Uretek Deep Injection Method
Full Scale Test



Stefan Segers
Std.no. 9757113

M. Sc. THESIS, Civil Engineering
Uretek Deep Injection Method;
Full Scale Test
April 2006

URETEK

resina

TUDelft
Delft University of Technology

It does not matter how slowly you go so long as you do not stop.

Confucius

Colophon

Organisations

Delft University of Technology
Faculty Civil Engineering and Geosciences
Stevinweg 1
2628 CN Delft
Tel +31 15 278 5440

Resina Chemie BV
Korte Groningerweg 1A
9607 PS Foxhol
Tel +31 598 317 907
www.resina.nl

Uretek Nederland
Zuiveringweg 93
8243 PE Lelystad
Tel +31 320 256218
www.uretek.nl

Student

Stefan Segers
Achterom 16
2611 PP Delft

Tel +31 6 24274867 (cell phone)
Email ssegers@gmail.com
Std.no. 9757113

Committee

Chairman (TU Delft)
prof. ir. A.F. van Tol
Chair Foundation Engineering
Room 1.18 (Civil Engineering)
Tel +31 15 278 5478
+31 15 269 3584 (GeoDelft)
Email A.F.vanTol@citg@citg.tudelft.nl
A.F.vanTol@geodelft.nl

Daily supervisor (TU Delft)
ing. H.J.Everts
Chair Foundation Engineering
Room 0.10 (Civil Engineering)
Tel +31 15 278 5478
+31 15 269 3769 (GeoDelft)
Email H.J.Everts@citg.tudelft.nl
H.J.Everts@geodelft.nl

Supervisor (TU Delft, separate department)
dr. ir. P.J. Visser
Chair Hydraulic Engineering
Room 3.96
Tel +31 15 278 8005
Email P.J.Visser@citg.tudelft.nl

External supervisor (Uretek)
ing. A. ter Huurne
Executive Uretek Nederland en Duitsland
Tel +31 320 256218
Email Alwin.ter.Huurne@uretek.nl

External supervisor (Resina Chemie BV)
dr. ir. A. van der Wal
Head Research and Development
Tel +31 598 317 907
Email A.Wal@resina.nl

Preface

The final thesis of the Civil Engineering Master at Delft University of Technology consists of research on a civil engineering subject. This research is on an experiment performed in the context of the injection method of Uretek, so called Uretek Deep Injection Method. Settled, and especially tilted foundations can be lifted with this method. The possibility exists also to increase the strength of the soil using this method.

A full-scale experiment was carried out on the Deep Injection Method to increase the understanding of the technique. The experiment was performed in Wolvega, a village in the Northern part of The Netherlands, in the province of Friesland. The Uretek and Resina Chemie BV companies fund the experiment. Uretek is the company who carried out the intervention method and Resina Chemie BV is the supplier of the two-component resin used.

Reader's guide

Chapter 1 is an introduction into the resin and the Deep Injection Method. Chapter 2 describes the problems encountered during the research into the injection method and what the difficulties where during this study. Chapter 3 illustrates the geology of The Netherlands. The soil investigation that was done on the test site is described in chapter 4. The results of the laboratory research are described in chapter 5. The prediction on the settlement is given in chapter 6. The design and a description of the test facilities are presented in chapter 7. Chapter 8 describes the execution of the tests. The results of the test are presented in chapter 9. The report finishes in chapter 10 with the conclusion and recommendations.

Acknowledgments

I would like to thank my counseling team made up of Professor van Tol, Mr. Everts, Mr. Ter Huurne and Mr. Van der Wal for their support and for coaching me during the execution of the test and the writing of my report.

My appreciation goes to the employees of Uretek, who gave me, and my counseling team, the required practical information for the execution of the experiment. As well as the employees of Punter BV, who gave me the required support for the realization of the test facility and the execution of the test.

Finally, I want to thank my friend Jacinthe who supported me on the English language, Jelke for pointing out the relativity of ordinary life. And last but certainly not least, my girlfriend Mignon who did not only support me on finishing my study; she also gave me essential support, which is indispensable when accomplishing a final thesis.

Delft, 18 April 2006

Stefan Segers

Summary

Introduction
The Uretek Deep Injection Method is an intervention method for settled and or tilted foundations. Structures settle because of the increased effective soil stress, below foundation level, due to the weight of the structure. The magnitude of the settlement depends on many aspects, like soil characteristics, stress level below the foundation, stress history, etc. In cases with large amounts of settlement, damage to the constructions may occur. Once in existence, this settling can be undone through a method introduced by the Uretek Company. This method uses two-component polyurethane resin provided to Uretek by Resina Chemie B.V.

The main objective of this thesis is to set up an experiment where, by means of the Uretek Deep Injection the following aspects can be analysed;

- the possibilities of lifting structures;
- the improvement of the strength and stiffness properties of the soil (sand as well as clay)
- the run of the resin during and after injection;
- the durability of the heave of the structure and improvement of the soil characteristics.

The following thesis deals with the results of this test and does further research to determine the characteristics of the soil and the excavated resin.

Central Part
The geology of The Netherlands and the used materials for the research are described as background information for this thesis. From the executed soil investigation and the laboratory research the following stratification of the soil has been deduced;

Top [m]	Bottom [m]	What	
0	-1.00	Sand	Above water table
-1.00	-2.20	Sand	Underneath water table
-2.20	-3.50	Loam	
-3.50	-9.00	Sand	

Table 1 Average result of the composition of the soil

The laboratory research proved that Uretek could use their Deep Injection Method in the soil of the test site in Wolvega. With the knowledge of the composition of the soil a settlement prediction was carried out. This prediction is later checked with the measurement results. That showed that the prediction was in the right class.

The Test Facility consists out of two concrete foundation strips with a length of 5 meters. These strips are fixated to each other with two UNP240 profiles. On the concrete foundation strip three steel HE260B profiles are placed. Perpendicular to these profiles dragline floor plates are placed to carry the load which consists of sand filled big bags. The load is placed in two stages to determine the increase of the stiffness of the soil. The strength of the soil is tried to determine with the use of soundings. In total three test facilities have been build, a Reference Test Facility, a Short Term Test Facility and a Long Term Test Facility. Each facility has its own purpose. The ReferenceTest Facility was for the determination of the stiffness of the soil without injection. The long term behavior of the treated soil will be deduced by comparing the settlement of the Long Term Test Facility and the Reference Test Facility. The Short Term Test Facility was meant for the determination by excavation of the strength of the soil and the visualisation of the in the soil formed resin. Only underneath the Long Term- and the Short Term Test Facilities there was injected.

The measurements that were executed are dynamic soundings, leveling measurements with the help of leveling spirit equipment and a marker and the excavation of the Short Term Test Facility.

Conclusion

The conclusion can be drawn that the main objective of the research has been achieved.
A test facility was built under which a two-component resin could be injected, could be dismantled and excavated in order to examine the results. A cheap, and easy in use, leveling instrument was used that performed according to expectation. The injections were performed in two stages. The first stage consisted of injecting underneath all the foundation strips of the Short-Term Test Facility and the Long-Term Test Facility. The second stage consisted of injecting underneath foundation strip 3.3 – 3.4 of the Short Term Test Facility. With the aid of the levelling instrument, used before and after the injections, conclusions could be drawn on the amount of heave and stiffness of the soil. The location and shape of the resin in the soil could be examined after the excavation of the Short-Term Test. The gathered data will result in an increase of knowledge about the UDI.

The UDI is a method with a high potential but further research will be necessary towards a better understanding of the consequences of the UDI and towards perfecting the injection method, as well as some research on the resin itself.

Content

Colophon 1
Preface 3
Summary 5
Content 7
1 Introduction 9
1.1 The Thesis 9
1.2 The Uretek Method 10
2 Problem Analyses and Objectives 11
2.1 Aspects to consider 11
2.2 Objective 12
3 Geology 13
3.1 Pleistocene 14
3.2 Holocene 16
4 Soil Investigation 19
4.1 Materials used 20
4.1.1 CPT truck 20
4.1.2 Hand Sounding Device. 20
4.1.3 Dynamic Sounding Device Pagani DPML 30/20 20
4.1.4 Dynamic Sounding Device Stitz DPL 21
4.1.5 Observation Well 21
4.2 Execution of the Soil Investigation 21
4.2.1 Test Site Orientation 21
4.2.2 Second Soil Investigation 21
4.2.3 Hand Soundings 22
4.3 Soil Investigation as a Measurement 22
4.4 Soil Investigation Conclusions 22
5 Laboratory Tests 23
5.1 Grain Size Distribution 23
5.2 Atterberg Limits 23
5.2.1 Liquid Limit with Casagrande Method 23
5.2.2 Plastic Limit Test 24
5.2.3 Plasticity Index 24
5.3 Bore Logs 24
5.4 Laboratory Test Conclusions 24
6 Settlement Prediction 25
7 Test Facility Design 27
7.1 The Reason for a Full Scale Facility 27
7.2 Types of Test Facilities 27
7.3 Test Facility Construction 27
7.4 The Test Facility Elements 28
7.4.1 Foundation Strip 28
7.4.2 Steel Profiles 29
7.4.3 Dragline Floor Plates 29
7.4.4 Big bags 30
7.5 Positioning of the Test Facility 30
8 Execution 31
8.1 Positioning of the Test Facility 31
8.2 Test Facility Construction 31
8.3 Injections 32
9 Results of the Test 35
9.1 Measurements 35
9.1.1 Levelling Instrument 35
9.1.2 Levelling NAP Results 36
9.2 Levelling measurements 36
9.3 Dynamic soundings 40
9.3.1 Dynamic Sounding Devices 40
9.3.2 Dynamic Sounding Conclusions 41

9.4 Injection 41
9.4.1 Piquet's 41
9.4.2 Observation Wells 42
9.5 Excavation 42
10 Conclusions and Recommendations 45
10.1 Conclusions 45
10.1.1 Set-up 45
10.1.2 Levelling measurement 45
10.1.3 Settlement 45
10.1.4 Dynamic soundings 47
10.1.5 Excavation 47
10.2 Recommendations 47
10.2.1 Research 47

Appendices 49
Appendix 1 The Resin 51
Appendix 2 *Intervention with UDI in Germany* 53
Appendix 3 *Tests on the resin by the University of Padua* 55
University of Padua Report 1 55
University of Padua Report 2 56
University of Padua Report 3 56
University of Padua Report 4 58
University of Padua Report 5 58
University of Padua Report 6 59
University of Padua Report 7 59
Appendix 4 *Bell Tower* 61
University of Padua: Intervention to consolidate the foundation ground of a bell tower with high expansion pressure resin 61
Appendix 5 Hand Sounding Equipment 63
Appendix 6 Pagani 65
Appendix 7 CPT's Fugro 67
Appendix 8 CPT's Wiertsema and Partners 75
Appendix 9 Laboratory Tests 77
Appendix 10 Sieve Curves 79
Appendix 11 Bore Stats 81
Appendix 12 Settlement Prediction 83
Appendix 13 Detailed Schematic 87
Appendix 14 Calculations for the Test Facilities 89
Appendix 15 Test Facility Positions 91
Appendix 16 Schematic of the Test Facilities 93
Appendix 17 Weight Determination 101
Appendix 18 Test Facility Construction Pictures 103
Appendix 19 Injection Schedule 109
Appendix 20 Renewed Injection Schedule 113
Appendix 21 Control Measurement 115
Appendix 22 Dynamic Soundings 117
Appendix 23 Marker Pot 123

References 125

1 Introduction

Structures settle because of the increased effective soil stress, below foundation level, due to the weight of the structure. The magnitude of the settlement depends on many aspects, like soil characteristics, stress level below the foundation, stress history, etc. In cases with large amounts of settlement, damage to the constructions may occur. Once in existence, this settling can be undone through a method introduced by the Uretek Company.
This method uses two-component polyurethane resin provided to Uretek by Resina Chemie B.V.
Testing of the characteristics in terms of volume increase and stiffness of a two-component resin has, so far, only been done in a laboratory environment. Both companies, Uretek and Resina Chemie B.V. are interested in the in-situ behaviour of the resin, in other words, the behaviour in a realistic on site situation.

This research is divided between two graduate students who performed the test together but have their own specific areas of research. The other research deals with the properties of the resin and this research deals with the aspect described in the following paragraph.

Figure 1 Control Test Facility

1.1 The Thesis

In total, three identical test facilities were designed. These test facilities consist of two concrete foundation strips, fixated by steel profiles. Three steel profiles were placed on the foundation strips and three dragline floor plates were placed perpendicular to these profiles. The final load is placed in two stages. The choice was made to use two stages. The reason for that was that the resin had to be injected under normal stress conditions, which were realized during the first loading step. The original soil stiffness is determined during this first and the second loading step as well. Comparison of these stiffnesses is an indication for the increase of stiffness due to the injection of the resin. The difference in soil strength, before and after the injections, will be determined with the aid of static and dynamic soundings. The durability and the creep characteristics of the resin will be determined by comparison of the behaviour of the Long Term Test Facility and the Reference Test Facility.
Injections are done underneath the foundation strips of the Short Term Test Facility and the Long Term Test Facility as well in order to determine if the injected resin has an impact on the strength and stiffness of the soil. The resin is made up of two components solidifying in less than 2 minutes, see appendix 1 *The Resin* for the details.

1.2 The Uretek Method

A foundation should be capable of bearing a certain load with a settlement that stays within certain limits. In order to do that the foundation transfers its forces to the sub-soil. This leads to increased effective stresses beneath foundation level. If these stresses differ strongly or if in-homogeneities in the soil exist, differential settlements might occur. These situations provide the possibility for Uretek to apply their product. By injecting the resin into the soil the possibility arises to reinforce the ground below the foundation. The injection uses the Uretek Deep Injections, UDI for short.
The distances between these injections differ from project to project, depending on the required depth of the injection. The soil investigation also determines the type of resin used. A stabilisation plan is made based upon the results of the soil investigation. Uretek uses a 3-stage plan for every project they contract to do.
1. Drilling: Holes are drilled at equal distances through the base of the foundation, in diameter varying from 14 to 30 mm. In order to reach deeper lying ground layers (up to 5 m) drilling takes place alongside the foundation. In both situations copper tubing is inserted into the drilled hole to keep the injection path clear. The injection gun can be mounted easily onto this copper tubing. At greater depths special equipment is used to push the copper tubing with a wider diameter, 30mm, into the ground.
The injection tube is placed within this so-called jacket tube.

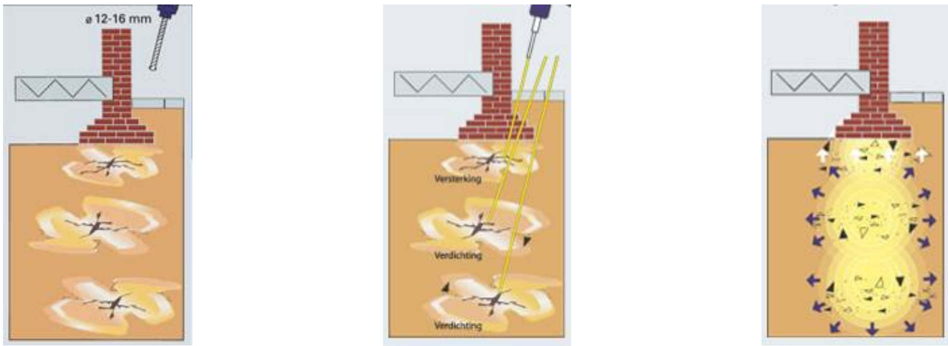


Figure 2 The three stages of the UDI

2. Injection: The injection gun is connected to the injection tube, weather or not in a jacket tube.
The injection gun contains a special gripping mechanism that connects the copper tubing and the injection gun. The resin is injected, both components mixing just before they leave the gun. The chemical reaction takes place within 10 seconds. The injected mixed resin propagates in the soil in a direction of least resistance.
3. Expansion: The resin can expand to 30 times its own volume in less than 120 seconds. As soon as the soil resists the pressure of the resin, a deformation of the soil mass will take place. A laser receiver mounted onto the construction will register this motion, however subtle. At that point the suggestion can me made that ground improvement is evident, according to Uretek. When the injections are then continued, a settled foundation can be lifted; if possible to its original level.

