Diaphragm walls, a reliable solution for deep excavations in urban areas?

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In recent years, there have been major problems with deep excavations in urban areas supported by diaphragm walls. In some cases, like the fatal Nicoll Highway collapse in Singapore and probably also the collapse of the Archives in Cologne, the quality of the D-walls was not the cause. However, the major leak in the walls of the Boston Big Dig was related to poor D-wall quality. Recently in Belgium and the Netherlands, leakage through the D-walls at stop end joints in deep excavations led to very serious settlement behind the walls. In the case of the North-South line in Amsterdam, there was severe damage to historical buildings. Most cases have generated considerable additional costs and delays, and subsequently public and political concern. This constitutes a threat to the feasibility of future underground projects because of administrators' concern about the related risks.
This paper describes some cases in which the quality of the D-walls failed to meet the requirements and analyses the most probable reasons. The authors discuss the lessons learnt and the measures needed to restore confidence in D-walls.

INTRODUCTION

General

Delta areas throughout the world are appealing places to live and work, with considerable economic potential, leading to the intensification of construction activities for transport infrastructure, buildings and parking facilities, especially in the urban centres of these densely populated areas. This situation represents a major challenge for the construction industry and in particular for deep foundation contractors, designers and manufacturers because of the increasingly intimate interaction between construction activities and urban living. Globally, prospects for the deep foundation sector are very promising, especially in densely populated areas where modern society is pressing for innovative solutions for congested inner cities and for the reduction of nuisance during the execution of underground works. If the sector succeeds in reducing nuisance, in managing the risks that regularly result in project budgets being exceeded and in minimising the impact on neighbouring buildings, it will meet most of society's demands. It is therefore essential for the entire sector to continue with technical innovations and also to manage, in particular, the areas of risk management, quality control and monitoring.

In the case of deep excavations in urban areas close to neighbouring buildings, diaphragm walls are, in particular in saturated sands the best feasible solution for the retaining structure. Although the costs of D-walls generally exceed the costs of other alternative wall solutions, the advantages are decisive in many situations. An advantage of D-walls is the limited impact of the installation on the built environment, the high stiffness and bearing capacity and the retention of water, even at considerable depths. This makes D-walls the preferred solution for deep excavations in saturated non-cohesive soil conditions. In these conditions steel sheet pile walls risk leakage due to declutching and secant pile walls have proven to be not watertight in many cases (Korff et al., 2007). In particular, the formation of a reliable barrier for water incursion means that D-walls have been chosen worldwide for deep excavations in urban areas.

State of the art

The authors have been involved in many D-wall projects from the eighties to the present. In many of these projects, the reliability of the water barrier was a vital issue. After the teething troubles with D-walls in the sixties and seventies, things went well. There were very few cases involving leaks, with the exception of some minor seepage from panel joints. The development of stop ends containing water-retaining profiles was a major step forwards. Thereafter, the message was that, for deep excavations in urban areas, where impact on the built environment and water-tightness were important, the D-wall was the reliable solution. This was confirmed by the experiences from the Netherlands listed in table 1. Prior to 2007, severe leakage was a feature in about 1 in 2000 panel joints.

However, in the past ten years, there have been several projects involving the collapse of D-walls

or D-walls with major problems. Brandl (2007) describes the case of the Europlex-Building in Warsaw in 1998.
In the case of the Nicoll Highway in Singapore in 2004, which resulted in fatalities, the quality of the diaphragm walls was not the cause. The inquiry relating to the Cologne Archives event in 2009 is still ongoing. The major leakage in the diaphragm walls of the Boston Big Dig, however, was related to poor D-wall quality (McNichol, 2000)

Project	**No. of joints**	**Year**	**Known problems**
Rotterdam Willemsspoor	100	1985 / 1990	None
Rijswijk Station	250	1990 / 1995	None
The Hague "Sousterrain"	400	1995 / 2000	1 small leak
Rotterdam HSL Tunnel	300	2000 / 2005	Several small leaks
Venlo – Maasboulevard	150	2008	1 crack, 1 sand inclusion
Maastricht Markt-Maas	100	2006	None
Almelo – Railway	295	2008	None
Rotterdam – Lightrail	70	2007	1 small leak
Rotterdam Central Station	450	2007	1 severe leak
Subtotal	***2115***		***1 severe leak 2 small leaks 1 sand incl.***
Amsterdam Metro Rokin Vijzelgracht Ceintuurbaan	 120 115 120	2008	 Unknown 3 severe leaks None
Total	***Approx. 2460***		***4 severe leaks.Several small leaks***

*) still under construction

Table 1: Overview of projects in the Netherlands with known problems with leaks close to or at stop ends.

