Bentonite cavities in diaphragm walls

Case studies, process decomposition, scenario analysis and laboratory experiments

Thesis
THE MORE YOU KNOW,
THE MORE YOU KNOW THAT YOU DON’T KNOW.
Bentonite cavities in diaphragm walls

Case studies, process decomposition, scenario analysis and laboratory experiments

THESIS

TO OBTAIN THE MASTER OF SCIENCE DEGREE

DELTFT UNIVERSITY OF TECHNOLOGY
FACULTY OF CIVIL ENGINEERING & GEOSCIENCES
DEPARTMENT OF GEOTECHNOLOGY
SECTION OF G EO-ENGINEERING
CHAIR OF UNDERGROUND SPACE TECHNOLOGY

BY AUKE LUBACH
Preface

This report is part of my master thesis research for the master ‘Underground Space Technology’ at the faculty of Civil Engineering and Geosciences at Delft University of Technology. Recent calamities e.g. at Vijzelgracht station, part of the North/South line in Amsterdam, triggered the need for research on diaphragm walls and the occurrence of bentonite cavities. By the recommendation of PhD student Ir. J. van Dalen and Prof. Ir. J. Bosch, I started this research in April 2009.

Reader’s guide
The research ‘Bentonite cavities in diaphragm walls’ consists of four parts, an introduction, problem analysis, laboratory experiments and conclusions & recommendations. The introduction comprises an introduction to recent problems which shows the necessity for new research on diaphragm walls and an introduction into diaphragm wall technology in general. The second part is a further analysis with respect to bentonite cavities in diaphragm walls, causal relations with construction techniques, parameters involved and scenario analysis regarding diaphragm wall construction in general as well as some specific cases. The results from the second part urge for additional research i.e. experimental laboratory testing, which is described and analysed in the third part of this report. In the final fourth part some conclusions and recommendations regarding further analysis or study are provided. Key findings of interviews are added as appendix 1 and referred to by round brackets i.e. (de Kort), whereas literature references are indicated with by square brackets i.e. [de Kort, 2006]. Quotes in this report are italic.

Acknowledgements
This research was performed with help of my supervisor, the committee, laboratory employees, diaphragm wall experts and others persons who gave support or advice during the process:

• The committee with Professor Johan Bosch, Professor Frits van Tol, Claire Chassagne, Alex Fraay and in particular Jan van Dalen for his primary guidance and support.
• Claire Chassange for her contribution in understanding electrokinetic phenomena, colloidal suspension behaviour and for providing the opportunity to do some experiments at Deltares.
• Laboratoria employees; Han de Visser, Arno Mulder, Wim Verwaal, Henny van der Meulen, Ruud Hendrikx, Arjan Thijssen and Ellen Meijvogel. Especially Han for his input and expertise in experimental laboratory testing, and Arno for his hard work at Rokin station, Amsterdam.
• Diaphragm wall experts; Professor Jan Martens, Peter de Kort, Bartho Admiraal, Anton Smits, Laurent van Mansfield and Marc van den Eynde
• CEBO Holland; Jaap Groen and Marcel Bijleveld for providing the necessary bentonite and for receiving me for a guided tour in the factory and laboratory facilities.

The last thanks are for a special lady, my girlfriend Filicia Mulder, who gave me support in every aspect.

Delft 16 June 2010,

Auke Lubach
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Part I: Introduction
1 Introduction

1.1 History

The construction of the East Line in Amsterdam in the ‘70’s resulted in negative associations with the construction of a metro system in a build up area by the people living in Amsterdam. In that time the main construction technique (pneumatic caissons) was inextricably bound up with demolishing houses. Massive protests and a thwarted bomb attack were an incentive for the municipality of Amsterdam to abolish the initial plan to construct a metro system which connects all districts of Amsterdam.

figure 1: construction of the East Line

In the late 90’s new developments in underground construction, and especially the new developments in tunnelling in soft soils, the North/South Line could be build without demolishing any houses in the area and with a minimum disturbance for the surroundings using the cut and cover technique for the stations. Especially in the city centre designer’s requirements related to the protection of the historic structures and limitation of disruption resulted in tailor made design solutions.

Construction of the North/South Line started in April 2003. Like other large construction projects time and budget overruns occurred and more people are complaining about the hindrance of the project. On the 19th of June 2008 a calamity occurred at station Vijzelgracht (one of the deep stations), a leakage in the wall of the building pit resulted in the inflow of groundwater and sand into the building pit. As a result the adjacent historical houses settled and were damaged. After extensive research for other possible leakages, the project continued on the 9th of September 2008. One day later the worst nightmare became reality; another incident at the same station. Six adjacent historical houses were damaged and some declared unsafe to live in (figure 3).

figure 2: demolishing houses secured by military police

figure 3: ‘Wevershuisjes’ - damaged historical houses at Vijzelgracht
The leakage in the diaphragm wall was caused by a bentonite cavity. The size of this cavity is rather unusual and there is no univocal explanation for its existence. Experts from Deltares [2008] concluded that the bentonite enclosure is most likely arisen due to a combination of various suboptimal conditions during execution. This accident and the measures to prevent another calamity resulted again in a budget and time overrun. As a result the alderman responsible for the North/South Line resigned and a commission was arranged to investigate how the project should proceed. The paradox is the fact the project was feasible due to the development of new (technological) developments in the tunnel boring techniques, but was almost stopped due to the consequences induced by the tried-and-true technique.

1.2 Aims of this research

Diaphragm walls used as ground- and watertight retaining structures are common practice and due to increasing multiple space use in build up areas this technique is used more intensively nowadays. The construction technique for these walls is rather straight forward and the walls were proven to be reliable only with some minor leakage or sweating which is inherent to the technique. Calamities at a construction site in Rotterdam just before Christmas 2007 and at Vijzelgracht station in June/September 2008 changed the confidence in this technique. Bentonite cavities encountered at these construction sites are unusual in both the size and the quantity of occurrence. The phenomenon of bentonite cavities in diaphragm walls and the possible leakages due to these cavities is a call for new research in order to prevent this in future projects.

The societal relevance of this study is evident; constructing a metro line or any other construction work in a city centre requires robust and reliable techniques, without the risk of damaging adjacent (historical) buildings. These infrastructural projects are stimulating to the economy and enabling more comfort in travelling in urban environments. The cut and cover technique using diaphragm walls should fit this purpose, but unfortunately the technique of diaphragm wall suffers from a bad reputation due to the calamities in Rotterdam and Amsterdam.

Statements regarding the bentonite cavities in diaphragm walls show the need for extensive research in order to determine the important aspects to make the necessary adjustments in the design and execution of this type of retaining structures;

- Bentonite cavities in diaphragm walls are a major risk regarding the water tightness of the retaining structure
- There is no univocal explanation for the existence of bentonite cavities in diaphragm walls
- Mechanisms and processes which could be held responsible for bentonite cavities are not clear
- Attempts to predict the occurrence of bentonite cavities based on analysis of the diaphragm walls’ panel records (in Dutch: geboortekaarten) have not been successful
- Texplor measurement technique cannot predict leakages due to bentonite cavities, since the leakage increases significantly during excavation

Diaphragm walls which will be under construction in the future should be without the risk of bentonite cavities, the possibility of leakages and the negative consequences induced by this phenomenon. The objective of this research is to determine processes, parameters and mechanisms which could be held responsible for the bentonite cavities. Furthermore some laboratory experiments are performed in order to determine adequate measurement techniques and external influences on bentonite suspension. The outcome of this research is a basis for more extensive (PhD) research in order to prevent calamities in the future and improve regulations in order to minimize anomalies.
1.3 Thesis outline

The research consists of four parts, an introduction, problem analysis and laboratory experiments and conclusions & recommendations. The introduction comprises the first two chapters; an introduction to this thesis and diaphragm wall construction technology in general. The chapter on diaphragm wall construction technology is based on literature study and experts’ interviews which is also used for the process decomposition and scenario analysis.

The second part comprises case studies and a further analysis with respect to bentonite cavities in diaphragm walls, causal relations with construction techniques, parameters involved and scenario analysis regarding diaphragm wall construction in general. The calamities in recent cases are in fact an incentive for this research, with Vijzelgracht station in particular, and with the knowledge available from the literature and the experts’ interviews in this case can be evaluated. Information from the literature provides more insight in the construction of diaphragm walls and the critical aspects in the design and execution phase. By interviewing people involved in the execution of diaphragm walls, information is gathered regarding the execution aspects. The experts’ opinions are essential to gain more insight in understanding the origin of leaking joints, bentonite cavities and other problems during construction. The literature study will give an overview of the available knowledge, the phases during the construction of the diaphragm walls and the different processes and techniques which are used in practice. The construction processes will be studied during a process decomposition to be able to study the separate processes more intensively. Mechanisms and parameters which are important to the problem definition will become clear at this stage.

The result of this part is the basis for the third part; laboratory experiments. This part describes the performed experiments, backgrounds, some additional theory and results of the experiments. The experiments focus on both integral improvement of the technique as well as the testing of hypotheses on what has gone wrong on construction sites with calamities and in particular at Vijzelgracht station.

The final part of this research comprises conclusions and recommendations based on first three parts, enabling more research in order to gain more knowledge and prevent further calamities in future projects. The urge for better solutions for underground construction in urban areas such as Spoorzone Delft is indispensable and although the contribution of this research will be limited, others may use some conclusions and recommendations provided for future research.
2 Diaphragm walls

Diaphragm wall technique was developed in the 1960’s mainly in Germany, France and Italy after the use of bentonite suspension in the oil and gas industry to make deeper boreholes. Diaphragm walls are underground structural elements used as retaining structures, foundation systems or as groundwater barrier. Different production processes, terms and definitions are distinguished around the globe. In the Netherlands diaphragm walls generally find their application in urban infrastructure projects requiring strict specifications with respect to lateral soil deformation generated by excavating these deep building pits. Diaphragm walls are frequently used as permanent walls for underground parking garages, start and end shafts for tunnel boring machines and cut-and-cover tunnels. This chapter is an introduction to the diaphragm wall technique, where terms and definitions, construction technique and special solutions are discussed in order to provide a state of the art overview.

2.1 Terms and definitions

For the design and construction of diaphragm walls there are codes available providing a guideline, requirements and obligations in the process of diaphragm wall construction. For Dutch diaphragm wall practise the German codes are important in addition to the Dutch version of the international EN 1538 codes for diaphragm walls. Because the international codes are not much elaborated, German DIN 4126 is often consulted. The codes are constantly under development and therefore draft standards are additional to the current standard. Extensive international literature regarding the construction of diaphragm walls is limited to work of Xanthakos and bundled articles by Paul et al. Dutch literature is limited to single chapters in lecture notes from van Tol and COB ‘Handboek ondergronds bouwen’. At this moment a CUR commission is writing a state of the art report on diaphragm walls in the Netherlands with new obligations and recommendations.

A list of codes used for this research:

- NEN-EN 1538: 2000-01 Standard ‘Execution of special geotechnical work - Diaphragm walls’
- NEN-EN 1538: 2009-02 Draft standard ‘Execution of special geotechnical work - Diaphragm walls’
- DIN 4126: 2004-08 Draft Standard ‘Cast-in-situ concrete diaphragm walls; design and construction’
- DIN 4126: 1986-08 Standard ‘Cast-in-situ concrete diaphragm walls; design and construction’
- DIN 4127: 1986-08 ‘Earthworks and foundation engineering; diaphragm wall clays for supporting liquids; requirements, testing, supply, inspection’

In practise different types of diaphragm walls are distinguished based on the construction method and the type of support fluid. In order to prevent confusion, differences between the types of diaphragm walls and the scope for this research are elaborated. The Dutch code NEN-EN 1538 distinguishes:

- Cast in situ concrete diaphragm walls; wall made of plain concrete or reinforced concrete, which is constructed in a trench excavated in the ground. The excavation is carried out in discrete length to form panels and the concrete is placed through tremie pipes most of the time immersed in a support fluid. In some cases the excavation and the concreting may be carried out in dry conditions.
- Precast concrete diaphragm walls; wall made of precast elements which are lowered into a trench excavated in the ground containing hardening slurry.
- Slurry walls; wall made from hardening slurry. In most cases, the excavation is carried out using hardening slurry as the support fluid. Sealing elements such as membranes or sheet piles may be inserted
• **Reinforced slurry wall:** wall made from hardening slurry reinforced by steel beams, steel mesh or other suitable elements.

• **Plastic concrete walls:** wall made of plastic concrete (a low strength, low Young's modulus concrete capable of sustaining larger strains than normal concrete), which is constructed in a trench in the ground. The excavation is carried out in panels and the concrete is placed through tremie pipes most of the time immersed in a support fluid. In some cases the excavation and the concreting may be carried out in dry conditions.

Dutch engineers refer ‘diaphragm wall’ to ‘cast in situ concrete diaphragm walls’ because this type of wall is common practice and other types are not often used or known by another name. Therefore the scope of this study is bounded by this ‘cast in situ concrete diaphragm walls’ and diaphragm wall should refer to this type of wall from this point on.

Until now diaphragm walls have been constructed to depths of 180m, although the majority of these walls are constructed to a depth of 15 to 30 m. The wall thickness varies from 0.4 to 3.2 m depending on the project requirements. The length of the excavation varies from 2.0 to approximately 10.0 m depending on the stability requirements determined by the vulnerability of and distance to surroundings. In the NEN-EN the geometry of a diaphragm panel is depicted with the terms for different dimensions and levels conform the international codes with an addition of the excavation depth of the building pit (12).

![Figure 4: Geometry of a Panel](image)

**Key:**
1. Wall thickness
2. Horizontal length reinforcement cage
3. Cage with
4. Length of panel
5. Platform level
6. Casting level
7. Guide-wall
8. Cut-off level
9. Vertical length of reinforcement cage
10. Reinforcement cage
11. Depth of diaphragm wall
12. Depth of excavation building pit

### 2.1.1 Special solutions

Special solutions in diaphragm walls can be found in the geometry and arrangement of the panels and in the applications of the diaphragm walls. Diaphragm walls for cut and cover or building pit constructions are usually rectangular shaped, while special solutions are needed in corners or when connections with existing structures are designed. Nowadays diaphragm walls are also constructed in circular building pits for parking garages or LNG tanks.

Construction for a building pit by means of a diaphragm wall requires special attention to the corner solutions and elements with two or more panels with an angle to each other. Different forms of elements (figure 5) are distinguished and require special excavation sequences and solutions for the reinforcement. Furthermore, special attention should be paid for the stability of the excavation and the corners pointing into the building pit because these joints are susceptible for opening and leakage. Sometimes circular diaphragm walls constructed for large
storage tanks or parking facilities, special attention is required to the wall thickness and the positions of panels in relation to each other in order to gain ring forces.

![figure 5: forms of elements [Paul et al., 1992]](image)

Foundation elements by means of the diaphragm wall technique, barettes or barette-piles, could consist of an individual panel or more panels interconnected and constructed at the same time. Barettes gain their bearing capacity by the wall friction of the panel, which could be as high as 20,000kN. The load-settlement behaviour of these elements is known to be quite good but depends on the quality of execution and the properties of the soil. Especially the bottom of trench should be cleaned adequately in order to decrease the load settlement behaviour. The use of diaphragm walls for the cut and cover technique requires a sound connection between the walls and the roof, the floor and sometimes an intermediate level. For the connection of the roof, the diaphragm wall is usually partly demolished to clear the reinforcement and thereby to make a connection with the reinforcement of the roof. Connecting the floor or an intermediate level is achieved by installing prefab reinforcement connections in the reinforcement cage of the diaphragm wall (box-out) in order to make the slab connections. Another special solution is the possibility to install part of a sheet pile in the reinforcement cage of the diaphragm wall to make a connection with a sheet pile wall later on. Sometimes pipes are installed in the reinforcement cage, plastic tubes for grouting purposes or steel pipes for inclinometers to monitor the verticality of the diaphragm wall during excavation or other construction activities. For the station boxes of the North/South Line the steel reinforcement is replaced with glass fibre reinforcement polymers (GFRP), because the tunnel boring machine has to penetrate the diaphragm walls. The damage to the cutter wheel is significantly reduced by replacing the steel for these soft-eyes.

2.2 Construction technique

The construction technique of diaphragm walls is based on individual excavated and concreted panels constructed adjacent to each other in the underground. Discrete panels are constructed by excavating a trench, using a support fluid (usually a colloidal bentonite suspension) with a higher level and density compared to the groundwater in order to stabilise the excavated trench. When the designed depth of the excavation is achieved, the support fluid is replaced by fresh or regenerated support fluid before reinforcement is placed and the trench is concreted from the bottom up displacing the support fluid which is pumped out at level. A schematisation of the construction phases is depicted in figure 6.
bar. The telescopic Kelly-bar is a guide bar which can transfer torque/steering and crow force on the grab to a considerable depth. These advantages are of minor importance for deep diaphragm walls because the steering capabilities diminish with depth and downward pressure can also be generated using a heavy grab. A hydrofraise is a frame with two drums equipped with cutters which rotate in reverse to break up the soil. Excavated soil is suspended in the support fluid and directly above the rotating drums pumped to the surface. The verticality of the excavated panel should be better compared to the cable suspended grabs; 1:200 for the hydrofraise opposed to 1:100 or 1:120 for grabs [Salet et al., 2006], for the Dutch situation. The discontinuity of the excavation process with a grab is considered to be a disadvantage of the technique since the hydrofraise is able to excavate and transport the soil at the same time. The continuously excavating and transporting process results in a higher quality of the diaphragm wall since there are no turbulent currents in the bentonite suspension induced by the lifting and lowering of the grab. These turbulent currents can erode the soil, especially in case of non cohesive soils. Despite the theoretical advantages of the hydrofraise the technique has not been applied in the Netherlands until now.

2.2.2 Joints

Joints in between the panels require special attention because joints are weak in transferring shear stresses while they are conductive to moisture transfer. The surface of concreted panels is in contact with the support fluid for quite some time, until the support fluid is displaced by fresh concrete. Support fluid cannot be replaced entirely and therefore the panels are regarded as individual constructive objects, there is no monolith construction and the joints are susceptible for leakages and deformations. In order to decrease the susceptibility to leakages and increase shear transfer in the joints, joint formers/stop ends are used. Joint formers should provide some basic requirements:

- The joint should be ground- and watertight
- The joint should transfer shear between discrete panels
- Physically the stop end should not disturb the previously concreted panel and on the other hand accommodate the excavation of adjacent panel
- While the joint is constructed leakage of fresh concrete should be kept as small as possible
- Stop ends should withstand high pressures from the concrete without deforming
• Little or no collection of bentonite (or other materials) on the concreted surface and the stop end should facilitate cleaning and removal of residual material from hardened concrete surfaces
• The joint should be constructed with simple methods and simple equipment

There are many lateral joint methods and types of joint formers, according to Sanno [1992] with or without overlapping of rebar in horizontal direction. The Dutch practice is restricted to mainly two types of joint formers: permanent and temporary joint formers and the temporary stop ends two types are distinguished; stop end pipes and stop end sheets. The permanent joints are highly exceptional and are consisting of steel or concrete beams, which become part of the diaphragm wall. Stop end pipes have to be removed vertically directly after or even during concreting, while the stop end sheets are normally laterally removed after excavation of the adjacent panel. In case of the hydrofraise no stop ends are used because an overlap between the panels is removed by the fraise, when the hydrofraise is cutting on both sides into concrete of a primary panel.

The variance in stop end sheets is mainly due to improvements in the waterproofing properties by an increased travelling path of groundwater. For stop end pipes this path is longer than for stop end sheets. To improve waterproofing, stop end sheets are equipped with one or more water stops, rubber strips to be installed in the diaphragm wall, or with injection tubes in order to be able to inject the joints after the new panel has been concreted (figure 8).

In the Netherlands stop end sheets are nowadays common practice especially the CWS (Coffrage avec Water Stop) type formally patented by Bachy or similar types. These are usually with one two rubber water stops. Round stop end pipes are also applied for diaphragm walls with a thickness up to 1,0 meter until a depth of 60m. Besides Bachy types the ‘cascade’ or ‘organ’ type of stop end is sometimes used. This type of stop end has a bad reputation regarding waterproofing capacities due to experiences in Berlin. In front of the historical Amsterdam Central station a new metro station is constructed with this type of stop end [Linder, 2006] for panels with a thickness of 1,2m. Until now the performance of these joint formers in this project is not known, since the building pit is not fully excavated until now.
2.2.3 Panel type

Within diaphragm wall construction different types of panels are distinguished based on the execution sequence. Two systems are distinguished primary (starter) and secondary (closure) panels and a system where in between a starter and closure panel, intermediate panels are constructed. In case of the first system two stop ends are required for the construction of a start panel and no stop ends are required for the construction of a closure panel. The system with intermediate panels (I) in between a starter and a closure panel requires the use of one stop end for an intermediate panel. In figure 10 both systems are depicted for two types of stop ends; primary and secondary panels on the left and starter, intermediate and closure panels on the right.

The sequence of construction of these panels is determined by the surroundings of the project, layout of the diaphragm wall and the amount of space available on the construction site. The sequence in which the panels are constructed are for instance 1-4-2-5-3-for the construction with primary (1,2,3) and secondary (4,5) panels. Using intermediate panels the sequence could be for instance 1-3-5-7-2-4-6, with 1 and 2 as start panels, panel 7 is a closure panel and 3, 4, 5, 6 are intermediate panels. Sometimes panels are constructed one after the other: 1-2-3-4-5-etc, where there is one start panel, a lot of intermediate panels and one closure panel.

2.3 Support fluid

To prevent the collapse of the excavated trench a support fluid is used, usually in the form of a bentonite suspension. Bentonite suspension has a plastering effect on the sides (bentonite cake) of the excavated trench, the bentonite clay particles stick between the soil grains because the water is squeezed out true the pores, due to the relatively higher hydrostatic support fluid pressure compared to the hydrostatic water pressure. The higher hydrostatic pressure of the support fluid against bentonite cake creates a pressure which should be larger than the horizontal earth and water pressure in order to maintain stability.
The main function of the support fluid is supporting the trench during excavation until the trench is filled with concrete. Support of the excavated trench is based on three aspects; a higher density, a higher the level of support fluid compared to groundwater level and the plastering effect. Besides the supporting function of bentonite suspension, it should be able to suspend sand and soil particles during the period between the cleaning of the bentonite and the end of concreting to prevent settlement of this material onto the concrete.

The level of the support fluid should be kept about 2m higher compared to the highest groundwater encountered during the excavation. This increased head in combination with the slightly higher density than water results in the net flow of water from the support slurry to the groundwater plastering the trench surface. This plastering effect of the bentonite forms a ‘cake’ on the surface of the excavated soil. During excavation of the trench the density of bentonite increases due to the sand particles suspended in the slurry which results in an increased thickness of the bentonite cake.

This plastering effect is observed for permeable soils because filtration effect can occur and the protective cake can be formed. With low permeable soils like clay this filtration effect is minimal and no filtration cake will form. In gravels or highly permeable ground the bentonite is not able to form a cake, because the void between the individual grains/particles is significantly larger than the particle size of the bentonite. For this situation the bentonite flows into the soil until the friction between the bentonite and the individual grains is equal to the pressure difference between the groundwater and the bentonite pressure. Adding wood-, mica chips or crushed stone particles should increase the plastering effect in this type of soils and thereby reduces the amount of suspension which will be lost in the underground. The cake or cohesive soil transfers the pressure created by the support fluid on the soil skeleton, which provides the stabilising effect. The stability of the trench can be increased by increasing the volumetric weight of the suspension, increasing the level of the support fluid or lowering the level of ground water.