2 Problem Analyses and Objectives

As discussed in the introduction, not much is known about the Uretek Method. As far as is known, only one in-situ test using the Uretek method has been performed, this test was conducted in Germany, see appendix 2 *Intervention with UDI in Germany*. The other tests are performed on the resin itself and not on the effect it had on the ground, see appendix 3 *Tests on the resin by the University of Padua*.
Documentation exists about tests done on a Bell Tower foundation in Italy, but the conclusions drawn can be interpreted as guesswork, because only the surface results where studied, see appendix 4 *Bell Tower*.
This thesis has the intention to conduct a study that approaches reality as closely as possible. This is attempted using a model with long foundation strips under which, by means of a heavy load, a foundation pressure is generated matching that of a real building.

2.1 Aspects to consider

The aspects that have to be taken into consideration or that are of a determining factor to the design or the execution of the tests are:

Burden and the foundation strip: shallow foundations have an impact on the stresses in the soil. Stresses will be high directly below the foundation and deeper down (at a larger depth relative to the foundation) stresses will be less. The problem with injecting below shallow foundations is that there is little coverage making it possible for the injected resin to escape. Injecting at great depth is difficult to execute because of the equipment limitations.
The stresses before, during and after the injection make up important data. These stresses immediately show the effect of the injection on the ground as well as on the run of the resin.
Soil conditions: in soft soil, such as clay containing ground layers, the injection will create excess pore pressure. The excess pore pressures dissipate slowly (consolidation), so the water pressure will adapt slowly to the new situation. The speed of this process depends on the permeability of the ground. The dependence of the water pressure on time can cause a problem for the UDI, for consolidation means loss of soil volume and therefore loss of soil stresses. Therefore the method is mostly used on sandy soil, because in these types of soil the water pressure will not increase during the injection and no volume loss occurs after injection.
The time-depending behaviour of the water pressure can lead to time-depending settlement after the injection. The layers of soil underneath the injected layer can be cohesive and therefore responsible for settling to occur. Different types of soil have their own specific effect on settling. Uretek confirms this statement through practical experience:

- Injections into completely permeated soil types with a granular diameter < 60um (cohesive soil types) seldom render the desired result. During, or directly after the injection settling occurs and therefore increasing the problem instead of solving it.
- Clay and peat layers, thicker than 50cm, containing a high percentage of organic material, will not be injected by Uretek because of poor results in the past.
- Problem areas, with a depth of over 5 m, relative to the injection level, cannot be treated with the UDI. In those cases another solution will have to be found.

The subject of this thesis is to find out what influence the Deep Injection Method has. The Uretek method can be an effective method to lift settled foundations to their original level and improve the soil characteristics. This renders it necessary however to understand what really happens in the soil and things are made easier when the results and behaviour of this method can be predicted and that requires further study. This research is necessary to better understand the behaviour of the injections and to make clearer predictions about the use of the two-component resin.
This will be of great value when the method can be optimised for both production of the components and the method used in the field.

Problem statement: Does the UDI really improve the strength and stiffness of the layer of soil into which it is injected and is the method usable in clay-containing soil for levelling settled foundations and what is the long term behaviour?

2.2 Objective

The tests will be executed towards a fairly broad objective; the points that will be taken in to account are;

- The run of the resin
- The soil improving properties
- The effected area
- The applicability within clay-containing soil.

So the final objective for the tests reads as follows:

Objective: Design of a test facility to conduct tests using the Uretek method in clay containing soil and ascertaining the improvement of the soil properties, strength and stiffness, of the method as well as researching the run and the creep characteristics of the resin.

3 Geology

The geological history of The Netherlands can be derived from the construction of its soil. Research has shown that The Netherlands, as part of the European continental plate, made a migratory voyage around the globe. On this voyage The Netherlands migrated through all of the temperate climates. Each climate zone has it's own characteristic types of sedimentation that allow us to determine, through examining the layers in the soil, which sedimentary deposits and climates have been known to The Netherlands.

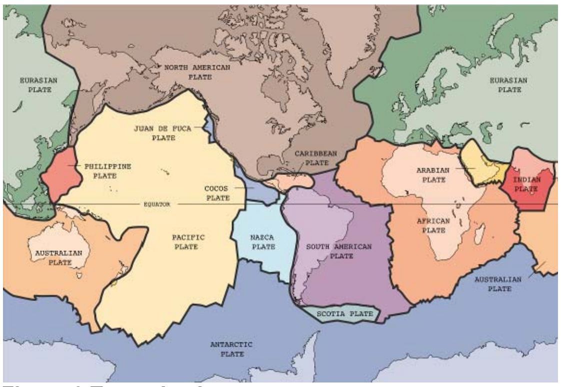


Figure 3 Tectonic plates

Tectonic movements together with fold- and fracture structures raised minerals such as coal, oil and natural gas to depths that are accessible to mankind. Mining of these minerals has a great impact on the natural landscape.
The geological history of the Netherlands can be studied starting from the Devonian contrary to that of the Ardennes and the Eiffel where older types of sedimentation are to be found closer to the surface of the soil.
The periods that have shaped the globe into its current form are:

• Paleozoic: 570 – 230 Ma	Devonian	(395 – 345 Ma)
	Carbonic	(345 – 280 Ma)
	Perm	(280 – 230 Ma)
• Mesozoic: 230 – 65 Ma	Trias	(230 – 195 Ma)
	Jura	(195 – 141 Ma)
	Chalk	(141 – 65 Ma)
• Cenozoic: 65 Ma – present	Tertiary	(65 – 2,4 Ma)
	Quatrain	(2,4 Ma – present)

The Quatrain is the most important to the geotechnical engineer because the layers formed during this period are close to the surface. The Quatrain can be subdivided into two periods of very different lengths (geological perspective). The oldest period, the Pleistocene, over 2 million years in length encompasses the glaciations. The latter period, the Holocene, 10.000 years old started after the last of the glaciations.

Glaciations are characteristic of the Quatrain, the climate varying from moderately warm to polar.
These climate changes caused continuous changes to the landscape.
From a tectonic point of view the position of The Netherlands stayed the same, on the tip-line of the descending North Sea basin and the rising Ardennes and Eiffel. Thick layers of sediment, caused by erosion in the rising hinterland where deposited in the descending basins. Here the sedimentation was predominant and therefore protected from erosion. Consequently a large part of the history of the Quatrain is well documented in the soil of the Netherlands.

3.1 Pleistocene

The Pleistocene (2,4 – 0,1 Ma) is subdivided into periods each starting with a climate change. The Pleistocene is the period in which frost, thaw and ice where strongly involved in the geological processes that contributed strongly to the shaping of the relief and many of the deposits that can be found above N.A.P 0 m. in The Netherlands.
During the Saline the land ice caused differences in elevation of over 100 m. Tipping processes and ecological deposits caused the relief to level off during the Eemien and Weichsel. The nature of the soil determines to a great extend the degree of change a glacier can cause to the landscape. The soil in The Netherlands consisted mostly of sand and clay. The ice put pressure on the soil, froze to it, broke the permafrost down into shale's and displaced the soil. The glacier tectonics resulted in causing folds and shale's in fine grain sediments. In coarser grained sediments steeper shifts or shale's where formed. The cause of this behaviour was the amounting pore pressure in the fine grain sediment under the influence of the glacier. The particles of the sediment where packed less close together. Because of the glacier the pores became tighter. These pores couldn't drain the water fast enough causing a layer of sludge, consisting of easily mixed ice and sediment, to form. All deposits underneath this layer of sludge where shaved off their base and displaced. In the coarser grained sediment the water could drain more easily causing the water pressure to build up less or not at all. The sediment packet remained more homogenous and served as a block in which fractures and shale's where formed instead of flattened folds.

Figure 4 Boundaries of the glaciers during the Pleistocene

Landscape
Looking at the glacial landscape, two areas can be distinguished, the accumulation area and the ablation area.
In the accumulation area a lot more snow fell than there could disappear through evaporation or melting. Here the flow speed of the ice increased until a balance was struck between supply and discharge. The path of the ice was elongated.
In the ablation area the ice mass diminished and the flow speed decreased towards the edge of the ice. The ice was compacted horizontally.

Development of Pleistocene deposits
Initially sand and clay belonging to the Oosterhout Formation was deposited in the sea and along the coast, later transforming into the Maassluis Formation.
In the Southeast of The Netherlands the Kiezolie Formation was deposited by the Rhine and the Meuse as packages of clay or sand and gravel (Pretiglien).
During the next period the marine and coastal deposits forming the Maassluis Formation continued. From the South, the Rhine, Meuse and Schelde shaped a delta in the North Sea. The sand, gravel and clay in these deposits belonged to the Tegelen formation. A lot of coarse, grey-white fluvial sand and fine grain gravel was added to this delta from the East, Harderwijk/Maassluis Formation. Eventually the coastline was situated outside of The Netherlands; this situation remained until the Late Cromerien (Tiglien).

Subsequently a glacial period followed in which mainly fine sand and clay from the Kedichem Formation was deposited by the Rhine and Meuse onto the Tegelen Formation.
Locally these formations fell pray to the peri-glacial processes, forming, amongst others, brook sedimentation or peat. The Harderwijk formation was still under construction (Eburorien).
The subsequent period was interglacial in which the shaping of the Kedichem and Harderwijk Formations continued (Waalien)
The Kedichem Formation was built on and extended by the Rhine and Meuse rivers. Rivers from the East, at first building on the Harderwijk Formation, continued by adding coarse gravel and stones that started the Enschede Formation. This Formation consisted mostly of coarse, gravel containing, grey-white sands (Menapien)
The Early Pleistocene ended with alternate glacial's and interglacial’s, adding to the Enschede Formation from the East. The Rhine and Meuse continued building on the Kedichem Formation towards the end of this period, in the mean time depositing gravels and coarse gravel containing sands locally in the Sterksel Formation (Bavelien).
The Mid-Pleistocene started with a period encompassing four glacial's and four inter-glacial's. The supply of sediment shaping the Enschede formation seized because of changes in the Eastern rivers drainage pattern at the Baltic and North-German places of origin. The flow areas of the Rhine and Meuse shifted apart due to tectonic influences leaving the Rhine to shape the Urk Formation and the Meuse to continue shaping the coarse sanded Veghel Formation (Cromerien).
The subsequent period consisted of a glacial and an inter-glacial. Permafrost existed in The Netherlands during these Mid-Pleistocene glacial’s. New Formations formed as a result of a glacier, just reaching the Wadden area in The Netherlands. Channel-like depressions, filled with fine sand and dense clay, existed in the soil, probably as a result of tunnel forming under the ice cap.
The Eindhoven Formation also contained drift-sand and peri-glacial sand together with peat and brook sedimentation. The Rhine and Meuse continued building on the Urk and Veghel Formations. The land was covered in Forest’s, at first mainly pine, gradually more deciduous forests appeared, only to disappear again towards the end of the period, giving way to flora adaptable to the cold. The construction of the Urk, Veghel and Eindhoven Formations continued steadily (Holsteinien).
The subsequent period consisted of three stadials and two interstadials, each with their own vegetation. This is also the time that offshoots from the Scandinavian glacier reached into the Centre of The Netherlands. Barrage walls and deep glacial basins where formed. The great rivers where forced to flow ahead of the ice towards the west. The Drenthe Formation was shaped underneath and alongside the ice. Together they shaped the Kreftenheye Formation, during the later Saline Period.
Eological deposits shaped the Eindhoven Formation, also containing peri-glacial brook deposits and peat, to the South of the ice, an area without vegetation. The soil formed during the Holsteinien has largely disappeared due to scattering or glacial erosion (Saalien).
Raised temperatures hailed in the start of the Late Pleistocene. The Eem Formation was formed along the Dutch shores. Coastal cliffs appeared in the West in places where the clay from the Kreftenheye Formation protected the underlying sands. In the interior of the Netherlands, extensive moor land developed, The Asten Formation. The great rivers continued building up the Kreftenheye Formation (Eemien).
The Late Pleistocene ended when the Scandinavian glacier was split in two. That was, for the time being, the last time permafrost existed in the Netherlands. Although the ice no longer reached The Netherlands, the temperature reached very low values during the stadials, even lower than during the Saline. The Netherlands turned into cold, sandy high ground, relative to today, an arctic dessert. Forceful winds displaced great amounts of sand and silt across the dry planes, these turned into windborne sand deposits and loess of the Twente Formation, also containing peri-glacial brook deposits and some peat. This Formation is lothologicaly hard to distinguish from the older Eindhoven Formation. The Rhine and Meuse formed the upper deposits of the Kreftenheye Formation, culminating in isolated river dunes. Likewise, many of the Eemien soils disappeared during this period (Weichselien).

3.2 Holocene

The Holocene was distinguished by a climate change setting on the melting of the Land ice cap. From a geological point of view the Holocene was a relatively short period, though long enough to be of great importance to the Netherlands. Bog, maritime clay areas, dunes, the river deltas and the vast (former) moor land derive from this age. During this period humans became a geological factor.
The Holocene is an interglacial from a geological standpoint, interglacial's lasting for approximately 20.000 years, meaning another ice-age is to be expected in another 10.000 years.
The sub-division of the Holocene is based on the change in the flora. The Holocene can be sub-divided into the following periods:

Holocene (10.000 – present):	Early:	Pre boreal (10.000 – 9.000)
		Boreal (9.000 – 8.000)
	Middle:	Atlantic (8.000 – 5.000)
		Sub boreal (5.000 – 2.900)
	Late:	Sub Atlantic (2.900 – present)

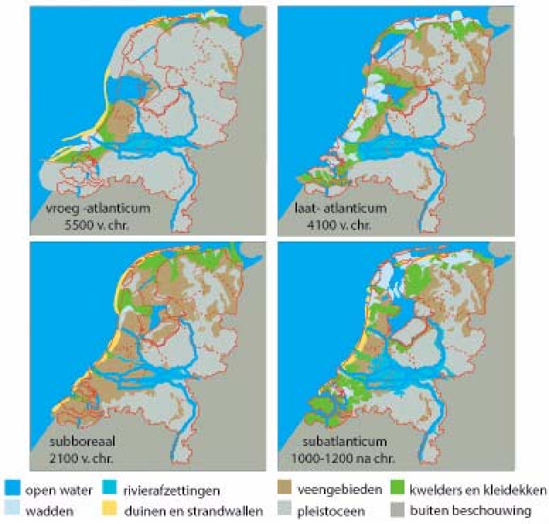


Figure 5 Landscapes of The Netherlands during the Holocene

Sea-level movements

The movements of the sea levels during the Holocene resulted, for the Netherlands, in a rising of the sea level.
The rising of the sea level was a result partly of the melting of the Land-ice caps and partly of the tectonic movements. The elevation of the Ardennes and the slate mountains of the Rhine, in conjunction with the reseeding North-sea basin, determined the situation of the coast line and with that the relative rising and lowering of the sea level. The average rise over the last 2000 years is approximately 0,02 m per century. A rise of 0,15 m per century, as can be observed now, coincides with the climate swings melting the polar ice caps and mountainous glaciers and the expansion of the warming sea water.
The Holocene transgression fazes appear in cycles of 300 to 600 years. Four of such cycles existed between 8000 and 3700 BP.
The genesis of the Holocene deposits in The Netherlands is marked by six formations linked to five sedimentary environments.