In 2008, serious questions were asked in the Netherlands about the reliability of diaphragm walls after one incident during the reconstruction of the Rotterdam Central Subway Station (December 2007) and two major events in 2008 during the construction of the North-South Subway line in Amsterdam when historic buildings were severely damaged due to the inflow of water and sand through the walls. Including these cases, there were 4 severe leaks in approximately 2500 joints. There is therefore a risk of a serious leak in 1 joint in 600. In a medium-sized project with 100 panels, this means that the probability is 16%. A risk this high means that a responsible design needs to take steps in advance to prevent leaks. This would impose an unacceptable claim on the projects' budget and so the effort must focus on determining the cause of the events and improving the D-wall installation process rather than spending large amounts of money on preventive measures.

The cases described here have led to considerable additional costs and delays, and so the public and administrators have started to become concerned. This development constitutes a threat to new underground construction projects.

In this paper, the authors describe four cases that have adversely affected confidence in diaphragm walls and attempt to formulate conclusions about what steps will be needed to restore that confidence.

DESCRIPTION OF RECENT CASES

North-South Line, Amsterdam

The North-South Line in Amsterdam is 9.5 kilometres long. This metro line starts at the surface in the North of Amsterdam, and passes under the historical centre of the city in a twin shield tunnel. In the south of the city, the line re-emerges at surface level between the RAI and South/WTC stations. Five underground stations are under construction. This paper looks at three of them: Rokin (RKN), Vijzelgracht (VZG) and Ceintuurbaan (CTB).

At Vijzelgracht Station, there are fill deposits and soft Holocene clay deposits to a level of about (Dutch reference level) NAP –12.5m (ground level is around NAP +1.5m). These are underlain by the 1st sand layer, from NAP –12.5m to NAP –14/–15m, which is on top of a 2.5m thick sandy silt stratum (the Allerod). The 2nd sand layer is found at about NAP –17/–18m, extending to NAP –26m. Below the 2nd sand layer, there is a stiff clay layer that is approximately 15m thick (the Eem clay). The piezometric head in the 1st and 2nd sand layer is about NAP –2.0 m. Details of the construction and soil profiles can be found in Kaalberg et al. (2005).

The stations are being built in a top-down construction approach on a depth of about NAP –31 m, with 1.2m thick diaphragm walls extending to a depth of approximately NAP (Dutch reference level) –45m. The diaphragm walls consist of panels with lengths of approximately 2.8m and 5.2m. Traditional grabs and steel stop ends with water bars (PVC strips)are used to a depth of NAP –36m to provide waterproofing. The Eem clay layer below NAP –26m provides the water barrier. Before installing the walls ground improvement was done by replacing obstacles with a soft mix in the Holocene deposits.

At Vijzelgracht station in particular, and to some extent at Rokin and Ceintuurbaan stations, numerous joints in the D-wall panels leaked during excavation up to about NAP –12 m. These leaks varied from damp patches to more significant water flows, but up to that depth the wall did not leak much.

There was severe inflow of water and soil through a panel joint for the first time on the 19th of June 2008 in the west wall of Vijzelgracht Station. The excavation depth was approximately NAP –12m at that time. The leak was attributed to a steel stop end which could not be removed at this location and the failure of the jet grouting repair method. This leak of water and soil resulted in substantial settlement – up to 140mm – in the adjacent buildings. The settlement was mainly the result of ground loss into the excavation; consolidation effects due to pore pressure reduction were minor. It was possible to stop the inflow only after substantial backfill and polyurethane injection.

On the 17th of June, two days before the described event, a large bentonite inclusion (measuring approximately 0.4x1.0m) was discovered during excavation just next to a panel joint in the east wall. Immediately after the discovery, water with soil or bentonite started to flow in. Fortunately, the contractor was able to stop the intrusion of water by immediate backfilling in front of the joint.

After these events geophysical leakage detection was carried out by the multi-sensor survey system (ECR®) with application of spatially targeted electrical impulses (EFT®). According to these measurements there were only many small leakages.

