbstone is in very bad shape which makes it impossible to reuse. Therefore, a newly mined Belgian blue curbstone is placed, marking this very special place in the city of Antwerp.

5 CONCLUSIONS

Almost 150 years ago, the new quay walls required cutting edge technology to be constructed. After several attempts over the last century to alleviate the instabilities resulting from serious design defects at the time of building, the present renovation project intends to preserve the iconic Scheldt quays for the generations to come. To reach this goal in a cost effective way, innovative and site-specific techniques are required once more.

Originally estimated as works for the next 15 years, the conclusion of the entire renovation project of the Scheldt quays, including all works for the storm surge barrier and the urban facelift of the quay area, might take (much) longer. By tackling the most unstable zones first, our understanding of the Scheldt quay wall structures has greatly improved. This gives ground for optimism for the future renovation works.