Stability of the diaphragm wall itself is an important aspect since the wall should be stiff and strong enough to act as retaining structure and the wall should be stable in order to remain functioning. This holds for any retaining structure, but for a diaphragm wall the stability of the excavated trench is special. Over the full height of the diaphragm wall the stability should be checked both for the stability of individual grains and soil particles, micro stability, as for the wedge of soil which could collapse into the trench, macro stability. Individual grains in the wall of the excavated trench are not stable by itself and could also collapse into the trench even with the use of support fluid. The snowball-effect of this local instability could result in a global instability with the collapse of the trench as a result. Macro stability is determined by the safety against the collapse of a wedge of soil into the trench. The weight of the wedge of soil should be kept stable by the horizontal pressure of the support fluid, the wall friction of the wedge and the arching effect of the soil.

![figure 11: micro stability (left) and macro stability (right)](image)
Part II: Problem Analysis
3 Recent problems

In the Netherlands diaphragm wall technology was regarded as a safe and sound technology until calamities occurred in a build up environment. Although this technology is inextricably bound up with minor leakages in the joint of the diaphragm walls and even sweating of concrete in the middle of the panel, these calamities were not accounted for in daily practice and triggered the urge for new research in this technology. Three cases are briefly discussed to provide an overview of calamities and their origin in order to determine important parameters and scenarios regarding bentonite cavities in diaphragm walls; central station Rotterdam, Vijzelgracht metro station Amsterdam and a land tunnel from the Diabolo project in Antwerp, Belgium.

3.1 Rotterdam

The first major incidents regarding leaking diaphragm walls in the Netherlands were in Rotterdam at a building pit near central station and at the building pit where the tunnel boring machine started, Sint Franciscus Driekhoek (SFD). The latter incident involved a large leakage with approximately $1000m^3$ water and soil flowed into this building pit. Although this incident was larger compared to the one at Rotterdam CS, there is not enough information on the background of this incident and therefore not further elaborated. At Rotterdam CS two leakages have occurred, one without consequences, the other larger incident endangered the damage of the monumental Groothandelsgebouw.

3.1.1 Project

In the center of Rotterdam in the period 2005-2010 worked on the extension of the old subway station near Rotterdam CS. The two-track station with a center platform is extended to a three-track station with two platforms, with a connection for the new light rail to The Hague through the Statenwegtunnel. The old and new situations are depicted in figure 12 and figure 13.

The building pit is consisting mainly of a diaphragm walls (indicated with green in figure 14) with a panel depth of -38 m NAP into a clay layer with low permeability in order to have a watertight building pit. Where no diaphragm walls could be installed temporary freezing of the ground was performed in order to create a permanent construction around the tunnel passing underneath Delftse Poort indicated turquoise in figure 14. With the diaphragm walls already installed in 2005-2006 the freezing of the soil started in the spring of 2007 and consequently the building pit could be excavated.
For the construction of the diaphragm walls stop end sheet were used and only corner panels were equipped with a rubber water stop. During the normal production procedure joints were not cleaned prior to concreting. Some panels were standing open for quite some time without additional measures. It is probably the joint of this panel which caused the leakage.

3.1.2 Leakage

When the excavation of the building pit was at maximum excavation dept (-14m NAP) a leakage occurred in the joint of two diaphragm panels (indicated with black arrow). As a result a large amount of water flowed into the building pit at a distance of approximately 20m from Groothandelsgebouw, a large monumental building. The amount of water flowing into the building pit was estimated at approximately 100-125 m³ per hour.

Almost immediately after the discovery of the leak, large sand bags were placed in order to limit the flooding of the building pit. The leakage was also injected with a polyurethane resin in order to stop the leakage.

Due to this leakage large settlements occurred at level in an area of approximately 25x25 m with a settlement of 0.3 m along Groothandelsgebouw. In total 300-500 m³ sand flowed into the building pit due to this leakage.

3.1.3 Measures

Injection of polyurethane resin didn’t stop the leakage because before hardening of this material is was washed away by the large amount of water flowing into the building pit. Steel plates were installed at the joint, reducing the water flow but didn’t stop it. The already installed groundwater wells from the Weena project in combination
with new installed wells were able to lower the hydraulic head from -6m NAP to -9m NAP and therefore significantly reduced the water flow to approximately 50 m$^3$ per hour and more importantly without sand. At this point the polyurethane resin stopped the leakage, while at the outside of the diaphragm wall Jet grouted columns were installed to provide a more robust solution for this situation.

3.1.4 Conclusion

Although a significant amount of sand was flowing into the building pit with major settlements at level, the consequences of this incident were limited. The foundation of Groothandelsgebouw; driven piles with an enlarged pile base just into the first sand layer, is susceptible to settlements in this layer but fortunately was left undamaged.

The cause of this leakage is believed to be a vertical flow of water through the joint of the diaphragm wall, probably caused by excess bentonite material in the joint. The water from the first sand layer at -16m NAP was therefore able to flow through the joint into the building pit at -14m NAP. The importance of a clean joint surface before concreting and could prevent this from happening.

3.2 Vijzelgracht

This paragraph involves an introduction to the North/South Line project in general, the station design and more in particular the problems encountered at Vijzelgracht station. The Amsterdam North/South Line project is the second metro line partly under the historical city centre. With an overall length of 9 km with 6 km underground, it will run from North to South from Buikslootermeeplein underneath the river IJ connecting the Central railway station to its final destination of Zuid/WTC. The line will pass under the river IJ and the Central railway station by means of submerged tunnel elements. From this point two bored tunnels with a diameter of 7 m passing through 3 deep stations; Rokin, Vijzelgracht and Ceintuurbaan to station Europaplein where the tunnel boring machines finish and the line surfaces to its final station Zuid/WTC.

![figure 16: part of cross-section North/South Line with the deep stations](image)

3.2.1 Station design

The design of the deep stations is based on the cut and cover technique with a top down method, which reduces the amount of disturbance at level. During the construction of diaphragm walls as permanent walls for the station boxes, a jet-grouted strut is installed to reduce ground movements during the excavation phase. The roof slab is constructed after the walls and the grout strut are finished in order to reinstate the situation at level. The excavation of the station boxes if performed underneath the roof construction. The depth of the three stations in combination with adjacent houses requires a robust design with respect to the deformations and the vertical equilibrium of the stations. Diaphragm walls with a thickness of 1,2 m and 1,5m up to 45m deep where used in these extreme conditions. For station Ceintuurbaan where the two tunnels are situated above each other the deeper excavation is allowed by means of pressurized air. Rokin station is special because of the parking facilities installed between level and the platforms.
Once a canal Vijzelgracht is nowadays one of the busiest main roads in the centre of Amsterdam. On both sides of the street historical houses are situated including the formally unknown Wevershuisjes. Since the platforms are relatively deep the space between level and the roof of the platform could be used for other purposes. For Vijzelgracht a detention-and-settling tank was designed to serve as overflow area for excess rainwater drainage. Furthermore there a small car and bicycle parking are located in this area. After the underground station has been finished, the situation on level is restored with a three lined promenade. In figure 18 a cross-section of Vijzelgracht station with an indication for the global soil profile is depicted.

3.2.2 Phases

Constructing the diaphragm walls and the roof of Vijzelgracht station was performed in two phases to reduce the amount of hindrance for the traffic. First the East side of the station was constructed (phase 1) with the traffic in both ways on the west side of the Vijzelgracht. Phase 2 was just the other way around, with the difference of the type of contract for the subcontractor involved with the construction of the diaphragm walls. In order to make more progress a Lump sum contract was pushing the subcontractor to increase production in order to make more benefit. Some experts conclude the quality difference between the East and Westside of the station. For the second phase a new contract was made with the sub-contractor in order to speed-up and make higher production (Erik Stoll). Due to this new form of contract information on the execution of the diaphragm walls was not provided.

3.2.3 Subsoil

In this paragraph the de properties and parameters of the subsoil are discussed in order to provide an overview of the different deposits dealt with. The subsurface of Amsterdam is composed of sand, clay and peat deposits, which are deposited in the subsoil during the Pleistocene (2.5 million to 10,000 years ago) and the Holocene (from 10,000 years ago until now) geological periods. During the Pleistocene there was a succession of ice ages (glacial) and between glacial (interglacial). By sea and wind the three san layers are deposited, the sands are separated by clay layers.

In the Holocene was warmer, sea levels rose and shifted the coastline to the east. t Amsterdam successively deposited: a layer of peat (Dutch: Basis Veen), clay layers, sandy clay and clay-bearing sands (or deposits of Calais `wadafzettingen`) and another set of thick peat (Holland Veen, from ca NAP - NAP to 3 m - 4 m). The wadafzettingen have a heterogeneous structure, ranging from fine sand with some clay layers of clay with some sand layers.

The subsurface of Amsterdam is composed of sediments, originated from marine-, glacial-, eolian- and river deposits. Underneath the anthropogenic two main geological important sediments are distinguished, Holocene and Pleistocene deposits. Holocene deposits consist mainly of peat, clays and sand layers, while the Pleistocene deposits consists of marine clays (‘Eem’-clay) and fine grained sands. For the construction of the North/South Line extensive soil investigation has been carried out; in total 125 boreholes, 400 CPT’s sometimes up to 70
meters deep. In appendix 3 a qualitative geological and geotechnical description of the soil layers is given for the typical Amsterdam situation, composed from the Geotechnical Baseline Report [2001]. The very weak Holocene deposits are unable to support buildings or other constructions hence Amsterdam was build on piles founded on the first (Pleistocene) sand layer. Nowadays constructions are still founded in the first or second sand layer, while sometimes the third sand layer (NAP -60 m) is used for the foundation, for example vertical micro tunnelling wall at Central station where every second or third pile is pace into the third sand layer. Special interest should be paid to the layers close to the bentonite cavity induced leakages in order to determine a causal connection between bentonite cavities and soil layers. It’s obvious that water leaking into the building pit are from an aquifer with high permeable layers because in this layer has the least flow resistance. If there is a bentonite cavity nearby permeable layers a leakage will occur, while a bentonite cavity in a less permeable layer won’t cause these problems. The question is if there is a relation between the existence of bentonite cavities and the bentonite cavities.

3.2.4 Obstacle removal and ground improvement

Old quay wall constructions in the trajectory of the diaphragm wall have to removed before construction of the diaphragm walls can take place. Brickwork, piles and wooden constructions were removed by constructing a building pit by making a small sheet piled wall to be able to excavate the brickwork and to remove the wooden piles by pulling them out. Because the Holocene layers are disturbed by the removed piles, a sand backfill of the excavation could result in instability of the diaphragm trench. To remediate these risks additional measures were taken in the form of ground improvement on both sides of the trench of the future diaphragm wall. In the first phase the obstacle removal and the ground improvement was performed by means of small building pits in order to remove obstacles and were backfilled with stabilized sand (a mixture of sand and cement). The ground improvement over the trajectory of the diaphragm wall was performed by a wet “mixed in place” (MIP) method, where the ground is mixed with cement by three rotating augers. These augers are inserted approximately 13 m into the ground where over and area of 1,0 by 0,4 m the soil is mixed with cement creating primary trenches, this procedure is repeated for each primary trench. Between the primary trenched spaced centre-to-centre 1,5 m a secondary trench is constructed in between the primary trenches with an overlap of approximately 0,25 m at both sides. For secondary trenches half of the amount of cement is mixed, while after
the secondary trenches the primary trench is mixed again without inserting cement. Approximately 250 kg cement is used for each cubic meter of ground improved soil [de Kort, 2005]. Due to some problems in the first phase with broken piles a new method is chosen in order to remove obstacles and improve the ground in the trajectory of the diaphragm wall. Obstacles in the trajectory of the diaphragm wall were removed and the soil was stabilised using a bucket-bore (in Dutch: appelboor). A steel casing is inserted into the ground until NAP-13 into the alléröd layer and subsequently excavated. After removing the obstacles and the soil the excavation was backfilled with a softmix (low strength concrete <1 mPa). This process is repeated with a certain pattern (figure 19) covering the trajectory of the diaphragm wall. The alléröd layer should be low permeable in order to have a dry excavation and to be able to insert the softmix in dry conditions. The composition and properties of the soft mix are given in table 1.

<table>
<thead>
<tr>
<th>Material</th>
<th>Amount</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement CEMIII B42,5</td>
<td>25 kg/m³</td>
<td></td>
</tr>
<tr>
<td>Fly ash</td>
<td>250 kg/m³</td>
<td></td>
</tr>
<tr>
<td>Sand 0-0,1mm</td>
<td>525 kg/m³</td>
<td></td>
</tr>
<tr>
<td>Sand 0,1-0,4mm</td>
<td>1160 kg/m³</td>
<td></td>
</tr>
<tr>
<td>Water</td>
<td>220 kg/m³</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>2180 kg/m³</td>
<td></td>
</tr>
<tr>
<td>Water-Cement Ratio</td>
<td>8,80</td>
<td>-</td>
</tr>
<tr>
<td>Water-Binder Ratio</td>
<td>0,80</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 1: Composition and properties softmix

3.2.5 Diaphragm wall

Construction of diaphragm wall at Vijzelgracht station is performed by means of a hydraulic grab equipped with a rotator which enables the precision positioning of the grab with respect to the guide walls. On average one out of five panels was a primary panel, which means that the majority of the panels is constructed and an intermediate panel, which were about 5,1m wide with a wall thickness of 1,2-1,5m to a depth of NAP -45m. Because the grab was 3,3m wide two overlapping excavation trenches were used to excavate a trench of 5,1 m width. The excavation of a panel adjacent to a concreted panel starts with the trench located at the far side of the concreted panel. Bachy type of stop end sheets were used as joint formers equipped with a CWS rubber strip of 150 mm wide. The length of the stop ends was approximately 35m for Vijzelgracht station while for Rokin station 36m stop ends were used. According to some experts these lengths are pushing the technological limit [de Kort, 2006] while Pöllath et al. [2006] constructed diaphragm walls to a depth of 42m with these stop ends. Marc van den Eynde with Soletanche Bachy installed diaphragm walls in Belgium using stop end sheets with a length of 50m without significant problems. Reinforcement cages for the diaphragm walls at station Vijzelgracht are complicated and quite heavy. Special provisions (in order to connect the reinforcement of the roof, intermediate and basement slabs) are installed in the cages. In combination with the deep excavation, thereby the large bending moments, requires dense reinforcement bars in the front and backside of the reinforcement cage. At some locations three layers (for one side of the cage) 50 mm FEB 500 steel was required to cope with the bending moments. On average 280 kg/m² reinforcement used and on certain levels even up to 500 kg/m² (figure 20). The width of reinforcement cages varied from 2,0 to 2,4m and the heaviest cage was 62 tons.
3.2.6 Concrete

From contractual documents the type of concrete was prescribed to be a B35 with a maximum aggregate size of 20 mm. Furthermore the requirement for the concrete under the level of NAP -2m should be of environmental class II while the upper part of the diaphragm wall should be environmental class III. Concreting by means of a tremie makes it impossible to apply different concrete mixtures for different levels, the total concrete volume should be high quality concrete. According to codes the density of the reinforcement in the cages prescribed an aggregate size of 16 mm, with 400 kg cementitious material. [de Kort, 2006]

At Rokin station major problems occurred due to the concrete mixture because the workability of the concrete was too high and as a result excessive flow behind stop ends exacerbated by overbreak in the sand layers. The removal of stop ends became problematic due to the high amounts of concrete behind the stop end sheets. It was decided to reduce the fine aggregates by 50% by using 32mm in order to reduce the workability of the concrete. According to the codes this grain size should not be acceptable.

3.2.7 Bentonite

The bentonite for the construction of diaphragm walls of Vijzelgracht station was processed at the bentonite installation at Singelgracht in between Vijzelgracht and Ceintuurbaan station. The pumping system and the bentonite separation plant were especially designed for this project. Because the pipes were laid primarily out of sight on the canal bed it was very important that the pipes did not become blocked. Pre-treatment on site was necessary to keep the sand content smaller than 5% before pumping to the Singelgracht. For this reason it is not possible to replace the bentonite before concreting only after the bentonite is desanded to <5% at the construction site.

At first a polymer bentonite from Sud-Chemie was used as support fluid but it became clear that the polymers strings were destroyed by the pumps. The polymers provided the sand suspending properties which were thereby significantly reduced. After the construction of some panels the sub-contractor decided to use Trenchcontrol from CEBO.

The cleaning procedure of bentonite suspension prior to concreting was in the first phase performed with the regeneration technique, while in the second phase replacement of bentonite suspension was used in order to save
time. The pipe which was used to pump bentonite suspension to level was placed in the middle of the panel and was not progressively displaced during the cleaning procedure (de Kort).

### 3.2.8 Bentonite cavities

During the excavation of the building pit at Vijzelgracht station three calamities occurred due to leakages in the diaphragm wall. Leakages in the diaphragm wall resulted in the settling of adjacent houses because the water flowing into the building pit eroded the sand from the layer where these old houses were founded. All the historical buildings in Amsterdam are founded by means of wooden piles on the first sand layer approximately 15 meters below ground level. Locations of the calamities are depicted in figure 21 and some data is provided in table 2.

**figure 21: top view Vijzelgracht station location calamities [Google Earth]**

<table>
<thead>
<tr>
<th>#</th>
<th>Date</th>
<th>Address</th>
<th>Panels</th>
<th>Settlement</th>
<th>Depth</th>
<th>Cause</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>17-6-2008</td>
<td>41-43</td>
<td>24/25</td>
<td>10 mm</td>
<td>NAP -13,9 m</td>
<td>Bentonite cavity</td>
</tr>
<tr>
<td>2</td>
<td>19-6-2008</td>
<td>20-26</td>
<td>89/90</td>
<td>140 mm</td>
<td>NAP -16 m</td>
<td>Stop end sheet left behind</td>
</tr>
<tr>
<td>3</td>
<td>10-9-2008</td>
<td>4-10</td>
<td>69/70</td>
<td>230 mm</td>
<td>NAP -13 to -16 m</td>
<td>Bentonite cavity</td>
</tr>
</tbody>
</table>

**table 2: calamities at Vijzelgracht station**

The first calamity at Vijzelgracht station was on June 17\textsuperscript{th} 2008 at west side of the station at the depth of the sand layer as a result of a bentonite cavity. Sand was eroded from the sand layer and as a result of this the houses 41 and 43 settled 10 mm. The second calamity was much worse: large settlements with a maximum of 14 cm for Vijzelgracht 24 and 26 just on the other side of the station two days later on the 19\textsuperscript{th} of June 2008, while the first leakage was not even stopped yet. The location of the leakage was marked as suspicious because the stop end sheet didn’t come out after a week of struggling. In order to prevent leakages several measures were taken but proven to be unsuccessful. The third leakage on September 10\textsuperscript{th} 2008 was the worst, not only because of the shape of the bentonite cavity and the settlements of the houses, but the results of that calamity. The aldermen responsible for the North/South Line resigned and a commission was arranged to investigate how the project should proceed. The bentonite cavity was unusual in size shape and there was no univocal explanation to its existence, see figure 23 and figure 18.
The location of the bentonite cavity was between panel 69 and 70 and was discovered in an inspection pit, a separate excavation inside the building pit in order to be able to inspect the joint and to be able to take measures in case of a joint with insufficient quality. The discontinuity started from a depth of NAP -15 m in the first sand layer to the excavated level of NAP -16,6m into the Alleröd layer. The depth of the bentonite cavity (in horizontal direction) was estimated by inserting an ruler into the cavity which indicated 0,6m, the vertical length of the cavity was about 1,5m to 2m by a maximum width of 0,2m (figure 23).

Both panels are intermediate panels with a working direction from right to left as can be concluded by the shape of the joint it panel 69 was constructed first. The joint of panel 69 looks quite normal: a straight but not perfectly vertical joint was constructed for this panel. Three days after concreting panel 69 the excavation of panel 70 started. The removal of the stop end of panel 70 started on Friday 17th of November 2006 and was problematic; according to the panel records the removal of the stop end lasted for 4 working days. This in combination with the excavation time allowed the bentonite filter cake to increase in thickness for 10 days. The problems have occurred during the construction of the secondary panel number 70, the joint was frayed because the concrete didn’t replace the bentonite (sand mixture).

<table>
<thead>
<tr>
<th>Properties</th>
<th>Panel 24</th>
<th>Panel 25</th>
<th>Panel 69</th>
<th>Panel 70</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness</td>
<td>1,20m</td>
<td>1,20m</td>
<td>1,20m</td>
<td>1,20m</td>
</tr>
<tr>
<td>Width</td>
<td>5,10m</td>
<td>5,10m</td>
<td>5,10m</td>
<td>5,10m</td>
</tr>
<tr>
<td>Length</td>
<td>44,5m</td>
<td>44,5m</td>
<td>44,5m</td>
<td>44,5m</td>
</tr>
<tr>
<td>Length stop end</td>
<td>35,4m</td>
<td>35,4m</td>
<td>35,4m</td>
<td>35,4m</td>
</tr>
</tbody>
</table>
Duration excavation | 3 (work) days | 4 (work) days | 4 (work) days | 8 (work) days
Filter cake formation | 3 days | 4 days | 4 days | 10 days
Start concreting | 23-06-2004 15:00 | 02-04-2004 16:40 | 10-11-2006 16:00 | 22-11-2006 17:30
Duration concreting | 5.5 hours | 3 hours | 4 hours | 4 hours
Speed concreting | 8.1 m/hour | 14.8 m/hour | 11.2 m/hour | 11.2 m/hour
Bentonite density | 1.06 | 1.10 | 1.04 g/ml | 1.04 g/ml
Bentonite viscosity | 35 | 34 | 35 sec | 35 sec
Bentonite pH | 9.5 | 9.4 | 9.1 | 9.1
Bentonite sand content | 1.8 % | 2.0 % | 0.5% | 0.5%

Table 3: Data on the construction of panel 69 and 70 [Panel records]

3.3 Diabolo

The Diabolo project comprises a railway connection between Brussels and Antwerp with a direct link to Brussels National Airport. At the same time the area around the airport is redeveloped in order to decrease congestion and improve public transport. In the figure below a schematisation of the works is given, the railway connection between Brussels and Antwerp in magenta, the railway connection to Brussels National Airport in orange and the new highway connections in green. The complexity of this project is evident since all the roads and highways have to be in use all the time and a lot of crossings have to be realised.

![figure 24: Diabolo project](image)

3.3.1 Geology

Soil conditions in the area mainly consist of some loamy alluvium and underneath the Brussels’ sand is found. This is tertiary, well-compacted sand, containing sandstone boulders and layers of various size and thickness. The largely flat sandstone formations have a small thickness compared to surface and are predominantly horizontally oriented (figure 25). There are also sculpture like sandstone structures found in the subsoil (figure
26). A slightly inclined horizontal stratification can be seen on the excavated side of the diaphragm wall. Brussels’ sand is very fine with limestone containing traces.

Most Dutch soil conditions show the weak layers in the diaphragm wall, the high pressure from the fresh concrete compresses/displaces these weak layers for concrete. This is seen as bulging concrete sticking out on the predominantly vertical diaphragm wall. In this project the sandstone layering is shown at the excavated diaphragm wall as small bulges of concrete sticking out (figure 25). These bulges are formed due to the removal of sandstone blocks which are situated both in and out of the trajectory of the diaphragm wall.