On the 10th of September 2008, there was another severe leak of soil and water, resulting in settlement in some adjacent buildings of up to 250mm. This leak was caused by a large bentonite inclusion next to panel joint 69/70 in the west wall during a trial excavation from NAP -13 to -17m. The maximum width of the inclusion was approximately 0.2m and the height was at least 2m. When the contractor noticed the inclusion, it was still dry. In the next 4 hours the contractor made preparations for covering up the bentonite inclusion with steel plates. After holes had been drilled to anchor the third plate, water suddenly started to flow. Within half an hour the flow of water and soil was so large that it took hours to stop it. After 12 hours the contractor, municipal officials and back office concluded that the situation was stable. During those 12 hours, almost 700 litres of polyurethane had been injected and approximately 450m3 of earth had been backfilled.

Unfortunately, the precise cause of the bentonite inclusions is not known. A visual inspection (of the excavated area) indicates that the quality of the D-walls at Vijzelgracht Station is significantly worse than at Rokin and Ceintuurbaan, even though they were installed by the same contractor. The overall quality of the walls at Vijzelgracht is worse than might reasonably have been expected. The presence of so many large bentonite inclusions has led to doubts about the workmanship and quality control.

The bentonite inclusions are most likely caused by a combination of suboptimal circumstances during the installation of the walls, such as delays after removing the steel stop ends, after cleaning the bentonite slurry, the inclination of the stop ends and very thick reinforcement bars relative to the aggregate seize of the concrete. Moreover the bentonite slurry had to be replaced entirely in several panels probably because of an unfavourable interaction with the soft mix, In addition, cleaning the trench with the 5.2m-wide panels from just one pump position and concreting with only one tremie pipe could have contributed to the development of a large bentonite inclusion. All the causes listed here mean that the trench was not fully cleaned with fresh bentonite just before concreting (Figure 1a). During concreting, the thick bentonite was therefore not removed by the concrete, as seen in Figure 1b. The result is a bentonite inclusion in the concrete, as shown in Figure 1c.

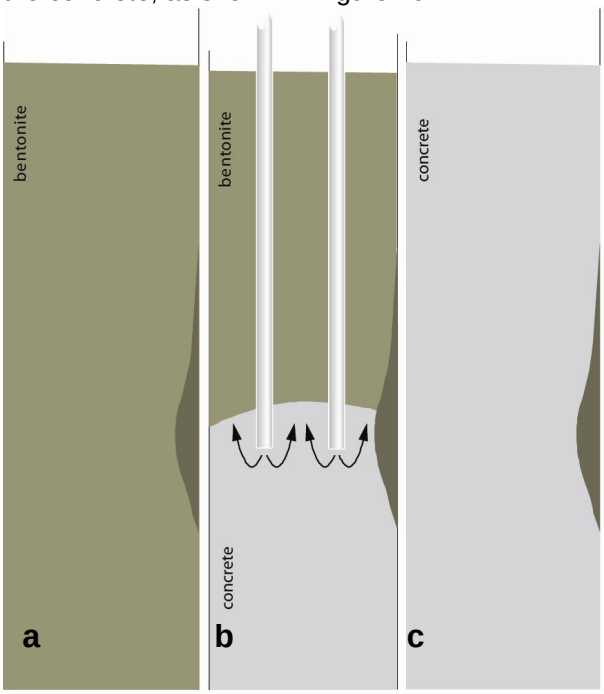

Figure 1: Creation of a bentonite inclusion during the concreting of a panel in three subsequent stages.

Central Railway Station Rotterdam

For the reconstruction of the metro station in front of the central railway station, excavation is required down to 14m below the surface. The retaining structure is composed of D-walls installed to NAP –36m through soft Holocene clay and peat layers and the first medium to coarse sand layer (from NAP –17 to –33m) into a stiff clay layer. So, the D-walls function as cut-off walls. When the excavation had almost reached the required depth, water and sand started to flow into the pit through a joint. The flow was estimated to be 100m^3 per hour and it lasted for more than a day. Outside the D-wall, pavement settlement started immediately. Ultimately, a roughly semi-circular settlement trough resulted measuring about 200m^3. The sand that flowed into the excavation was most probably from the top of the first sand layer, some metres below the excavation level. Fortunately, there were no buildings in the immediate vicinity.

