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Deep excavations for Amsterdam Metro North-South line: An update and lessons learned

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ABSTRACT: The North-South line in Amsterdam is being built underneath the historic centre of the city. Three deep stations are constructed in deep excavations supported by diaphragm walls. During the excavation for Vijzelgracht Station, leakage through the wall resulted in large settlements of and damage to monumental buildings, which threatened the support of the authorities for the project. With the application of robust preventive measures at two of the deep stations, it was possible to continue the project. This paper reports on the events, the counter measures, the monitoring, the repair of the buildings and the lessons learned for the project and for the foundation sector to avoid future failures in D-walls. Moreover, effort is made to improve quality assessment and measuring techniques to detect potential leakages in advance.

1 INTRODUCTION

1.1 *North South line Amsterdam*

The North-South Line in Amsterdam is 9.5 kilometres long. This metro line starts at street level in the North of Amsterdam and passes under the historical centre of the city in a twin shield tunnel. South of the historic centre, the line re-emerges at street level between the RAI conference centre and the existing railway station South/WorldTradeCentre. Five underground stations are under construction. Diaphragm walls support the excavations of the three deep inner city stations: Rokin (RKN), Vijzelgracht (VZG) and Ceintuurbaan (CTB). This paper deals with the leakages that occurred at Vijzelgracht resulting in severe settlement of monumental buildings, exceeding project budgets and time schedule. After a short description of the events the paper will focus on the lessons learned.

1.2 *Ground conditions*

At Vijzelgracht Station, there are fill deposits and soft Holocene clay deposits to a level of about (Dutch reference level) NAP –12.5 m (ground level is around NAP +1.5 m). These are underlain by the 1st sand layer, from NAP –12.5 m to NAP –14/–15 m, which is on top of a 2.5 m thick sandy silt stratum (the Allerød). The 2nd sand layer is found at about NAP –17/–18 m, extending to NAP –26 m. Below the 2nd sand layer, there is a stiff clay layer that is approximately 15 m 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 Salet et al. (2006).

1.3 *Construction method*

The stations are being built top-down to a depth of about NAP –31 m at Vijzelgracht, with 1.2 m thick diaphragm walls extending to a depth of approximately NAP –45 m. See Figure 1. The diaphragm

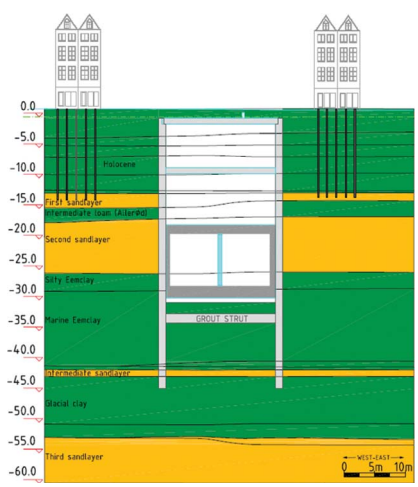

Figure 1. Cross-section of Vijzelgracht Station.

walls consist of panels with lengths of approximately 2.8 m and 5.2 m. Traditional grabs and steel stop ends with water bars (PVC strips) are used to a depth of NAP –36 m to provide waterproofing. The Eem clay layer below NAP –26 m provides a seal off for the bottom of the excavation. Ground improvement works with lean concrete columns took place before installing the walls to remove obstacles in the Holocene deposits.

2 EVENTS

2.1 *Leakages of D-walls*

At Vijzelgracht Station in particular, and to some extent at Rokin Station, numerous joints in the D-wall panels leaked during the 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. A standard procedure of drilling in the wall and polyurethane injections stopped these leakages.

2.2 *First event*

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 –12 m 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 140 mm—and damage in the adjacent buildings. To stabilise the historic buildings a timber framework was installed, see Figure 2. More details of the damage to the adjacent buildings are described in Korff et al. (2009). The settlement was mainly the result of ground loss into the excavation causing a strong reduction of the cone-resistance of the first sand layer, the bearing strata of the wooden pile foundations; consolidation effects due to pore pressure reduction were minor. It was possible to stop the inflow only after substantial backfill and polyurethane injection.