3.3.2 Bentonite cavities

In the partly excavated tunnel some remarkable appearances were observed which are usually not seen after the excavation of the diaphragm wall. Besides the sandstoned layering on the diaphragm wall bentonite cavities were extraordinary both in qualitative as in quantitative perspective. In one panel there has been a delay in concrete supply, which resulted in very large cavities, while there were more cavities with a more ordinary appearance in other panels without concreting problems.

The panel with a delayed concrete deliverance in one of the last charges of concrete shows the consequences of this irregularity which resulted in large bentonite cavities in both joints and minor bentonite cavities in the middle of the panel. Due to the delay in deliverance the already poured concrete progressively hardened and thereby failing to flow upward when new concrete finally arrived at the construction site. This plastic concrete forces the fresh concrete along the tremie pipe over the older concrete and thereby creating large cavities.

During the excavation of the concreted diaphragm wall these consequences became visible, and showed that at some places the entire thickness of the diaphragm wall was without concrete. Exposed reinforcement and cavities in the diaphragm wall where the daylight on the other side which was temporary excavated for repair works (figure 27 and figure 28). The results of the break in the concreting process caused extremely large bentonite cavities, luckily without damaging adjacent buildings and quite easy to repair.
The bentonite cavities are found in various diaphragm panels and can be divided into (combinations of) two appearances: surface cavities and in depth cavities. Bentonite cavities of the first type situated horizontal direction of the panel and are a result of concrete failing to flow through the reinforcement cage. The depth cavities occur in the joints of the panels and are characterized by a high and narrow shape situated at the joint of the panel. The cavity is quite deep into the thickness direction of the panel and sometimes there is no concrete at all in this direction. A combination of a surface and a deep cavity is seen in figure 27 and figure 28.

The outside of the tunnel is usually not excavated, but due to the bad quality both sides of the diaphragm wall have to be repaired and therefore the other side of the diaphragm wall is also excavated. At this side the bentonite cavity is clearly seen and without the additional removal of concrete which is necessary for the repair works. In figure 29 the horizontal oriented bentonite cavity is clearly visible with at the front the original situation (figure 27 is a more detailed picture) and at the back the repair works with the additional removal of concrete. The horizontal cavity is situated almost over the full length of the panel 5-6m long with a varying height and a depth of approximately 10cm.
The other panels with bentonite cavities in this part of the tunnel had a more common type of appearance. The in depth bentonite cavities are always at the joints of an intermediate or closure panel. In general the shape and size of these cavities are comparable; about 80-100cm high and 15-20 cm wide. The depth of the cavities is varying between a quarter to the full thickness of the panel. The cavities shown in figure 30 and figure 31 are examples of the in depth cavities in intermediate or secondary panels.

3.3.3 Water gullies

Another remarkable appearance is the presence of water gullies on the excavated diaphragm walls and baretes which indicates that there is something wrong in the amount of water in the concrete mixture. These water gullies suggest a flow of water during the hydration process of concrete. A low water-cement factor (WCF) could initiate this flow of groundwater to the concrete by the cement which requires a certain amount of water for the
hydration process. Bleeding is another mechanism leading to the water gullies on the concrete surface. Bleeding occurs when the solid materials settle in the fresh concrete mass and excess water migrates to the surface, excessive bleed water is an indication for a (too) high WCF. In figure 32 and figure 33 some of these gullies are shown which were several meters long and 1-2 cm deep.

3.3.4 Bentonite cake

On the interface between the concrete diaphragm wall and the soil naturally present just in front of the panel some interesting discoveries were made. The bentonite directly in contact with the concrete was very much dehydrated. In between bentonite and natural soil, the Brussels’ sands, layers of this sand were present with their grains cemented together resulting in a significantly harder than surrounding soil.

The dehydrated remains of bentonite used during the concreting process contained no sand grains or traces that indicated that this bentonite was used during the excavation sequence. This thin layer of dehydrated bentonite (approximately 1 mm thick) shows a crackle pattern and indicated the water from the bentonite to be disappeared after concreting. Normally the bentonite remains plastic even after the lowering of the groundwater table and excavation of the diaphragm wall. Some laboratory experiments were performed and are elaborated in appendix 13.

figure 32: water gullies

figure 33: detail water gullies

figure 34: crackle pattern
4 Process decomposition

Construction of a diaphragm wall consists of sequential installation of individual diaphragm panels adjacent to each other. Construction of one individual panel comprises different phases and each phase involved different processes and parameters. The phases in the process decomposition of a single panel are divided into excavation, cleaning, installation reinforcement and concreting with the scope of cast in situ diaphragm walls executed by means of grabbing equipment. Each construction phase contains processes which are analysed in order to determine processes in relation to the possible appearance of bentonite cavities. Within the construction of a diaphragm wall using grabbing equipment, there are different execution methods and sequences of processes depending on the type of stop end and the type of panel (starter, intermediate and closure). In general three types of stop ends and thereby three types of execution methods are distinguished, which are described in the relevant construction phase. An overview of the phases for different types of panels and different types of stop ends is depicted in figure 35.

Preliminary works to make the construction of a diaphragm wall possible are not incorporated in the process decomposition. The underground of the project should be without obstacles such as old sewer systems, foundation elements or old quay walls. Since diaphragm walls are commonly used in a build up area, grey or (former) brown field locations, historical investigation is needed for the soil investigation in order to determine and remove obstacles. Sometimes ground improvement is also needed to secure the stability requirements of the excavated trench. With a fairly clean underground the organisation of the construction site and the guide wall installation can start. The function of this guide wall is not only to guide the grab during excavation, but also as a reference level and support for the installation of the reinforcement, a buffer for the support fluid, to prevent caving in of upper soil layers, marking the panels and as a reaction frame for the jacks which remove the stop ends.

![Process decomposition schematic](image-url)

**Figure 35: Schematisation process decomposition construction diaphragm wall section**
4.1 Excavation

Excavation of the individual panels for the construction of a diaphragm wall is in the Dutch situation performed by means of grabbing equipment. The excavation sequence for a single panel depends on the type of panel (starter, intermediate or closure) and the dimensions of this panel. But there are also other processes during this phase essential for the construction of a diaphragm wall. Monitoring/correcting for verticality/twist/position (in relation to adjacent panels), installing/maintaining support fluid (level), removing obstacles encountered during excavation and the removal of the stop ends. Processes within the excavation construction phase are described and analysed in this paragraph.

4.1.1 Excavation trenches

A panel is excavated in a number of trenches depending on the width of the panel, the dimensions of the equipment, working direction and type of stop end. A good verticality of the panel obtained with symmetrical grabbing resistance for the claws of the grab during excavation. This symmetrical excavation resistance is determining the sequence and number of trenches in the excavation of a single panel. In practice panels are often excavated in two or three trenches and in case of strict settlement requirements in a single trench. The sequence of the trenching is depending on the type of stop end, the type of panel and the working direction and dimensions of both the panel and the equipment. The excavation sequence for the different trenches per type of panel and per type of stop end is given in table 4 and depicted in figure 36.

Stop end sheets require single, triple or any uneven number of excavation trenches for all panel types. For a triple trench panel, the outer trenches are excavated first and subsequently the dam in the middle of the panel is excavated. The use of a stop end pipe requires a single or triple excavation trench(es) for starter- and closure panels, while the number of trenches in an intermediate panel should be even in order to obtain symmetrical excavation sequences. In case of a triple the sequence of a closure panel is from the inside out; first the trench in the middle of the panel and then the outer trenches.

4.1.2 Positioning

The position of the individual panels in relation to each other is an important to obtain a high quality watertight diaphragm wall. Therefore the verticality of the individual panels and the excavation trenches in both directions should be monitored and corrected during the excavation sequence. Using a device mounted on the grab the crane operator monitors the position and rotation of the grab during excavation and if necessary corrects for
deviations. Correcting the position or rotation of the grab is difficult since there is no steering mechanism on the grab. Twist of the panel is prevented by rotating the grab 180° after each excavation sequence in order to eliminate the asymmetrical behaviour of the grab. The verticality of the panel/trench should be accomplished by the high self weight of the grab, which provides the downward vertical force. Encountering a harder soil layer or an obstacle in the trajectory of the panel deflection of the grab is likely to occur.

4.1.3 Obstacles
When the verticality is at stake due to a local hard soil layer or an obstacle in the ground, a chisel could be used to provide the necessary force to remove the obstacles and restore verticality. Chiselling may be necessary for excavation through hard deposits or concrete from an adjacent panel which has flown round the stop ends. The process of chiselling has a negative influence on the quality of the diaphragm walls, because the high turbulent currents around the chisel are eroding the ground around the excavated trench and the chisel itself can damage the excavated trench. People living in the vicinity of the trench can feel the vibrations of the grabbing equipment in their houses not to mention the vibrations caused by the chisels when obstacles are removed.

4.1.4 Verticality
Usually the grab is equipped with an instrument which is able to measure position, rotation and verticality of the excavated trench in order to be able to correct if the deviations are too large. The verticality of the diaphragm panels is important especially with increasing excavation depth; hence the deviations are also increasing with depth. At surface two adjacent panels with a thickness of 1m are perfectly aligned, but both of them have a deflection of 1% of the depth, at 10m there is already a deflection of 0.2m. If both panels have a deflection of 1% in opposite directions the connection between these panels 10m deep is on 0.6m and with 20m excavation depth the connection could be 0.2m which becomes unacceptable especially when circular building pits are constructed. In general a deflection under 1% is acceptable. Verticality of a panel is depending on the soil layers, the crane operator and the excavation method, as specified in table 5.

<table>
<thead>
<tr>
<th>Excavation method</th>
<th>Verticality</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cable excavator</td>
<td>1/300</td>
</tr>
<tr>
<td>Kelly excavator</td>
<td>1/500</td>
</tr>
<tr>
<td>Hydrofraise excavator</td>
<td>1/2000</td>
</tr>
</tbody>
</table>

Table 5: vertical accuracy main types of excavators [Sanno, 1992]

4.1.5 Support fluid
During the excavation the level of the support fluid is changing due to the volume of the grab which is lifted out of/lowered in the trench and therefore there should be enough buffering capacity. These fluctuations should be limited and not fall below the level required for excavation stability and not below the base of the guide walls in order to prevent soil from caving into the trench. Not only is the level of support fluid important to monitor but also the properties of the support fluid itself. If the properties of the bentonite are influenced the plastering effect of the bentonite could be influenced and the stability of the excavated trench could be at stake.

4.1.6 Erosion
During the excavation process the trench depth is increasing and consequently decreasing the excavation speed. To increase the productivity the crane operator sometimes gains time by lowering the excavation tool with a free-fall and lifting the tool with higher speeds than allowed resulting in turbulent currents and erosion of the excavated trench. This erosion has a higher impact in coarse grained layers and is negligible in cohesive soils.
4.1.7 Stop end sheets

Trenches excavated adjacent to concreted panels are guided either by the cavity left behind by the stop end pipes or by the guiding the grab along the stop end sheet facilitating a good connection between panels. This is done by using the claws mounted on the grab (for instance) every fifth grab to guide the grab along the stop end sheet and thereby pulling the stop end loose from the concreted panel. When the excavation trench closest to the stop end has reached the bottom of the stop end sheet it is removed by means of crane. Some contractors don’t use the claws during the excavation to avoid maintenance grab by replacing the broken claws. The stop end sheet is removed when the trench is at the depth of the stop end sheet.

Sometimes problems occur regarding the removal of stop end sheets because concrete from the adjacent panel is blocking the stop end sheet. Due to this the claws mounted on the grab collapse and the stop end cannot be removed with the claws on the grab. The claws can be easily replaced, but will almost certain collapse again. A chisel or jacks can be a solution to this problem and if not the stop end sheet has to stay in the trench and a high leakage risk is created.

4.2 Cleaning

When the designed depth of the excavation is reached the excavated trench is prepared for concreting, by creating optimal conditions for the concrete to replace the support fluid and improve the quality of the watertight connection between individual panels. This construction stage is important for the quality of a diaphragm wall and usually takes quite some time, without the need of effort from the workers. In this paragraph cleaning processes and requirements of the support fluid before concreting are elaborated.

4.2.1 Support fluid

The properties of the support fluid are changing during the excavation of the panel, for instance the density and viscosity increased by the higher sand contend, which reduces the ability of bentonite to be replaced by concrete. For this reason requirements are set for the support fluid. In order to accomplish design criteria two methods can be used: replacing all the support fluid by a clean ‘fresh’ support fluid, or regenerating the support fluid. The advantage of the first method is that this process is less time consuming compared to the regeneration method because the volume pumped in and out of the trench is similar to the volume of the trench, while the amount of support fluid pumped in and out of the trench with the regenerating technique is in the order of 2 or 3 times the volume of the trench. The disadvantage is the necessity of two storage basins, one for clean or fresh bentonite and one to store bentonite used during the excavation sequence. By lowering a pump into the trench support fluid is pumped out from the bottom and fresh or regenerated support fluid is pumped in form surface. This process can also be done by an airlift system, using a pipe and compressed air to lift the suspension to ground level. The
required properties for bentonite suspensions are stated in the codes, and the following parameters are monitored: density, viscosity, fluid loss, pH-value, and contend, filter cake.

![Image](image.png)

**Figure 38: replacement of support fluid**

**4.2.2 Bottom**

Sand will settle down from the suspension and will deposit on the bottom of the trench and will be mixed with concrete resulting in a reduced quality of the concrete if no measures are taken. Cleaning the bottom of the excavation either after replacing or regenerating the support fluid by means of the grab or by displacing the pump or airlift over the bottom of the trench during the replacing or regeneration of the support fluid should prevent these problems. If there is no sand deposit on the bottom of the trench the bearing capacity of the panel will be better compared to trenches where the bottom is not adequately cleaned.

**4.2.3 Joints**

Depending on the type of panel, starter (0), intermediate (1), closure (2), a number of connections is formed during concreting. For a starter there are no connections made, only the rubber water stop is concreted into the panel, but for intermediate or closure panels one or two connections are created. The quality of this connection depends on the joint system and the amount of bentonite or other material on the surface of the already concreted panel. To obtain a high quality joint with a low leakage probability the surface of the connection with the adjacent concreted panel is cleaned sometimes. Cleaning the joints is performed by intermittently lifting and lowering a heavy eccentric suspended con-mould chisel (stop end pipes) or a heavy eccentric suspended steel brush (stop end sheets) in the excavated panel alongside the adjacent concreted panel in order to remove bentonite cake or other inconsistencies.

**4.3 Inserts**

After the cleaning process usually one or more reinforcement cages and if necessary stop ends are inserted prior to the concreting process. During the placement of these inserts it is important not to damage the excavated trench and not to disrupt the support fluid. If the cleaning procedure is finished the amount of time between the end of cleaning and the start of concreting should be kept as small as possible. Sand which was in suspension in the support fluid can settle down and reduce the quality of the concrete as mentioned earlier and bentonite can adhere to the cleaned surface of the concreted panels.
4.3.1 Joint formers
In case of a start or intermediate panel 2 or 1 stop end respectively should be inserted in the trench. Stop end sheets are suspended on the guide walls in order to create a distance between the bottom of the trench and the lower end of the stop end sheet. Stop end pipes should be inserted into the trench penetrating the bottom of the trench to ensure its position and prevent concrete flowing into the stop end pipe. The best way to do this is by using the weight of the pipe and the free-fall mechanism of the crane. For this operation the bylaw obligated safety mechanism against free fall should be turned off (Admiraal).

4.3.2 Reinforcement
Installation of the reinforcement is performed by means of a handling crane which can lift the long reinforcement form a horizontal position into vertical position in order to be able to lower the cages into the panel. When the reinforcement of the panel is long compared to the transport length, lifting capacity (length and weight) the reinforcement can consist of more than one cage. The first cage is lowered into the trench and suspended on the guide wall to couple the upper cage to the lower cage of the reinforcement. Placement of reinforcement cages could damage the excavated trench and contamination of support fluid by the reinforcement cage. By damaging the excavated trench parts of the filter cake from the trench will be suspended in the support fluid and or stick to the reinforcement. Difficulties during the coupling of reinforcement cages sometimes requires welding activities while the reinforcement cage is partly in the bentonite suspension, which can damage the filter cake due to electrophoresis and could result in a collapse of the trench.

4.4 Concreting
Concreting of the panel is the last step in the construction of a diaphragm panel. The trench is filled with concrete from the bottom upward in order to replace the support fluid. During this process bentonite cavities are formed due to a combination of some suboptimal circumstances. In this paragraph the process of concreting is elaborated together with points of attention with respect to the reduction of the probability of bentonite cavities.

4.4.1 Tremie
Concreting is performed by means of the tremie method, which means that the concrete is poured in a funnel which is connected to a pipe positioned just above the bottom of the excavated panel. The concrete replaces the bentonite from the bottom upwards where it is pumped away at surface. To prevent segregation of the concrete in the tremie it must be kept separated from the support fluid, which is achieved by putting a plug of vermiculite, wood or a plastic ball in the tremie before the first charge of concrete is poured. This will float on the suspension and will keep the concrete separated from the support fluid until the concrete reaches the end of the tremie. Now the concrete is in contact with the support fluid, where some minor mixture will occur which results in a reduction of the concrete quality. The first meter of concrete is regarded as contaminated and cannot be used as concrete for the diaphragm wall and will be removed when the wall is excavated. To prevent further quality loss it is important to that the tremie is kept in the concrete for at least 6m before a section (3m) is decoupled in order to shorten the tremie. Codes for the construction of diaphragm walls require the tremie submerged for at least 3m into the fresh concrete. During the concreting process the level of concrete is measured after each truck on different locations in the trench.

The number of tremie pipes to be used for one panel depends on the properties of the concrete and the dimensions and the shape of the panel. At least one tremie should be used for each reinforcement cage and usually a corner or T shaped panel is concreted with respectively two and three tremie pipes. When more than one concreting pipe is used the supply of concrete has to be arranged in order to obtain a uniform upward flow of concrete. Preferably the number of tremie pipes should match the number of concrete mixers pouring concrete at
the same time. The location of the tremie pipe to the proximity of the joint former further improves the quality of the concrete near the joint as it can provide more pressure to force the slurry away from the joint area. [Tsai et el., 1991]

4.4.2 Concrete flow

Tremie concrete replaces the bentonite on the bottom of the trench progressively upwards due to the difference in volumetric weight and viscosity. The density of concrete is more or less fixed depending on the composition, while the density of bentonite depends on different variables. When the cleaning process is carefully performed the density of the support fluid should be significantly lower compared to concrete 10.5kN/m³ and 24kN/m³ respectively.

Besides replacing bentonite in the vertical direction concrete has to flow horizontally to cover the entire cross section of the panel. The reinforcement cage becomes an obstacle for the flow of concrete depending on the structure of the reinforcement cage which should be kept as open as possible. On the other hand the viscosity of the concrete should be low to achieve high workability.

From research by Gerwick et al. [1986] under water tremie concrete can flow in two different patterns; a layered flow and a bulging flow pattern (figure 39). Layered flow patterns are a result of concrete with a high internal shear resistance forcing the new concrete to flow upward around the tremie over a the slope of the previous concrete. The bulging flow pattern produced a general displacement of concrete resulting in much platter slopes.

The quality of the end product was significantly better compared to the concrete with high shear resistance; this was mainly due to less laitance.

figure 39: layered and bulging flow pattern of underwater tremie concrete

4.4.3 Monitoring

At the start of concreting a panel the concrete delivered on the construction site should be tested for material properties before it is put into the panel. Consistency of the concrete is tested by means of a flow table test or a slump test before starting concreting and the concrete for each truck should be checked visually according to Dutch codes.

Monitoring the rise of the concrete level after every delivery information on the consistency of the shape of the wall is obtained hence over consumption of concrete is an indication for irregular forms of the diaphragm wall. With the rise of concrete level the length of the tremie can be progressively shortened. It is recommended that the minimum immersion depth should be at least 3 m, but can be reduced to 2 m when the level of concrete is accurately known. Monitoring the angle of the rising concrete could be checked during the process to ensure a good concrete process.
4.4.4 Stop end pipes

Start and intermediate panels equipped with stop end pipes are not finished when the concrete is poured. The stop ends have to be removed slowly during the hardening process of the concrete. The process of stop end removal is a delicate procedure, hence there is no or little information on the behaviour of the concrete in the panel. The required forces for the jacks to pull the stop end pipes is a measure for the hardening process of the concrete. For diaphragm walls to a great depth the forces will be quite high and the removal of the stop ends could be problematic, which is why during the concreting process the pipes are pulled out a little to reduce the accretion of the concrete to the stop ends. Normally the team or part of the team has to stay for 4-5 hours after the last concrete truck.

figure 40: removal of stop ends pipes
5 Support fluid & filter cake formation

Support fluid used to construct diaphragm walls is usually a colloidal bentonite suspension. This bentonite suspension is made by mixing dry bentonite powder with water. This colloidal bentonite suspension is used to support the panel during excavation until the panel is concreted, by formation of a relatively impermeable seal. During excavation and concreting the bentonite suspension is contaminated with soil particles and affected by the environment which changing properties as a consequence. Monitoring the properties is during the process is performed using laboratory and in field measurement techniques. This chapter elaborates on bentonite material- and colloidal suspensions properties, bentonite filtration cake formation, bentonite suspension testing and the consequence of environmental influences.

5.1 Bentonite

Bentonite is the name of a clay named after the location where it was discovered in 1898, Fort Benton in the state of Wyoming (U.S.). This clay consists mainly of the mineral montmorillonite which is part of the smectite group. Montmorillonite and was discovered more than 50 years earlier (1847) in Montmorillon, France. Bentonite is formed by chemical a geological processes on volcanic deposits in salty lakes hence it is found in Europe along the Mediterranean sea from Sardinia to Turkey, but also in Germany and Great Britain calcium bentonite is mined. The most famous and also the largest natural sodium bentonite mine is found in Wyoming. Mining of bentonite is usually in large open pit mines where the bentonite chunks have a moisture content of about 30-40 % (m/m).

5.1.1 Bentonite type

In natural form bentonite consists for about 80-90% of the mineral montmorillonite and some closely related materials such as feldspar, mica’s kaolinite, vermiculite, quartz, chlorite, illite, clinoptilolite, calcite, gypsum and cristobalite. Montmorillonite has a layered molecular structure with in between the crystal lattice of montmorillonite interchangeable cations such as sodium (Na), calcium (Ca) or kalium (K). The different types of bentonite are determined by these elements and also named after these elements. Sodium and calcium bentonite are used in civil engineering purposes but the properties of sodium and calcium bentonite are quite different. Given the fact that natural sodium bentonite has the best properties this type of bentonite is preferable in civil engineering. The problem is that sodium bentonite is very rare and calcium bentonite is quite easy to get and therefore cheaper. But the industries have found that the natural calcium bentonite can be adapted in order to create an artificial sodium bentonite by interchanging the cations. This process is performed by interchanging the calcium by sodium cations using soda (sodium carbonate) resulting in activated (calcium) bentonite and calcium carbonate:

\[
\text{Ca-bentonite} + \text{Na}_2\text{CO}_3 \rightarrow \text{Na-bentonite} + \text{CaCO}_3
\]

5.1.2 Application

Application of bentonite varies from civil engineering to cosmetics and medicines, from oil and gas industry to food and winemaking purposes, but most commonly known as cat litter. An overview of the properties and applications is given in figure 41. In civil engineering bentonite is used as bentonite suspension for instance as drilling fluid in horizontal directional drilling, in (micro) tunnel boring machines and as support fluid in diaphragm wall construction. Due to the swelling capacities of bentonite and the ability to form a watertight seal bentonite is used as groundwater barrier in a bentonite-cement wall or in a soil-bentonite wall.
5.1.3 Structure

Bentonite clay minerals are usually found as small particles, with a typical hexagonal plate form which has a large surface area to mass ratio. The diameter of these plates is often less than one micrometer and has a length-width and thickness ratio of 500:1. In dry conditions these plates are stacked together with some water molecules in between the plates (figure 42).