Westland Formation:	Coastal area (Tide influence, coast- and per marine area)
Betuwe Formation:	River area (no influence of tide)
Formation of Kreftenheye:	River area (older than BF, weaving river system)
Formation of Singraven:	Creek valleys
Formation of Griendtsveen:	Peat
Formation of Kootwijk:	Land dunes

The Netherlands was a somewhat undulating sand plate with a slight elevation towards the North-West with protruding lateral moraines. The permafrost disappeared from the soil restoring the permeability of the soil. In places with higher levels of ground water, peat formed (Pre boreal).
Subsequently the braided system of the great rivers changed into a meandering system of rivers with a landscape fully covered in vegetation. The coastal planes became moist, developing bog. The climate remained largely the same (Boreal).
The sea level rose to approximately its current level. The first beaches and dunes formed with lagoons behind them with mudflats and reed-swamps. A number of breaches marked this phase, so-called transgressions. During these transgressions, maritime sand and clay was deposited with intervals of peat reclaiming ground towards the sea. Locally sand dunes appeared due to human intervention. The climate reached an optimum (Atlanticum).
The maritime activity decreased, reducing the amount of breaches. The peat was able to expand and the lagoons where taken over by bog. In the worst drained areas of the bog, moor land started to develop evolving into large peat cushions. Even de badly drained parts of the Pleistocene deposits in the hinterland developed large areas of moor land. Sand drifts appeared only in the beginning of this period. The human influence, through cattle breeding and agriculture becomes noticeable (Sub boreal).
The current period is marked by human activity, maritime influences and reshaping of new, steep dunes. Many moor land areas where drained of water and destined for agriculture or mined for turf. The, not always intentional, lowering of the Greenfield level initiated new breaches in the lower part of The Netherlands, allowing maritime sand and clay sediments to be deposited. The intruding seawater and meandering river system where controlled through the erection of dikes. The climate went through some warmer and cooler fazes (Sub Atlantic).

4 Soil Investigation

Various types of soil investigation were conducted at the test site in the north of The Netherlands in Wolvega (Friesland). The soil investigation was conducted to make sure the soil was homogeneous enough at the test site for the test facility to be positioned there.

Firstly the terms CPT and SPT will be explained.
CPT (Cone Penetration Test). A cone is driven into the soil with the aid of steel rods. The cone resistance q_c is defined as the vertical soil pressure against the top of the cone during penetration of the soil, divided by the surface area of the cone (10 cm^2). The friction is the total amount of shaft friction of the sleeve divided by the surface area of the sleeve (150 cm^2). The friction number is the friction resistance divided by the cone resistance, multiplied by 100%. An electrical cone is used. The advantage of this cone is that the resistance and force are measured continuously. The cone consists of three parts that can move a little in relation to each other. The sounding results generate a clear impression of the composition and strength characteristics of the soil. Layers consisting of clay have a lower resistance than layers consisting of sand. Differentiating, being made even clearer when the friction is measured. The friction number of clay is being much higher than the friction number of sand through low cone resistance and high friction.

Soil type	Friction number	Resistance [q_c]
Sand medium – coarse	0,4 %	
Sand fine – medium	0,6 %	5 – 30 MPa
Sand fine	0,8 %	
Sand, Silty	1,1 %	
Sand, Clayey	1,4 %	5 – 10 MPa
Sandy Clay of Loam	1,8 %	
Silt	2,2 %	
Clay, Silty	2,5 %	
Clay	3,3 %	0,5 – 2 MPa
Clay, Humus	5,0 %	
Peat	8,1 %	0 – 1 MPa

Table 2 Friction number and cone resistance for various types of soil

The possibility exists to equip the cone with a water pressure meter, the piëzometric cone. Because not much is known about the relation between the changing water pressure, by means of the inserted cone, and the initial water pressure, the piëzometric cone doesn't generate much usable information. However a clay lens within a sand pocket can be revealed with the use of a piëzometric cone. The advantage of a CPT is the continuous data. The disadvantage being the great force put on the measuring device making it necessary to use a lot of weight or anchoring the device to the ground.

SPT (Standard Penetration Test) is mainly used in the Anglo-Saxon countries. The SPT consists of a cylindrical sample tube that, with the aid of a standard weight, is driven into a borehole. The number of blows necessary for the complete insertion of a 300 mm long sample tube is monitored, blow number (N).
The advantage being that a lighter construction can be used. A disadvantage being that the results cannot be reproduced accurately and that there is a smaller distinguishable difference between sand and clay, see table 2.

Sand		Clay	
N	Density	N	Density
<4	Very Loose	<2	Very Soft
4 – 10	Loose	2 – 4	Soft
10 – 30	Normal	4 – 8	Normal
30 – 50	Dense	8 – 15	Stiff
>50	Very Dense	15 – 30	Very Stiff
		>30	Hard

Table 3 Interpretation of the SPT according to Terzaghi and Peck

4.1 Materials used

Different materials where used for the soil investigation. This material is described in the next sub paragraphs.

4.1.1 CPT truck

A CPT truck is a standard 6x6 truck with an overall weight of 21 tons. The truck is equipped with a 200 kN sounding installation, that can be used to its full extend. The truck needs to meet specific requirements. The requirements being:

- A dump truck chassis
- A vertical exhaust behind the cabin
- In order to reach the right weight distribution in combination with good terrain accessibility, the right wheelbase has to be chosen.

Alterations to the truck are necessary but are usually limited to relocating the batteries, the fuel tank, the air pressure tanks and the extension of the primary chassis.
The primary chassis is weighted down with a sub chassis made of UNP300 profiles. The sub chassis forms a stable frame and generates the majority of the required extra weight. The base of the hydraulic system consists of a hydraulic pump operating in two phases. The hydraulic 200 kN penetrometer press is a double cylinder with a compression reach of 1350 mm. During the manual sounding the sounding speed can be adjusted variably from 0 to 24 mm/s. In the semi-automatic mode the speed is 24 mm/s. The uncharged press speed can be adjusted from 0 to 165 mm/s and the draw speed from 0 to 125 mm/s. The CPT truck is furthermore equipped with hydraulic legs that can lift the truck off the ground and level the truck along the horizontal, important to sounding.

4.1.2 Hand Sounding Device.

The hand-sounding device is a construction weighing in at 100 kg. Because of its low weight the construction can be moved by hand. To prevent the construction from lifting off the ground during sounding, ground anchors need to be used. Furthermore the hand-sounding device uses electricity, needed for the electronic cone as well as the printer that visualises the generated output. Otherwise it is an ordinary CPT. See appendix 5 *Hand Sounding Equipment* for the visuals.

4.1.3 Dynamic Sounding Device Pagani DPML 30/20

Dynamic sounding is a form of SPT, driving a cone instead of a sample tube into the ground. It is a penetrometer with a dynamic hammer that drives a cone into the ground with the aid of steel rods, following the International Standard Procedure. The dynamic sounding device is a portable device with the following properties:

- Cone with a surface area of 10 cm^2, an angle of 60^0 and a diameter of 3,56 cm.
- Steel rods with a length of 100 cm, a diameter of 2 cm and a weight of 2,4 kg.
- A hydraulic hammer with a blow weight of 30 kg and a drop height of 20 cm.
- An electric motor that powers the hydraulic pump.
- A jack to retract the inserted rods from the soil.

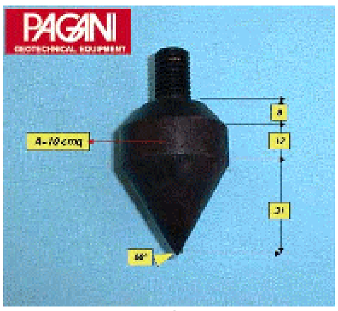


Figure 6 Pagani Cone DPML 30/20

See appendix 6 *Pagani* for the re-formulating of the N_{20} values into the N_{spt} values.
A disadvantage is that the output is hard, if not at all, comparable to the other penetrometers, because of:

- The effect of friction on the rods
- The average efficiency of the dynamic hammer.

4.1.4 Dynamic Sounding Device Stitz DPL

The Stitz DPL is a lot more manageable then the Pagani, due to the lesser weight of the hammer, reducing the overall weight to 19 kg. The Stitz uses a cone with a smaller surface area raising the effect of friction on the rods. The weight of the hammer is 10 kg. The differentiating ability of this device is a lot less. The device does meet the demands set in the DIN4094 for dynamic sounding devices.

4.1.5 Observation Well

An observation well is a tube with a filter at one end that is inserted to a certain depth. The filter allows water to enter the tube up to the height of the water level present in the sand layer the tube is inserted to. Two tubes where installed, one in-between the control test and the Long-term test at locations 2.1 and 1.4 see appendix 7 *CPT's Fugro*, filter depth is at 4 m below ground level, installed by FIBV. The graduates placed the other next to the Long-term test at location 1.1. The filter is situated at approximately 1,2 m below ground level.

4.2 Execution of the Soil Investigation

This paragraph will elaborate on the execution of the soil investigation. Each stage of the soil investigation will be another paragraph.

4.2.1 Test Site Orientation

During the first visit to the test-site location on august 2nd, 2005 Wiertsema en Partners executed four soundings, with friction and resistance measurement. The soundings where done to determine the soil condition at the test site location. The soundings where done with the use of a 21-ton sounding truck. The results of these soundings can be found in appendix 8 *CPT's Wiertsema en Partners* as well as the sounding locations. The results of these soundings show that the composition of the soil at the test site is heterogeneous. For a better look at the soil composition at the test site more soundings would need to be done. The average sounding shows the following composition of the soil of the test site:

Top [m]	**Bottom [m]**	**What**	
0	-1.00	Sand	Above water table
-1.00	-2.20	Sand	Underneath water table
-2.20	-3.50	Loam	
-3.50	-9.00	Sand	

Table 4 Average result of the CPT done by Wiertsema en Partners

4.2.2 Second Soil Investigation

The second soil investigation at the test site consisted of 23 soundings at up to 4 m deep, with friction, installing of an observation well, performance of 2 hand borings and taking of un-disturbed samples. This research was done at September 27th and 28th, 2005.
The results of this investigation can be found in appendix 7 *CPT's Fugro*. The overview also contains the locations of the earlier soundings, the hand boring locations as well as the locations of the observation well.

These soundings too, show that the soil is heterogeneous, but because of the smaller grid of 8 x 8 meter a better determination of the best locations for the test site could be made. Laboratory testing was to be done on the soil samples by FIBV. These tests entailed the following aspects:

- Visual inspection boring logs.
- Eight grain size diagrams.
- Determining the Atterberg limits.

Eventually the Atterberg limits where determined at the laboratory of Technical Earth Sciences because FIBV had claimed that it couldn't be done. The results of these tests can be found in appendix 9 *Laboratory Tests*. This shows that the loam contains a lot of sand but is very much present in the soil and that the Atterberg limits can be determined.

4.2.3 Hand Soundings

Two hand soundings were done on the day (November 7th, 2005) prior to the start of construction of the test facilities. These hand soundings were done in the centre of both the short-term test site and the long-term test site. The results of these hand soundings can be found in appendix 5 *Hand Sounding Equipment*.
Hand soundings were done on February 9th and March 16th, 2006 to determine if improvement of the soil has taken place after the injections.
The hand sounding results show a great similarity with the earlier soundings done by Wiertsema en Partners and Fugro.
The soundings after the injections show that there is little to no improvement of the strength of the loam. But there is some improvement in the sand layers.

4.3 Soil Investigation as a Measurement

During the period that levelling measurements were done several dynamic soundings were done as well. These soundings were performed using two different kinds of dynamic sounding devices. Each comparison that is made is made with measurements from the same kind of dynamic sounding device. The Pagani was used on November 14th and 17th, 2005. The measurement on November 17th was taken right after the injection of the two-component resin. The Pagani was also used right before excavation of the short-term test on February 12th, 2006. The chapter on measurements details on the results of these measurements.
The Stitz was used before and after the second batch of injections on the second foundation strip (locations 3,3 and 3,4) of the short-term test. The soundings were done on December 15th, 2005 and January 2nd, 2006 respectively. The chapter 9 *Results of the Test* details on these results as well.

4.4 Soil Investigation Conclusions

The overall result of the soil investigation is that the soil composition at the test site is very Heterogeneous. The test facilities were placed at locations where the construction of the soil showed the least amount of differences, the chapter on construction of the test facilities will detail on this. The soil investigation that was outsourced went well, with the exception of the laboratory testing.

5 Laboratory Tests

During the second time Soil research was done, samples where taken for laboratory testing. This research consisted of determining the grain size distribution, the Atterberg limits and a visualisation of the samples taken.

5.1 Grain Size Distribution

The physical properties of soil are usually determined by analysing the grain size distribution. The result of this analysis is called the sieve curve. This term is derived from the visual representation that can be made of the different fractions.
Firstly the total dry mass of a soil sample is determined. This sample is sieved through a 2 mm sieve, after which the fractions smaller than 2 mm receive further treatment. This soil undergoes a peroxide rinse to oxidise the organic material present. The carbonates are removed by boiling the soil in Hydrochloric acid. The treated soil is than sieved through a 35 µm sieve. The particles larger than 35 µm are dried, weighed and divided by sieves with various mesh widths into several fractions. These fractions are weighed. The particles smaller than 35 µm however are too small to be sieved. These fractions are determined with the aid of sedimentation, by which the sedimentation velocity is the determining factor, the hydrometer test.
The sieves used to determine the sieve curve have the following mesh widths:

2 mm	1 mm	0.5 mm	0.25 mm	0.125 mm	0.063 mm

Table 5 Sieves used for the determination of the grain size diagram

The fractions smaller than 0.063 mm are not specified any further. The grain size distribution results can be found in appendix 10 *Sieve-curves.*
The tested soil samples contain a lot of sand; the loam and/or clay percentage varies in eight of the examined samples from 5 to 35%.

5.2 Atterberg Limits

The Atterberg limits are determined by means of two tests. The Casagrande Method and the Plastic Limit Test.

5.2.1 Liquid Limit with Casagrande Method

De liquid limit (LL) is determined with the aid of the Casagrande Method. This method uses a cup, see figure 7. De cup is filled with a mixture of soil sample and distilled water. A small slit is drawn across the sample. Next, the cup is tapped by a mechanism in the bottom plate of the test facility. This has to be done at a pace of 2 taps per second. When the slit in the sample closes between 15 and 35 taps, the sample can be weighed and placed into an oven to dry. After 24 Hours in the oven the sample can be weighed again.
The moisture content of the sample is determined using the following formula:

$$W_n = \frac{\text{water mass}}{\text{mass of oven dried sample}} \cdot 100\%$$

Figure 7 Casagrande device

The data of the different tests (the amount of taps) on samples from the same boring are marked in a graph, the moisture content on the y-axle and the number of taps on the x-axle. A linear line is drawn across the marked points from the tests done. From this graph, the liquid limit (LL) can be determined by reading the moisture content at 25 taps.

5.2.2 Plastic Limit Test

The plastic limit is determined by means of the plastic limit test. For this test a 20 g sample is used, divided into two 10 g balls, each of which has to be divided into four equal parts. These parts have to be rolled into cylinders with a diameter of 3mm. Small tears are allowed to be present in the roll but the roll is not allowed to fall apart. Several attempts have to be undertaken to reach the stage between the roll staying intact and falling apart. When all four rolls are finished they are placed on a tray and weighed. The tray is then placed into an oven. The same procedure is performed on the other ball. After 24 hours when the cylinders are dry they are weighed again. Subsequently the moisture content is calculated. The moisture content of the two samples is not allowed to differ more than 0,5 %. The numbers of the two samples are middled and rounded off to the nearest whole number.

5.2.3 Plasticity Index

Determining the Plasticity Index is done using the LL en PL, in which PI = LL – PL.
The results of these tests vary quite a bit; Fugro was, according to themselves, not capable of determining these values. Two trial determinations where done at the laboratory of Technical Earth Sciences, yielding the following results:
The liquid limit (LL) for B1 is 18, with a plasticity limit of 13 making the PI = 5.
The liquid limit (LL) for B2 is 19, with a plasticity limit of 12 making the PI = 7.
See Appendix 9 *Laboratory Tests* for calculations.

5.3 Bore Logs

The bore master produces visuals of the bore logs. He describes what is found at which depths and what it looks like; this data is then reviewed in a laboratory. Furthermore the volumetric weight (wet and dry), moisture content, porosity, degree of saturation and the un-drained shear stress are determined. See Appendix 11 *Bore Logs.*

5.4 Laboratory Test Conclusions

The samples selected by Fugro in order to determine the grain size diagram are illogical, considering the construction of the bore logs. The results as depicted in the grain size diagrams show a good quality soil.
The two trial determinations show that, contrary to Fugro's claims, determining the PI is possible. The data from these determinations show that it has to be possible for Uretek to use their method in these types of soil. The visualisation of the bore logs was done very accurately but doesn't match the executed CPT's. However, the heterogeneous soil has to be taken into account.