Figure 2. A timber framework was installed to stabilise the historical buildings after severe settlements.

On the 17th of June, two days before the described event, a large bentonite inclusion (measuring approximately 0.4 × 1.0 m) 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.

2.3 *Geophysical leakage detection*

After these events occurred geophysical leakage detection was carried out by the multi-sensor survey system (ECR®) with application of spatially targeted electrical impulses (EFT®). Analysis of these measurements showed that there were many small leakages to be expected along the wall, but none of the joints showed major leakage. The work resumed with trial excavations.

2.4 *Second event*

On the 10th of September 2008, another severe leakage of soil and water occurred, resulting in settlement in adjacent buildings of up to 250 mm. 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 –17 m. The maximum width of the inclusion was approximately 0.2 m and the height was at least 2 m. When the contractor noticed the inclusion, it was dry (no leakage). In the next 4 hours the contractor made preparations for retaining the bentonite inclusion with steel plates. After holes had been drilled to anchor the third plate in the wall, water suddenly started to flow. Within half an hour, the flow of water and soil was almost impossible to control and it took hours to stop it. After 12 hours the contractor, municipal officials and back office consultants concluded that the situation was stable. During those 12 hours, almost 700 litres of polyurethane had been injected and approximately 450 m^3 of soil had been backfilled.

3 TECHNICAL ANALYSES

3.1 *Quality of the D-walls*

These serious events originate technically to two main causes, the first is the presence of a large bentonite inclusion in the D-wall and second the fact that the bentonite inclusion was not classified as a possible big risk. As a consequence, the resulting inflow of water and sand could not be stopped

directly or in a restricted time span to prevent serious erosion effects outside the excavation.

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

3.2 *Bentonite inclusions*

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 size of the concrete. Moreover the bentonite slurry had to be replaced entirely in several panels when it became too viscous probably because of an unfavourable interaction with the soft mix, used to stabilise the very soft top-layers and to replace bored obstacles. In addition, cleaning the bentoniet in the trench with the 5.2 m-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 attributed to the trench being not fully cleaned with fresh bentonite just before concreting (Figure 3a). During concreting, the thick bentonite was therefore not removed by the concrete, as seen in Figure 3b. The result is a bentonite inclusion in the concrete, as shown in Figure 3c.

Up to the first event no specific measures were taken to control the situation in case of a serious leak. The normal procedure with PU-injection and placing of steel plates in case of more severe leakage was applied. After the first event and the geophysical leakage detection, which showed many (probably) small leakages it was decided to start the next stage of excavation (from NAP –13 to –17 m) with small inspection pits at the joints between the panels in order to be able to fill back in case of severe inflow of water and to obtain in such a way a validation of the (ECR®)/(EFT®) measurements. The second event occurred when the mentioned inclusion was found in the inspection pit but as no water flow was observed it was not considered as a serious problem and in stead of back filling the contractor started drilling holes in the adjacent concrete to connect steel plates. The inclusion probably liquefied due to the drilling, resulting in the mentioned inflow of water and sand that could not be stopped for many hours.

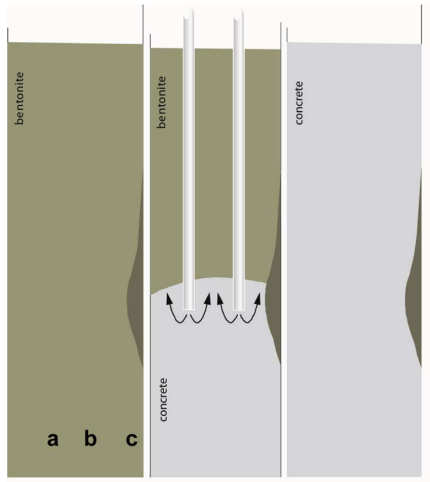

Figure 3. Forming of a bentonite inclusion during the concreting of a panel in three subsequent stages.