![Figure 42: Structure of Montmorillonite](image)

Individual platelets are consisting of two tetrahedral silicate sheets sandwiching a central octahedral aluminium sheet. The tetrahedral silicate sheet is made out of four-sides pyramidal geometrical shapes; tetrahedrons. A tetrahedron is made of four oxygen atoms packed around a single silicon atom. The single silicon-oxygen tetrahedron is a complex ion with a formula of SiO$_4^{4-}$, silicon has a charge of $+4$ and the four oxygen ions have 8 negative charges ($-2$ for each oxygen atom). An octahedral aluminium sheet is made out of eight sided geometrical shapes; octahedrons. An octahedron is made of oxygen atoms packed around a single aluminium atom. The single aluminium octahedron has the formula of Al(OH)$_6^{3-}$, aluminium has a charge of $+3$ and the six hydro ions have 6 negative charges.
The typical smectite 2:1 three-layered clay mineral forces the two tetrahedral silicate sheets sandwiching a central octahedral aluminium sheet to share atoms in order to be balanced enough to be stable. The tips of the tetrahedral point towards the centre sheet where some of the oxygen atoms at the tips are shared with the central octahedral sheet. This structure of sandwiched sheets has a net negative charge which is filled in by the interchangeable sodium or calcium cations in between the platelets.

In montmorillonite some aluminium ions (Al$^{3+}$) or silicium ions (Si$^{4+}$) have been replaced by ions of similar size but of lower valence (Fe$^{2+}$, Mg$^{2+}$ or Al$^{3+}$) respectively. This isomorphous replacement in the crystalline structure providing a net negative charge which is compensated by the cations which lie between one unit layer and the next. [Tomlinson] & [de Vries, 1992] The chemical formula of montmorillonite becomes 4SiO$_2$·Al$_2$O$_3$·H$_2$O [de Vries, 1992].

In dry conditions the montmorillonite sheets form a layered structure with water molecules and exchangeable cations in between. Layers of montmorillonite and water/cations are each approximately 1nm in thickness but both the crystal structure of the montmorillonite as well as the intermediate layer of water and cations can swell when moisture content is increased, resulting in a double thickness of the layers.

5.2 Colloidal bentonite suspension

A bentonite suspension is a colloid system which is a type of chemical mixture in which one substance is dispersed evenly throughout another. A colloid system consists of two separate phases, a dispersed phase and a continuous phase. Bentonite particles are not dissolved in the water; they are small enough to be evenly dispersed and large enough to scatter light. This colloidal system is a typical form of a sol, which consists of a solid dispersed phase and a liquid continuous medium.

The colloidal bentonite suspension is prepared by mixing 30-80 kg of dry bentonite particles inherently with one cubic meter of water. This water should match the criteria for concrete and mortar, and an additional requirement
for the conductivity of this water <900µS/cm. The conductivity is a measure for the amount of dissolved salts, which could result in the flocculation of bentonite suspension. [de Vries, 1992]

### 5.2.1 Card house structure

The bentonite plates are negatively charged at the surface and slightly positively charged at the edges surrounded by a cloud of cations, they form a structure comparable to a card house, depicted in figure 45a. The colloidal bentonite suspension has certain strength due to the structure of the individual clay platelets which is formed by positive and negative charges on the particle surfaces and forms a kind of network (figure 45).

![Card house structure diagram](image1)

**figure 45:** clay minerals in (a) card house (b) band type structures and (c) transition between a and b

### 5.2.2 Intracrystalline swelling

Between the weak unit layers there are only weak bonds and therefore can be easily severed enabling water molecules to enter between the unit layers causing the crystal to swell. The sodium montmorillonite undergoes crystalline swelling and continues with a rapid uptake of a large quantity of water which results in a swelling of 5-15 times its own volume. This ripen process can take up to 24 hours, where individual mineral platelets create a layer thickness from 1-2 nm up to a theoretical infinite thickness. As a result of this swelling and de solvability of sodium cations in water, repulsive forces between montmorillonite plates are created and force them to move farther apart. As a result the layered structure is demolished and a suspension of individual plate particles.

![Intracrystalline swelling diagram](image2)

**figure 46:** intracrystalline swelling of calcium and sodium bentonite [Koch, 2002]
5.2.3 Zeta potential

Under the influence of the physical (secondary) Van der Waals forces bentonite particles stick together, causing a coagulation of the individual particles in water. The Van der Waals force is an attractive or a repulsive force between dipole molecules (or between parts of the same molecule) and is relatively weak compared to primary bonds. Besides Van der Waals forces the particles dispersed in water are electrically charged and similar particles have the same charge and therefore have repulsive forces. If the range of these forces is larger than the amount of Van der Waals forces the coagulation of particles is impossible.

The amount of the electrical charge is called the zeta potential and depends on the amount of interchanged atoms in the crystalline structure; the range of this zeta potential is the Dabye length. In colloidal systems the electro kinetic potential or zeta potential plays an important role. From a theoretical viewpoint the zeta potential is an electric potential in the interfacial double layer or electrical double layer (EDL). This potential difference is formed in between the dispersed bentonite and the stationary layer of water attached to this dispersed particle. The range of the electrical surface charge depends on the salinity of the suspension and the valence of the cation (calcium or sodium). A high salinity and a high valence of the cation reduce the range of the electrical forces. A low range and a low surface charge results in a low zeta-potential. The zeta potential level determines the stability behaviour of the colloidal bentonite suspension, at low zeta potentials coagulation or flocculation of the colloid occurs rapidly. Zeta potential higher (positive or negative) than 25 mV are free from coagulation or flocculation [CUR 189].

![Diagram of colloidal bentonite suspension, electrical double layer and zeta potential](image)

**figure 47:** colloidal bentonite suspension, electrical double layer and zeta potential

5.2.4 (Rheological) properties

The card-house structure extends throughout the bentonite suspension which results in a gel structure of the suspension. The bonds between the edges and surfaces of the plates are relatively weak, but if the gel is stirred these bonds are broken and the system behaves like a fluid. At rest bonds between the bentonite plates are formed again and the gel structure is restored again. This phenomenon is called thixotropy and is an important property regarding the use of bentonite.

Due to the thixotropic behaviour the viscosity of a bentonite suspension decreases over time at a constant shear rate. The longer the fluid undergoes shear stress, the lower the viscosity will become. When a certain change in shear stress is applied to a thixotropic fluid it takes finite time to attain an equilibrium viscosity. During the
excavation of the diaphragm wall, the support fluid is constantly in motion by the movements of the grab, resulting in a low viscosity, while during concreting the suspension is at rest resulting in a higher viscosity. Due to the higher viscosity, the soil particles coming loose from the wall or falling into the panel are carried by the high viscous bentonite suspension. The thixotropic properties are sensitive to changes in the pH values of the suspension.

The density of a bentonite suspension for civil engineering purposes is slightly higher than the density of water depending on the amount of dry bentonite added to the water and the sand content of the suspension. A fresh bentonite suspension, usually between 10.2 and 10.6 kN/m³, consists of 1 m³ water mixed with 30-80 kg dry bentonite powder. During the excavation sequence the bentonite will be contaminated with sand and other soil particles which increases the density; values up to 14 kN/m³ are normal.

5.3 **External influences**

Rheological characteristics of a bentonite suspension can change by external influences. During the application of bentonite as a support fluid the medium will be contaminated with soil particles. Due to the contamination with for instance sand the density, viscosity and hence the rheological properties are changing. But chemical changes in the suspension are also resulting in a change in rheological characteristics; the behaviour of dispersions of montmorillonite in water depends on pH and conductivity of the suspension.

A colloidal bentonite suspension has alkaline properties depending on the type of bentonite used, usually a pH value of 8.5 to 9 is observed. In an acidic environment (at low pH values) kalium (K⁺), sodium(Na⁺) and calcium (Ca²⁺) cations are displaced by hydrogen. The internal structure of montmorillonite is also affected by the acidic conditions because the Al³⁺, Fe²⁺, Mg²⁺-ions are leached out. The consequence is a chemical more active and therefore less stable clay, with reduced water absorbing properties and increased permeability. The contamination by a saline groundwater environment or by acidulous peat layers is influencing the rheological properties in a chemical way.

5.3.1 **Calcium**

A calcium rich environment causes the sodium cations being exchanged by the calcium cations which are less soluble in water, causing the bentonite to coagulate and flocculate. Since the construction of a diaphragm wall take place by installing individual panels adjacent to each other, bentonite is not only interacting with ground and groundwater. During the hydration process of hardening concrete, cementitious water is expelled from the concrete which interacts with the bentonite.

Cement consists for 40 to 60% of calcium silicates (3CaO.SiO₂ and 2CaO.SiO₂) which dissolve in water and thereby creates an alkaline fluid due to the hydroxide ions CaO + H₂O → Ca²⁺ + 2OH⁻. The Ca²⁺ ions are attaching to the montmorillonite and as a result the charge of the plate is reduce the repulsive forces between the particles resulting in a creating of attractive potentials in the contact regions between the particles causing formation of band-like structures (figure 45).

As a result of these chemical reactions the rheological properties of bentonite are changed, shear stress, thixotropic behaviour and permeability are influenced. The reduction of the card-house structure and flocculation of the fluid can be observed in the microscopic images depicted in figure 48.
5.4 Support fluid preparation and testing

5.4.1 Sample preparation

Preparation of a colloidal bentonite suspension is based on inherently mixing dry bentonite particles, the dispersed phase, with the continuous water phase. After the mixing process bentonite should ripen for a couple of hours, enabling the maximum swelling of the particles and thereby maximizing the properties of the suspension.

The type of bentonite used was CEBO Trenchcontrol which is a commonly used bentonite for the construction of diaphragm walls. This bentonite was also used for the construction of the diaphragm walls for the three deep stations at the North/South line project: Rokin, Vijzelgracht and Ceintuurbaan. Trenchcontrol is composed of a high quality sodium activated bentonite which has been treated in a kept secret process. Properties of Trenchcontrol according to the supplier (CEBO) are stated in table 6.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Method</th>
<th>30 kg/m³</th>
<th>40 kg/m³</th>
<th>50 kg/m³</th>
<th>60 kg/m³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield point</td>
<td>Kugelharfengerät DIN 4126</td>
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<td>3</td>
<td>4</td>
<td>5</td>
</tr>
<tr>
<td>Density</td>
<td>Mud balance</td>
<td>1,02 g/ml</td>
<td>1,03 g/ml</td>
<td>1,03 g/ml</td>
<td>1,04 g/ml</td>
</tr>
<tr>
<td>Filtrate water</td>
<td>DIN 4127</td>
<td>9,5 ml</td>
<td>8 ml</td>
<td>7 ml</td>
<td>5,5 ml</td>
</tr>
<tr>
<td>Marsh funnel</td>
<td>API RP 13B (1 litre)</td>
<td>31,5 s</td>
<td>36 s</td>
<td>41 s</td>
<td>45,5 s</td>
</tr>
</tbody>
</table>

Table 6: properties of Trenchcontrol according to CEBO

All laboratory experiments were performed using CEBO Trenchcontrol bentonite with a ratio of 30 kg of “dry” bentonite for each m³ tap water, unless stated otherwise. For some experiments bentonite was used twice and therefore the properties could have changed but only a significant change in colour was measured.

5.4.2 Tests and requirements

Properties of a colloidal bentonite suspension are usually tested with in the field equipment which are not very precise, therefore some experiments have been replaced or both executed, table 10 provides an overview of properties and testing methods.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Method</th>
<th>Code</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density</td>
<td>Mud balance</td>
<td>ASTMD 4380</td>
</tr>
<tr>
<td></td>
<td>Aerometer Hydrometer</td>
<td>BS 1377: part 2:1990</td>
</tr>
<tr>
<td>Sand content</td>
<td>Sand content kit</td>
<td>API RP 13B</td>
</tr>
</tbody>
</table>
Fresh bentonite suspensions have different properties compared to bentonite used for excavation which are contaminated with soil particles. The parameters of bentonite suspension should comply with certain values before concreting and are stated in table 8. Bentonite testing procedures indicated in table 7 are further elaborated in appendix 12.

<table>
<thead>
<tr>
<th>Property</th>
<th>Unit</th>
<th>Stages</th>
<th>Fresh</th>
<th>Ready for re-use</th>
<th>Before concreting</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density</td>
<td>g/cm³</td>
<td>&lt;1.10</td>
<td>&lt;1.25</td>
<td>&lt;1.15</td>
<td></td>
</tr>
<tr>
<td>Marsh value</td>
<td>s</td>
<td>32-50</td>
<td>32-60</td>
<td>32-50</td>
<td></td>
</tr>
<tr>
<td>τ</td>
<td>Pa</td>
<td>&lt;10</td>
<td>&lt;10</td>
<td>&lt;10</td>
<td></td>
</tr>
<tr>
<td>Fluid loss</td>
<td>cm³</td>
<td>&lt;30</td>
<td>&lt;50</td>
<td>not applicable</td>
<td></td>
</tr>
<tr>
<td>pH</td>
<td></td>
<td>7-11</td>
<td>7-12</td>
<td>not applicable</td>
<td></td>
</tr>
<tr>
<td>Sand content</td>
<td>% mass</td>
<td>not applicable</td>
<td>not applicable</td>
<td>&lt;4</td>
<td></td>
</tr>
<tr>
<td>Filter cake</td>
<td>mm</td>
<td>&lt;3</td>
<td>&lt;6</td>
<td>not applicable</td>
<td></td>
</tr>
</tbody>
</table>

5.5 Filter cake formation

To provide stability the hydrostatic pressure from the bentonite suspension should be transferred to the soil skeleton by means of a seal on the surface. The formation of a bentonite seal can occur according to several mechanisms under varying soil conditions, like cohesive and non cohesive soils. Filter cake formation is larger in non cohesive soils and from fine grained soils to very open structured soils like gravel the thickness of this bentonite filter cake increases. For cohesive less permeable soils no or little cake formation is observed.. There are three mechanisms in the formation of a seal in permeable soils; surface filtration, deep filtration and rheological blocking (figure 49).

Surface filtration occurs when the particles suspended in the colloidal support fluid are relatively larger than the pores of the soil. Surface filtration will only occur in cohesive soils or very fine grained soils with relatively low permeability.
Coarse grained and high permeable soils allow the bentonite suspension to penetrate relatively deep into the pores and voids of the soil and thereby clogging the pores slowly. Eventually a seal is formed and the bentonite suspension is no longer penetrating the soil only filtration water which causes even more clogging of pores and thereby a decrease in permeability. The penetration depth of the bentonite suspension can be up to several millimetres. It is obvious that this type of filtration is less effective due to the high fluid losses and the time required to establish the soil stabilizing mechanism.

Rheological blocking occurs in highly permeable grounds or other open structured soils. This mechanism is based on an equilibrium in shear stresses between slurry and soil particles and the pressure difference. The bentonite slurry penetrates the ground until it is restrained by the total amount of shear stress in a channel. Penetration distance due to this mechanism is described in the following equation:

\[
l = \frac{(\gamma_s h_s - \gamma_w h_w) \pi r^2}{2 \pi \tau} = \frac{\Delta P r}{2 \tau}
\]

where the penetration length (l) is determined by the pressure difference (\(\Delta P\)) and the pore radius divided by two times the shear strength (\(\tau\)) of the suspension.

Filter cake formation from bentonite suspensions is a combination of the three types of filtration mechanisms, occurring at the same time. The dimensions of bentonite particles enable the suspension to penetrate into the soil and consequently the pores will be blocked partly due to rheological blocking and particles blocked in small pores of the soil skeleton. Since the soil skeleton including the intruded bentonite suspension is still permeable and the pressure gradient is unchanged, the filter cake starts in between the pores and on the surface of the soil skeleton according to figure 50.

![Figure 50: Formation of filter cake](image)

5.5.1 Cake thickness

Thickness of bentonite filter cake depends on several parameters involving the properties of the bentonite suspension and the filter medium, but more importantly the pressure difference and the amount of time. In principle Darcy’s law of flow through porous media in combination with a volume material balance the thickness of bentonite filter cake can be calculated.
The total volume of bentonite suspension ($Q_m$) which is filtered generates filtrate water ($Q_f$) and cake thickness $hA$, a volume material balance can be written as:

$$Q_m = Q_f + hA$$

and

$$Q_s = f_{wc} hA$$

where the volume of solids deposited in filter cake ($Q_s$) and volume fraction of solids in cake ($f_{wc}$)

The volume fraction of solids in mud ($f_{sm}$) can be written as:

$$f_{sm} = \frac{Q_s}{Q_m} = \frac{f_{wc} hA}{Q_f + hA}$$

Solving for $h$ gives:

$$h = \frac{Q_f}{A (f_{wc}/f_{sm} - 1)}$$

The latter formula is only valid for a specific moment in time while the formula below holds for cake thickness in time:

$$h = \sqrt{\frac{2k_c (1-n_s) (\gamma_w z_1 - \gamma_w z_2)}{(n_j - n_s) \gamma_w}} \sqrt{t}$$

$h_s$ slurry head
$\gamma_w$ density water
$h_w$ water head
$h_c$ cake thickness
$h_b$ bentonite slurry level
$h_g$ ground water level
$k_c$ bentonite filter cake permeability

Figure 51: formation of surface filtrate by penetration of suspension
\( n_s \)  porosity bentonite slurry
\( n_c \)  porosity bentonite filter cake
\( \gamma_s \)  bentonite slurry density
\( \gamma_w \)  ground water density
\( t \)  time
6  Scenarios

Expert interviews, case studies and process decomposition have gained understanding in the construction of diaphragm walls and some possible influences connected to the formation of bentonite cavities in diaphragm walls. This chapter provides a further elaboration on possible scenarios resulting in bentonite cavities. Scenarios are subdivided; execution method, particular circumstances and special attention is paid to the scenarios related to Vijzelgracht.

6.1  Execution method

The execution method of the contractor building diaphragm walls is important to the formation of a bentonite cavity arising during the process. Contractors emphasize on the importance of team commitment and accuracy of every production step to the occurrence of errors including bentonite cavities in diaphragm walls. Besides an accurate execution of every production step, special attention should be paid to certain aspects elaborated on in this paragraph.

6.1.1  Cleaning support fluid

The method of regeneration or replacement of support fluid is a delicate procedure and when not properly executed this procedure has a significant influence on the quality and properties of the bentonite suspension in the trench. Replacement of support fluid with fresh support fluid is preferable over the regeneration of support fluid because the quality of the replaced bentonite is better than regenerated support fluid and this method is less time consuming. Regeneration of support fluid requires the entire volume to be circulated several times in order to fulfil the requirements of the suspension before concreting, whereas for the replacement of bentonite suspension the exact volume should be replaced.

More important is the procedure of regeneration or replacement of support fluid. Both methods are equally sensible to execution errors. The cleaning procedure is based on the difference in density of the ‘contaminated’ and fresh or regenerated bentonite; bentonite with a higher density is pumped away from the bottom of the trench while fresh relatively low density bentonite is pumped in from the top of the trench. Thixotropy prevents support fluid from flowing to the location where support fluid is pumped away and therefore is not replaced or regenerated. This is compensated for by displacing the location of the pipe from the one end of the trench to the other and thereby shearing this thixotropic bentonite in order to be pumped away.

6.1.2  Joint surface

Cleaning the joint surface of already concreted panels is a procedure reducing the risks of bentonite cavities and increasing the quality of the joint. For panels adjacent to already concreted panels cleaning of the surface of the joint is sometimes performed depending on the contractor and the type of stop end. Traditionally panels constructed using a stop end pipes are always cleaned by means of a contra mould grab or chisel, but with the introduction of the stop end sheets equipped with the rubber water stops the cleaning procedure is often omitted in order not to damage this rubber.

Excavated trenches which are standing open longer than necessary have an increased thickness of bentonite filter cake simply due to the time effect, while the adhesion of bentonite suspension to concreted surfaces also increases. Mineral slurries that remain in the drilled hole for too long can form a filter cake that is too thick for the fluid concrete to scour off the sides [Foundation Manual, 2008]. With this time depending effect the thixotropic properties of bentonite suspension also increases and therefore make it more difficult to completely
regenerate or replace the bentonite suspension which contributes to the phenomena stated in the previous sub-paragraph.

Cleaning the joint surface is nowadays not adequately performed while (NEN) codes for the diaphragm wall construction are maybe not clear enough about the cleaning procedure of the joint: The bottom of the excavation and the surface of the joints shall be cleaned and if necessary, the supporting fluid shall be desanded or replaced. New Dutch code under development NEN-EN 1538: 2009 Ontw.: When a support fluid has to be replaced by concrete or another material, the bottom of the excavation shall be cleaned and the joints shall be cleaned if necessary. In my opinion this means that the joints shall be cleaned if the panel is concreted adjacent to an already concreted panel hence there is no joint to clean if there is no adjacent panel. German codes are more elaborate on that specific and very important subject. German code DIN 4126: “Unmittelbar vor dem Bewehren (bei unbewehrten Wänden vor dem Betonieren) sind die Anschlußflächen an bereits hergestellte Schlitzwandelemente zu reinigen, die Schlitzsohle von dem dort abgelagerten Schlamm zu befreien und die Flüssigkeit im Schlitz zu homogenisieren.” English version: Immediately prior to reinforcing (in case of unreinforced walls, prior to concreting), the joint faces of prefabricated diaphragm wall sections shall be cleaned, the bottom of the trench cleared of mud which has been deposited there, and the liquid in the trench homogenized.

6.1.3 Fluttering stop end sheet

When stop end sheets are removed during excavation by means of the grab using claws, stop end sheets are fluttering until they have are completely removed. This back and forth movement of the stop end sheet in the trench may result in scraping off filtration cake/soil particles from the sides of the trench. This material of filter cake/soil particles will be plastered against the joint surface by the fluttering movement of the stop end sheet. Material from the grab falling in between the concrete and the stop end sheet will also be plastered against the face of the joint during the excavation. This process could result in a significant amount of material on the joint surface, in case joints are not cleaned, cannot be removed by scouring concrete. The effects of this scenario, excess bentonite or plastered soil on the joint surface, can easily be dealt with if contractors clean the joint surface prior to the concreting procedure as described in the previous sub-paragraph. Therefore this scenario needs no further research to prevent future cavities caused by this mechanism.

![figure 52: fluttering stop end sheet during excavation](image)
6.1.4 Verticality

When the deviation in verticality of the excavation becomes too large problems can occur with respect to the connection of the joints of individual panels. During the excavation the grab will be mounted on the stop end sheet using claws and will be able to loosen the stop end sheet in order to remove it. With the stop end sheet removed bentonite and soil particles could be plastered against the joint and now the verticality of the excavated trench disables the removal of this substance by a joint cleaner. This scenario involves a combination of conditions which will probably not occur during the construction of a diaphragm wall. Although the verticality of the trench is monitored and corrected for during the excavation a large deviation in verticality could occur, while at the same time soil or bentonite cake should be plastered against the joint.