6 Settlement Prediction

After the first soil investigation, a prediction was made towards the expected settlements. This was done after the results from the soil investigation where known, see appendix 8 *CPT's Wiertsema en Partners*.
The soil construction as explained in Chapter 5 *Soil Investigation* was used for the prediction of the expected settlement. The calculations where done on an instant load of 50 kPa and 100 kPa. Furthermore the calculations where done on 50 + 50 kPa
The following parameters where used for the calculations:

Level	Top	$h_{T,layer}$	[m]
	Middle	$h_{M,layer}$	[m]
	Bottom	$h_{B,layer}$	[m]
Pressure	Top	$\sigma_{T,layer}$	[kN/m^2]
	Bottom	$\sigma_{B,layer}$	[kN/m^2]
Specific Weight		γ	[kN/m^3]
Soil Pressure	Middle	$\sigma_{M,layer}$	[kN/m^2]
Water pressure	Middle	$\sigma_{W,layer}$	[kN/m^2]
Effective soil pressure	Middle	p_0	[kN/m^2]
Increase of eff. soil pressure		Δp	[kN/m^2]
Coefficient of volume compressibility		C	[-]
Settlement		z	[m]
Total time		t_e	[s]

The calculations are done for each layer using the following formulas. The total settlement is the settlement of each layer combined.

$$h_{M,i} = \frac{h_{T,i} + h_{B,i}}{2}$$

$$\sigma = \left|h_{M,i}\right| \cdot \gamma$$

$$p_0 = \sigma - \sigma_w$$

$$\Delta p = \left(\frac{B}{h_{M,i}}\right) \cdot Q$$

$$z = \left(\frac{\left|h_{B,i} - h_{T,i}\right|}{C}\right) \ln\left(\frac{\Delta p + p_0}{p_0}\right)$$

Delta p is the width of the foundation divided by the width plus the distance of the bottom of the foundation and the centre of the calculated layer. The denominator is in this case because of the inclination of 2:1 equal to the depth of the centre of the calculated layer.

These formulas applied the following values:

Q = 50, 100 or 50+50 kPa
C = 1000 (sand) or 86 (loam)

According to the theory, the following settlement is to be expected underneath the test facilities:

Q = 50 kPa	→	z = 0,0054 m
Q = 100 kPa	→	z = 0,0095 m
Q = 50 + 50 kPa	→	z = 0,0075 m

See appendix 12 *Settlement Prediction* for the calculations that where done.

7 Test Facility Design

This chapter will elaborate on the design of the test facility. The first paragraph will explain the choice of a full scale test facility followed by an explanation in paragraph two of the steps taken. The third paragraph finally, is divided into sub-paragraphs. This subdivision is made according to the parts making up the test facility.

7.1 The Reason for a Full Scale Facility

The choice of constructing a full scale test facility was made because of the wish of doing tests as realistic as possible. This reduces the chances of making mistakes, meaning that no scaling errors can be made. Meanwhile injecting underneath a scale model is very hard, if not impossible, to do because there is a chance of a blow out. A blow out is a situation where the injected material escapes towards the ground level. In that case high swelling pressures will not occur. It has been decided that a full scale test facility is the best and easiest approach to reality.

7.2 Types of Test Facilities

The test program contains three test facilities, each of which has its own purpose. Ahead of the first load being put on the facility, hand- and dynamic soundings are made to get a clear picture of the composition of the soil and to find the best (mostly homogeneous) location of the test facility. The first loading stage will generate a foundation pressure of 50 kPa, which can be considered as a standard foundation pressure of normal houses. The first loading stage consists of the weight of the facility itself and an applied load of big bags filled with sand. The second loading stage consists solely of big bags filled with sand and generating a foundation pressure of 100 kPa in total, coinciding with 1 kg/cm^2. The final foundation pressure will match the pressure of a regular low-rise building.
The choice for three different testing facilities was made for the following reasons:
The Reference Test is the test that will not undergo any injections. This test will be used to compare the other two test facilities to. The construction of the Reference Test Facility will be exactly the same as that of the other two test facilities.
The Short-Term Test is one where, after placing the first loading stage, an injection will be performed after which the soil beneath the foundation beam will be excavated and a visual inspection will be made of the propagation of the injected resin. In the time span between the injection and the excavation, measurements will be taken of the amount of settling taking place. This time span will cover about three months.
The Long-Term Test has the same objective as the Short-Term Test only covering a longer time span. This time span will cover 2 to 3 years.

7.3 Test Facility Construction

The test facilities consist of two concrete foundation strips that are dug into the ground. A construction bearing the loads will be placed upon these strips. This construction consists of three steel profiles bearing three dragline floor plates. The loads consisted of big bags filled with sand and the test facility's own weight. The loads generated the final foundation pressure in two stages.
The reason for using two loading stages is to create a possibility to determine the stiffness of the soil. This is done by comparing the difference in settling after placing the first load and the amount of settling after placing the second load. The strength of the soil was determined with the aid of dynamic soundings. The choice for dynamic soundings has been made because abroad the use of dynamic soundings is common sense. The differences between the soundings before and after the injection determined the improvement of the strength of the soil.

Construction schedule of the test facilities:

- Soil investigation: executing CPT's to locate the most homogeneous position for the test facility.
- Soil investigation: taking hand soundings of the precise positions of the test facilities.
- Construction right up to the first loading stage: to be elaborated upon in the chapter on test facility realisation.

- Level measurements of the test facilities.
- Soil investigation: taking dynamic soundings using the Pagani DPML 30/20.
- Injecting the two-component resin according to a pre-determined injection plan.
- Soil investigation: taking dynamic soundings using the Pagani DPML 30/20
- Level measurements of the test facility.
- Placement of the second loading stage.
- Soil investigation: taking dynamic soundings using the Pagani DPML 30/20.
- Level measurements of the test facilities over a time frame of three weeks.

The following aspects have a bearing solely on the Short-Term Test Facility;
- Disassembly of the short-term test facility.
- Drainage
- Excavation of the injected resin.

7.4 The Test Facility Elements

Each test facility consists of the same parts. These parts are designed, calculated, constructed or ordered. These parts are covered in the next paragraphs.

7.4.1 Foundation Strip

The foundation strip was designed to mimic reality. Since, in reality, the foundation strips, under which Uretek injects, are about 600 mm wide. The choice was made to use this width as well for the foundation strips of the test facilities.
The minimum length of the strips needed to get a good impression of the injections in reality is 5000 mm, this to facilitate enough space to inject under. In order to, amongst other things, save time a choice was made to use pre-fabricated elements.

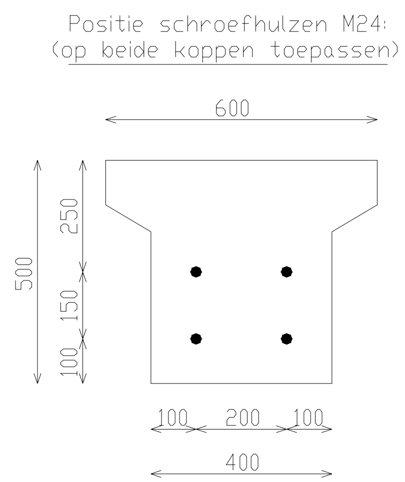


Figure 8 Front view of the foundation strip

This choice does limit the range of measurements that can be used for the foundation strips. Therefore the following minimal measurement requirements have been set.

- Length 5000mm minimum
- Width 600 mm minimum, 700 mm maximum
- Height 500 mm minimum

Ringvaart B.V. with their T-bar can meet these sizes. However, the bar has to be used upside down making the widest part end up at the bottom. The realisation has been discussed with the designers from Ringvaart, see figure 8.
18 open vertical holes have been created in the body of the foundation strip, needed to make the dynamic soundings and the injections beneath the foundation strip. These holes have a

diameter of 50 mm and are placed evenly spaced along the length of the foundation strip. See appendix 13 *Detailed Schematic* for a detailed schematic of the bar.
The foundation strips were made to measure with 18 slots along the length and 4 Screw-thread housing per short end of the foundation strip. These housings will be used to affix an UNP-profile, this to be explained in the paragraph on steel profiles.
The designers from Ringvaart B.V. designed the concrete foundation strips; a total of 6 strips were ordered.

7.4.2 Steel Profiles

Two types of steel profiles were used in total per test facility, UNP240 and HE260B.
The UNP240 profiles were used to fixate the two foundation strips. Fixation occurred using 4 bolts divided over 2 slots of the profile, see figure 9.

Figure 9 Slots in UNP240 profiles

So per profile, 8 bolts and 4 slots are used. The slots have been carved by a local black smith. Per test facility two UNP profiles were used, one at each short end. Furthermore three HE260B profiles were used, these profiles were chosen after having calculated the bend and rotation. The reason for using a B profile being that its width equals its height, minimal sing the chances of tilting of the profile and the pressure within the contact surface being enough, see appendix 14 *Calculations for the Test Facilities*. The calculations were done on the profile bearing the heaviest load, the centre profile. The calculations show that the values are within the material boundaries.

7.4.3 Dragline Floor Plates

To guide the forces, generated by the big bags, through the steel profiles onto the concrete foundation strips, dragline floor plates are placed on top of the steel profiles. The calculations for the maximum load are different than those for the steel profiles, see appendix 14 *Calculations for the Test Facilities*. The calculations show that the dragline floor plate dimensions were oversized, but they were available, so why order other material.

7.4.4 Big bags

The added loads consist of sand filled big bags. Big bags are made of polypropylene and have the following dimensions; 0,91 x 0,91 x 1,10 m^3. The surface dimensions of 0,91 x 0,91m^2 were chosen because, once filled, they cover a surface of 1,00 x 1,00 m^2; see figure 10 for an example of a filled big bag. The big bags were filled with roughly 0,81 m^3 of sand. A total of 44 big bags were used per test facility divided over two loading stages.

Figure 10 Filling up of the Big bag

7.5 Positioning of the Test Facility

The determination of the exact position of the test facilities is done after the examination of the soil investigation. The soil investigation showed that the results of the soundings with numbers 20, 22 and 23 are similar. These soundings show the following results of the structure of the soil;

Top [m] NAP	**Bottom**	**What**
0	-1.75	Sand
-1.75	-3.25	Loam
-3.25	-4.00	Dense sand

Table 6 Average result of the CPT's around the position of the test facility made by Fugro

So the positions of the different test facilities are;

Short Term Test	Between CPT's 20 and 21
Long Term Test	To the right of CPT 23
Reference Test	Between CPT's 22 and 23

For the real results of the CPT's see appendix 6 *CPT's Fugro*. And for the exact positions of the test facilities see appendix 15 *Test Facility Positions*.

8 Execution

This chapter details on the construction phase of the different test facilities and the execution of the injections. This chapter contains three paragraphs; the first one deals with the positioning of the test facility, the second paragraph deals with the construction of the test facility and the third paragraph describes the execution of the injections in detail.

8.1 Positioning of the Test Facility

The different test facilities were positioned in relative close proximity to each other. For details of the exact positions see appendix 15 *Test facility Positions* and paragraph 7.5.
The distance between the Long-Term Test Facility and the Control Test Facility is approximately 4 meters. This distance was chosen because it was expected that the injections underneath the Long-Term Test Facility would have a small area of influence. The distance between the Control Test Facility and the Short-Term Test Facility is greater because of the necessity to excavate the Short-Term Test Facility. The excavation required extra space in order to facilitate a sloping wall for the excavation pit and room for well point drainage. The positioning of the test facilities was done on the basis of the results of the soil investigation and the preference of the owner of the terrain. Furthermore it became evident that a ditch used to run right underneath the planned location of the foundation strip of the Short-Term Test Facility at locations 3.1 and 3.2. A different type of sediment was found at the location of this former ditch compared to the surrounding area. This resulted in the relocation of the Short-Term Test Facility along a sideways path and out of alignment with the Control Test Facility, see appendix 15 *Test Facility Positions*.

8.2 Test Facility Construction

The construction of the test facilities took 4 days. This paragraph explains the construction details.

The terrain was made better accessible by levelling, where possible, in advance to digging the concrete foundation strips into the soil. This was achieved by excavating the top layer of humus from 0,1 to 0,6 m deep and filling in the excavated area with sand, creating paths. These paths ended up outside the test facility boundary at a distance of 1,5 m. Subsequently 0,60 m deep slots were dug at pre-marked positions using a laser guided digging machine. This had to be done very precisely in order to make the concrete strips fit directly level onto the soil. If large differences in height were present, the concrete foundation strips would not be able to make uniform contact with the soil. The slots were 1,00 m wide and 0,6 m deep. After the slots were dug, connecting slots were dug at either end of the already dug slots. The area in between was excavated to a depth of 0,2 m. Subsequently the foundation strips were placed in their consecutive slots using a crane, a steel four-way chain and two straps, see figure 11.

Figure 11 Foundation strip positioning

Special attention was paid to make sure that the foundation strips were laid parallel to each other and at the right distance to each other (3,80 m measured from the outside of the foundation strip at the top). The UNP Profiles were attached after the foundation strips for each test facility were in position, using a crane, a four-way chain and two straps. The UNP profiles were bolted down using four M24 bolts per foundation strip. Before placing the HE260B profiles unto the foundation strips, 0,30 x 0,40 m^2 wooden sheets were placed on the foundation strips to prevent the concrete from chipping under the heavy burden. Three HE260B profiles (each 3,80 m in length) were placed on each pair of foundation strips, one at each end and one in the middle, see appendix 16 *Detail Schematic of the Test Facility*. Three dragline floor plates, 6,00 m long, 1,00 m wide and 0,20 m high, were placed on top of the profiles, with the aid of a crane and the four-way chain. The big bags were placed on top of the floor plates after weighing them on a weighbridge in order to determine the exact amount of weight put on the test facility, see appendix 17 *Weight Determination*. The truck was weighed in empty and then again when fully loaded revealing the weight of the big bags. The big bags were filled with the aid of a hydraulic crane situated on a truck, a shovel with spoons and two people on the ground, sliding the big bags onto the spoons and guiding the big bags in place, expediting the process. The second loading stage, consisting of 24 big bags, was placed on the test facility on the evening after the injections were done. Visuals of the construction of the test facility can be found in appendix 18 *Test Facility Construction Pictures*. The next table shows the different activities in the construction phase of the test facility;

	07/11/2005	08/11/2005	09/11/2005	10/11/2005	17/11/2005	18/11/2005
Marking						
Leveling						
Fill						
Foundation ditch						
UNP						
Wood						
HE260B						
DLS						
Filling big bags						
Load stage 1						
Load stage 2						

Table 7 Execution schedule

8.3 Injections

On Tuesday, November 15th 2005, a start was made with the injections under the first beam. The first injections were made difficult by the bad weather conditions, rendering the test site barely accessible. At the start the decision was made to fill in the boreholes in the strip with resin to prevent the resin from escaping during the injections because it is easier to drill through the resin for the next injection than to drill through the concrete to create the next injection hole. Subsequently the injection tubes where placed according to the injection schedule, see appendix 19 *Injection Schedule*. When the time came to withdraw the injection tubes it became clear that this was not possible due to too much friction, prohibiting the removal of the end cap and making the injection impossible. In order to proceed, the decision was made to inject the first layer directly underneath the foundation strip, using short injection tubes reducing the amount of friction. Next the deepest layer was injected at 3,5 m below ground level followed by the layer at 1,7 m deep and finally at 2,5 m below ground level, the injections being done right after the tubes were placed; see appendix 19 *Injection Schedule* for details and the number of kilo's injected. This appendix shows that, except for two, all injections into the first layer were successful. At the two places where no or minimal injections could be done, the risk of a cross-over was too great. A cross-over occurs when the outside pressure is equal to or higher than the pressure generated by the injected material. When this happens, the resin is pushed back into the injection gun where the chemical reaction then takes place, blocking the gun.
On Wednesday, November 16th 2005, Uretek arrived with two teams, making up for time lost.