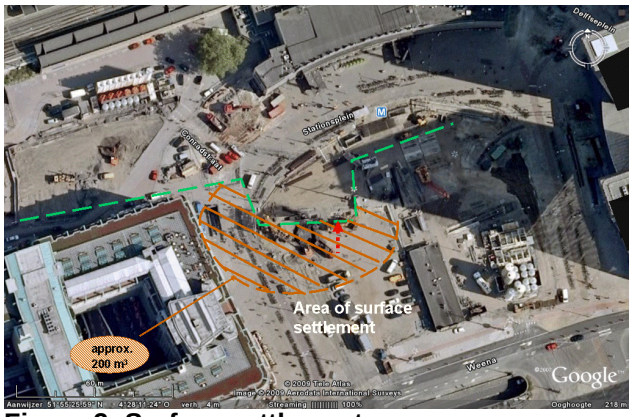

Figure 2: Surface settlement area

Once again, the exact cause of the bentonite inclusions in Rotterdam is not known. There were serious delays during the installation process, both at the panel with the major leak and at panels without any defects. So the cause was probably a combination of different conditions.

Deinze

In Deinze an underground car park of 30m x 37m and a depth of 12m was built for a hospital. The construction site is situated in the alluvial plain of the Scheldt River. The subsoil consists of relatively heterogeneous alluvial soils about 20m thick with some very compressible layers a very thick tertiary clay layer. The groundwater level is between –2 and –3m.

A diaphragm wall 0.80m thick was installed to a depth of 24m in order to create a hydraulic barrier. The groundwater level was lowered to a depth of about –5m in order to limit bending in the diaphragm wall. Horizontal support for the diaphragm wall was provided with a row of ground anchors.

The diaphragm wall was excavated with a classic hydraulic grab and CWS-type joints were installed.

During excavation, water inflow through one joint was observed before the final excavation depth was reached. Excavation was immediately suspended and a polyurethane-type material was injected. Excavation and injection then continued stepwise. Just after the final depth was reached, there was a sudden inflow of water and soil. At least 30m^3 of soil flowed through the diaphragm wall, causing serious settlement behind the diaphragm wall. Some existing minor structures had to be demolished. Steel plates and a sand plug were installed in front of the joint.

To remediate the problem with the joint, 12 jet-grout columns with a diameter of 0.90m were installed behind the joint to a depth of 24m. The large number of jet-grout columns was due to the fact that the main contractor and the insurance companies wanted to do everything possible to limit the occurrence of new problems and new delays.
During the installation of the first jet-grout columns, grout flowed through the sand plug. However, no further problems were encountered.
After the excavation, it was possible to see that, above a certain height, neither side of the water bar was anchored in the concrete of the diaphragm wall. It should be pointed out that, during the execution of the 2 adjacent panels, nothing unusual had been observed.
Since only very general records were kept during the execution of the diaphragm wall, it was not possible to determine the reason for the lack of concrete around the water bar.

Vorst - Brussels

In Vorst Brussels, a stormwater storage basin with a diameter of 30m and a depth of 36m was built within a circular diaphragm wall. The soil was excavated to a depth of 28m.

The subsoil consists of:

- alluvial soft soils of the Zenne River containing some very soft clays and, locally, peat, thickness about 6m, layer L1;
- alluvial sand and gravel with a thickness of about 9m, L2 and L3;
- a tertiary clay layer with a thickness of about 8m, L4
- medium to dense sand with a thickness of 12m, L5
- very dense sands with a thickness of 18m, L6 (see also Figure 3).