6.1.5 Concrete level versus tremie level

Monitoring and maintaining concrete and tremie level is very important during the concreting process. The level of concrete determines the progressively shortening of the tremie pipe during the process. When the tremie level is close to the concrete level, the flow pattern of concrete is different compared to a deeper tremie level which could result in a higher risk of bentonite cavities. A more extreme situation occurs when the tremie level is accidently higher than the concrete level and consequentially the rise of concrete is stopped and a horizontal bentonite cavity is formed in the panel. A sudden lowering of concrete level is also a scenario which has a causal connection to the formation of a bentonite cavity. This sudden drop can be caused by passive soil failure with a concrete flow into the stop end pipe.

6.1.6 Curing time previous panel

Curing time of an already concreted panel determines the amount of interaction in between this concrete and bentonite suspension in the new panel. Interaction with bentonite suspension is based on the chemical reactions from ionic substance and alkalinity of concrete. The degree of chemical reaction is determined by the amount of substances interacting with bentonite in combination with the possibility of these substances to migrate to the bentonite suspension. Properties of concrete are strongly related to time and for instance permeability of concrete is also playing a key role in this scenario because permeable concrete enables substances from concrete to flow to the surface of the concreted panel. When the time between concreting a panel and excavating the adjacent panel becomes smaller, the amount of adhesive bentonite on the joints becomes larger due to these reactions. This reaction causes bentonite to flocculate forming a thicker bentonite cake on the joint surface.
6.1.7 Cement type

The type and amount of cement used in concrete determines the properties of the concrete itself as well as the amount and degree of substances involved during the hydration and hardening of concrete. Development of calcium hydroxide, an important hydration substance, is directly related to the amount of Portland cement clinker and indirectly related to the amount of blast furnace slag and fly ash. The generated amount of cement water including calcium hydroxide by Portland cement clinker is reduced in presence of blast furnace slag and fly ash. An interaction in between bentonite suspension and calcium hydroxide is resulting in a degradation of filtrate properties as the bentonite will flocculate and therefore generate more filtrate water and more cake thickness. The thickness of bentonite adhering to concrete surface is also increasing with the presence of more calciumhydroxide. The susceptibility of bentonite suspension to cementitious water depends on the properties of bentonite suspension. In general concrete based on Portland cement is more problematic compared to blast furnace cement (in Dutch: hoogovencement) and fly ash cement as the amount of Portland cement is reduced.

6.2 Flow behaviour

Flow behaviour of bentonite suspension and fresh concrete during the concreting process is one of the most important aspects regarding the occurrence of bentonite cavities in diaphragm walls. This flow behaviour is a complicated phenomenon since there are many parameters involved are these parameters are also due to the hardening of concrete and the influence of concrete to bentonite suspension.

6.2.1 Flow resistance

Flow resistance of concrete relative to the geometry of the excavation and relative to the support fluid determines the occurrence of bentonite cavities arising during concreting. In horizontal direction the concrete should have a symmetrical flow resistance by the shape of the excavation and by the layout of the reinforcement in order to obtain a horizontal layer progressively flowing upward. When the flow resistance is not symmetrical the level of concrete could have a inclination and therefore a laitance can occur on the lowest level of the concrete which can be plastered at the joint of the adjacent panel (figure 54). In this case the eroding capacity of concrete flowing upward is not enough to remove this plastered layer and a bentonite cavity is created. When the flow resistance becomes to large the use of two tremie pipes can reduce the flow distance and thereby resistance of concrete flow. Again the problem of an inclination in the concrete front can result in a bentonite cavity.

![figure 54: inclination of concrete front](image)
Flow resistance relative to the bentonite suspension is another mechanism susceptible to the occurrence of bentonite cavities during concreting. Viscosity of bentonite suspension in increasing with increasing shear rates and so is the viscosity of concrete. At low velocity gradients the differences in shear stress of both materials is decreasing and which makes the displacement of bentonite suspension by concrete more difficult and therefore could result in a bentonite cavity. Higher concreting speeds and therefore higher velocity gradients results in a larger difference in between support fluid and concrete (figure 55).

![Figure 55: shear stress and velocity gradient during concreting](image)

6.2.2 Scouring effect

The scouring effect of the concrete rising in the panel should remove a significant amount of bentonite cake if this is present during the concreting procedure. The scouring effect depends on parameters regarding the concrete, position of the tremie and the rate of rising concrete, furthermore the properties of the bentonite cake and the thickness determine the residual bentonite cake thickness on the joint surface and thus the possibility of an bentonite cavity between both panels.

According to [Tomlinson, M.J.] this scouring effect is negligible; *The order of shearing resistance of bentonite filter cake formed on a diaphragm wall is several times greater than the shear resistance of bentonite slurry or fresh concrete. Therefore the scouring effect of rising concrete is not sufficient to remove the bentonite cake partly or totally from the trench.*

6.2.3 Concrete break

Flow behaviour of concrete is influenced by the curing time of the specific charge of concrete. If a calamity occurs during the process of concreting and consequently the deliverance of concrete is delayed, the already poured concrete is progressively hardening and major difficulties can occur when finally fresh concrete arrives at the construction site. The hardened concrete will be much more viscous and therefore the fresh concrete follows the path of the least resistance, probably along the tremie and consequently flowing over contaminated bentonite suspension creating a large horizontal bentonite cavity in the diaphragm wall.

At the Diabolo project this calamity has occurred with larges consequences to the quality of the diaphragm wall. Fortunately there were no other constructions nearby and groundwater level was lower than the bentonite cavity, therefore both sides of the diaphragm wall could be repaired quite easily. The break in the concreting process was only an hour and a half, while consequences involve major bentonite cavities.

6.2.4 Contaminated concrete

The first charges of concrete are in contact with bentonite and therefore have a lower quality than concrete which hasn’t been in contact with bentonite. Depending on the soil suspending capacity of the bentonite sand (or soil) particles will be deposited on the concrete front and will also reduce the quality of the concrete. The theory of
the tremie method is to let the low quality concrete rise to the top of the panel where it can be removed, resulting in a panel of high quality concrete. In case of a dense reinforcement cage the flow of concrete is reduced around the steel form the reinforcement and consequently the contaminated concrete sticks to the reinforcement. As a consequence the low quality concrete doesn’t flow to the top where it can be removed.

6.3 Particular circumstances

Particular circumstances during construction of a diaphragm wall can lead to a bentonite cavity when bentonite suspension is influenced by external influences or calamities occur during the construction. This paragraph contains less obvious scenarios involving bentonite cavities in diaphragm walls.

6.3.1 Stray voltage/current

The effect of stray voltage or stray current on the stability of excavated trenches supported by bentonite suspension could result in a collapse of the excavated trench. This phenomenon may occur in the presence of cables and ducts in the underground or due to a railway at level. The consequence of electricity in the underground in the vicinity of the excavated trench could be severe with a total collapse of the excavated trench or a local soil collapse (consequences elaborated in next sub-paragraph). Although this scenario is highly unlikely to occur there should be accounted for during diaphragm wall construction.

6.3.2 Soil collapsing

Collapsing ground or soil particles during concreting are another mechanism which could be held responsible for the occurrence of bentonite cavities. When a wedge of soil collapses it either gets stuck in the reinforcement cage or it settles down to be deposited at the concrete. Due to the small density difference between concrete and saturated sand or clay, there is a large possibility that the soil is not replace by concrete. The collapse of soil can occur as a large scale micro instability or soil caving in as a result of vibrations by concrete tucks in combination with a low head of support fluid or soil from level that falls in the trench.

6.3.3 External influences

Besides influences from ground improvement of from previously cast panels, soil and groundwater could also influence bentonite suspension. Acid peat layers or saline groundwater could influence the properties of bentonite suspension. Again viscosity changes could result in problems during regeneration/replacement of
bentonite suspension or during concreting. In this scenario the susceptibility of bentonite suspension to acid or saline influences should be aim of further research.

6.3.4 Permeability of concrete

If the concrete of the already concreted panel has a relatively high permeability and there is a high potential with respect to groundwater pressure and the bentonite pressure, bentonite cake will be formed on the concrete surface as it happens on the permeable sand layers in the ground. The amount of water and cement in concrete, usually denoted as W/C-factor, determines the permeability of the concrete and therefore the susceptibility to the formation of bentonite cake. The permeability of the concrete increases with an increasing W/C-factor while the permeability is very small for the minimal W/C-factor for concrete to have all the cement hydrated; 0.3 according to the Dutch Centre for Cement and Concrete. Furthermore the quality of the concrete at the joint is determined by the properties of the concrete itself, the distance the concrete has to travel and the amount of obstacles in the reinforcement. When there are not enough fine particles on this location the permeability and thereby the quality of the concrete becomes critical and hence susceptible to filter-cake forming.

![figure 57: W/C ratio and permeability of concrete](Cement & Beton Centrum)

For this scenario it is important to gain understanding in the parameters influencing the permeability of concrete besides the initial water cement ratio. Furthermore the change of this water cement ratio in concrete due to the application in diaphragm walls with the tremie technique, hypothetically it could be that this ratio increases due to the influence of a high groundwater head or due to contact with bentonite suspension. In the prefab concrete industry in conditioned environment demixing of concrete occurs on a daily basis. This demixing of concrete results in permeable concrete because there is only a skeleton of gravel is formed without smaller particles in between.

6.4 Vijzelgracht

The Vijzelgracht situation is separately elaborated in this chapter. There are several important scenarios which could have played a role in the occurrence of bentonite cavities in the diaphragm walls of this station. First of all the cement bentonite interaction with respect to the ground improvement, whether or not in combination with the method of cleaning support fluid, whether or not in combination with the (omitted) joint surface cleaning.
6.4.1 Ground improvement

Ground improvement in the trajectory of the diaphragm wall is sometimes used when obstacles are removed prior to the installation of a diaphragm wall. For Rokin and Vijzelgracht station ground improvement was used after obstacles have been removed in the trajectory of the diaphragm wall. For this ground improvement different techniques were used with in principle the same objective; stabilising the soil without vibrations to the surrounding buildings. Cementitious material was used to stabilise the backfilled (softmix) or mixed in place (MIP) soil prior to the excavation of the diaphragm walls.

These cementitious materials of the ground improvement are consequently excavated and therefore become in contact with bentonite suspension. The influence of cementitious material to the bentonite suspension can cause a change of properties of this suspension. Changed rheological properties of this suspension could provide problems during the regeneration or replacement of bentonite prior to concreting as described in sub-paragraph 6.1.1 and the next paragraph.

To check this hypothesis for the situation of Vijzelgracht station, the composition of ground improvement at this particular situation should be determined as well as the age of this material in relation to the construction of the diaphragm wall. The influence of cementitious material to the bentonite suspension should be determined in At Vijzelgracht station this procedure was omitted and in combination with possible effects from bentonite softmix interaction could have caused viscous bentonite to be left in the trench.

6.4.2 Cleaning support fluid

Cleaning or regeneration of support fluid as described in paragraph 6.1.1 was also performed at Vijzelgracht. Although this procedure was carried out according to the requirements of the codes, the displacement of the pumping pipe from the one end of the panel to the other was omitted. The scenario depicted in figure 59 could have resulted in a bentonite cavity leading to the calamity occurred at Vijzelgracht station. For this scenario important parameters regarding to the viscosity of bentonite suspension are important to conclude something about the cleaning failure as well as the behaviour of suspension during concreting. The viscosity parameters of both bentonite phases as well as concrete should be the input for a 3 phase flow model simulating the concreting process.

![figure 59: cleaning failure](image)

6.4.3 Joint surface

This cleaning procedure is nowadays usually not performed, except for the English contractor (Skanska Cementation) who builds the diaphragm walls at Ceintuurbaan station. In the Netherlands this procedure is not
well known as a part of the diaphragm wall construction by designers or (consultants of) the client (Driesse). On the other hand the majority of the contractors are not cleaning the joint surface and rely on their experience without cleaning the joints. By performing this step the risk of bentonite cavities in joint between panels is significantly reduced, while there isn’t a lot of effort to be made. For Vijzelgracht station no joint cleaning has been performed at all. If panels were standing open for a long time no additional measures were taken to prevent or remove bentonite from the joint surface. An interesting remark emphasizes the high quality of diaphragm walls at Ceintuurbaan in comparison to Rokin and Vijzelgracht station [Deltares, 2008]. This higher quality could have been obtained by the joint surface cleaning which was used in a structural way at this station.

6.5 Conclusions

This chapter contains the scenarios which are possibly responsible for the occurrence of bentonite cavities in diaphragm walls. These scenarios are based on execution methods, flow resistance, particular circumstances and the Vijzelgracht situation. This paragraph contains the conclusions regarding these scenarios and scenarios urging for additional research.

- Cleaning the joint surface of diaphragm panels could contribute to water tightness of the joints in diaphragm panels. This hypothesis is supported by the fact that quality of diaphragm walls at Ceintuurbaan, where joint surfaces were cleaned, were significantly better compared to Rokin and Vijzelgracht, without joint surface cleaning [Deltares, 2008]. Apart from remaining filter cake from the joint, the cleaning procedure is also shearing the bentonite suspension and consequently reducing thixotropic properties before regeneration/replacement of support fluid which facilitates the cleaning of bentonite suspension in the next production step.

- Insufficient cleaning of support fluid might enable viscous bentonite suspension to stick to the sides of the trench. This viscous material on the joint surface of the trench could be easily concreted in between two panels; a bentonite cavity is created. This scenario is extra important if the previous cleaning procedure (joint surfaces) is omitted.

- From literature it is known that flow behavior of bentonite suspension and concrete in the panel with a reinforcement cage and one or more tremie pipes can contribute to the occurrence of bentonite cavities:
  - A highly dense reinforcement cage decreases flow resistance in between concrete and bentonite suspension as well as the flow resistance of these materials relative to the surfaces of the panel and the reinforcement cage.
  - Concreting speed, the rate of the rising concrete front, contributes to the flow resistance in between both materials; higher concreting speed results in a larger difference in flow resistance and therefore minimizing the risk to bentonite cavities.
  - Contaminated or externally influenced bentonite suspension can increase the risk of bentonite cavities due to changed rheological properties. The possible external influences urge for additional research in order to determine the significance of this scenario. Especially the influence of cementitious material and softmix on bentonite suspensions urges additional research since half of the diaphragm wall at Vijzelgracht was constructed in softmix.
  - Time dependent viscosity of concrete during the pouring of concrete is also important in understanding the behavior of different charges in the panel. In fact bentonite suspension and concrete with different viscosity parameters can be present at the same time making the understanding of this process more difficult.

- Time in between testing of support fluid and concreting of the diaphragm wall could be significant since viscosity changes of bentonite suspension are time dependent.
Part III: Laboratory experiments
7 Bentonite filter cake formation

Literature study, experts’ interviews and case studies have provided input for laboratory experiments in order to gain more understanding in physical processes of bentonite filter cake formation. The build-up rate of bentonite filter cake and the progressively changing properties of this filter cake are parameters of interest, especially for future experiments with erosion of this filter cake either by replacement/regeneration of bentonite suspension as well as during the concreting process. This chapter comprises of experiments regarding the formation of filter cake and measurement techniques.

7.1 Aim of the test

For this test different measurement techniques are applied in order to obtain insight and experience with bentonite filtration cake formation, monitoring and measurement techniques. For future experiments adequate measurement techniques are needed and therefore several methods are tested. This gives an opportunity to study the bentonite cake formation in time. Filter cake thickness and density of the bentonite cake are properties to be monitored, measured and calculated.

7.2 Test set-up

The process of bentonite cake formation takes place within a vertical Plexiglas cylinder with a drainage layer, a sand layer and bentonite suspension. At the top of the tube a removable flange is installed in order to pressurize the bentonite suspension once it has been applied into the cylinder. Furthermore a pressure meter (manometer) is installed to be able to measure the pressure in the air bubble and another rubber gasket is installed to measure the pressure with hydrostatic water pressure. The flange at the bottom of the tube is connected with a reservoir with can simulate the groundwater level in the sand layer. This reservoir is equipped with an overflow to measure filtrate water. In between the sand and the gravel drainage layer a circular disk connected to a pole which stops...
just below the top flange of the cylinder is installed prior to the placement of sand. This circular disk is provided with holes and a filter material enabling filtrate water to flow to the drain layer and prevent sand particles to flow with the filter water. On top of this disk an oversized rubber slab is placed to ensure the connection between the Plexiglas tube and the circular disk. A schematization of this test setup is depicted in figure 60. Cake thickness Monitoring development of bentonite filtration cake thickness over time, different measurement techniques have been used in the past. Filtrate water and cake thickness are related since the amount of filtrate water is directly related to the amount of clay particles deposited at the sand/bentonite interface. Monitoring the amount of filtrate water during the filter cake formation provides information on the increasing cake thickness, decreasing permeability and decreasing filter cake growth. Tien [2006] describes different techniques to measure cake thickness; method of Murase et al., a method based on pressure measurements, non/intrusive methods of cake thickness measurement. Furthermore visual observation of cake growth through transparent test cells is a method, but often difficult since the demarcation of the cake and the suspension is not clear.

- Method of Murase et al. is based on a sudden change in filtration rate due to a reduction of filtration area. This method requires the placement of a disk (an inserted plate) with a sharp central hole within an experimental cell at a specified height (or distance from the filter medium) [Tien, 2006]. When the cake thickness increases and reaches the circular disk, the sudden change in filtration rate indicates that the filtration area is reduced when reaching the disk. The cake thickness is now equal to the difference between the circular disk and the filter medium.
- The method based on pressure measurements is performed by measuring the pressure at different levels in between the liquid phase and the filter medium, respectively bentonite suspension and sand, one should be able to determine the cake thickness. The pressure drop across the cake increases as the cake thickness increases and therefore the first pressure meter which doesn’t show a pressure drop indicates that the cake thickness is lower than that pressure meter.
- Non-intrusive methods of cake thickness measurements are based on acoustical or optical signals through or onto the cake are changing in amplitude or intensity as the cake thickness increases. These measurements need to be calibrated first and require for different suspensions different calibrations. In 1991 Takahashi et al. developed a method of determining cake thickness based on ultrasonic measurements [Tien, 2006].
- Visual observation method of cake thickness measurement is based on the density changes in between the filter medium and the liquid phase. Bentonite cake consists of hydrated bentonite particles stacked in a dense packing in relation with the bentonite suspension there is a contrast in between both phases. With an ordinary digital camera and a calculation program (e.g. Matlab or MathCAD) this can be quantified and the amount of pixels can be related to the cake thickness.

7.3 Electrokinetic phenomena

During bentonite filter cake formation electro kinetic phenomena are involved. Electrokinetic phenomena occur when two phases move with respect to each other while the interface is an electrical double layer [van Olphen, 1963]. Electric potential difference in between the slurry and the saturated sand is caused by the differential pressure in between the sand layer and the bentonite suspension. The pressure drop in the developing bentonite cake causes the electric potential due to the streaming potential. This also works the other way around: applying an additional direct current potential across electrodes inserted in the bentonite suspension and in the saturated sand a filter cake will form or cause its destruction. The physicochemical phenomena responsible for this behaviour are electro-osmotic flow and electrophoresis [Nash, 1974]. According to Xanthakos [1979] the filter cake sometimes acts as a membrane which is selectively permeable to anions or cations and thereby causes an electric potential difference. According to Veder (1961) an electric potential difference in between the bentonite
suspension and the saturated sand could be measured and this phenomenon can be used for the monitoring cake thickness and density. There are four electro kinetic phenomena which could be involved during the formation of bentonite filter cake, streaming potential, sedimentation potential, electro-osmotic flow and electrophoresis.

7.3.1 Helmholtz-Smoluchowski theory

Theoretical electrokinetic phenomena were first described by Helmholtz in 1879 and later refined by Smoluchowski in 1914 and still widely used. The streaming potential $V_s$ under influence of the pressure difference applied across the bentonite samples of which the pores carry an electrical double layer is given by:

$$V_s = \frac{\varepsilon_0 \varepsilon \zeta \Delta H}{\eta K_p}$$

where $\varepsilon_0$ is the permittivity of a vacuum, $\varepsilon$ is the relative dielectric constant of the medium, $\zeta$ is the zeta potential, $\Delta H$ is the pressure difference across the sample, $\eta$ is the dynamic viscosity, and $K_p$ is the specific conductivity of the suspension. The relative dielectric constant of water, the medium of bentonite suspension is 80.10 at 20 °C, whereas the permittivity of a vacuum is defined by

$$\varepsilon_0 = \frac{1}{\mu_0 c_0^2}$$

where $\mu_0$ is magnetic constant or often called vacuum permeability, defined at $4\pi \times 10^{-7}$ Hm$^{-1}$ and $c_0$ is the speed of light in vacuum defined at 299792458ms$^{-1}$. From this it follows that $\varepsilon_0$ also has a defined value approximated by $\varepsilon_0 = 2.985418\ldots \times 10^{-12}$ Cm$^{-1}$.

7.3.2 Streaming potential

Water flowing through a soil, or any other porous medium, due to a hydraulic head difference causes double layer charges from the soil, or in this case the bentonite cake, to be displaced in the direction of the flow. This results in an electrical potential difference proportional to the hydraulic flow rate which is called streaming potential. Streaming potentials between opposite ends of the soil mass can be as large as several tens of millivolts [Mitchell, 1976].
7.3.3 Sedimentation potential

The movement of charged bentonite particles relative to a solution due to gravitational forces lead to generation of potential. This potential is caused by the viscous drag of water that retards the movement of the diffuse layer cations relative to that of the particles [Mitchell, 1976]. The viscous flow around a bentonite particle drags ions of the diffuse layer away from the particle and thereby causes a slight displacement between the surface charge and the electric charge of the diffuse layer. The bentonite particles are gaining a dipole moment which generates an electric field over the sedimentation height which is called sedimentation potential.

7.3.4 Electro-osmosis

If an electric potential is applied across a wet soil mass, cations are attracted to the cathode and anions to the anode. There is an excess of cations in the system to neutralize the net negative charge on the soil particles. As these cations migrate to the cathode, they drag water with them, causing water movement towards the cathode as shown in figure 64. The anions also drag water with them as they migrate to the anode. This flow is usually much less than the flow to the cathode; therefore a net flow to the cathode results [Mitchell, 1976].

7.3.5 Electrophoresis

If a direct current is applied across a colloidal suspension, charged particles are attracted electrostatically to one of the electrodes and repelled from the other. Negatively charges clay particles move towards the anode as shown in figure 65. This is called electrophoresis. Electrophoresis involves direct particle transport through water; electro osmosis involved water transport through a continuous soil particle network [Mitchell, 1976].

7.3.6 Zeta potential

Under the influence of the physical Van der Waals forces bentonite particles stick together, causing a coagulation of the individual particles in water. This Van der Waals force is an attractive or a repulsive force between dipole molecules (or between parts of the same molecule) and are relatively weak compared to chemical bonds. Besides Van der Waals forces the particles dispersed in water are electrically charged. Similar particles have the same charge and therefore have repulsive forces. If the range of these forces is larger than the amount of Van der Waals forces the coagulation of particles is impossible. The amount and the range of the electrical charge is called the zeta potential and depends on the amount of interchanged atoms in the crystalline structure.
In colloidal systems the electro kinetic potential or zeta potential plays an important role. From a theoretical viewpoint the zeta potential is an electric potential in the interfacial double layer or electrical double layer (EDL). This potential difference is formed in between the dispersed bentonite and the stationary layer of water attached to this dispersed particle. The range of the electrical surface charge depends on the salinity of the suspension and the valence of the cation (calcium or sodium). A high salinity and a high valence of the cation reduce the range of the electrical forces. A low range and a low surface charge results in a low zeta-potential. The zeta potential level determines the stability behaviour of the colloidal bentonite suspension, at low zeta potentials coagulation or flocculation of the colloid occurs rapidly. Zeta potential higher (positive or negative) than 25 mV are free from coagulation or flocculation [CUR 189]. The zeta potential of CEBO Trenchcontrol bentonite was determined using the Malvern Zetasizer defined for different conductivities of bentonite suspension in appendix 4 and summarized in table 9.