Despite the two teams, it became clear that not all planned injections could be done that day. Therefore the decision was made to approach Uretek Belgium to inject the last foundation strip on Thursday November 17th 2005.
On Thursday Uretek Belgium injected underneath the last remaining foundation strip. The difference in injecting and the amount of resin used between Uretek Holland and Uretek Belgium is substantial as well as the time used; see table and appendix 19 *Injection Schedule*.

Differences between Uretek Holland and Uretek Belgium:

Holland	Belgium
Copper tubing, diameter 12 mm	Metal Tubing, diameter 15 mm
Insertion using a hammer drill	Insertion using a hammer
When in doubt, Immediate termination	When in doubt, wait and see
Relatively view kilo's per injection	Relative high amount of kilo's per injection

Table 8 Differences between Holland and Belgium

Because, according to Resina Chemie BV and Uretek, not enough resin was injected, the decision was made to further inject underneath one of the two foundation strips of the Short-Term Test Facility. These injections were used to apply different methods. The foundation strip was divided in half and two types of injection methods were used on either half.

The methods used:

- Tight Matrix Method; the normal injection method, though injection points closer together.
- Drawn Injection Method; injecting a certain amount of kilos per meter while withdrawing the injection tube.

Furthermore, Uretek Holland switched to using metal tubing. These metal tubes are more rigid than copper, making them easier to insert and they are wider, see appendix 20 *Renewed Injection Schedule* for the visualization of the used types of injection methods.

9 Results of the Test

This chapter discusses the test results from the test facility at Wolvega (Friesland). The chapter consists of five paragraphs. The first paragraph contains a description of the measurements taken, the second paragraph discusses the results of the level measurements, the third paragraph describes the results from the dynamic soundings and the fourth paragraph contains the piquet's and the observation wells and finally the fifth paragraph details on the excavation.

9.1 Measurements

Different measurements were taken during the execution of this test. These measurements consisted of measuring the settlement of the facility as well as the dynamic soundings, determining the soil improvement.

9.1.1 Levelling Instrument

The measurements to determine the vertical position of the test facility were taken using a levelling instrument and a marker. A nut was attached to the marker, fitting into a custom-made marker pot. The marker pot consists of a 48 mm cylinder mounted on a stainless steel plate. The cylinder holds a space for the nut to fit into, see figure 12 and appendix 23 *Marker Pot*.

Figure 12Top view of the Marker Pot

The circumference of the cylinder is designed to fit into the holes of the concrete foundation strip, limiting the deviation in position, rendering the measurements as accurate as possible. The levelling instrument was used according to the following rules:
The sight with lenses consists of: an objective, a central focussing lens and a focussing ocular. Looking through the ocular, a vertical and a horizontal cross line are visible, perpendicular to each other. Above and below, two distancing lines are situated. The central focussing lens is used to focus the image. Rotating it, like on a pair of binoculars, can also focus the ocular. Focussing the ocular is done to compensate for the users vision impairment and is done only once per user.
The measurements are done by two individuals: The first carries the marker around, the second takes the measurements using the levelling instrument. After placing the levelling instrument level to the horizontal, the position of the marker is determined. When the marker is found through the sights of the levelling instrument, the height of the three cross lines is read in centimetres, using the marker. The height is read in centimetres while millimetres need to be estimated. More accurate measurements, up to a tenth of a millimetre, are possible using a more detailed scale on the levelling instrument. The marker can be placed at a distance of several dozens of meters from the levelling instrument because the sight of the levelling instrument doubles as a pair of binoculars, making the scale clearly visible and larger than the eye is capable of distinguishing. The difference in the reading on the marker represents the actual distance in meters. For example: difference = 22,4 centimetres, making the distance between the levelling instrument and the marker 22,4 meter. The difference in height between the measuring points can be determined by looking at the difference in height

reading between those two points. The accuracy of the measurements lies around 10ppm (1cm/ measured km). See figure 13 for a visualisation.

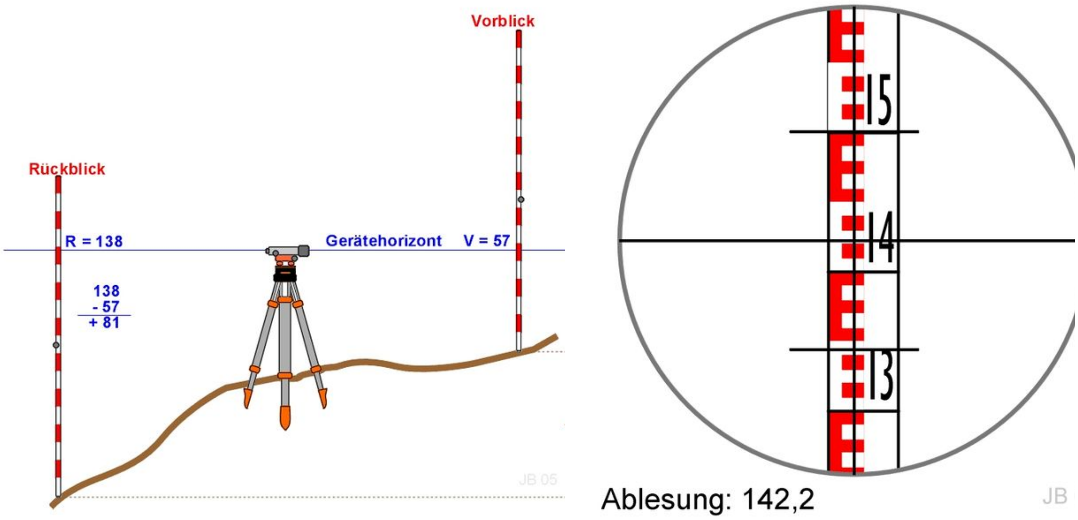


Figure 13 Levelling Measurement Equipment

9.1.2 Levelling NAP Results

The measurements are taken from a culvert in the vicinity of the test facility, see appendix 15 *Test Facility Positioning*. The culvert is marked out using a known NAP point, a well located bolt in the Schuttevaerstraat at NAP + 0,80 m. This results in the following NAP levels for the culvert:

Location	Marker	Top	Bottom	Centre	Difference	Distance [m]	Height difference	NAP [m]
Well	1.560							0.800
Lantern	1.688	1.823	1.550	1.687	0.273	27.30	-0.1280	0.672
	1.593	1.725	1.463	1.594	0.262	26.20		
Bricks black	2.010	2.101	1.918	2.010	0.183	18.30	-0.4170	0.255
	1.848	1.976	1.720	1.848	0.256	25.60		
Centre heap	1.995	2.143	1.852	1.998	0.291	29.10	-0.1470	0.108
	1.460	1.640	1.282	1.461	0.358	35.80		
Observation Well	1.115	1.220	1.000	1.110	0.220	22.00	0.3450	0.453
Fugro	1.127	1.174	1.081	1.128	0.093	9.30		
Culvert	1.689	1.766	1.610	1.688	0.156	15.60	-0.5620	***-0.109***

Table 9 Levelling measurement data of the culvert

Therefore the difference in height between the set starting- and finishing point of the level measurements equals NAP - 0,109 m.
In order to get more accurate measurements from the marker a more detailed scale was mounted on the levelling instrument. Rendering the results accurate to within a tenth of a millimetre
The dynamic soundings were done using various types of equipment, because the equipment wasn't under our own management. The dynamic soundings were done using a dynamic sounding hammer, driving a metal rod with cone, with a decimetre scale on the rod, by means of a falling weight into the soil. The number of hits necessary to drive the rod in 10 cm determines the strength of the soil; see the chapter on soil investigation.

9.2 Levelling measurements

The levelling measurements were done beginning right at the start of test facility construction. The first measurement consisted of determining a set reference point relative to NAP. The culvert mentioned earlier was selected, being located in the direct vicinity of the test facility

(roughly at 15m). The measurement and the control measurement differed by only 2 mm, well within the tolerated 5 mm margin, see Appendix 21 *Control Measurement*.
Right from the moment the foundation strips were placed, the settlement for each corner of the foundation strip was monitored and recorded. This rendered the following settlement, up to the first loading stage being placed:

Point	1.1	1.2	1.3	1.4	2.1	2.2	2.3	2.4	3.1	3.2	3.3	3.4
Difference [mm]	-5.8	-8.1	-2.7	-4.5	-7.9	-1.8	-4.2	-5.1	-4.5	-4.5	-1.5	-1.5

Table 10 Settlement up to the first load stage

These settlements are due to the self weight of the test facility. The lift created by the injections is the following:

Point	1.1	1.2	1.3	1.4	3.1	3.2	3.4
Lift [mm]	18.7	17.4	25.3	12.4	9.5	14.2	18.0

Table 11 Lift after the first injection

Point 3.3 wasn't market out because of bad weather and the impossibility of placing the marker at a point visible to the levelling instrument.
The settlement from the day after the injections is recorded in the following table, as is the net amount of lift.

Point	1.1	1.2	1.3	1.4	3.1	3.2	3.4
Settlement [mm]	-2.8	-1.2	-1.3	-2.1	-2.5	-1.5	-1.1
Net lift [mm]	15.9	16.2	24.0	10.3	7.0	12.7	16.9

Table 12 Net lift after the first injections and a foundation pressure of 100 kPa

The net lift is the created rise as a result of the injection minus the direct settlement after placing the second loading stage. At the control test the settling continued right through the whole process, yielding the following settling results:

Point	2.1	2.2	2.3	2.4
Settlement [mm]	14.2	3.9	9.4	7.9

Table 13 Total amount of settlement

This leads to the following graph, showing the net amount of settlement. The net amount of settlement is the amount of settlement starting from 0 at every point.

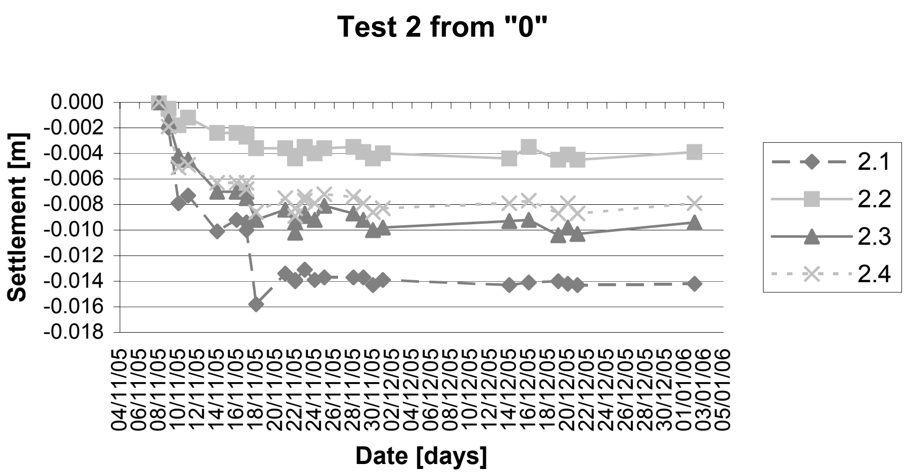


Figure 14 The settlements of the Reference Test Facility

This graph clearly shows that mistakes were made when the points were marked out. The measurements on November 18th 2005 are clearly 2 mm too low. However, it does show that point 2.1 endures a larger amount of settlement than point 2.2 though on the same foundation strip. Points 2.3 and 2.4 settle evenly at the same rate.

The settlement up to the first loading stage leads to the following graph. This graph clearly shows that in the period up to the first loading stage, a settlement between 2,4 and 10,1 mm occurred, divided over the entire area between these points. The amount of settlement according to the prediction should have been 5,4 mm. This shows that the prediction was within the same range and that the right soil characteristics where used (NEN 6740 table 1). The values within table 1 of NEN 6740 are of a conservative nature.
Furthermore, the graph shows that full consolidation isn't reached, due to the fact that the graph shows downward sloping lines.

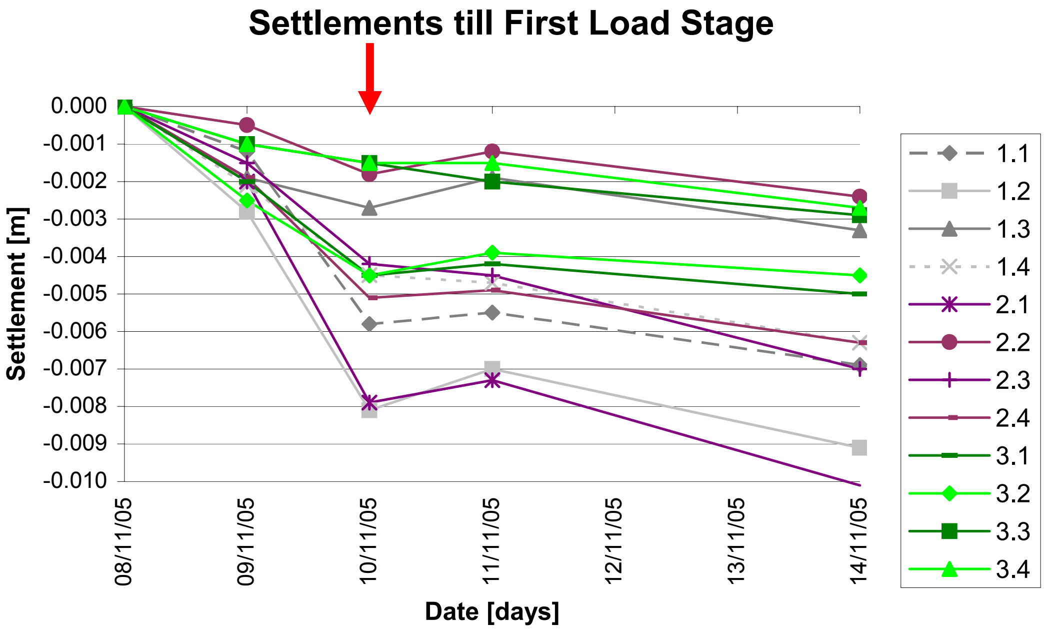


Figure 15 Settlements till the First injection; the red arrow marks the day of the first load stage

The next graph shows the amount of lift created by the UDI. It is clearly visible that lift has been achieved. Every graph contains an injected facility.

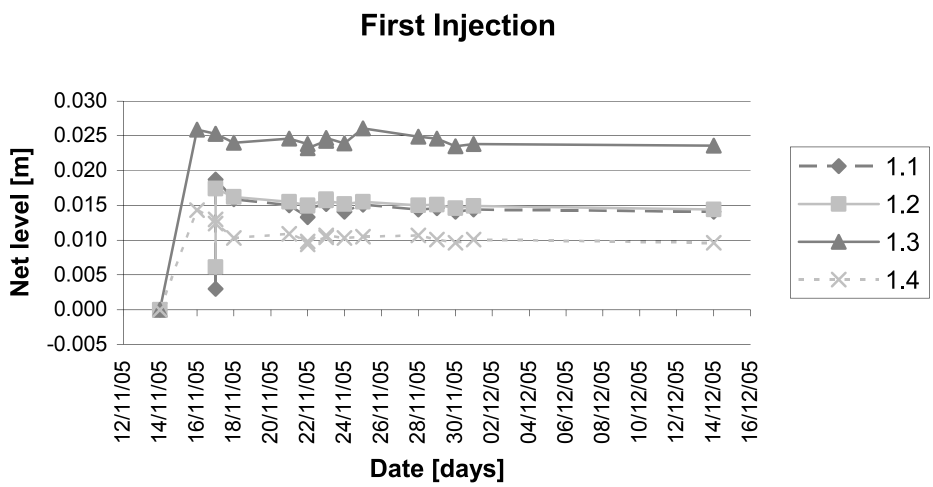


Figure 16 Lift after the first injection Long Term Test Facility

This graph clearly shows that there are large differences in settlement at the various points. Both Uretek Belgium and Uretek Holland injected this facility. The foundation strip injected by Uretek Belgium at points 1.1 – 1.2 is lifted evenly according to the measurements. The foundation strip injected by Uretek Holland shows an uneven amount of lift between either ends of the foundation strip.