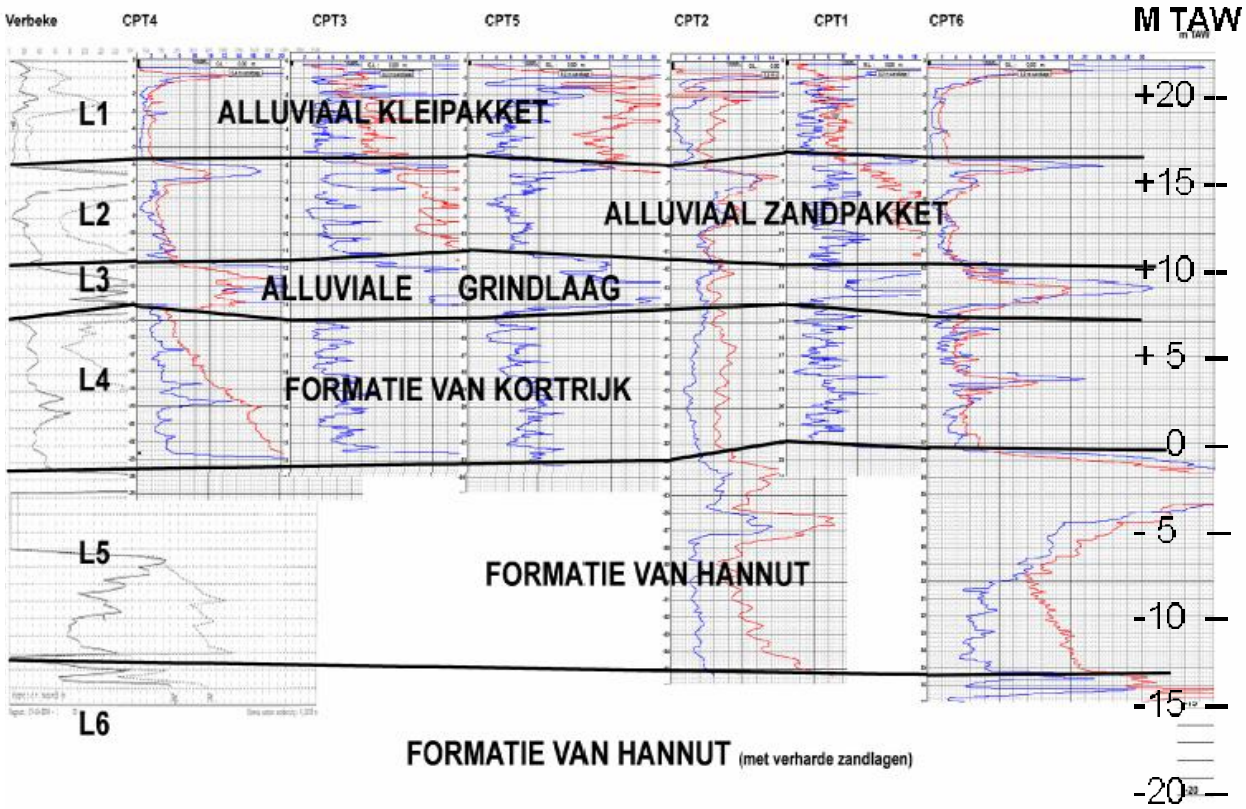

Figure 3: Soil profile at Vorst construction site

A 1m thick diaphragm wall was installed to a depth of 1m in the very dense sand layer and the groundwater level was lowered in the deep sand layers. To prevent settlement of the adjacent building, groundwater recharging was necessary in the alluvial layers.

The diaphragm wall was excavated with a classic hydraulic grab and CWS-type joints were installed. Just before the final excavation level was reached, a sand-bentonite inclusion was observed at the location of one joint with a height of about 2m. As the amount of water flowing through this inclusion was rather small, a sand berm was installed in front of the joint before the site was closed down in the evening. Employees working for the main contractor at night on an adjacent jobsite were asked to check on the sand berm. Shortly after midnight, they observed large amounts of water flowing into the basin through the joint.

As it was impossible to stop the flow of water at that point, the basin was filled with water up to the normal groundwater level and the sinkhole behind the joint was filled with sand. About 800m3 of soil flowed into the basin. Fortunately due to the shape of the 30 m circular coffer dam the overall shaft stability was not an issue. The collapse could also have been worse.

The groundwater level in the lower sand layers had been lowered to the excavation level. It is therefore clear that the large amounts of water flowed through the clay layer, which everybody had thought was a horizontal hydraulic barrier (L4 in Figure 3). At present, it is still unclear how such large amounts of water could flow through the clay layer.
Once again, only very general records were kept during the execution of the diaphragm wall, and so it is not possible to determine the reason for the sand bentonite inclusion in the joint.

DESIGN AND EXECUTION CONSIDERATION

A general and essential requirement when making D-walls is that the bentonite slurry must be completely replaced by concrete. Severe problems can result if any bentonite is left behind. Both the design and the execution of the wall must focus on compliance with this requirement. The following areas must be taken into consideration:

- Guiding walls
- Accurate excavation
- Shape, dimensions and stiffness of stop ends
- Cleaning and replacing the bentonite suspension
- Reinforcement and concreting

Many of these factors are listed in EN1538. However, in addition to EN1538, the authors will now provide some guidelines that may improve the D-wall installation process because we see

the requirements in EN1538 as a minimum; they do not ensure the satisfactory quality of the D-walls.