4 CONTINUATION OF THE EXCAVATIONS

4.1 *Freezing of all joints*

An attempt has been made—using an analysis of the panel production data, the construction log books, the (ECR®)/(EFT®) measurements and the observations during the performed excavation—to obtain reliable information prior to further excavation about the quality of the wall, and about the severity and the locations of possible bad spots in the diaphragm walls. The analysis focused particularly on determining in advance which suspect locations (i.e. anomalies) are so serious that they will fail immediately upon excavation, resulting in a breakthrough of water and sand, since this will dictate how the work can be resumed safely.

It followed from the analysis that qualitative relationships could be established between the quality of the joints on the one hand, and the (ECR®)/(EFT®) measurements, construction data and logbooks on the other. However, it was not possible to establish any direct, unambiguous relationship between the construction data, the (ECR®)/(EFT®) results and the observed quality of the joints up to the excavation level. In order to ensure an adequate level of certainty during the subsequent excavations, each joint had to be considered a severe potential leak. It was decided that it was necessary to seal all joints before further excavation could start.

Because of the impossibility or unacceptability to carry out works from street level, it was decided to seal the joints from inside the excavation by freezing the joints. Adjacent to every joint two freezing pipes were installed. After reaching a frozen body—consisting of frozen soil, D-wall and

Table 1. Overview of measured width of D-wall joints at three deep excavations.

Joint width [mm]	Ceintuurbaan number	Rokin number	Vijzelgracht number
0–20	126	65	32
20–50		28	69
50–100			2
100–300			5
>300			1

possible inclusions in the wall—that fulfilled the requirements, the staged excavation was continued in layers of about 4 m. During such a stage, the joints were excavated first and covered with steel plates, while the freezing process continued deeper down for the next stage of excavation.

4.2 *Present situation*

This process started in the spring of 2010 and is ongoing at present, May 2011. This preventive sealing covers all sand layers in the height of the excavation. Excavation in the Eem-clay is considered to pose no severe leakage risk.

The same procedure was adopted for Rokin Station as this excavation was at the same level at the time of the events and the risk analysis was similar to the one at Vijzelgracht. At Rokin Station, the excavations are finished and the concrete floor slab has been installed. At Ceintuurbaan Station, the excavation already reached the Eem-clay layer before the Vijzelgracht incidents and therefore no additional measures were taken and no severe leakages occurred.

During the excavations of the frozen joints the width of the joint (=thickness of bentonite cake or inclusion) on the inside face of the wall were measured after freezing the cover of the D-walls. Table 1 presents the results of the three excavations. The results from Rokin Station and Ceintuurbaan Station are based on the full excavation, while Vijzelgracht is at 75% of the final excavation depth. It appears that at Ceintuurbaan all joints have a width of 20 mm maximum and at Rokin of 50 mm. At Vijzelgracht 8 joints exceed the width of 50 mm. The maximum width exceeds 250 mm.

5 BUILDINGS

5.1 *Effect on the soil around the excavation*

Investigations were undertaken, including Cone Penetration Tests (CPTs) to determine the amount of soil disturbance, which reached about 20 m away from the joint where the soil inflow occurred, see Figure 4. Examination of CPTs performed immediately after the first incident (see Figure 5) indicates that there is clear evidence of the 1st sand layer (between NAP −12 and −14) having been disturbed in the region of the panel joint 89/90. The second sand layer also shows disturbance.

The CPT's show cone resistance, q_c, values of the 1st sand layer to have reduced significantly adjacent to houses 4 to 8 Vijzelgracht. The largest reduction to 6–7 MPa was found at the location of the houses 4 and 6 Vijzelgracht. Other CPTs at greater distance from this region show q_c values in excess of 20 MPa.