<table>
<thead>
<tr>
<th>Sample [#]</th>
<th>KCl molarity [mM]</th>
<th>Zeta Potential [mV]</th>
<th>Error [mV]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0,0</td>
<td>-15,2</td>
<td>±0,5</td>
</tr>
<tr>
<td>2</td>
<td>1,5</td>
<td>-16,6</td>
<td>±0,6</td>
</tr>
<tr>
<td>3</td>
<td>3,0</td>
<td>-16,0</td>
<td>±0,4</td>
</tr>
<tr>
<td>4</td>
<td>15</td>
<td>-22,7</td>
<td>±0,5</td>
</tr>
<tr>
<td>5</td>
<td>30</td>
<td>-30,7</td>
<td>±1,8</td>
</tr>
</tbody>
</table>

Table 9: zeta potential for different salinities

7.4 Density

Density measurement is performed after the bentonite filtration cake has been formed on the sand layer in the Plexiglas cylinder, by progressively removing the sample using the sample retraction tool by the CPT apparatus. On the retraction tool the filter medium sand with on top bentonite filter cake and finally bentonite suspension can be removed stepwise from the Plexiglas tube. Bentonite suspension is removed in large steps progressively, measuring each charge of suspension removed weighed, and the retraction height is denoted. Closer to the bentonite filter cake the step size is decreased in order to obtain a higher
measurement resolution of the bentonite cake itself. These measurements provide a density distribution over the sample, from bentonite suspension to the filter medium.
7.5 Measurement equipment

Filtrate water measurement equipment is consisting of an overflow and a measuring cylinder of 1000 ml. By denoting the amount of filtrate water several times a day the total amount of filtrate water in time is constructed. The amount of filtrated water lost by evaporation is neglected because these values are not significant for these measurements.

The visual observation equipment consists of a digital camera connected to a computer with remote capture software. This software enables one to make pictures with a preselected interval. Two halogen light sources make sure the area of interest is luminous and no brightness drop over the area where filter cake is expected.

For the first three tests potential measurements have been performed using a laboratory multimeter (Philips, PM 2534 system multimeter) and an operational amplifier (TL074CN). This operational amplifier (OpAmp) increases the impedance of the measurements and thereby disables leaking electrons from the multimeter to influence the process. To power the operational amplifier a laboratory power supply (Delta Electronika Dual Power Supply E018-0.6D) is used provide +15/0/-15 V, which was also used to provide a solid ‘ground for the reference electrode’. A schematization of the electric circuit is shown in figure 68.

During the last test the measurements were performed in between the measurement electrodes relative to the reference electrode instead of measuring in between the reference and measurement electrodes. There was also compensated for a 50 Hz interference originating from environmental electrical using a differential amplifier (DiffAmp) in combination with two OpAmps according to the schematization in figure 69.
Bentonite cake density is measured by progressively removing the retraction tool, determining the removed height and the weight of the material in the reception vessel (figure 71). The sample is retracted using a CPT apparatus which is connected with a rod to the sample retraction tool. By rotating the crank, using the lower gear setting of the CPT apparatus, a ‘click’ from the safety mechanism indicates a retraction of 1.8mm. During the removal of the bentonite suspension the density measurement is performed every 10 or 20 ‘clicks’ while the amount of clicks is decreased for density measurements closer to bentonite cake. Measurements in the bentonite cake are performed using 5 or 2 clicks in order to increase resolution of the measurements. Material on the surface of the tube is removed by a steel ruler making the surface of the material equal with the surface of the tube each time a measurement is performed. During the retraction of the bentonite cake shear strength measurements were performed using the Vane test (figure 72).

**7.6 Results**

In total four tests have been performed in which the filter cake formation is monitored by visual observation and filtrate water measurement and electric potentials are measured at several moments during the test. The bentonite used for this test is CEBO Trenchcontrol from two different batches. For the first three tests bentonite from the first batch was used with a bentonite to water ratio of 3% and for the last test a new batch with a ratio of 4% was used. Bentonite from the first test was also used during the second test, while for the first, third and fourth test freshly made bentonite was used. The pressure gradient was for all the tests the same at approximately 20 kPa and slightly decreasing over time except for the last test. A short summary of the test is given in table 10.
<table>
<thead>
<tr>
<th>Test</th>
<th>Time [hours]</th>
<th>Actual cake thickness [mm]</th>
<th>Cake thickness from image processing [mm]</th>
<th>Filtrate water [ml]</th>
<th>Pressure [kPa]</th>
<th>Bentonite</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>42</td>
<td>8</td>
<td>7.6</td>
<td>-</td>
<td>±20</td>
<td>Fresh 3%</td>
</tr>
<tr>
<td>2</td>
<td>68</td>
<td>12-14</td>
<td>14</td>
<td>815</td>
<td>±20</td>
<td>Reused 3%</td>
</tr>
<tr>
<td>3</td>
<td>503</td>
<td>15*/17**</td>
<td>30.4</td>
<td>2450</td>
<td>±20</td>
<td>Fresh 3%</td>
</tr>
<tr>
<td>4</td>
<td>227</td>
<td>-</td>
<td>-</td>
<td>1270</td>
<td>±20</td>
<td>Fresh 4%</td>
</tr>
</tbody>
</table>

*Table 10: test properties (* actual bentonite filter cake, ** high density bentonite slurry*)

### 7.6.1 Image analysis

Image analysis of bentonite filter cake formation using colour intensity over a period of time provides knowledge on the rate of cake growth and cake density. Bentonite suspension contains very fine particles dispersed in water and therefore this suspension has a colour. Increasing concentrations of bentonite suspensions have a higher density and therefore have a different colour. Bentonite filter cake is developing on a filter medium, the density (amount of bentonite particles per unit volume) increases with a change of colour or specific colour intensity. During filter cake formation pictures are taken from the interface of the filter medium (sand) and the bentonite suspension, where this filter cake is formed.

Analyzing these images for three basic colours; red, green and blue (RGB) provides different results which are depicted in figure 73, figure 74 and figure 75 respectively. These graphs, from left to right, provide different materials; filter medium (sand), interface (bentonite cake) and bentonite suspension due, to different colour intensities. Over time, in these graphs from 80 minutes to 56 hours after the application of bentonite suspension to the filter medium, intensities are changing due to filter cake formation. Intensity changes in the red light are more severe compared to changes in green and blue light intensity, which makes red colour intensity more suitable for analysis, appendix 10 contains all the colour intensity analysis.

![Figure 73: red light intensity changes at sand-bentonite interface in time (from 80 minutes to 56 hours)](image-url)
figure 74: green light intensity at sand-bentonite interface in time (from 80 minutes to 56 hours)

figure 75: blue light intensity at sand-bentonite interface in time (from 80 minutes to 56 hours)

figure 76: RGB light intensity at sand-bentonite interface in time (from 80 minutes to 56 hours)
7.6.2 Cake thickness

The parts of the graphs representing the sand layer are all overlapping during the time period, indicating that there is no significant change in this part of the image. A sudden increase of intensity indicates the position where bentonite filter cake build-up starts, whereas the local minimum of the individual lines indicates the transition between cake and suspension. This location of local minimum is progressively shifting over time, indicating filter cake growth for all individual colour intensities as well as the combined colour intensities (figure 76). After the first test the relation between the graph and the final bentonite cake thickness was checked by taking a sample from the actual cake and measuring the thickness (figure 77).

figure 77: cake thickness first test, approximately 8mm
figure 78: dark layer in between bentonite cake and suspension

7.6.3 Filter water

Filtrate water measurements in combination with filter cake thickness from the image analysis are plotted in figure 79. The measurements for the first test are not recorded and therefore not available in this graph. Both filter cake build-up and the amount of filtrate water regarding tests 2 and 3 are correlated to the square root of time and to each other. Both tests were performed under similar conditions. The first test which was carried out at lower differential pressure results in a lower rate of filter cake build-up.

figure 79: filter cake and filtrate water in time
Filtrate water and bentonite cake thickness are both related to the square root of time by the variation of a constant. The cake thickness \( h_c \) can be calculated with the amount of filtrate water \( V \) for bentonite support fluids when these constants \( n \) and \( m \) respectively are known. The slopes in figure 77 determine these constants and with the following relations both parameters can be expressed into each other:

\[
h_c = \sqrt{\frac{2k_c(1-n_v)(\gamma_v \epsilon_z - \gamma_v \epsilon_s)}{(n_v - n_c)\gamma_v}} \sqrt{t} = n \sqrt{t}
\]

\[
V = m \sqrt{t}
\]

\[
h_c = \frac{n}{m} V
\]

The higher \( n/m \) ratio of the first test indicates a lower filtrate value; on average more filter cake is formed with the same amount of filtrate water compared to the third test. Although both tests were performed under similar conditions this small deviation has occurred and could be related to the reuse of bentonite from the first test for the second test although this is not theoretically supported.

### 7.6.4 Density

Density of bentonite filtration cake over the sample height was determined by removing the sample progressively and weighing the mass of the bentonite cake removed from the test setup. While the density of bentonite suspension should be constant and only increase close to bentonite cake, measurements were fluctuating with a large bandwidth with impossible values. Densities recorded from tests 2 and 3 are depicted in figure 80 with an indication for the bentonite cake thickness relative to the sand layer and the actual density of bentonite suspension. An error analysis (appendix 5) showing this method to be inaccurate for the resolution of interest in density 1.02-1.30 gr/cm³ for the bentonite suspension and bentonite cake respectively. Although a direct density measurement of bentonite cake over the height of the sample is was not measured accurately using retraction method from the test setup, the average density at 1.25 gr/cm³ of bentonite cake was measured in the first test from the sample in figure 77.

From the image analysis this density change from high density sand particles to low density bentonite suspension is visible in the colour intensity of the image. The transition in between sand and suspension is a sharp increasing line until a certain maximum value, then drops to a minimum value (transition between solid cake and liquid bentonite) to finally increase to the intensity of bentonite suspension. The sudden drop of intensity is probably caused by dark particles observed during the tests (figure 78). Due to this interference the observed intensity cannot be correlated to the actual density of bentonite suspension. Furthermore the actual density of bentonite cake should be determined in order to conclude whether or not this method can be used to correlate density to colour intensity, apart from the dip caused by darker non bentonite particles.

<table>
<thead>
<tr>
<th>Test</th>
<th>m</th>
<th>n</th>
<th>n/m</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>14.09</td>
<td>0.22</td>
<td>0.0156</td>
</tr>
<tr>
<td>3</td>
<td>17.14</td>
<td>0.24</td>
<td>0.0140</td>
</tr>
</tbody>
</table>

Table 11: filtrate water and cake thickness parameters
7.6.5 Potential measurements

The potential measurements during the filter cake build-up should provide insight in the location of the interface in between bentonite filter cake and bentonite suspension due to the streaming potential, which is present over the height of the filter cake. For the first three tests the relation of electronic potential and filter cake thickness was not found. The first two tests resulted in a relatively stable measurement of potentials over time; each electrode provided more or less the same potential over time with an average bandwidth of 100 mV, while the third test had a capricious course in the beginning and stabilized later on. Electric potentials from the first three tests related to the distance of the top of the sand layer do not show a significant deviation in the filter cake area or in the suspension. At least not in the analysis made in appendix 9.

Results for the second test are depicted in figure 81 and figure 82 for the distance to the top of the sand layer and potentials in time for each electrode and respectively. The first graph indicates that after an establishing time the potentials are more or less constant over time. The bandwidth of these measurements is also quite constant while the distribution of the potential measurement doesn’t correlate with the distance to the sand layer. The second
The graph shows an increasing potential for each electrode in time, without any significant deviation in the area of filter cake growth.

Figure 82: Test 2; potential measurements in time

The last test with adapted measurement instruments, to eliminate environmental interference, and a different measurement method, measuring potential difference in between electrode did not provide any improvements. In contrary to the expectations measurements fluctuated more in comparison to the first three tests, graphs depicted in figure 83 and figure 84.

Electric potential in between the first two electrodes, situated in the sand layer, show an increasing electric potentials while the other electrodes are not significantly changing in time and relative to each other. During this experiment the water reservoir providing the pressure on the bentonite suspension is filled several times during the test. This correlates with the three points (at 2400, 6500 and 9200 minutes after the start of the test) where nearly all individual electrodes indicate an abrupt change.

Figure 83: Test 4; potential measurements for each electrode with a certain distance to the sand layer
7.7 Conclusions

For this experiment different objectives were set; the first objective was to find out if this test setup was suitable to make bentonite filter cake. The second objective was to test various measurement techniques to determine density and rate of filter cake build up.

- Although some recommendations can be made for the test setup bentonite filter cake was produced immediately after the application of bentonite suspension on the filter medium.
- Build up of bentonite filter cake can be seen with the naked eye in the first hours of filtration, while the transition in between solid and liquid bentonite disappears after some hours.
- This transition of solid and liquid phase was observed due to dark colored particles settling down on top of the filter cake.

General conclusions regarding the measurement techniques of image analysis, cake thickness, density analysis, filtrate water and electric potential measurements are provided:

- Image analysis from experiments in the Plexiglas cylinder focused on the rate of filter cake build up, density and correlation to be able to correlate this with actual bentonite cake thickness, filtrate water and density measurements.
  - Cake thickness measurement during filter cake formation using image analysis has proven its value because the cake thickness cannot be observed with the human eye after several hours. Using image analysis the transition in between the solid and liquid phase of bentonite can be used accurately up to 3 days.
  - Density correlation from the image analysis was not possible due to interference with dark colored particles in the suspension settling down in the interface between solid bentonite cake and liquid bentonite suspension, and thereby significantly influencing the color intensity over the whole spectrum. This effect seems to diminish over time, but also the visible transition of solid and liquid bentonite in the image analysis. Analyzing the images using another method (unknown at this point in time) might provide the parameter of interest. Although this test setup was vertical, this interference might not occur if bentonite filter cake is build-up in the horizontal direction as this occurs in the actual situation construction a diaphragm wall.

- Density measurement of the actual bentonite cake using the retraction tool and progressively removing the sample, calculating density from the mass and volume of the removed bentonite has proven to be inaccurate (see appendixfigure 13&14). During density measurements observations on the consistency
and the density of both the liquid and the solid phase indicate a smooth transition from liquid to solid phase and also in the solid phase, a transition in density was visually observed. Since the measurements require high accuracy in a small area of density varying from 1.02 gr/cm$^3$ for bentonite suspension to 1.30 gr/cm$^3$ for dense bentonite cake, this method is susceptible to measurement errors and therefore not able to show this relation.

- Filtrate water measurement in combination with filter cake thickness from image analysis confirmed the relation of filtrate water and cake thickness to the square root of time. Although this relation was clearly observed, the parameters for several tests under the same conditions were slightly different, probably due to differences in porosity of the filter medium.

- Electric potential measurements for the first two tests showed similar results; gradually increasing early in the process and then stabilized during the rest of the test. The third test, on a larger time scale, showed a capricious development in electric potential measurement during the first period of time, where tests 1 and 2 stabilized already. After quite some time electric potentials also stabilized but for a negative value. These tests didn’t indicate different phases of bentonite in potential measurements for individual electrodes. The fourth and last test with adapted measurement instruments compensating for environmental interference didn’t provide better results.

  - In comparison to Veder’s tests (1961) these electric potential measurements were performed in a closed cylinder with pressurized bentonite suspension forming a horizontal cake, while Veder’s tests were performed in an open container forming a vertical bentonite cake. Although the direction and pressures are different, principles are similar to tests in literature.
  - The only big difference is the type of electrodes, which are most likely the cause of the failure of these measurements. Veder used long brass rods (20-30 mm with a diameter of 3-5mm) instead of stainless steel points to measure electric potentials. The surface area of the rods from Veder’s test is 50-1300 larger than the surface area of the stainless steel points used in this research. In addition to the surface area the long brass rods penetrate deeply into the bentonite cake, whereas the stainless steel points are just in contact with the outer surface of bentonite cake. In literature similar tests were performed using gold-plated large surface area electrodes. The electric conductivity or resistivity of these materials is also playing a role. I.e. for gold this is 22.1 nΩm, for brass this is 3 times higher (64 nΩm) and for stainless steel ten times higher than for brass (720 nΩm).

The experiments in this chapter have been performed in order to determine measurement techniques for bentonite cake thickness and density. Successful and reliable measurement techniques regarding the bentonite cake thickness are limited to image analysis and filtrate water measurement. Filter cake density measurements have proven to be inaccurate, and due to interference with dark colored particles within the bentonite suspension no correlation with image analysis can be found. Electro kinetic phenomena have the potential to be used as a measurement technique, but should be adapted in order to be working for this test setup.
8 Bentonite softmix interaction

8.1 Aim of the test

Bentonite softmix interaction research is performed with three different methods; analytical analysis from theory, physical modelling and core boring material analysis, all focussed on the interaction of cement and hydrated lime (calcium hydroxide) with bentonite suspension. The analytical method is based on the possible amount of hydrated lime originating from cement and fly ash in contact with bentonite slurry. Physical modelling method this process is simulated and bentonite is tested for the degree of interaction and influence on the properties. The material analysis on the core borings of the softmix columns is performed to investigate the physical evidence of the interaction and a composition analysis of the samples.

8.2 Theory

Composition and the degree of hydration of the softmix determine degree of interaction with the bentonite suspension. Hydration processes of the various materials and the possible interactions are elaborated in this paragraph in order to determine the amount of influence caused by the cement. The softmix is basically a stabilised sand mixture consisting of sand, cement, fly ash and water (composition stated in table 12). Cement and fly ash in combination with water create a ‘glue’, binding the granular sandy material to a solid softmix. This is the same principle as for ordinary concrete only with less glue resulting in a softer solid material.

<table>
<thead>
<tr>
<th>Softmix composition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material</td>
</tr>
<tr>
<td>Cement CEM III B42,5</td>
</tr>
<tr>
<td>Fly ash</td>
</tr>
<tr>
<td>Sand 0-0,1mm</td>
</tr>
<tr>
<td>Sand 0,1-0,4mm</td>
</tr>
<tr>
<td>Water</td>
</tr>
<tr>
<td>Total</td>
</tr>
<tr>
<td>Water-Cement Ratio</td>
</tr>
<tr>
<td>Water-Binder Ratio</td>
</tr>
</tbody>
</table>

*table 12: composition and properties softmix [de Kort, 2009]*

8.2.1 Cement

The cement is a blast furnace slag cement (BFSC), indicated by CEM III which means it is a mixture of Ordinary Portland Cement (OPC) clinker material and blast furnace slag (BFS). Type B42,5 gives the composition and the strength respectively. The strength class 42,5 is usually applied when a compressive strength of 30 N/mm² within 28 days is requested. The actual strength of the mixture is strongly dependent on the water-cement or water-binding ratio (WCR and WBR respectively) and the composition of aggregates. The composition is based on least 2/3 (BFS) and 1/3 (OPC) clinker. The cement clinker material in OPC is composed of many oxides, with major oxides of calcium oxide (CaO) or lime, silicon dioxide or silica (SiO₂) and aluminium oxide (Al₂O₃). These oxides exist in the form of four minerals; Alite, Belite, Aluminate and Ferrite. Besides cement clinker 5% gypsum is added to obtain OPC to control the control the reaction. The main components and of OPC are stated in table 13.
<table>
<thead>
<tr>
<th>OPC component</th>
<th>% mass</th>
<th>Chemical formula</th>
<th>Cement chemist notation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alite (tricalcium silicate)</td>
<td>50%</td>
<td>3CaO·SiO₂</td>
<td>C₃S</td>
</tr>
<tr>
<td>Balite (dicalcium silicate)</td>
<td>25%</td>
<td>2CaO·SiO₂</td>
<td>C₂S</td>
</tr>
<tr>
<td>Aluminate (tricalcium aluminate)</td>
<td>10%</td>
<td>3CaO·Al₂O₃</td>
<td>C₃A</td>
</tr>
<tr>
<td>Ferrite (tetracalcium aluminoferrite)</td>
<td>10%</td>
<td>4CaO·Al₂O₃·Fe₂O₃</td>
<td>C₄AF</td>
</tr>
<tr>
<td>Gypsum (calcium sulfate dehydrate)</td>
<td>5%</td>
<td>CaSO₄·2H₂O</td>
<td>C₃S·H₂</td>
</tr>
</tbody>
</table>

Table 13: chemical composition of OPC

BFS is a non-metallic co-product in the production of iron. It consists primarily of silicates, aluminosilicates, and calcium-alumina-silicates [TFHRC, 2010], composed of the same oxides as OPC. The BFS particles are latent hydraulic, which means activation is necessary before hydration occurs. The activation comes from a reaction product from OPC and therefore both are usually combined in BFSC.

Fly ash is a finely mineral residue resulting from the combustion of powdered coal in electric generating plants. The particles are often perfect spheres with a ranging in size from 0.5µm to 100µm. The chemical composition of fly ash shows a wide diversity, with mainly silicon dioxide, aluminium oxide, iron oxide and calcium oxide.

These three cementitious materials, cement clinker, BFS and fly ash, are plotted in the phase diagram in figure 85 and show and the ratios of the main oxides.

Figure 85: phase diagram of three cementitious materials

8.2.2 Hydration

The hydration of cement clinker is mainly responsible for the strength development of softmix, whereas BFS and fly ash are contributing to the development of strength through a so called pozzolanic reaction with some of the components of the hydrated cement clinker. These three components of softmix, blast furnace slag, fly ash and cement, are developing strength in this mixture under certain conditions. These reactions can occur only if there is enough water for hydration.