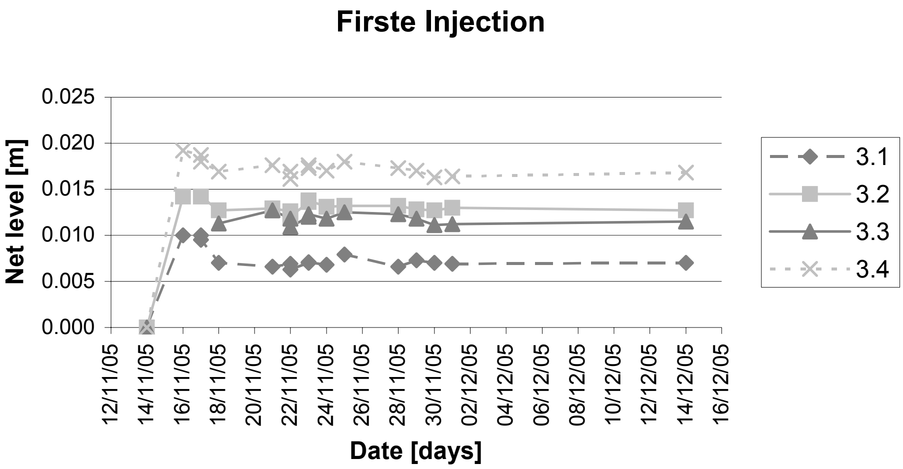


Figure 17 Lift after the first injection Short Term Test Facility

The lift created in this test facility ranges from 10 to 20 mm. Despite not being able to measure at point 3.3 directly after injecting, the assumption can be made that the amount of lift was 13 mm. This can be checked against the settlement process after injecting.

After placing the second loading; the test facilities clearly show a further amount of settlement. This was 2mm for the Reference Test and 2 to 4 mm for the injected facilities. It is likely that part of the settlement was created by excess pore pressure due to the injections. Measurements have demonstrated that the consolidation process is accelerated by the injections, as can be derived from the difference in the course of the settlement lines between the injected facilities and the reference facility. The settlement line for the Reference Test Facility continues to drop for a period while the settlement lines for the injected facilities, after the first settlement, due to the second loading stage, curve towards the horizontal showing some secondary effect at best.

The decision was made however to further inject. Foundation strip 3.3 – 3.4 was chosen for these extra injections. This was done to be able to immediately visualize the differences between the drawn injection method and the tight matrix injection method because this facility was to be excavated. Both methods generate their own amount of lift, made visible in the next graph for foundation strip 3.3 – 3.4.

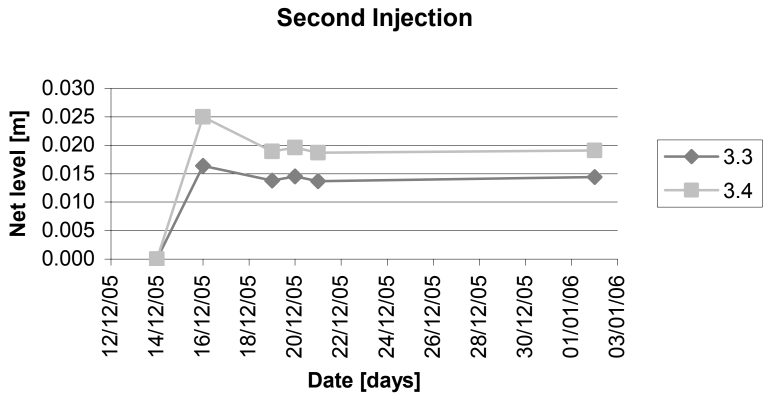


Figure 18 Lift of the second injection

The net lift is displayed, in other words, the lift as measured from 0 for both points.
The graph clearly shows what the differences in lift are, knowing that the same amounts of resin where used underneath both foundation strips. The lift created with the tight matrix injection method (at 3.3) is about 9 mm less than the lift created by the drawn injection method (at 3.4). The settlement also differs for both methods. The amount of settlement for the drawn injection method is greater than the amount of settlement for the tight matrix injection method, see the next graph.

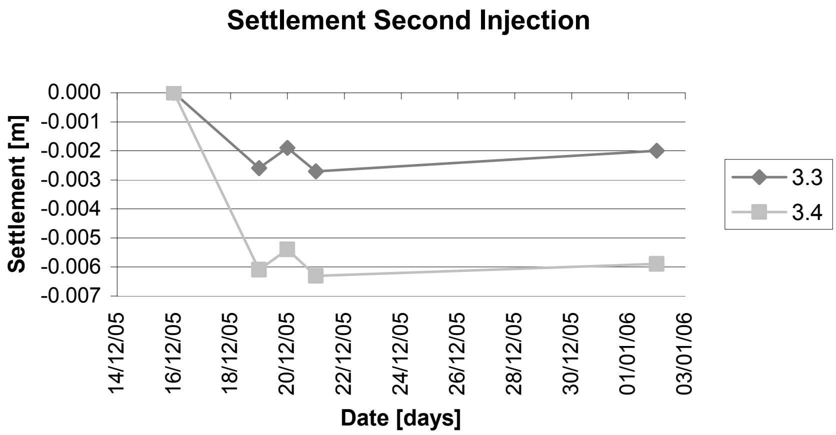


Figure 19 Instantaneous settlements after the second injection

The difference in the amount of settlement is a factor 2.5. The total amount of lift for the Tight Matrix Injection Method is 15 mm and for the drawn injection method 19 mm.

9.3 Dynamic soundings

Different dynamic soundings were executed to see if the bearing capacity of the soil was improved by the injections. Due to the fact that none of the funding companies owned a dynamic sounding device, the dynamic sounding equipment from Uretek Belgium was used as well as rented equipment.

9.3.1 Dynamic Sounding Devices

Two different devices were used, making any comparison between the two impossible. The results of the different soundings can be seen in Appendix 22 *Dynamic Soundings*.
The results of the dynamic soundings do not deliver a conclusive answer to the question of the soil being improved. Some parts of the soil show improvement while other parts show a

decline in soil strength. One of the problems with sounding, static as well as dynamic, is that the soundings can never be performed at exactly the same location, revealing the differences in the soil. On the location of this test it is hard to say, due to the heterogeneous nature of the soil, if the soil was affected by the UDI. It is likely that the dynamic soundings of November 17th 2005 were performed too soon after the injections were done. Contrary to the dynamic soundings of January 2nd 2006, the soundings of November show no immediate improvement.

9.3.2 Dynamic Sounding Conclusions

However, it is possible, as a result of this test, to say that each type of equipment has its own specific advantages. From geotechnical point of view the Pagani is preferred because of its superior distinguishing capability.

9.4 Injection

The injections that were done have a different lifting effect. The reason, generating this difference in lift, is the amount of kilos of resin injected underneath the foundation strip. The next figures show lift marked against instantaneous settlement as well as lift against the amount of injected kilos of resin per foundation strip.

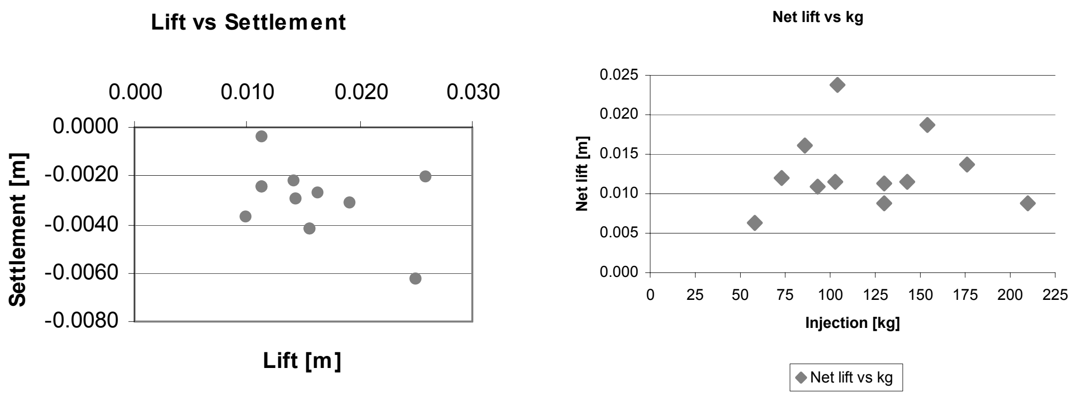


Figure 20 Lift vs. Settlement and Net Lift vs. kg

From this cloud of dots can be derived that the average amount of settlement directly after injecting ranges generally between 2 and 4 mm.
From this can be derived that the average amount of lift ranges from 10 to 15 mm and that this amount of lift can be achieved using an injected mass of between 90 and 180 kg per foundation strip. See Appendix 19 *Injection Schedule* for the injection schedules. This appendix also contains the injected mass per location drafted in a table.

9.4.1 Piquet's

For the second round of injections, piquets were placed next to the foundation strip that was going to be injected. These piquets were placed in a row perpendicular to the centre of the foundation strip 3.3 – 3.4. The distance to the centre of the strip is 0.50 m, 0.85 m and 1.32 m respectively. The graph shows that the lift on piquet's 1 and 3 is nearly identical. The fact that the lift on piquet 2 is half a centimetre more is strange. This could be because of piquet 1 is located closely to the foundation strip, which is load and lifts only 50% of piquet's 1, 2 and 3. The restricted lift of the foundation strip might reduce the lift of piquet 1, due to shear stresses in the soil. That restriction for lifting is less the case near piquet's at larger distances to the foundation strip.

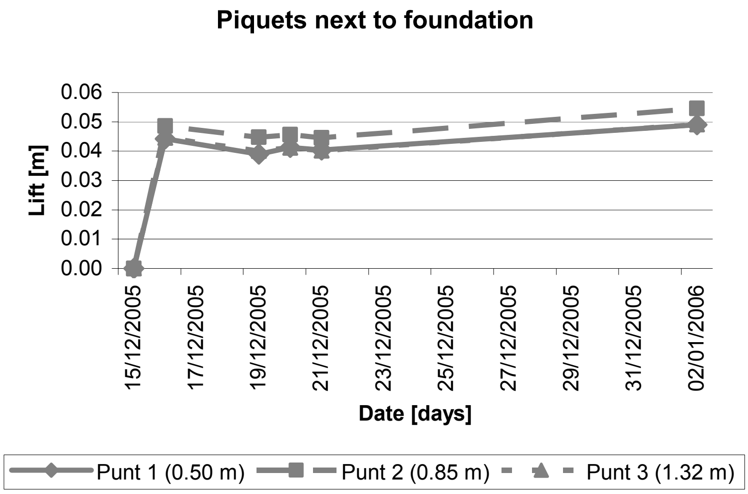


Figure 21 Piquet's next to foundation

9.4.2 Observation Wells

The reading of the observation wells revealed the following data that shows a high ground water level. The top of the observation wells was placed above ground level. Ground level is at roughly NAP + 0.15 m. Simultaneously; the graph shows that the ground water level fluctuates quite a bit. Both on the shallow observation well with a filter at 1.20 m below ground level and the deep observation well in the layer of sand at 4 m below ground level. The measuring was done from ground surface which is NAP + 0.15 m.

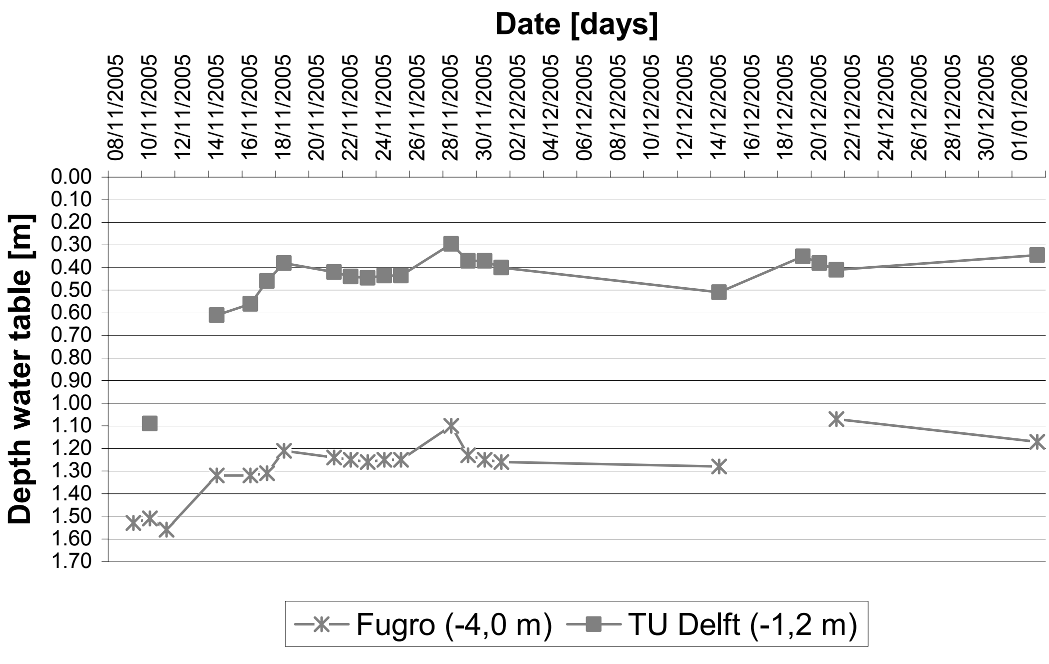


Figure 22 Depth of water table of the observation wells

9.5 Excavation

The excavation of the Short-Term Test Facility was done with the aid of a crane, a trowel, a shovel and brooms. The excavation was started in the centre between the Reference Test Facility and the Short-Term Test Facility. The first foundation strip to be excavated was foundation strip 3.3 – 3.4. Two injections were done underneath this strip. Every injected foundation strip underwent de first round of injections. The following amounts of resin were injected (in kg) underneath this foundation strip;

3.3-3.4 depth to ground surface [m]	1	3	5	7	9	10	12	14	16	18		Total
-0.6	2	1	1	1	0	1	1	1	1	2		11
-1.7	13		6		7		12		5			43
-2.5		3		10		10		7		21		51
-3.5	14		17		11		21		11			74
											Total	***179***

Table 14 Injected kg First injection round

During the excavation it was revealed that the injections created a beautiful pattern in the soil. It is most likely that the vertical scale developed first. This scale pre-stressed the soil, after which the

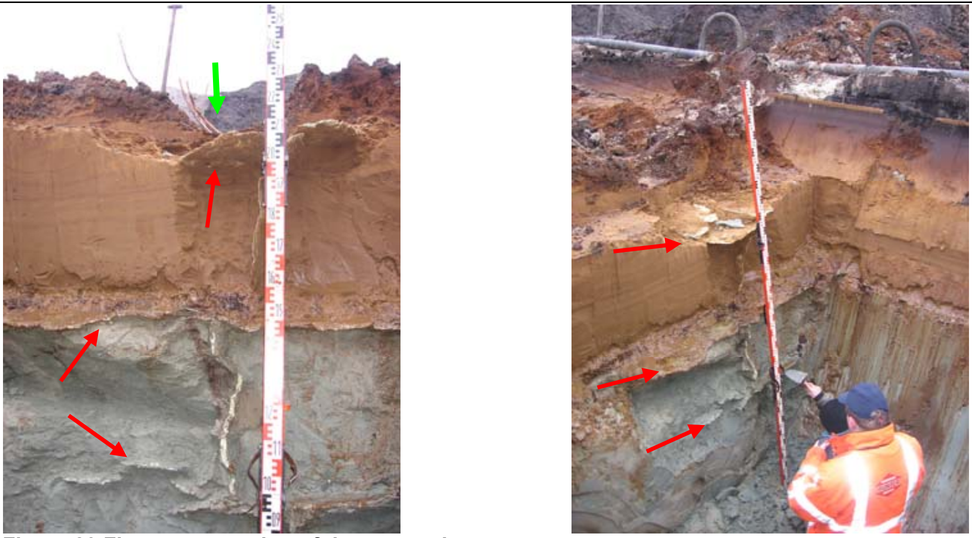


Figure 23 First cross section of the excavation

horizontal scale created lift. This is the theory that is used with compensation grouting and is pursued during the experimental research of grouting. The horizontal scales are located at the depth of the injection tube. The red arrows in the photographs point to the locations of the horizontal scales. The centre arrow points at a horizontal scale that is not located at the depth of the injection tube. The most likely explanation is that there is an “oer” layer situated above this scale that is hard to penetrate by the resin. These scales are likely to be results of the first round of injections. Shown beautifully in the photograph is that the vertical scales have formed perfectly underneath the foundation strip, the green arrow point at the injection tube which is inserted through the centre of the foundation strip, and that the run from the horizontal scales stayed nicely within 2 meters from the foundation strip.
The crane created the following cross section. The crane excavated about 1.5 m creating a cross section at the location where drawn injections were done. The cross section contains both the vertical face and the horizontal face, see figure 23.