Guiding walls

Guiding walls are used for the initial guiding of the grab and to support the soil close to the surface. They are needed to support the stop end and the reinforcement cages. In order to prevent settlement of the guiding walls, cast-in-place guiding walls on a stabilised sand foundation should be used unless the surface layers consist of competent material.

The guide walls should be long enough to create a large reservoir of bentonite slurry in order to control the slurry level during the excavation and casting process. Guiding walls can be used to set the level of the slurry above the surface and therefore ensure trench stability during excavation. According to EN1538 the level of the supporting fluid must always remain at least 1m above the highest piezometric head. However, with a variable slurry level during excavation (when the digging bucket is extracted) and concreting (when the volumetric weight of the slurry is relative low) the nominal slurry level should be 2m above the groundwater table. Where the groundwater level is relatively high, it is better to raise the top of the guiding walls than to lower the groundwater table.
The latter option involves lowering the head across the full height of the panel, and this is not always easy to achieve. This means that deep wells are the only satisfactory solution.

Excavation

During the excavation of a panel, the bentonite slurry plays an important role. The slurry has to form a filter cake at the wall of the trench, which is essential to the stability. The slurry must prevent sand sedimentation as much as possible, but during concreting, it must be easily replaced by the concrete. Despite the importance of the bentonite, the grounds for choosing a specific type of bentonite are not very clear. There are many different types of bentonite but their different properties are not always clear, and this goes also for the contractor who has to make the choice. The chosen bentonite must be tested on undesired interaction effects with the soil to be excavated.

EN1538 does not state any requirements about how long a trench may be left open. However, in the authors' opinion, this is a very important consideration. Both in Amsterdam and Rotterdam, severe delays have at least contributed to the presence of bentonite inclusions. During the total D-wall installation process, stagnation must be avoided as much as possible. The whole process of excavation, cleaning, desanding, placing the reinforcement and concreting must be uninterrupted and as short as possible. Stagnant bentonite slurry forms a thick bentonite cake that sticks to the soil and as observed at the joint between panels 69/70 at Vijzelgracht, also to the concrete surface of the adjacent already casted panel. This cake can hardly be removed during desanding before concreting. This may explain the bentonite inclusions that have been observed.

The verticality of the excavation is determined to a large extent by the experience and skill of the crane driver. Some tools are also available to improve verticality. Accurate excavation is important to achieve a good connection between the stop end and the soil behind / next to it and prevent enclosed concrete.

EN1538 requires the verticality of the panels (including their ends) to be within 1% in both the transverse and longitudinal directions. However, in many project specifications, the maximum tolerance is 0.5%. Although this is an improvement on EN1538, there is still a risk when the grab is not guided by the stop end.

The type of grab is important. In deep excavations, hydraulic grabs with a hydraulic correction system are better because it is possible to compensate for the deviation by turning the grab. To check verticality, most grabs these days are equipped with inclinometers so that drivers can check verticality during excavation. Excavation should be as vertical as possible and a conservative approach to correction is required since corrections can further the development of enclosed concrete as well as excess concrete consumption. If a driver waits too long before correcting an inclination during the excavation, a kind of excavated wedge will be created, allowing concrete to run around the stop end, as in figure 4.

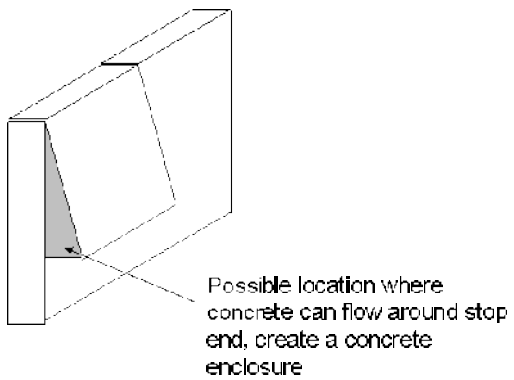

Figure 4: Effect of an inclined excavation corrected during excavation

The position of the inclinometer should also be measured and linked to a fixed point, preferably on the guiding wall. To analyse the quality of a diaphragm wall, accurate measuring of verticality in all directions is important. Analysis of the installation data of the D-walls of the North-South Line to determine possible openings between the panels suggested that two adjacent panels overlapped because the position of the grab had not been properly recorded. This data was therefore useless, which is particularly unfortunate since it would have been useful for identifying the risk of leaks.

Under normal conditions, a single-phase excavation or a three-phase excavation of a panel is required to achieve symmetrical loading of the grab during excavation. With a two-phase excavation, accuracy is only possible if the grab is fully guided by the stop end.