The high water-cement ratio could results in a high hydration degree of cement particles. Fly ash on the other hand will also be hydrated and therefore the water-binding ratio becomes 0.8 approximately ten times lower. These numbers are correct if the sand added to the mixture is dry and the softmix is applied in dry conditions. The soft mix is placed in the ground with a high groundwater level and therefore additional water could be attracted during the hydration process of the soft mix if necessary.
Once cement in mixed with water, particles start to dissolve and chemical reactions start. The reactions are leading to the formation of a gel of Calcium Silicate Hydrates (CSH), the ‘glue’ responsible for the hardening of cement. The development of this glue from cement is due to hydration of the minerals stated in table 13. The main reactions of the individual cement compound were described clearly by Lea [1970] and Older [1998]:

1. The C₃S and C₅S react with water to produce calcium silicate hydrate (CSH) and calcium hydroxide (CH), and these reactions can be written as:
   \[ 2C₃S + 6H = C₃S₂H₆ + 3CH \]
   \[ 2C₅S + 4H = C₅S₂H₆ + CH \]
2. The C₃A not only reacts with water, but also reacts with gypsum (CSH₂) to produce ettringite (AFt).
   \[ C₃A + 3CSH₂ + 26H = C₃A₂S₈H₃₂ \]
   Once the gypsum is used up, the AFt becomes unstable and reacts with remaining C₃A to produce monosulfate aluminate hydrate crystals (AFm):
   \[ 2C₃A + C₃A₂S₈H₃₂ + 4H = 3C₃A₂S₈H₁₂ \]
3. Like C₃A, C₅AF reacts with gypsum water to form ettringite, lime and alumina hydroxides:
   \[ CaAF + 3CSH₂ + 30H = Ca₄(A,F)SH₁₈ + (A,F)H₁₈ + CH \]
   The complete product is garnet, which is produced by the further reaction of C₄AF with AFt:
   \[ Ca₄AF + Ca₄(A,F)SH₁₈ + 2CH + 23H = 3Ca₄(A,F)SH₁₈ + (A,F)H₃ \]

8.2.3 Calciumhydroxide

The important reaction products formed with these reactions are CSH and for the interaction with bentonite CH. The formation of hydration products is a process strongly depending on the time elapsed after the exposure of cement to water and it may not proceed to 100% completion. In an early stage of hydration <2 hours only CH and Ettringite are formed and the porosity of the cementitious material is decreased. After 2 hours the formation of CSH starts and one day later the other elements C(A,F)H and Monosulfate are formed. In figure 86 the hydration process of cement clinker is given for a period from exposure up to 90 days.

[figure 86: hydration process [Kurtis, 2009]]

Given the fact the hardening and the hydration process of cementitious material is long lasting one can expect the hardened cement paste contains some unhydrated residues and mainly CSH, CH and Monosulphate (figure 86). According to this table the hardened cement pastes consists for 20 % vol. CH which seems to be a significant amount, this is for hardened cement paste. The composition of softmix, stated in table 12, consists of 1,1% mass of cement which is consisting of 66% blast furnace slag and 34% cement clinker. Therefore the
maximum amount of CH generated by the hydration of cement clinker material is roughly estimated at 1.5 kg/m$^3$, 0.7% of the total mass of the softmix.

8.2.4 Reduction

There are also other chemical reactions influencing the amount of CH in the softmix due to the presence of blast furnace slag and fly ash. Blast furnace slag and fly ash increase the amount of CSH but decrease the amount of CH by pozzolanic reactions with CH generated by cement clinker. The chemical reaction from fly ash takes place under certain conditions. The reaction starts after at least 28 days and only takes place in high alkaline (pH $>$ 12/13) environment [reference needed and according to Bumrongjaroen [2009] this is chemically denoted as:

$$H_4SiO_4 + 3Ca(OH)_2 + 3H_2O \rightarrow (CaO)_3SiO_2\cdot(H_2O)_8$$

$$H_4SiO_4 + 3CH + 3H \rightarrow C_3SH_8$$

The pozzolanic reactions from fly ash and blast furnace slag are reducing the amount of CH and therefore the minimum amount of CH in the softmix is difficult to determine.

8.2.5 Conclusion

The composition of softmix makes it difficult to determine the amount of CH generated during the hydration and curing time of the material. The amount of CH generated by cement can be calculated quite easily using earlier studies and hydration equations, while this amount could be reduced by the presence of fly ash and blast furnace slag. The result of this amount CH to the bentonite suspension is another problem elaborated further on in chapter 9.

8.3 Core boring analysis

Two softmix core borings have been taken from Rokin station Amsterdam at the level of the first sand layer (figure 87) up to the concrete diaphragm wall. The cores are taken from two different softmix columns with a length of about 50 cm and a diameter of approximately 12 cm. The presence of calcium hydroxide or unhydrated cement particles indicates that bentonite exposed to this material could be influence during the construction of diaphragm walls in the soft mix.

8.3.1 Literature

From literature several methods are known to investigate cementitious materials for unhydrated cement and chemical composition. According to Sawaki et al. [2007] X-ray analysis, scanning electron microscopy and analysis of acid-soluble parts are basic and main methods. An analysis of the acid soluble parts is used for cementitious materials to determine the amount of cementitious material in a sample. This test is not suitable because CH and fly ash (as the pozzolanic reaction progresses) are both acid-soluble and therefore this test is not performed.
Powder X-ray diffraction can detect the hydrates and unhydrated cement in the sample. With this technique a sample is exposed to X-rays and the diffraction of the radiation as a function of the diffraction angle is measured. This generates high and low peaks over a certain spectrum which can be linked to the specific fingerprint of a specific material. By comparing the diffraction results of the sample with the diffraction fingerprints, the materials in the sample a semi-quantitative of analysis can be made. An important issue is the crystalline structure from materials of interest in the sample. Low crystalline structures are difficult to detect. Calcium hydroxide is a crystalline material and therefore can be detected with X-ray diffraction. According to Sawaki et al. (2007) the main component of hydrated cement CSH cannot be detected by this method, however other hydrates from cement can:

“Although calcium silicate hydrates are the main hydrate of cement, they cannot be detected fully by XRD because of their low crystallinity. The family of hydrates of calcium aluminate such as ettringite (3CaO Al\(_2\)O\(_3\) 3CaSO\(_4\), 32H\(_2\)O), monosulfate hydrate (3CaO Al\(_2\)O\(_3\), 12H\(_2\)O) and Friedel’s salt (3CaO Al\(_2\)O\(_3\), CaCl\(_2\), 10H\(_2\)O) is good evidence for cementitious materials. They show the highest peak on the low-angle side where the overlap with the highest peak of aggregate mineral is less than the high-angle side.”

Images from scanning electron microscopy (SEM), especially backscattered electron images (BEI) are often used in the study of cementitious materials. BEI provides the compositional information of hardened material of the polished surface. BEI is an image visualized by the detecting backscattered electrons (BE) reflected from the surface of the sample. The larger the atomic number of the atom, the higher the intensity of (BE). Unhydrated cement appears bright on the BEI. To avoid the accumulation of electric charge on the surfaces of non-metallic samples, an environmental SEM (ESEM) is used in which the sample is placed in an internal chamber.

8.3.2 Samples

Samples from the core borings are analysed using X-ray diffraction and images from scanning electron microscopy (SEM). Samples are taken direct next to the softmix-bentonite interface, the softmix-soil interface and a reference sample in between. Because both cores have the same appearance one core is sampled at these positions and from the other core only one sample is extracted. The sample locations and cores are depicted in Error! Reference source not found.
The core borings are consisting of cementitious material which is hydrated somewhere in 2004 and therefore in theory all the cement could have become hydrated over time. XRD analysis to determine (un)hydrated cement is therefore more difficult and the operator requested additional samples of younger material which contains both hydrated and unhydrated cement. Two samples are obtained from the physical modelling experiments; one unhydrated sample (S4) with the exact composition of the softmix except for water and one +/- 15 day old sample (S4).

The picture in figure 89 shows the thickness of bentonite cake in between the concrete from the diaphragm wall on the left and the softmix on the right. The thickness of the bentonite layer is varying from 6 mm to more than 20 mm at some locations.

Another sample for the ESEM/BEI has been extracted at the interface of softmix-bentonite because there is an interaction visible (figure 90). In the bentonite cake on the face of the softmix, actually in between the soft mix and the concrete of the diaphragm wall, a cementitious layer of 1-2 mm is present. The samples for XRD and ESEM/BEI analysis are summarized in table 14.
<table>
<thead>
<tr>
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<th>Ref</th>
<th>Type</th>
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<th>Test</th>
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<tr>
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<td>Chunk</td>
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<tr>
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<td>Chunk</td>
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<td>Chunk</td>
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<td>ESEM/BEI</td>
</tr>
<tr>
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<tr>
<td>7</td>
<td>S1b</td>
<td>Powder</td>
<td>Powder middle core 1</td>
<td>XRD</td>
</tr>
<tr>
<td>8</td>
<td>S1c</td>
<td>Powder</td>
<td>Powder inside core 1</td>
<td>XRD</td>
</tr>
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</tr>
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<td>Hydrated softmix</td>
<td>XRD</td>
</tr>
<tr>
<td>11</td>
<td>S4</td>
<td>Granular</td>
<td>Unhydrated softmix</td>
<td>XRD</td>
</tr>
</tbody>
</table>

Table 14: XRD/ESEM-BSE samples

XRPD patterns were recorded in a Bragg-Brentano geometry in a Bruker D5005 diffractometer equipped with Huber incident-beam monochromator and Braun PSD detector. Data collection was carried out at room temperature using monochromatic Cu Kα1 radiation (\( \lambda = 0.154056\)nm) in the 2θ region between 5° and 90°, step size 0.038 degrees 2θ. All samples measured under identical conditions. The samples of about 20 milligrams were deposited on a Si <510> wafer and were rotated during measurement. Data evaluation was done with the Bruker program EVA.

8.3.3 Results

The results from the XRPD analysis show no signs of cementitious material; all samples mainly contain quartz (SiO₂) and mullite (Al₂Si₀.₆O₄.₈). The presence of quartz is not surprising because the softmix is mainly consisting of sand, while mullite indicates the presence of fly ash. According to Fraay [1990] this mineral is the main crystalline phase of fly ash. Furthermore sample S1a contains orthoclase (KAlSi₃O₈) and sample S4 cristobalite (SiO₂). The XRD results can be found in appendix 11 and in figure 91 the analysis of sample S1a is depicted.
The analysis of Sample S1a shows some very high (off chart) peaks which are not covered by the fingerprints of quartz, mullite and orthoclase. This is probably due to the random orientation of the crystalline structures in relation to the fingerprint of these materials and as a consequence of this have an increased intensity. There are also peaks that don’t match with the fingerprint of the three materials and some shared peaks. For example in the peaks at 10.5°, 22°, 28° and 45.5° on the 2θ scale are not matched by any fingerprint. The peak at 21° on the 2θ scale is shared by both the fingerprints of quartz and orthoclase. At 112.5° and 117.5° on the 2θ scale fingerprint peaks of quartz appear without a peak in the analysis of the material. The reason no cementitious material was found is probably due to the fact that these materials were below the detection limit of the apparatus.

The BE images clearly showed the microstructure of the material with mainly quartz, some fly ash (and the darker material which could be either epoxy or hardened cement paste. The structure and colour of quartz (A) and especially the spherical structure of fly ash (F) is easy recognisable. After an extensive search by an expert one partly hydrated (H) and partly unhydrated (U) cement particle was found in sample S1a. Three BE images in figure 92 are analysed and some particles determined (all BE images in appendix 8). CH was not present or not found in the samples.
The sample with a composition of softmix, bentonite and cementitious material turned out to be difficult to interpret. The transition between the cementitious layer and the other layers can clearly be seen on the images although the structure of these layers was quite similar, only the composition was different according to the local element analysis. This shows a quantitative analysis of the composition of the elements at different locations in this sample (table 15).
8.3.4 Conclusion

The conclusion for the XRD test results is that this method is not suitable for this type of cementitious material. There is just too much quarts and too less cementitious material to be detected with this method. Test results from the ESEM/BEI provide the evidence for unhydrated cement particles although the amount of material present in the samples is little. The fact that after more than five years still some unhydrated cement is present indicates that during the construction of diaphragm walls this unhydrated cement was also present and probably in larger quantities. The sample from the interface between the softmix and the concrete of the diaphragm wall shows some interaction but it isn’t clear whether this cementitious layer is developed due to the cementitious material from the soft mix or cementitious material from the diaphragm wall concrete. It also became clear that the softmix contained some organic material and particles other than sand such as pottery fragments.

8.4 Physical modelling

Softmix applied for the construction of the diaphragm walls for the North/Southline is forged and exposed to a bentonite suspension in order to determine the interaction. The amount of material exposed to the bentonite suspension is calculated for different scenario. The hydration degree of softmix is also varied in order to determine the curing time effect on the interaction.

8.4.1 Softmix preparation

Softmix was made according to the composition provided by the contractor responsible for the construction of the softmix piles (table 12). The composition of the binders in the softmix, blast furnace slag cement and fly ash are analysed (table 16) and plotted in the phase diagram (figure 93). The composition of blast furnace slag cement (CEM IIIB 42,5) in the phase diagram fits nicely in between the cement and blast furnace slag while the fly ash contains more silica than average fly ashes.

When the materials are properly mixed together the softmix is placed in 6 moulds of 4 litre each and stored in a acclimatised environment. Three of the six samples are submerged in water with a temperature of 20°C and the other three samples are submerged in water at 60°C in order to increase the hardening process.
8.4.2 Hydration

The hardening process of softmix in the first stage is in fact the hardening process of cement. The rate and degree of hydration depends on the temperature of the cementitious material (figure 94). After 45 days of hydration the samples are stored at room temperature and low humidity. Until now the three softmix samples have hydrated just over 80% and the other samples at an increased temperature have hydrated about 85%, which is comparable to 70 days of curing time for cementitious material at 20°C.

![Figure 93: Phase diagram of three cementitious materials](image)

![Figure 94: Simulated and measured degree of hydration for ordinary Portland cement (OPC)](image)

The time in between the application of softmix and the construction of diaphragm walls was at least three months and the temperature in the underground is fairly constant at about 10-12°C. The degree of hydration for the forged softmix is therefore comparable to the degree of hydration in the actual situation.
8.4.3 Surface area

The degree of interaction from softmix and bentonite suspension is time dependent and depending on the amount of softmix material in contact with the bentonite suspension. Excavating a trench for diaphragm wall construction can take up to several days and when a weekend is included the bentonite suspension is exposed to the softmix for quite some time. In case of the situation at Vijzelgracht station (table 3) the excavation of panel 70 had duration of in total 10 days and therefore the softmix bentonite interaction could have the same duration. Excavation dimensions of this panel, 1.2m thick, 5.1m wide and up to 44.5m length, provide a contact area of 12.6m$^2$ for each meter length. In excavated conditions a contact area of 21 cm$^2$ is exposed to one litre of bentonite suspension, while during the excavation of softmix the contact area is larger due to the high surface area of the loosened softmix. During the excavation of the trench the grab loosens the material in order to be able to bring a fully loaded grab to the surface. The maximum exposure during excavation is encountered when the material is loosened and the bentonite surrounds this material. Depending on the porosity of this situation the surface area for perfect identical spheres is between 0.50 and 0.25 for thinnest regular packing and for thickest regular pattern respectively, while a porosity of 0.4 fits a loose random packing. The surface area of the softmix material in contact with bentonite support fluid depends on the porosity and the diameter of the softmix material in case of spherical shapes. This can be calculated using formula:

$$C = \frac{2(w+t)}{1000wt} + \frac{3\pi r^2 (1-p)}{1000\pi r^3 p}$$

with contact surface in m$^2$/l (C), width of the trench (w), wall thickness (t), porosity (p) and sphere radius (r)

For several porosities and radii of softmix material in the contact area is plotted in figure 95. During the excavation of the trench softmix is crushed to small chucks which will partly be removed by the grab. The situation in which a layer of softmix chunks with a radius less than 10 cm stay overnight (or even during the weekend) in the trench exposed to the bentonite could actually occur. With a very conservative assumption of a porosity smaller than 0.3, the contact area of softmix and bentonite could be more than 1000cm$^2$/l. Due to practical reasons this upper bound was set to 1250 cm$^2$/l, the lower bound was already mentioned of an excavated trench the contact area could be 21cm$^2$.

![Figure 95: Contact area vs particle size](image-url)
8.4.4 Viscosity

Viscosity of bentonite suspension is changing due to chemical interaction with softmix, depending on the hydration rate of softmix, the exposed surface area and time of exposure. Flow charts of bentonite suspension with different surface areas and hydration degrees are depicted in figure 96, figure 97, figure 98 and figure 99 for different exposure times. Because bentonite suspension changes over time, the viscosity of a reference bentonite suspension is also indicated in each flow chart and the legend from figure 96 holds for the four flow charts.

Flow charts indicate that viscosity of bentonite suspension is not significantly influenced in 8 or 24 hours because viscosity changes are very small. After 96 hours a significant change in yield strength of bentonite suspension is detected for large surface areas of exposed softmix. The difference in hydration degree becomes visible for the viscosity of the tested samples and this influence is as expected higher with a lower hydration degree, this difference becomes larger with increasing time.
8.4.5 Cake thickness

On the softmix surfaces exposed to bentonite suspension a thin bentonite cake is formed. Unhydrated cementitious material attracts water and therefore the cake thickness develops on the exposed softmix surface. After 24 hours there was already layer of bentonite suspension adhering to the softmix surface while the thickness of this layer was not measured during the test visually no increasing thickness could be observed. At the end of the test, after more than 10 days the thickness was measured 5 mm (figure 100). Because a high surface area was required for the interaction test flat pieces of softmix, did not represent the actual situation with a softmix thickness of 40 cm adjacent to the trench of the diaphragm wall. If the thickness of softmix would be larger and the porosity of the softmix was high enough the unhydrated cementitious material could absorb more water from the bentonite suspension and therefore result in a thicker bentonite cake.

8.5 Conclusions

Bentonite softmix interaction is based on the chemical interaction of calciumhydroxide from cement in the softmix with bentonite suspension. The amount of calcium hydroxide is determined by the total amount of cement in the softmix minus the reduction induced by blast furnace slag and fly ash. The exact amount of calciumhydroxide cannot be calculated because the amount of hydrated cement is unknown and the reduction of calciumhydroxide is also unknown. Core borings from Rokin station and experiments performed in this chapter gained knowledge on bentonite-softmix interaction processes, parameters involved and methods to determine these parameters. This paragraph will elaborate the conclusions regarding these experiments.

- Core borings of softmix (present on the outside of the diaphragm wall) provided information on the composition of this material and the thickness of bentonite (5->20mm) in between softmix and concrete from the diaphragm wall.
- Permeability tests indicate a very low permeability for softmix which cannot facilitate enough filtration capacity in order to enable bentonite filtration cake formation on the softmix. Since there was bentonite cake on the surface of softmix, another mechanism must be responsible for this cake formation.
- The amount of unhydrated cement in the softmix is an indication for the reactivity of excavated softmix in contact with bentonite suspension. Powder XRD and image analysis from backscattered electrons images (BEI) in the environmental scanning electron microscope (ESEM) were used to determine the composition.
Powder XRD analysis couldn’t provide the necessary information on the amount of unhydrated cement content and the presence of calciumhydroxide, because of a detection limit lower than the amount of particles present.

From backscattered electrons images (BEI) in the environmental scanning electron microscope (ESEM) unhydrated cement particles were observed. This indicates at the time diaphragm walls were constructed in the softmix unhydrated cement particles were also present probably in a higher concentration.

The presence of unhydrated cement particles alone is a reason to believe bentonite suspension is influenced while in contact with softmix. Unhydrated cement particles produce calciumhydroxide during the hydration process while in contact with bentonite suspension.

Experiments with bentonite suspension in contact with softmix have indicated a significant change in rheological properties of the bentonite suspension. The change of rheological properties is mainly due to an increased yield point or shear stress at low shear rates, while the viscosity is slightly increasing.

Rheological properties of bentonite suspension are progressively changing over time. A longer exposure time results in a larger change in rheological properties.

The change in rheological properties increases more with higher surface area of exposed softmix to the bentonite suspension.

A lower hydration degree of softmix (79%) results in a larger change of rheological properties in relation to a higher hydration degree (83%). This means the reduction of calciumhydroxide by fly ash and blast furnace slag is already of influence.

After 96 hours the yield point of bentonite suspension exposed to the largest surface area has increased almost 4 times for the lower hydration degree.

During the experiment a bentonite cake was formed on the softmix surface. After the experiment was finished all softmix surfaces showed more or less the same cake thickness of approximately 5 mm. This indicates that a difference in fluid pressure is not the only possible cause for formation of bentonite cake.
9 Bentonite flocculation and aggregation

Rheological properties of a colloidal bentonite suspension are depending on many parameters, for instance the type of (calcium or sodium) bentonite, quality of the continuous water phase, mix ratio, age of the suspension and external influences. In fact rheological properties of bentonite suspension are determined by the particle association of the individual bentonite platelets in the continuous phase. These particle associations are influenced by the adjustment of conductivity of and alkalinity of the bentonite suspension. Depending on both properties and the type of bentonite used the particles associations in bentonite suspension can change significantly. Van Olphen [1963] was the first to describe these associations depicted in figure 101.

![Figure 101: Modes of particle associations in clay suspensions](image)

**9.1 Aim of the test**

The composition of bentonite suspensions in civil engineering is usually 3-6% dry bentonite particles in a cubic meter water. Depending on the type of bentonite and the composition the platelet association in the suspension is changing with in- or decreasing conductivity and alkalinity. In order to determine the sensitivity of Trenchcontrol, an often used bentonite for diaphragm wall construction, some tests are performed. Influence is determined by visual inspection (free water) and/or rheological properties. To gain more insight in the bentonite interaction with cementitious material and the environmental influences of conductivity and acidity/alkalinity experiments are executed. During these experiments the change in properties of bentonite suspension at different pH and at different conductivities. Furthermore the bentonite cement interaction is simulated by both pH and the presence of Ca$^{2+}$ cations.

Hypothesis:
- In- or decreasing pH ratio of a bentonite suspension will cause flocculation of the platelets and therefore an increase in viscosity.
- Increased pH ratio caused by NaOH has less influence on bentonite suspension comparable to the same pH ratio caused by Ca(OH)$_2$ due to the double valence of calcium ions.
- Increased conductivity will cause bentonite to flocculate and therefore increase viscosity and decreasing filter cake formation properties.
9.2 Method and procedure

Bentonite suspension in water was prepared using an activated sodium montmorillonite, a commercial product developed for diaphragm wall construction by CEBO under the brand name Trenchcontrol. The cation exchange capacity (CEC) of ‘dry’ bentonite powder (moisture content 9.9 % m/m) is approximately 100 Meq/100 g, determined by the Methylene blue adsorption test. The dry particle size analyses provided 99% of the material to be smaller than 150 µm for 99%.

Influence of pH and conductivity on the viscosity and stability was determined for a 4% (m/m) bentonite suspension in deionised water. Bentonite suspension was prepared prior to the viscosity measurements enabling the bentonite to hydrate for at least 4 hours. Initially a 4.33% (m/m) (or 5.2% (m/m)) bentonite suspension was prepared in order to add 50 ml (or 150 ml) (deionised water with or without pH/conductivity influencing) to a 600 (or 500 ml) bentonite suspension to obtain a 4 % (m/m) suspension.

Viscosity measurements were performed using a continuously rotating Fann viscometer with varying rotational speeds. The inner cylinder diameter of 1.7245 cm and an outer rotating cylinder diameter of 1.8415 cm at a temperature of 18°C and speeds of 600, 300, 200, 100, 60, 6 and 3 rpm. At each shear rate the shear stress was denoted after 60 seconds before changing to the next lower gear and shear rate. Conductivity and pH were measured using a WTW Cond 340i conductivity meter and a Metrohm 744 pH Meter respectively. Stability of a bentonite suspension cannot be measured and is therefore visually determined, by monitoring the amount of free water.