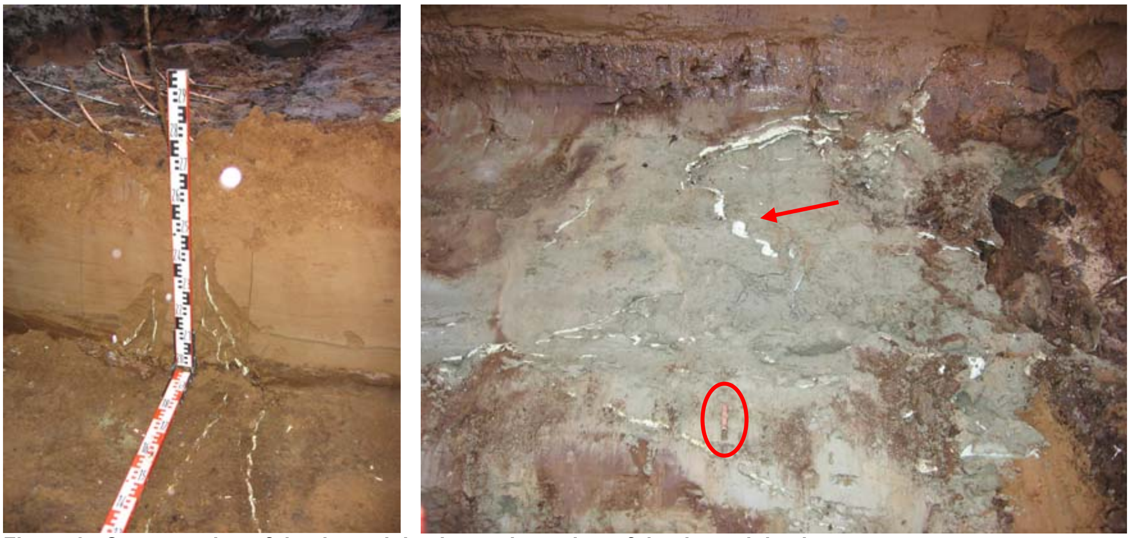


Figure 24 Cross section of the drawn injection and top view of the drawn injection

In figure 24 the oval marks the location of a bent injection tube and an arrow points to the crumbled edge of a horizontal scale, in a horizontal face. This scale is located just below the "oer" layer. However, the figures do show that the beautiful single pattern of a vertical with a crossing horizontal has seized to exist. Did this happen because of the drawn injection method? Or because of the fact that, a beautiful pattern was formed by the first injection and that this injection stirred the soil creating a different stress distribution in the soil. In order to answer this question, further testing must be done. This test consists of the drawn injection method in virgin soil. The resin of the drawn injection stayed nicely underneath the foundation strip.

Figure 25 Cross section and detail of the drawn injection

At the next cross section the Tight Matrix Injection Method was examined. The result is near identical to the drawn injection method. Here the shape of the resin is more irregular because the scales from the different injection depths invade each other's space.
Here too the question is if the shape of the resin will be the same as the shape found when the injection method is applied to virgin soil. With this injection method it is clearly visible that the scales from different injection methods search each other out and form next to each other; see figure 25. The figure on the right is an enlargement of the oval on the figure on the left. Here it is clearly visible that multiple scales formed next to each other

10 Conclusions and Recommendations

In order to formulate a well-founded conclusion of this research, this chapter reviews the objective as stated in the introduction.

Problem statement; Does the UDI really improve the strength and stiffness of the layer of soil in which is injected and is the method usable in clay-containing soil for levelling settled foundations and what is the long term behaviour?

Objective; Design of a test facility to conduct tests using the Uretek method in clay containing soil and ascertaining at the soil improving properties, strength and stiffness, of the method as well as researching the run and the creep characteristics of the resin.

Further research on the resin and the creep characteristics of the resin and the Long Term Test Facility are carried out by Van Reenen.

10.1 Conclusions

This paragraph contains the conclusions. Each part of the research yields its own conclusion.

10.1.1 Set-up

The conclusion can be drawn that the main objective of the research has been achieved.
A test facility was built under which a two-component resin could be injected, dismantled and excavated in order to examine the results. A cheap, and easy in use, leveling instrument was used that performed according to expectation. The injections were performed in two stages. The first stage consisted of injecting underneath all the foundation strips of the Short-Term Test Facility and the Long-Term Test Facility. The second stage consisted of injecting underneath foundation strip 3.3 – 3.4 of the Short Term Test Facility. With the aid of the levelling instrument, used before and after the injections, conclusions could be drawn on the amount of settlement or heave and the stiffness of the soil. The location and shape of the resin in the soil could be examined after the excavation of the short-term test. The gathered data resulted in an increase of knowledge about the UDI.

10.1.2 Levelling measurement

The conclusion can be drawn that the levelling measurements were done yielding results well within the pre-established margin of error. Due to the bad weather during the execution of the test, variations occurred in the levelling measurements, but overall it can be said that the levelling measurements using the levelling instrument and the marker worked well. The marker pot turned out to be a very adequate instrument for the precise positioning of the marker, though difficult to place due to the presence of dirt on the foundation strips. Some of the errors in the measurements were due to dirt being logged in between the marker pot and the foundation strip and in between the marker pot and the marker.

10.1.3 Settlement

The difference in settlement of the loaded Reference Test Facility and both the Short Term and the Long Term Test, underneath the soil that has been injected, can be attributed mainly to the injections.

The prediction of the settlements was in the same range as the measured settlements.

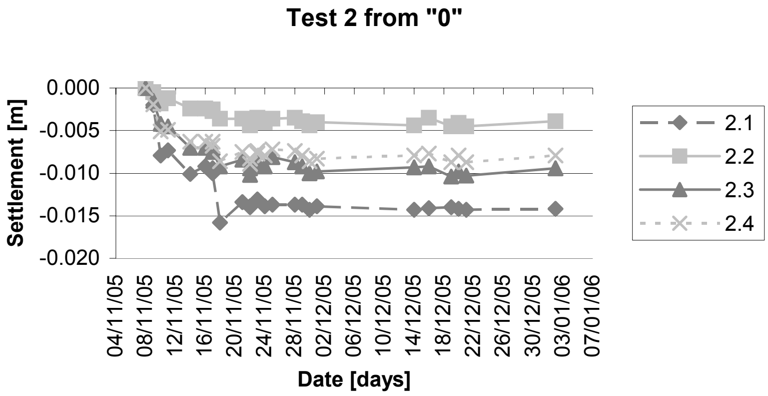
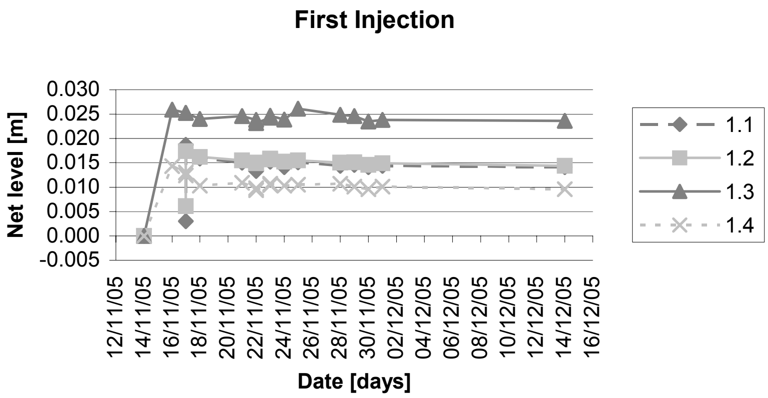


Figure 26 Settlement of the Reference Test Facility as plotted from the origin

---

The instantaneous settlement, immediately after heaving the structure by injecting is probably caused by the generated excess pore pressure during injecting.



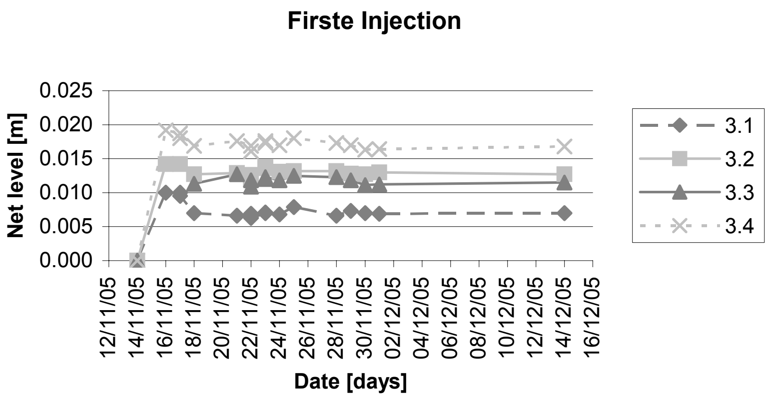


Figure 27 The horizontal path after the injections

10.1.4 Dynamic soundings

Conclusions on the dynamic soundings are hard to draw. However, it can be said that the upper sand layer and the lower Pleistocene sand layer are improved (increased strength) slightly by the injections. The heterogeneous soil conditions make it difficult to draw uniform conclusions on the dynamic soundings. Where the first dynamic sounding is performed on a loam pocket, the second could turn out to be performed on a sand pocket, see figure 28.

Figure 28 Sand and loam pockets

10.1.5 Excavation

The excavation revealed that the first stage of the injections created a beautiful pattern of vertical and horizontal scales. The vertical scales are formed underneath the foundation strip and the horizontal scales have a short run of about two meters besides the strips. This amount of run and the depth of the scales, exactly at the depth of the injection point, is a perfect result. The method can be controlled.
The excavation of the location of the drawn injections shows that the resin has a short run and that several scales formed next to each other. A problem is that the injection was performed in soil that had been previously injected; this makes it hard to determine the results of the drawn injection. The same applies to the tight matrix injections. The results with respect to creating scales of the tight matrix injections and the drawn injections are similar. However, the drawn injection method requires less execution time.
The injected resin is shaped like a thin scale. The high temperature, caused by the chemical reaction in the resin, results in expansion of the resin, creating these thin scales.

10.2 Recommendations

This paragraph contains recommendations for further research.

10.2.1 Research

The Drawn Injection generates more lift and a bigger instantaneous settlement than the Tight Matrix Injection. The net lift of the Drawn Injection is larger than the net lift of the Tight Matrix Injection. Because these injections were carried out in earlier injected soil a new test is recommended in order to confirm these preliminary findings. This research should be done on virgin, never before injected, soil to be able to determine the effects of the different injection methods.

Appendices

Appendix 1 The Resin

The composition of the resin for the Uretek UDI has the following qualities
URETEK RESIN 2409/HARDENER-10 is a special expanding polyurethane system for deep injection. URETEK RESIN 2409/HARDENER-10 is obtained from the chemical reaction between the two components URETEK RESIN 2409, the ready blended poly-ol and URETEK HARDENER-10, the isocyanate (MDI).
The mixing ratio of the two components is as follows:
100 parts by volume URETEK RESIN 2409
135 parts by volume URETEK HARDENER-10
100 parts by weight URETEK RESIN 2409
158 parts by weight URETEK HARDENER-10

The reaction time of the two components in a laboratory environment with a 5 ^{o}C cup test is:

- Cream time 9 seconds
- Gel time 23 seconds
- Tack free time 26 seconds
- Free rise core
- Density 37 g/m3

The typical data of the two components are:

- Viscosity, mPa.s, 25°C 510 160-240
- Density, kg/m3, 20°C 1045 1230
- Storage stability, months (in sealed drums) 6 – 12

The processing temperature in normal conditions is around: 35 – 40 °C.

Some safety precautions have to be taken into account:

- Raw material handling
  When handling the raw materials and components, care should be taken to prevent the liquids from coming into contact with the skin and eyes. Avoid also inhalation of their vapors.
- Resin handling
  When injecting, the reacting mixtures leaves the mixing chamber of the gun as a stream. In this process vapors of isocyanate are given off during the resin expansion owing to the reaction heat that is generated. These vapors can be removed by sufficient ventilation on the resin location (MAC-value URETEK HARDENER-10: 0.02 ppm).
- Resin Scorching
  To avoid scorching of the resin during injection the filling of large cavities (bigger than 30 liters) should be avoided. In practise this means that injection should be stopped if no lifting is recorded after 2 minutes of continuous injection. Fire risk:
  URETEK RESIN 2409/HARDENER-10 is an organic combustible product. If exposed to fire and/or heat it may present a fire risk in certain applications. Burning polyurethane resin does not cause more hazards than most other organic materials such as wood, cork, wool and leather.
- Resin Disposal
  URETEK RESIN 2409/HARDENER-10, which is completely formed as a resin, can be disposed as construction waste material.













RT 15 sec
CT 32 sec
RGT 65 sec
GT 125 sec
TFT 240 sec

Appendix 2 *Intervention with UDI in Germany*

This is a summary of the final report of the intervention in Germany:

ELH ingenieure Test injection
At a building at the Wilhelm-Leuschner-street 27 in Bremen are large and different settlements occurred. The opportunity was given to do a test injection with the Uretek method. In addition to the normal soil layers are there Holocene sands with Tonnestern which are under layered with Pleistocene sand layers.

Building:
Since 1983 there are different building activities with severe damage as result. Settlements at the North West wing of the building.

Damage:
The roof shows clearly the settlements of the building as well as the gap between the original building and the extension. Several cracks are found at the North side in the trace of the mortar.

Soil investigation:
Five pulse borings were done at the North West of the building, the results can be found in the original report on page 8 and 9. There is shown that borings 1 and 5 consists of an infill. Under this infill show the borings 2 – 4 Sand and under that consist the soil for all borings of loam with finally a sand layer. The Phreatic level is between 2,5 and 2,6 meter.
Laboratory tests show the sieve-curve and the results of the Atterberg limits (w_p = 34% and w_l = 71% and the I_c = 0,6)

Injection:
First the rotation laser will be placed at the building walls so a precise observation of the leveling is possible. The injection tubes are placed on each point at three levels, the distance between the different points is 80 cm. The expansion foam will be injected with a small overpressure. The injection is done in several intervals till some raise is seen. Some different injection methods are tested:
Variation 1: The injection tube is injected till the Schluff.
Variation 2: Injection tube is with some perforations at the end in the Schluff injected.
Variation 3: The injection tube is injected in the Schluff layer.
Variation 4: Injection tube is with some perforations at the end till the Schluff injected.