The reduced q_c values of the 1st sand layer are consistent with considerable loosening caused by a significant volume of sand being washed through the leaking panel joint. During the second incident, some disturbance of the second sand layer was also evident from the CPTs.

The settlement was therefore mainly the result of ground loss into the excavation resulting in loosening and strong reduction of the cone-resistance of the first sand layer, which is the bearing stratum of the wooden pile foundations. consolidation effects due to pore pressure reduction were minor.

5.2 *Damage to the buildings*

The houses influenced by both incidents are historic buildings from around 1670. The buildings all have a semi-basement, a raised ground floor,

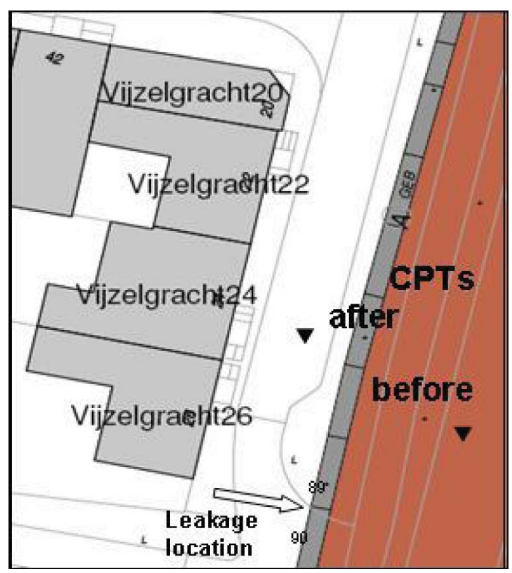

Figure 4. Situation of CPT's at first incident.

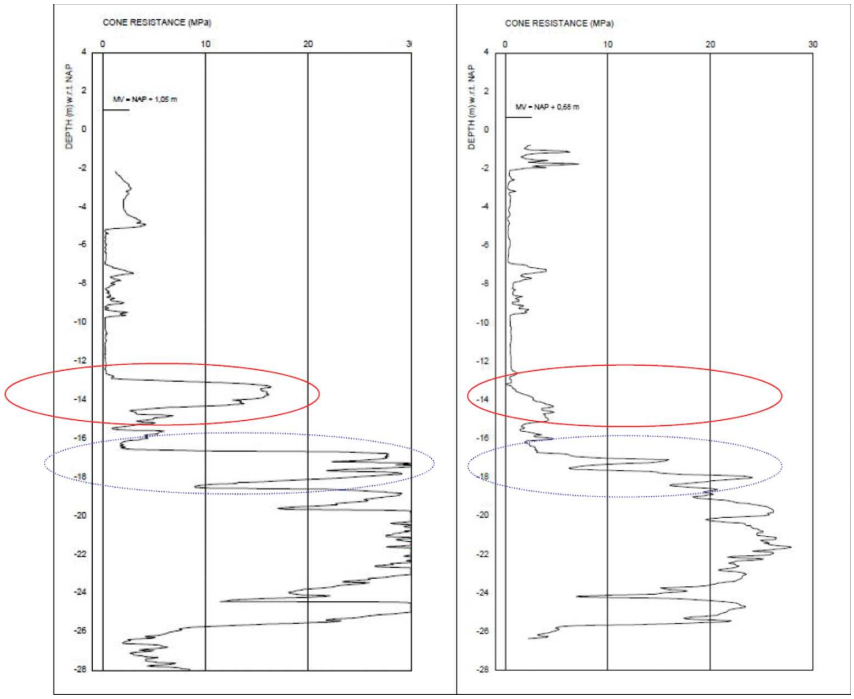

Figure 5. CPT's before (left) and after (right) first incident.

and a first floor with a vaulted roof. The height of the buildings is about 9 m. The original foundation consisted of 52 timber piles per house in rows of 2 wooden piles under the brick walls extends to the first sand layer at a level of around NAP –13 m. Before construction of the North South Line these buildings were equipped with monitoring instruments, comprising optical prisms for total station surveying at two levels and manual levelling bolts at street level.