9.3 Sample preparation

Samples are adjusted to different pH values using different solutions of NaOH, Ca(OH)\textsubscript{2} and HNO\textsubscript{3} with different molarities (6, 3, 2 and 1M). Conductivity of bentonite suspensions is adjusted by the addition of NaCl solutions with different molarities (1, 0.5, 0.25, 0.1, 0.05 and 0.025M). Hydrated bentonite suspension was poured in a flask and 150 ml deionised water or one of the latter solutions was added to the flask and inherently shaken for 60 seconds. The suspension was then poured in a measurement cylinder of 250 ml and the remaining suspension (of approximately 400 ml minus the suspension left behind in the mixing flask) was poured into a coverable container. This container was used to perform viscosity measurements, by first mixing the suspension for about 1 minute at the highest shear rate (600 rpm). After performing the viscosity measurements pH and conductivity of suspension in this container were determined.
The pH adjustment with acid and alkaline solutions automatically increase the conductivity of the bentonite suspension, due to the free (cat)ions from the solution. As a consequence bentonite properties are changing due to sodium/calcium exchange depending on the amount of initial ratio Na/Ca of montmorillonite. On the other hand conductivity adjustment does not influence pH while calcium sodium interchanging could to the original bentonite suspension.

The pH range of the test samples is in between 1.8 and 13 with a high concentration of samples around the natural pH of bentonite suspension of 10. Conductivity ranges from the natural conductivity of bentonite suspension of 1300-1400 µScm⁻¹ to high values of more than 75 mScm⁻¹ (75000 µScm⁻¹). The ranges of pH and conductivity for different samples are depicted in figure 103. The influence of pH should be determined within the pH range between 6 and 12, because the increased conductivity plays a minor role in within this range. NaCl is not significantly affecting the pH value; hence influence of NaCl on viscosity of bentonite can be directly concluded from these samples. Properties and results of all samples are summarized in table 17.

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<td>13.0</td>
<td>13.0</td>
<td>0.544</td>
<td>4.12</td>
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Table 17: Properties of bentonite suspension samples with rheological parameters.

### 9.4 Results

The flow curves of the 4% (m/m) bentonite suspension in deionised water at different pH values, adjusted by NaOH and Ca(OH)₂, show characteristics of Bingham and/or Herschel–Bulkley fluids and are depicted in figure 104 and figure 105. The first flow represents the viscosity changes due to NaOH adjusted pH, with a reference bentonite suspension with a pH of 10.2 without NaOH adjustment. Flow chart in figure 105 represents viscosity changes due to Ca(OH)₂ adjusted pH, with a reference flow chart of bentonite suspension with natural pH 10.1.

Due to the high increased viscosity of some NaOH adjusted samples, the typical Herschel-Bulkley model is difficult to observe for flow charts with lower pH values. A sharp increase of yield strength is observed with an increase of pH. Up to a pH value of 11.7 the flow curves show almost a Bingham model, while a further increase of pH the typical Herschel-Bulkley is observed with an increased plastic viscosity. Maximum viscosity of bentonite suspension somewhere in between pH 11.7 and 12.8, as the flow curve for a pH value of 12.5 shows the highest viscosity. Viscosity is slowly building up as the pH value increases until a maximum value has been reached where and increased in pH results in decreasing viscosity.

The increase in viscosity of Ca(OH)₂ adjusted bentonite suspension is lower compared to the increase in viscosity for the NaOH adjusted pH samples. Although a large amount of Ca(OH)₂ was added to the bentonite suspension the pH didn’t increase more than 12.8 with only a small change in yield strength and relatively large increase in plastic viscosity. The viscosity of bentonite suspension with pH adjusted by Ca(OH)₂ increased with an increasing pH but did not show a maximum value of viscosity at a certain pH value.
Flow charts for bentonite with HNO₃ adjusted pH show initially increased yield strength up to a certain maximum value while the plastic viscosity remained constant. The reference bentonite suspension with a natural pH of 10.2 is also indicated in this chart (figure 106). Decreasing pH results in increased values for the viscosity up to a maximum viscosity somewhere in the range of 1.8 to 8.2, while the highest pH value for this test is at 7.0. Further decrease of pH results in a relative decrease of plastic viscosity close to the plastic viscosity of reference bentonite only with an increased yield strength.

Flow curves of bentonite suspensions with NaCl adjusted bentonite suspension immediately show increasing yield strength in relation to reference bentonite (figure 107). Further increasing the conductivity by adding more NaCl a maximum viscosity is reached at a conductivity of 10.81 mScm⁻¹ (0.1 M NaCl) for a test sample, while the actual maximum could be in between 6.2 (0.05 M NaCl) and 22.8 mScm⁻¹ (0.25 M NaCl). Bentonite suspensions with higher conductivity (0.5 and 1.0 M NaCl) show an decreased and respectively increased viscosity which is remarkable. Yield strength of bentonite suspension is increased significantly while the plastic viscosity is remaining the same.
The samples with pH adjustment by NaOH, and HNO$_3$ show a maximum value for apparent viscosity in a range of pH, further in- or decreasing pH results in a decrease of viscosity and yield strength. While for the Ca(OH)$_2$ adjusted pH values no maximum was found for the apparent viscosity only an increase in viscosity and yield strength. Yield strength of pH adjusted and reference bentonite suspension is depicted in figure 108 and shows increasing yield strength if pH of bentonite suspension is adjusted.

The relation between yield strength and pH value of the bulk suspension is depicted in figure 108 and depending on the adjustment solution differences can be observed. It is not difficult to observe an enormous increase in yield strength due to the addition of NaOH at pH value of approximately 12.5, which is not observed by Ca(OH)$_2$ adjusted pH at the same value. Decreasing the natural pH of bentonite suspension by HNO$_3$ indicates some sort of maximum yield strength at natural pH, while further decreasing pH results in decreasing yield strength. Yield strength and bulk conductivity of bentonite solution is depicted in figure 109 for the tests with adjusted pH using NaOH, Ca(OH)$_2$ and HNO$_3$ and conductivity adjusted with NaCl.

Stability of bentonite suspension should be visible due to the amount of free water in relation to the total amount of bentonite suspension (figure 110). After viscosity, pH and conductivity measurements, the samples in
measurement cylinders were standing for at least 24 hours and then visually checked for free water. All 250 ml measuring cylinders were filled up to the maximum level in order to quantify the amount of free water. With the exception of two or three samples all bentonite suspensions were stable after 24 hours; no free water was observed. Some samples were standing longer, even up to one week and proven to be stable. High molarities of Ca(OH)$_2$ adjusted bentonite suspension resulted in a significant amount of free water after quite some time and therefore less stable than other pH or conductivity adjusted suspensions. In figure 112 the situation of 0,9 M Ca(OH)$_2$ bentonite suspension after one week is depicted; almost 10% free water and a formation of two unknown layers is visible. White substance is probably due to the presence of Ca(OH)$_2$ while the layer in between this white substance and bentonite suspension is unknown.

![Figure 111: gas development in 0.167M HNO$_3$ bentonite suspension](image1)

![Figure 112: free water in 0.9M Ca(OH)$_2$ bentonite suspension](image2)

### 9.5 Conclusion

This chapter comprises experiments in order to determine influence of pH and conductivity adjusted bentonite suspensions. Proceeding on the experiments regarding the interaction of bentonite and softmix from the previous chapter calcium hydroxide contaminated bentonite suspension was also tested. This paragraph elaborates on the main conclusions from these experiments.

- During the experiments it was found that rheological properties of fresh bentonite suspension are changing over time due to the 'hydration' of bentonite particles; this process can take up to several days. Therefore it is important to use a reference sample to compare results during testing.
- The influence of pH on the bentonite suspension is tested by the addition of different acid or alkaline solutions such as NaOH, NaCl, CaOH$_2$ and HNO$_3$. Bentonite suspension adjusted pH by the addition of NaOH or CaOH$_2$ provide different rheological properties at the same pH level. This means that the type of acid or alkaline solution influences the rheological properties as well. Adjusting the pH of a bentonite suspension is inevitable bound up with a change in conductivity of the suspensions due to the addition of acid or alkaline solutions.
• Viscosity of pH and/or conductivity adjusted bentonite suspension is slightly changing on the majority of the tested samples. Yield point is the rheological parameter which is susceptible to external influences from pH and conductivity.

• Yield strength of bentonite suspension is highly depending on the conductivity of the bulk suspension; a small conductivity increase results in significant increase in yield strength (figure 109). On the other hand a large increase in conductivity results in increased yield strength but significantly lower compared to a small conductivity increase.

• Viscosity increase is observed with high molarity NaOH and in lesser extend Ca(OH)$_2$ adjusted bentonite suspensions, with consequently increased pH values. High NaCl and HNO$_3$ molarities do not have significant influence on plastic viscosity.

• During the preparation of a Ca(OH)$_2$ adjusted pH bentonite suspension, an interesting phenomenon was observed: viscosity suddenly increased when suspension became in contact with the Ca(OH)$_2$ solution. This highly increased viscous suspension existed during 5 to 10 seconds and then became liquid again, a mechanical-chemical irreversible thixotropy could be an explanation to the low increase of viscosity for the Ca(OH)$_2$ contaminated solution. This increased viscosity during preparation was significantly higher than any other adjusted bentonite suspension.

• Yield point of bentonite suspension increased enormously for conductivities ranging from uncontaminated bentonite suspension at 1400 $\mu$Scm$^{-1}$ to an maximum value at conductivities of 3000 $\mu$Scm$^{-1}$. Further increasing the conductivity results in lower yield point values.

• Stability of this bentonite suspension over a period of 24 hours was good for all samples, no or very little free water was observed. Even for high molarities of calcium hydroxide contaminated bentonite suspensions little segregation and free water was observed and no instability occurred. The samples were tested for stability over 24 hours, and during the removal of calcium hydroxide adjusted pH samples, the viscosity had increased significantly over time and samples were difficult to remove form measurement cylinders.

• Regarding the stability can be concluded that Trenchcontrol is a stable bentonite suspension for a large range of pH. From pH 1.8 to 12 no or very little segregation of free water and bentonite suspension occurred, only at very high molarities of Ca(OH)$_2$ contaminated bentonite suspension little segregation was visible.
10 Bentonite contamination during concreting

10.1 Aim of the test

During concreting process of a diaphragm panel bentonite suspension is in contact with fresh concrete. Concrete will displace bentonite suspension from the bottom of the trench to level until the panel is completely filled with concrete. Bentonite suspension properties change immediately in contact with concrete. In order to determine the influence during concreting bentonite samples were obtained during the concreting process of a panel in Rotterdam where a new parking garage is build underneath Kruisplein.

10.1.1 Test procedure

Samples of bentonite suspension were taken at level at the same location where it is pumped back to the storage tanks during the process. The first sample (A) was taken before concreting and the last sample (H) when the process was finished. Other samples were obtained during the process with an increasing concrete level. In table 7 and figure 113 the sampling times and distances to the rising concrete front are summarized.

<table>
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<th>Sample</th>
<th>Time</th>
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<td>hh:mm</td>
<td>m-ground level</td>
<td>m</td>
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Table 18: samples with time and concrete level

During the process some unknown material floated up (figure 114) and formed a floating layer of this material. It was tried not to collect this during the sampling, but at some point this was inevitable. Samples F, G and H were contaminated with this material, especially sample F. The samples obtained during the concreting process are tested for density, sand content, alkalinity, filtrate water, cake thickness and viscosity according to paragraph 5.4.

Figure 114: floating material
10.2 Results

Each sample was tested in the laboratory under the same conditions and testing methods. The results for density, sand content and alkalinity versus the depth of the rising concrete level are depicted in figure 115. These results clearly show the bentonite suspension is affected by the concrete during the pouring of the concrete. Especially samples close to the concrete level show a difference in relation to samples taken at larger distance.

10.2.1 Density, sand content and alkalinity

The increasing density of the bentonite suspension during the rising concrete level does not correlate to the sand content since there was no difference observed in the sand content of the first five samples. Flocculation of bentonite could explain the density increase as well as the sudden decrease in the last sample. During flocculation the colloidal bentonite suspension becomes separated in two phases, flocculated bentonite and free water. The less viscous free water is easily pumped away leaving, flocculated bentonite in the trench, while there was a lot of free water during at the last sample. The sand content is rather stable for the majority of the samples and only for the last three samples suddenly increased and moderately decreased again. Apparently the floating material contains some sand particles. The alkalinity which is slightly decreasing could be explained by the interaction with the peat layer 12 m below ground level. The increased alkalinity for the last three samples is explained by the floating material, which is probably alkaline cementitious material (i.e. calcium hydroxide).

![Density vs concrete level](image1)

![Sand content vs concrete level](image2)

![Alkalinity vs concrete level](image3)

**figure 115:** density (a), sand content (b) and alkalinity (c) vs distance to concrete level

10.2.2 Filter cake and filtrate water

Filter cake formation and filtrate water loss show a consistent relation with the previous results. The fairly uncontaminated bentonite suspension samples A, B, C, D and E show a low filtrate water loss and very low filter cake thickness in the order of 0,5 mm. Bentonite suspension samples F, G and H are influenced by concrete resulting in significant changed properties. Sample G is mostly affected since the filtrate water and the cake thickness are very high. Sample H shows an interesting relation between the filtrate water and the filter cake thickness. Compared to sample F and G there is more filtrate water loss but less filter cake thickness, indicating a large amount of free water and flocculated bentonite suspension.
Filtrate water and bentonite cake thickness are both related to the square root of time by the variation of a constant. The cake thickness \( h_c \) can be calculated with the amount of filtrate water \( V \) for bentonite support fluids when these constants \( n \) and \( m \) respectively are known. The slopes in figure 116(a) and figure 116(b) determine these constants (table 19) and with the following relations both parameters can be expressed into each other:

\[
\begin{align*}
    h_c &= n \sqrt{t} \\
    V &= m \sqrt{t} \\quad \text{(Sample)} \\
    \frac{h_c}{m} &= \frac{n}{m} V 
\end{align*}
\]

The ratio of both constants represents the ratio of filtrate water and bentonite cake as the consequence of the filtration of a certain amount of bentonite suspension. A low number indicates a thin cake and a relatively large amount of filtrate water, while a high ratio produces a thick bentonite cake and relatively low filtrate water.

10.2.3 Viscosity

Viscosity of bentonite suspension is also influenced by the interaction of cement, changing properties significantly. Again samples A-E show a normal flow curve of bentonite suspension, almost as a Newtonian fluid, while the contaminated bentonite samples show an increased viscosity. The viscosity of sample F has increased enormously showing a typical Herschel-Bulkley flow curve. The shear stress at low shear rate and gel strength are important rheological parameters.

Gel strength a measure for the strength of a thixotropic fluid which develops over time. “Some drilling fluids are thixotropic, forming gelled structures when stagnant and liquefying when sheared. The specific gel strength of a drilling fluid is described as low-flat (most desirable), progressive or high-flat (both undesirable) according to its measured gel strength versus time. The shear stress measured at low shear rate after a mud has set quiescently for a period of time (10 seconds and 10 minutes in the standard API procedure, although measurements after 30 minutes or 16 hours may also be made.”
Conclusion

These experiments have indicated the significant influence of fresh concrete to bentonite suspension by changing rheological properties. Bentonite samples were taken during the concreting of a closure panel at the construction site of Kruisplein in Rotterdam and tested the next day in the laboratory. This paragraph elaborates on the main conclusions of this laboratory testing.

- Bentonite samples close to the fresh concrete are significantly influenced by concrete which manifests mainly in filtration and rheological properties.
- At a distance of less than 10 meter from the fresh concrete bentonite is contaminated by concrete substances running up from the concrete front to level and therefore have changed properties. The contamination and changed properties of bentonite suspension are generally increasing closer to the concrete front.
- Rheological properties are dramatically influenced for yield point and gel strength:
  - Yield points up to 50 times higher in comparison to uncontaminated bentonite suspensions.
  - Thixotropic properties have vanished in the three samples closest to the fresh concrete.
- Filtration properties are decreasing due to high filtrate water loss and the development of thick bentonite cake for samples closer to the concrete front.
- Density sand content and alkalinity show significant changes of bentonite suspension during concreting for bentonite suspension closer to the fresh concrete level.
Part IV: Conclusions & Recommendations
11 Conclusions

Conclusions regarding the research on bentonite cavities in diaphragm walls are elaborated in this chapter. These conclusions should provide an answer to the main objective of this research: “To determine important processes, parameters and mechanisms regarding the occurrence of bentonite cavities in diaphragm walls”. Conclusions from experiments and tested measurement techniques are also provided in this chapter. The main conclusion from this research connected to cement bentonite interaction.

11.1 Main conclusions

Conclusions from experiments regarding the occurrence of bentonite cavities in diaphragm walls are connected to bentonite cement interaction. The plausible causal relations of cement contaminated bentonite and the occurrence of bentonite cavities are stated:

- The causal relation in between cement contaminated bentonite and the change of rheological properties is demonstrated with experiments during this research. Viscosity, yield point and thixotropic properties of bentonite suspension are influenced by cementitious material in contact with bentonite suspension. (figure 117)
- Interaction of softmix and bentonite suspension is also demonstrated by experiments, with changed rheological properties as a result. A significant influence is observed at high surface area of softmix in contact with bentonite suspension. This high surface area is probably not representative for the actual excavation process as carried out at Vijzelgracht, but undeniably confirms the results of interaction.
- The causal relation in between bentonite with high yield point and/or increased viscosity and the occurrence of bentonite cavities is plausible, but not confirmed by experiments or otherwise. Additional experiments to confirm this relation are recommended in the next chapter.
- Experiments on the interaction of softmix and bentonite suspension have indicated that besides a difference in fluid pressure there are other mechanisms causing bentonite cake formation. The cake formation on the softmix is probably caused by cementitious material. Cake formation on fresh concrete surfaces is probably more severe due to the same mechanism and the higher cement content.
- Studying bentonite cavities at Diabolo and Vijzelgracht project during this research showed the remarkable fact that they were all situated at the joint of an intermediate (or a closure) panel.

11.2 Processes, parameters and mechanisms

Besides cement bentonite interaction there are important processes, parameters and mechanisms emphasized by experts and obtained from literature with a plausible causality to bentonite cavities:

- Regeneration/refreshing of bentonite suspension and the usually omitted joint surface cleaning are processes with a plausible causal connection to the occurrence of bentonite cavities. These processes haven’t been subject of research in this study, while the determination of these processes is in order to perform future research.
- Besides quality or type of bentonite, concrete mixture, influence of reinforcement density, number/position of tremie pipes, the concreting process in general is an important process regarding the occurrence of bentonite cavities in diaphragm walls. From literature it can be derived that the speed of concreting determines the flow resistance in between concrete and bentonite suspension. Higher concreting speeds increase the difference in flow resistance in between both fluids. This increased difference in flow resistance makes it easier for the concrete to displace the bentonite.
11.3 Measurement techniques

Conclusions regarding measurement techniques to monitor the rate of growth of bentonite filter cake and determine its density:

- Image analysis as a measurement technique to monitor the rate of filter cake formation is an adequate method. This method is able to indicate the transition point of liquid and solid bentonite over a significant amount of time. Image analysis used for density determination has not been successful.
- The transition in between liquid and solid bentonite couldn’t be determined using electric potential measurements. Environmental influences and/or inadequate measurement instruments are probable causes.
- From literature is known that filtration water and cake thickness are related to the square root of time. During the experiments this relationship was consistent and could be used as a measurement technique to determine cake thickness.
12 Recommendations

Recommendations from this research are subdivided in recommendations regarding ‘cement bentonite interaction’, ‘processes, parameters and mechanisms’, ‘measurement techniques’ and recommendations regarding the ‘adaption of codes’ for diaphragm wall construction. Recommendations regarding the performed experiments are provided in order to improve further study and future research. Recommendations regarding to the codes urge for adjustment in order to prevent the formation of bentonite cavities in future projects.

12.1 Cement bentonite interaction

Regarding cement bentonite interaction recommendations can be made for additional experiments to verify or reject the causality with bentonite cavities in diaphragm walls.

- More qualitative research regarding the interaction processes of cementitious material with bentonite is necessary to be able to quantify these interactions. With this information new or stricter regulations can be made for the use of cement in ground improvement in the trajectory of diaphragm walls and the time in between concreting one panel and excavating the next adjacent panel.
- The causal relation of increased yield point and increased viscosity with the occurrence of bentonite cavities should be determined, to gain more understanding in the replacement process of concrete and bentonite.
- Determination of the mechanism responsible for filter cake formation on cementitious surfaces should gain more insight in the cement bentonite interaction processes and to which extend this occurs at relatively fresh concrete surfaces.

12.2 Processes, parameters and mechanisms

Regarding the objective of this research “To determine important processes, parameters and mechanisms regarding the occurrence of bentonite cavities in diaphragm walls” some recommendations are provided. The processes and parameters with a plausible causal relation to the occurrence of bentonite cavities should be subject of new research.

- Interviewed experts stated that the cleaning process of bentonite suspension after the excavation could be an important process during the construction of diaphragm walls regarding the occurrence of bentonite cavities. During this process the bentonite suspension in the excavated panel should be replaced with fresh or regenerated bentonite to meet requirements before concreting. If the viscosity of this bentonite is (locally) very high this substance will possibly not fully be replaced by fresh or regenerated bentonite. This high viscous bentonite might not be replaced by concrete. Experiments to demonstrate this should be carried out.
- Although cleaning the joint surface of already concreted panels is not common practice in the Netherlands, this could help to prevent bentonite cavities. This procedure could enable the removal of high viscous bentonite material from the joints of the concreted panels. Furthermore this joint cleaning procedure is probably shearing the bentonite suspension prior to the refreshing/regenerating process. By shearing, the sometimes highly viscous bentonite suspension becomes more liquid and therefore contributes to the cleaning process. Experiments should demonstrate the importance of this process and make the cleaning of joint surfaces obligatory.
- The entire concreting process including properties of bentonite, concrete and reinforcement should be evaluated for weaknesses to the occurrence of bentonite cavities. Flow resistance in between concrete
and bentonite in combination with concreting speeds and dense reinforcement should be evaluated in order to develop new codes. This is necessary because at the north south line project dense reinforcement in combination with large gravel sizes in the concrete were not consistent with code. The influence of the concreting speed on the amount of cake thickness removal is an interesting experiment recommended to perform.

12.3 Measurement techniques
Measurement techniques tested in this research, urge for new techniques or different approach. In order to perform extensive research on bentonite filter cake, accurate measurements for density and shear strength should be easily obtained.

- An accurate bentonite cake density measurement technique should be used to determine the correlations with image analysis techniques and electro potential/current measurements and to obtain values for density over the height of the sample. A CT scan could provide relative density of bentonite cake, which could be linked to an absolute average density measurement over the height of the sample in order to obtain an indirect absolute density measurement. Direct absolute density measurement methods are unknown at this moment.

12.4 Adaption of codes
Recommendations from this research urging for adaption of the codes are elaborated in this paragraph. Adaption of the codes regarding certain processes can result in a significant reduction in the occurrence of bentonite cavities in future projects without additional risks or complicated processes.

- Cleaning the joints of previously casted panels prior to regeneration/replacement of bentonite suspension could result in an increased quality of diaphragm wall and reduced bentonite cavities. This cleaning process is recommended to perform for every intermediate and closure panel, because this process has potentially no negative influence on the quality of the diaphragm wall. Besides cleaning the joint surface, this process shears viscous bentonite close to the joint surface, making it easier to regenerate/refresh the suspension.

- Testing bentonite suspension properties should be performed using a bentonite slurry sampler and the location of sampling should at the bottom of the excavated trench close to the corner of the panel. This will provide actual properties of bentonite suspension in the trench.

- The cake formation on the softmix is probably caused by cementitious material. Cake formation on fresh concrete surfaces is probably more severe due to the same mechanism and the higher cement content. Therefore the time in between concreting a panel and excavating the panel directly next to it, should be as large as possible and the time in between excavation and concreting this panel should be as small as possible.
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