Stop ends

The function of stop ends is to establish a good connection between two panels. First of all, using a stop end ensures that the joint of the first panel will be straight after concreting. Secondly, the stop end should prevent enclosed concrete (concrete behind the stop end that hampers the excavation of the next panel). Stop ends should therefore be positioned carefully in a position close to the soil behind them. In this way and with a accurate excavation, the concrete cannot flow behind a stop end and create “enclosed concrete”. Enclosed concrete causes many delays and other problems with guiding the grab and/or during the removal of the stop end. Moreover, some stop ends like the CWS type can guide the grab during the excavation of the connecting panel.

There are many types of stop ends used by contractors. The most common are the open-ended pipes and the CWS-type joint. The second is preferable because it allows for the installation of a water bar in the wall joints to provide waterproofing. The water bar is partly fixed in the CWS stop end and the other end is cast in the concrete, as shown in Figure 5. Moreover, this profile can guide the grab.

It is very important for the stop end never to be removed by pulling it in a vertical direction if a water bar is used, since the bar will then no longer be watertight. Removing the stop end horizontally (like a zip) will leave the water bar in the hardened concrete.

It is now possible to install up to 3 water bars with one stop end but multiple water bars may impede concrete flow and encourage mud entrapment. Although there is no scientific evidence, it seems that one bar with a width of 150mm is the most effective option for waterproofing. EN 1538 does not contain clear information on the width of the bar.

Figure 5: CWS type stop end with the groove for two water bars.

Cleaning and replacement of the bentonite suspension

Easy replacement of the bentonite slurry with the concrete depends on maximising the difference in density. This means that the sand content must be as low as possible. A higher sand content also promote thicker cake development by making the cake much more permeable.

Contrary to the provisions of NEN-EN 1538, a sand content of 2% or even 1% just before concreting is preferable.

The trench can be cleaned up with a suction pump, a pump at the bottom of the trench or by airlift. It is also essential to run the grab in the excavation immediately before cleaning to remove as much as cake as possible from the walls if the trench has stood for any time with dirty mud in it. Regardless of which method is used, the pump or pipe must be placed at different positions across the full length of a trench and at the bottom to ensure that the trench is cleaned to the deepest level. In the authors' experience, the pump is sometimes placed in a single, eccentric position in the trench, with the possibility that bentonite slurry with a higher density is left in place in some places. During concreting this bentonite can form inclusions.

Paul et al. (1992) states that standard practice should include sweeping the joint to remove any possible accumulated slime/impurities before concreting the secondary panels. This can be

done using scrapers, brush wheels or water jets. Scrapers should not be used with rubber strips because of the risk of damage to the strip. This issue is not covered by the European standards such as DIN4126, DIN4127 or NEN-EN1538.

Reinforcement and concreting

Successful replacement of the entire bentonite slurry with concrete depends on the properties of the concrete and the skill of the workers.
NEN-EN1538 states requirements for the properties of fresh concrete. Slump value and workability and the concrete mix itself are particularly important. In general, there is no discussion of the values stated, but it is very important for every single truck mixer to be checked before unloading. In the case of large panels with more than one reinforcement cage, the concrete level in the trench must be as horizontal as possible. This means that all the tremmies should be used together, not one at a time. An inclined concrete surface can include bentonite. In this respect, EN1538 states a maximum distance of 2.5m for the concrete flow.

The maximum size of the aggregates depends on the clear space between reinforcement bars. EN1538 states a minimum horizontal distance of 100mm (or 80mm when the maximum size of the aggregate is 20mm). The minimum vertical clear space is 200mm, or 150mm when the maximum size of the aggregate does not exceed 20mm. The maximum size of the aggregate in Amsterdam was 32mm. In particular, the required clear space (the minimal cover was 75mm) to the front and back of a cage was critical (in relation to the aggregate size). It resulted in a very sandy concrete crust with almost no larger aggregate. In the direction of the joint, the clear space fulfilled the EN requirement. Weele (1996) states that a factor of 7 between the maximum aggregate size and the clear space is optimal for a good concrete flow. A factor of 4 or lower is problematic and should be avoided.

SUPERVISION OF EXECUTION AND MONITORING

Many of the factors described in the previous chapter can be monitored during the execution process. EN1538 provides a list of the parameters which should be monitored as a minimum. the next section looks at some additional factors.