The block of houses Vijzelgracht 20–26 settled a maximum of 150 mm as a consequence of the first incident in June 2008. The building tilted towards the corner of VZG 26, and towards the excavation and the location of the leakage. A slight sagging was found between VZG 26 and VZG 24 and hogging towards VZG 22.

Block Vijzelgracht 4–10 settled a maximum of 240 mm directly after the second leakage incident. The foundations of Vijzelgracht 4–8 had been renewed before the construction activities started. The new steel piles and the old timber piles are both founded in the first sand layer.

The most important damage indicators are presented in Table 2. According to Korff et al. (2011) the actual damage derived from the observations for both blocks (VZG 20–26 and VZG 4–8) would be category 5, very severe (Burland et al. 1977).

Table 2. Deformations after first and second incident.

Damage indicators	VZG 20–26	VZG 4–8
Max slope	1:111	1:51
Max relative rotation	1:160	1:125
Deflection ration	0.47%	0.40%
Horizontal strain	0.04%	0.12%

5.3 *Repair of the buildings*

5.3.1 *Corrective grouting of the buildings affected by the first incident*

After bracing the buildings, it was decided to apply corrective grouting to increase the bearing capacity of the sand and to lift the buildings. For the final restoration of the buildings a new foundation will be installed. It was implicitly demonstrated that the end bearing capacity of the pile foundations had been restored, when lifting of the buildings proved possible and stability of the building had been assured. Moreover, it showed that it was possible to apply further grouting to compensate for future settlements from ongoing construction of the station.

Although lifting of the buildings was successful, it appeared that for these conditions (loose sand

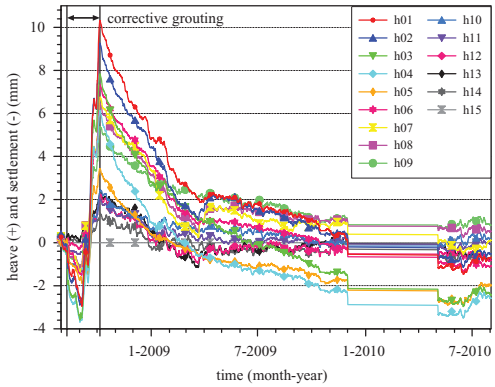

Figure 6. Results of corrective grouting underneath Block VZG 20–26, from Bezuijen et al. (2011).

due to ground loss) the efficiency of the corrective grouting process was very low. Ongoing settlements were found up to at least 5 months after finishing the grouting, see Figure 6. The buildings responded to the grouting in a relatively stiff manner, probably due to the temporary stabilizing timber cross beams, which made analysis of the grouting process difficult. Details of the corrective grouting can be found in Bezuijen et al. (2011).

5.3.2 *Lifting the buildings affected by the second incident*

Since the buildings of the second incident already had foundation renewal prior to the start of construction, it was decided to test the remaining capacity of the piles by pile load tests. Most of the piles had an ultimate capacity above 400 kN, which was the test level. The original pile load is about 250 to 280 kN. For a group of piles close to the leakage the ultimate capacities ranged from 300 to 350 kN. A few piles further away had capacities as low as 125 up to 225 kN, which could either be due to an irregular shape of the leakage effect or due to an originally low capacity. From the load tests it appeared that only a few piles had insufficient bearing capacity. It was decided to lift the whole block by jacking the piles; 5 piles were added in Vijzelgracht 6 to compensate for the loss of bearing capacity of the affected piles. The maximum lift to be obtained was 220 mm at the corner of Vijzelgracht 4. The procedure for lifting the buildings is described in De Nijs & Kaalberg (2010).

6 MONITORING

6.1 *Settlements and pore pressures*

One of the questions after the events occurred was the role of the monitoring system. Public as well as political concern arose as it became clear that the monitoring system did not warn for the severe incident. At the start of the project, during the political debate about impact, costs and risks, the monitoring system was presented as a tool that warned for deviations from the predicted behaviour.

The large settlements during the first event were however not preceded by smaller settlements that could have indicated the coming calamity. First the data from the robotic total stations, monitoring the prism's at the buildings required a processing time of 4 hours. After the first event it was decided to shorten that period to 2 hours.

Another possible early warning could be the readings from the piezometers installed in the different aquifers. These data were recorded every hour. Figure 7 shows the pore pressure readings (left axis) in the first and second sand layer during the first events as well as the total station reading as the manual levelling system (right axis). These measurements can be compared to the data from the diaries of the supervisors during the first incident:

13:10 h	First inflow, at excavation NAP –14 m
14:00 h	Start of grouting in joint, mounting steel plates and backfilling
16:00 h	Steel plates fixed (NAP –12 to –14 m). Inflow decreases; more back filling
18:45 h	Flow stopped almost
21:30 h	Increase of flow
22:00 h	Grouting from outside
01:00 h	Grouting in second sand layer
03:15 h	No more flow visible

At 13.00 the pore pressures first show signs of leakage by a draw down of nearly 1 m in the closest observation wells. Ten minutes later (13.10) the first inflow was recorded. So in this case neither the prism's nor the pore pressure transducers showed any early sign of the coming event.

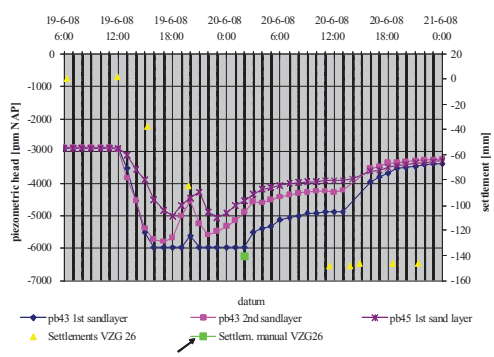

Figure 7. Monitoring data during first event.

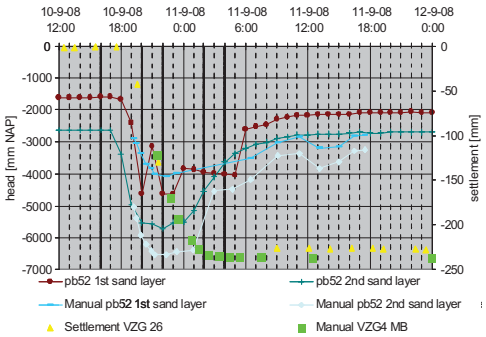

Figure 8. Monitoring data during second event.

Figure 8 shows the monitoring data during the second event.

The observations from supervisors are as follows:

12:00 h	At NAP –17 m big inclusion found. No leakage was observed!
16:30 h	Start mounting steel plates
18:00 h	1st plate was mounted, 2nd at 18.30 h and the 3rd was mounted at 18.45 h
18:45 h	Small leakage from behind plates
18:50 h	Water breakthrough ➔ injecting in D-wall,
7:00 h	After back filling 450 m^3, and massive PU-injection, leakages is stopped

During the second incident, the pore pressure transducers in the second sand layer show a first draw down of nearly 1 m at 18.00 hours, while the first inflow of water was observed at 18.45 h. This could have been a signal for immediate back filling instead of going on with mounting plates.

At one other deep excavation for the North-South Line, at Central Station a leakage in the D-wall was detected prior to the excavations due to careful monitoring of piezometers outside the excavation during test pumping inside the excavation. Due to the somewhat lower permeability of the sand layers at that location and a tight grid of observation wells it was possible to localise a leakage prior to the excavation and to mitigate it.