Verticality of the excavation

Project specifications often require a maximum deviation in both transverse and longitudinal directions. The normal procedure during the excavation is to measure verticality when excavation has finished. However, this makes it impossible to predict the risk of enclosed concrete due to inaccurate digging.

Drivers correct vertical inclination on the basis of real-time parameters. So it is advisable to present not only final verticality (in two directions) but also the range of the maximum deviations during the excavation process. There can be a considerable difference between maximum and final verticality, as shown in Figure 6.

Slump value of fresh concrete

A range of circumstances can result in truck mixer delays and so the slump value of every single truck mixer should be determined immediately before concreting. If the value is too low, the truck mixer should not be accepted. In addition, the loading and unloading times should also be noted.

Sand content of the bentonite slurry

In spite of the fact that the sand content in the bentonite slurry is always measured (maximum percentage often prescribed in the project specifications and otherwise in EN1538) during cleaning of the trench, it should be measured at every pump position. In case of multi bite panels there are at least three positions for the pump (left, middle and right side of the trench) this measurement must also be conducted at least three times.

Execution time

In order to check the continuity of the execution process, a timetable should be set up with all the working times, starting and finishing times, and delays to the entire process.

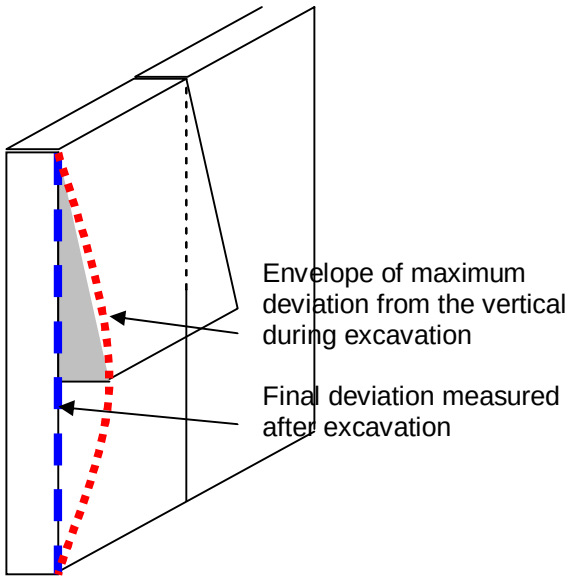

Figure 6: Final verticality and maximum deviation from verticality during excavation

RESTORING CONFIDENCE IN DIAPHRAGM WALLS

Lessons learnt

The accidents in Belgium and the Netherlands have taught us that:

- The observed leakages occur at or directly adjacent to the panel joints in the zones outside the reinforcement cages.
- The process of excavation, cleaning the bentonite, placing the reinforcement and concreting should be uninterrupted.
- A one-phase or three-phase excavation will lead to higher excavation accuracy. Two-phase excavation should be avoided except where the grab is fully guided on the stop end system.
- All the joints must be jetted or swept to remove the accumulated impurities as much as possible before concreting without demolishing the water bar.
- The openings in the reinforcement cages should be at least 7 times the maximum aggregate size.
- The trench must be cleaned up by moving the pump over the full length and to the bottom.
- During concreting, the slump value of every mixer load has to be determined immediately before concreting. The horizontal inclination of the concrete level must also be monitored.
- During excavation, steps must be taken immediately when water flows through the joints, even when small amounts of water are involved. The measures required should be included in the specifications.
- The present leakage detection systems like ECR® and EFT® are not yet able to detect bentonite inclusions. The feasibility of sonic logging and geophysics logging to detect weak spots in the D-walls, before starting the dig must be researched

Recommendations

The authors believe that diaphragm walls are still the best solution as retaining walls in deep excavations in urban areas. A more detailed monitoring of the execution process in accordance with EN1538, supplemented by the lessons learnt and described in this paper, is necessary. When only limited general information is available, good quality control is impossible, either by the contractor self or by the supervisors. If the execution process is monitored in detail, the quality of the product can be guaranteed and the risks of severe leaks will be minimised.

When a detailed monitoring is performed, it will at least be possible to learn a lot when new accidents occur. The case histories described in this paper clearly illustrate that due to a lack of information it was not possible to determine the reason of the difficulties.

The feasibility of sonic logging and geophysical logging of panels and across joints to check the wall for anomalies before the start of the excavation should investigated as the possibility to detect bentonite inclusions with geophysical leakage detection methods

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