6.2 *Conclusions regarding the monitoring*

1. Leakage through D-walls develops with ongoing excavation, so the inflow before excavation might be negligible.
2. Accurate analyses of continuous pore pressure registration may reveal a leakage at an early stage. This depends on flow and permeability of aquifers outside the excavation.
3. If such accurate monitoring does not reveal a leakage it has not been proven that there is none. But, if this monitoring shows a reaction outside the excavation, there definitely is a leakage.

7 LESSONS LEARNED

The lessons learned from the described events consider different levels. At the project level the organisational structure has been adapted, with other and new personal and new working methods as well. At the level of the foundation sector, clients, consultants and contractors united in a CUR-committee (CUR is comparable with CIRIA) to improve the D-wall installation and excavation process. At scientific level, Delft University of Technology started a research project into improvement of leakage detecting and quality assessment systems to detect bad spots, in particular in D-walls prior to the excavation.

7.1 *Project level*

The measures taken at organisation level are: (i) the appointment of project leaders for each of the three deep stations, (ii) more specific geotechnical expertise involved in both day-to-day construction control as well as independent reviewers (iii) application of risk management in a structured way in particular for soil related risks, (iv) open communication with the community; house owners, shop owners and other inhabitants.

7.2 *Sector level*

Van Tol et al. (2010) reported their findings related to D-wall problems in the Netherlands and in Belgium over the last years, including the Vijzelgracht events. Subsequently the foundation sector formed a CUR-committee and concluded a number of lessons learned and the following recommendations, which are reported in the recently published CUR-report 231 (CUR, 2010). Handbook Diaphragm walls:

1. Panel installation data should be reported in more detail. This will improve the quality of D-wall panels. After calamities, these data are extremely useful to determine the cause. The time schedule with all relevant start and stop times as part of the panel data: excavating, removing steel stop end, cleaning trench, concreting. Delays, weekend, and so on are indications for higher risk?
2. All the positions of the grab must be measured (and the envelope of all positions must be reported) during excavation instead of only measuring when the trench is at full depth,

in order to control the accuracy of the excavation and to reduce the risk of "enclosed concrete"

3. Bentonite must be tested with the soil from the site (and possible additives from ground treatment)
4. Time between removing steel stop end and concreting should be as short as possible to minimize the bentonite cake to the adjacent panel
5. All the joints must be jetted or swept to remove accumulated impurities, when the trench is open for more than 24 hours (the handbook states 'in case of delays')
6. Adequate risk management should consider the consequences of an inflow and prepare counter measures
7. The present leakage detection systems (EFT®/ECR®) are not yet able to detect bentonite inclusions and do not discriminate between small leakages and large bentonite inclusion
8. Also known facts from common practice, sometimes disregarded, are re-confirmed,:
   - A one-phase or three phase panel is more accurate than a two-phase panel
   - The trench must be cleaned by moving the pump over the full length of the trench
   - The slump value of every truck mixer load has to be determined immediately before concreting
   - Horizontal inclination of concrete level must be monitored
   - Openings in reinforcement cages should be at least seven times the maximum aggregate size.

Van Tol et al. (2010) argue that diaphragm walls are still a thorough solution for retaining walls in deep excavations in urban areas. A more detailed monitoring of the execution process in accordance with EN1538, supplemented by the lessons learned, is necessary. When only limited general information is available, good quality control is impossible, for both the contractor and 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.

7.3 *At scientific level*

Delft University started two research projects: one focuses on improvements of the installation process and the second investigates 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. Sonic logging is a standard procedure to check the consistency of the concrete for one panel or a bored pile. The feasibility of using this technique across the joint is relatively unknown. The first experiments on a test specimen (a piece of D-wall with an intended soft soil inclusion) show promising results.

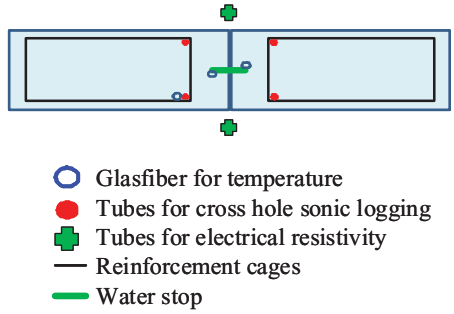

Figure 9. The position of monitoring systems (Spruit, 2011).

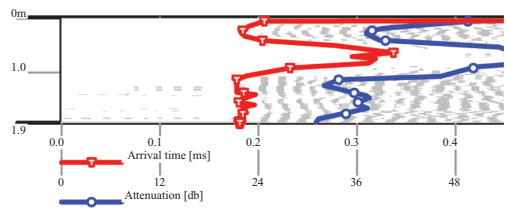

Figure 10. The response of Cross-Hole-Sonic-Logging over a D-wall joint, partly filled with an inclusion (Spruit, 2011).

Figure 9 shows the layout of different monitoring systems at the joint between two D-wall panels as recently applied in a test in a D-wall project in Rotterdam and on a regular base at a D-wall project in Delft, Netherlands. The cross-hole seismic logging method was validated using a D-wall test block with a height of 2 m with a known inclusion. The measurements, see Figure 10, showed a clear increase of arrival time and of attenuation of the sonic waves (Spruit, 2011).

The present leakage detection systems (ECR® and EFT®) proved to be successful in excavations supported with sheet pile walls. In case of D-walls it was not possible to distinguish between very small and insignificant leaks and very serious bentonite inclusions. The research at Delft University aims to improve this technique for D-walls.

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REFERENCES

Burland, J.B., Broms, B.B., de Mello, V.F.B., 1977. Behaviour of Foundations and Structures. *Ninth International Conference on Soil Mechanics and Foundation Engineering*, Tokyo, Japan. 495–546.

Bezuijen, A. Kleinlugtenbelt, R.E, Kaalberg, F.J. and Essler R.D. (2011), Settlement behaviour after compensation and corrective grouting, *Proceedings of TC28, International Symposium, Geotechnical aspects of underground construction in soft ground, Roma, Italy*.

CUR-report 231, 2010, Handbook Diaphragm-walls, design and installation, CUR/COB, Gouda, The Netherlands (in Dutch).

De Nijs, R., Kaalberg, F.J., 2010. Recovery of pile bearing capacity (In Dutch; Herstel paaldraagvermogen). *Cement* 2010/8. 48–53.

Korff, M., Mair, R.J. and Van Tol, A.F., 2009, Building damage examples due to leakage at a deep excavation in Amsterdam. *Int. Conference of Soil Mechanics and Geotechnical Engineering, Alexandria, Egypt*.

Korff, M., Veenbergen, V. & Van Tol, A.F., 2008, Summary Deltares reports for Vijzelgracht Station, Deltares report 435691-0003 (in Dutch).

Korff, M, Mair, R.J., Van Tol, A.F. and Kaalberg, F. 2011, Building damage and repair due to leakage in a deep excavation, *Proceedings of the Institution of Civil Engineers journal Forensic Engineering, accepted for publication*.

Salet, Th.A.M., De Kort, P.J.C.M. & Crawley, J.D., 2006. Amsterdam North South Line, Metro-construction of deep diaphragm walls for three underground stations. *Proc. Tenth International Conference on Piling and Deep Foundation, Linderberg, Bottiau & van Tol (Eds), Amsterdam. DFI, Emap*.

Spruit, R., 2011, Diaphragm walls Kruisplein, Proceedings of TC28, *International Symposium, Geotechnical aspects of underground construction in soft ground, Roma, Italy*.

Van Tol, A.F., Veenbergen, V. & Maertens, J. 2010, Diaphragm Walls, A reliable solution for Deep Excavations in Urban Areas? DFI/EFFC, *Proc. of the 11th International Conference (electronic version), DFI, London*.