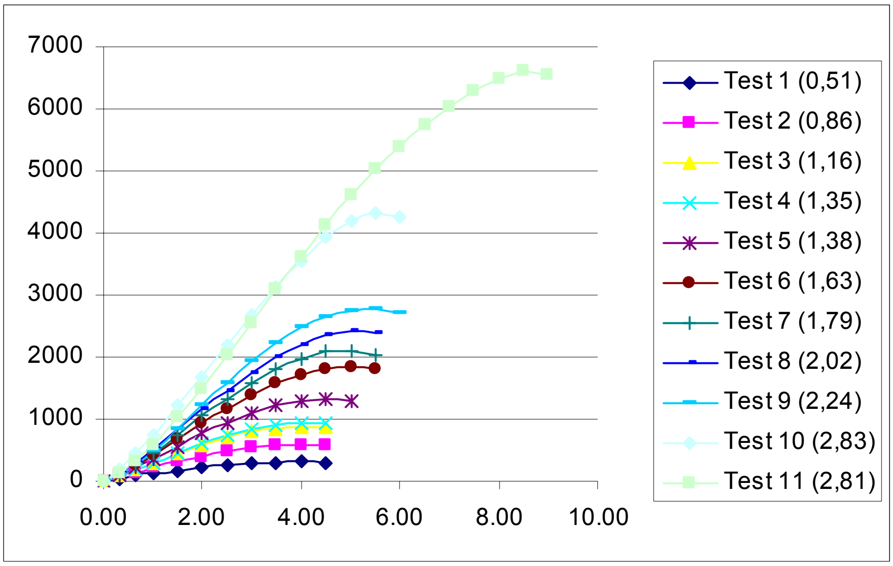
Excavation of the injections:
The excavation is very precisely done with mini excavators, there is started at the West side. A lot of roots were found near the foundation and a vertical slice of the injected material was found (depth 1,6 m) Different variations were used at both sides.
North side: there are different slices found (like fracture grouting).

Appendix 3 *Tests on the resin by the University of Padua*

This appendix contains the results of research done on the resin itself.

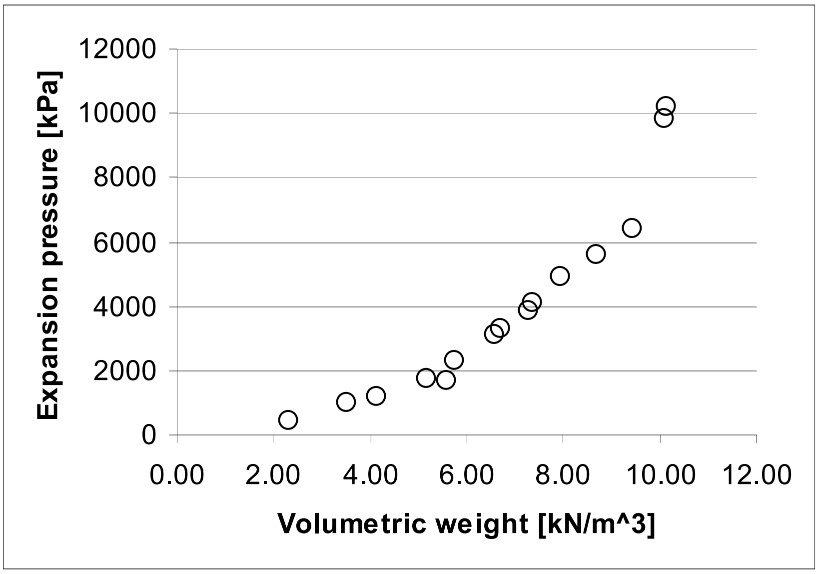
University of Padua Report 1

A press was used to apply a vertical load capable of maintaining a pre-established, constant feed rate of 0,5 mm/minute during the test. The test was conducted on sample cubes with 50 mm sides. Five samples were used for each density investigated, the range of volume weights between 0,5 kN/m^3 and 3,3 kN/m^3 were found.
The maximum compression resistance was defined as the ratio between the maximum loads encountered during the test and the initial surface area of the cross section at the right angles to the load direction. The sample bent to the areas of lower resistance because it was not perfectly homogeneous.
When the vertical applied force was eliminated, the sample reassumed to its original shape and its original dimensions were restored. The samples displayed a good level of isotropy because of maximum resistance is achieved regardless of load direction.



University of Padua Report 2

Vertical expansion tests under eudiometric conditions. The tests were conducted using a device constructed to an *ad hoc* design to allow the mix to be injected into a rigid metal cylinder. When the mix expands, it pushes a piston up that is blocked by a transverse counter plate with a pressure gauge, after traveling a few centimeters, so it is possible to measure the inflation pressure. At the end of the expansion, the samples were cylindrical with a constant diameter of 80 mm and a height ranging from 60 to 116 mm and a range of volume weights between the 2 and 10,5 kN/m^3.



University of Padua Report 3

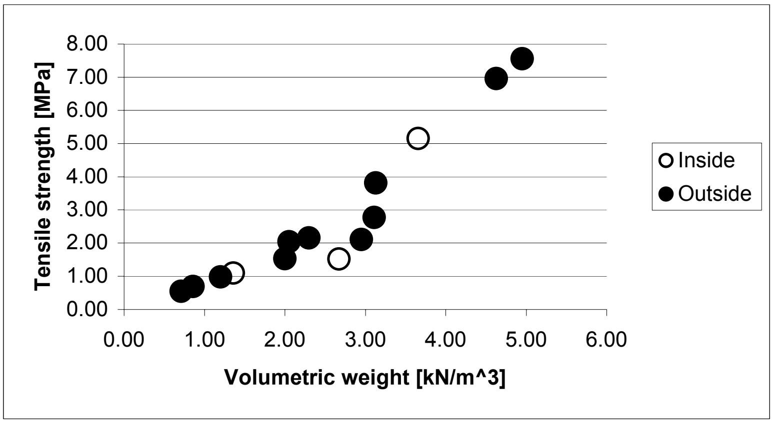
The resin samples supplied by Uretek s.r.l., for the environmental tests, came from a core drilling carried out on soil in which Uretek Geoplus had previously been injected and that compliance with limits imposed by Italian Ministerial Decree 47/99 were to be evaluated.
A yield test was conducted on the >2mm granule size fraction of the sample in water saturated with CO_2. The elute was examined for the parameters shown in the table (see excel Padua-test3) to ensure that the acceptable limit concentration values for subterranean waters were met.
There is no exceeding of the limits according to the Italian Miniteral Decree 47/99

Parameter	Symbols		Concentration	Limit	True / False
Metals					
Aluminum	Al	<	10	200	TRUE
Antimony	Sb	<	0.5	5	TRUE
Arsenic	As	<	1	10	TRUE
Silver	Ag	<	1	10	TRUE
Berylium	Be	<	0.1	4	TRUE
Cadmium	Cd	<	0.1	5	TRUE
Cobalt	Co	<	0.1	50	TRUE
Chrome IV	Cr	<	5	5	TRUE
Total chrome	Cr	<	1	50	TRUE
Iron	Fe	<	5	200	TRUE
Manganese	Mn		1	50	TRUE
Mercury	Hg	<	0.1	1	TRUE
Nickel	Ni	<	1	20	TRUE
Lead	Pb		1	10	TRUE
Copper	Cu		1	1000	TRUE

Selenium	Se	<	0.1	10	TRUE
Thallium	Ti	<	1	2	TRUE
Zinc	Zn		24	3000	TRUE
Inorganic Pollutants					TRUE
Boron	B		35	1000	TRUE
Free cyanides		<	5	50	TRUE
Fluorides		<	250	1500	TRUE
Nitrites		<	50	500	TRUE
Sulphates	mg/l	<	1	250	TRUE
Aromatic Organic Compounds					
Benzene		<	0.1	1	TRUE
Ethyl benzene		<	0.1	50	TRUE
Styrene		<	0.1	25	TRUE
Toluene		<	0.1	15	TRUE
Xylene		<	0.1	10	TRUE
Carcinogenic Aliphatic Hydrochlorides					
Chloromethane		<	0.1	1.5	TRUE
Trichloromethane		<	0.1	0.15	TRUE
Vinyl chloride		<	0.1	0.5	TRUE
1,2-Dichloroethane		<	0.1	3	TRUE
1,1-Dichloroethane		<	0.05	0.05	TRUE
1,2-Dichloropropane		<	0.1	0.15	TRUE
1,1,2-Trichloroethane		<	0.1	0.2	TRUE
Trichloroethylene		<	0.1	1.5	TRUE
1,2,3-Trichloroethane		<	0.001	0.001	TRUE
1,1,2,2-Tetrachloroethane		<	0.05	0.05	TRUE
Tetrachloroethylene	PCE	<	0.1	1.1	TRUE
Hexachlorobutadiene		<	0.1	0.15	TRUE
Summary of Organ halogenated compounds		<	10	10	TRUE
Non-carcinogenic Aliphatic Chlorides					
1,1-Dichloroethane		<	0.1	810	TRUE
1,2-Dichloroethylene	Cis + Trans	<	0.2	60	TRUE
Carcinogenic Halogenated Aliphatic Compounds					TRUE
Tribomomethane	Bromoformium	<	0.1	0.3	TRUE
1,2-Dibromoethane		<	0.001	0.001	TRUE
Dibromochloromethane		<	0.1	0.13	TRUE

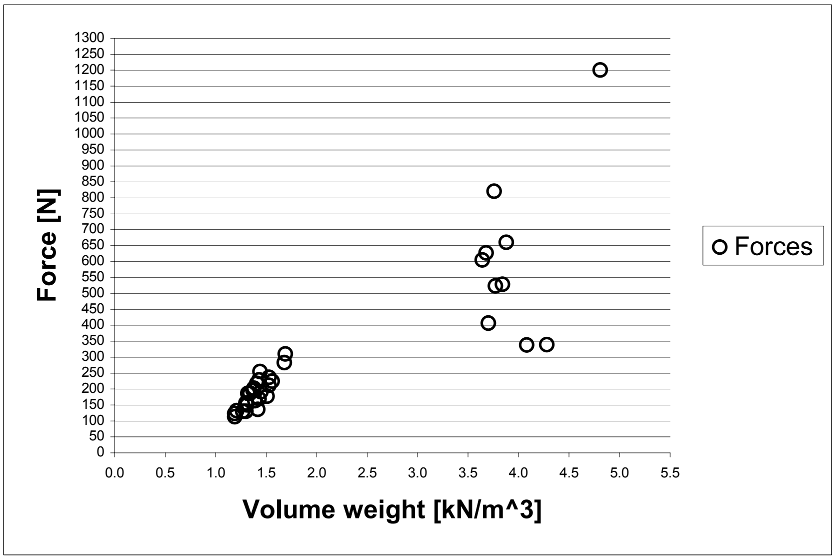
University of Padua Report 4

Determination of tensile characteristics was used as reference although the shape of the cross-section of the sample was round and not rectangular as indicated in the standard. A cross-section of the samples varied along its axis and tapered off in the centre of the sample. The diameter in the centre of the cross-section was 25 mm +/- 0,5 mm and 40 mm at the edge of the samples. A test device was used to apply a vertical load capable of maintaining a pre-established, constant feed rate of 5 mm/minute during the test. A range of volume weights between 0,7 kN/m^3 and 5 kN/m^3 were investigated.
Maximum tensile strength was defined as the ratio between maximum tensile load and the initial surface area of the cross-section at the right angles to the load direction measured in the tapered area. In 11 of the 14 tests the rupture took place outside the effective segment but still within the tapered area with the reduced cross-section.



University of Padua Report 5

Flexing tests on Uretek Geoplus resin samples, Determination of flexing load according to UNI standard 7031-72 which was used as the reference except for the sample length, which was less than the 120 mm +/- 1,2 mm as indicated in the standard and the distances between the supports, 8 cm instead of 10 cm. A machine for the flexion testing was used that was designed to advance the load application blade at a pre-established constant feed rate (10 mm/minute).
35 samples of varying density were tested, a range of volume weights between the 1,19 and 4,81 kN/m^3 were investigated. The flexing load is the load value applied at the time of rupture.



University of Padua Report 6

Uretek s.r.l. supplied with 3 cylindrical samples, of a resin known as Uretek Geoplus, of 38 mm in diameter and initial height of 76 mm. These samples were subjected to triaxial dynamic-cyclic compression tests. With these tests the dynamic and cyclic conditions of the Uretek Geoplus when it is used. The samples were subjected to cyclic-stress with amplitudes increased according to initial sample density. The confinement pressure was assumed to be zero because this type of injection will be used only at slight depths from the natural surface level.
The equipment used has a frequency of 2 Hz and a load-unload cycle number of 50000. The modulus of resilience is parameter that is used to characterize the soil under road surfaces. This modulus is defined as the ratio between the change in vertical applied pressure and the corresponding change in vertical deformation. For the graphs see the original report.

University of Padua Report 7

Uretek s.r.l. supplied 2 cylindrical samples of different densities. These were subjected to long-term vertical compression with free lateral expansion. The investigations were done in the elastic field. The samples were stressed with 4 vertical load increases, each load acted on the sample for 20-30 days. For the graphs see the original report.

Sample diameter	3,81 cm	Sample diameter	3,824 cm
Initial sample height	7,55 cm	Initial sample height	7,63 cm
Final sample height	7,435 cm	Final sample height	7,61 cm
Sample cross-section	11,40 cm^2	Sample cross-section	11,48 cm^2
Weight of ring + initial moist sample	21,4 g	Weight of ring + initial moist sample	27,2 g
Weight of ring	0 g	Weight of ring	0 g
Tare no.	0 g	Tare no.	0 g
Tare weight + dry sample	21,4 g	Tare weight + dry sample	27,2 g
Tare weight	0 g	Tare weight	0 g
Tare weight + dry sample	21,4 g	Tare weight + dry sample	27,2 g
Granular density to water	0.248	Granular density to water	0.31
Weight of initial moist sample	21,4 g	Weight of initial moist sample	27,2 g
Weight of final moist sample	21,4 g	Weight of final moist sample	27,2 g
Dry sample weight	21,4 g	Dry sample weight	27,2 g
Initial water content	0%	Initial water content	0%
Final water content	0%	Final water content	0%
Initial moist volume weight	2,44 kN/m^3	Initial moist volume weight	3,04 kN/m^3
Final moist volume weight	2,48 kN/m^3	Final moist volume weight	3,05 kN/m^3
Initial dry volume weight	2,44 kN/m^3	Initial dry volume weight	3,04 kN/m^3
Final dry volume weight	2,48 kN/m^3	Final dry volume weight	3,05 kN/m^3

Appendix 4 *Bell Tower*

This appendix contains the result of a suggestive research done in Italy.

University of Padua: Intervention to consolidate the foundation ground of a bell tower with high expansion pressure resin

Causes of cracking phenomena are differential settlements generated by enlargements or modifications made to the body of the building and variations in the distributions of he permanent loads applied; in other circumstances the cause of sinking must be sought in variations in the geotechnical properties of the foundation grounds, due, for example, to lowering or raising of the Phreatic surface, chemical degradation of some litho types, breakage of hydraulic and sewage systems, etc.
Due to the considerable expansion of the resin within the volume of the terrain treated, contact can also be restored between the terrain foundation interfaces, even where the values of solicitation are more modest. Thus improved distribution of the loads is obtained and peaks of tension are consequently limited.
Examination of the profile of cracking is an important source of information on the type of sinking that has taken place. The entity, form, inclination and evolution over a period of time of the lesions that have taken place on the bearing walls, dividing walls and floors of the structure can furnish a precious set of data, contributing to the accurate interpretation of the dynamics of the sinking.
The foundation terrain:
The geotechnical examination was carried out in the autumn of 2001 and consisted in the performance of probes in rotation, to a depth of -20 m, granulometric analysis of the re-managed samples taken during the probes. The terrain is mostly made up of sand and gravel with a muddy fraction that does not exceed 6%. At a depth of -14,2 -14,8 m from ground level, a layer of peat was discovered.

Depth (m)	Description	N_{spt}
0,0 – 1,8	Sand and brick fill	
1,8 – 3,0	Fine to average, slightly muddy sand	8
3,0 – 6,0	Medium to large sand with gravely layers	11
6,0 – 7,5	Large grain sand and gravel	14
7,5 – 12,0	Fine and medium sand	10
12,0 – 14,2	Large grain sand with gravel	-
14,2 – 14,8	Fibrous peat	-
14,8 – 16,5	Fine sand	10
16,5 – 20,0	Large sand with gravel	-

Intervention for consolidation:
The main cause of the sinking was identified as the different geometry of the foundations and the different loads are transmitted by the structure built on the terrain, therefore it was decided to intervene on the volume immediately underlying the bell tower foundation, solely wit the Uretek deep injection consolidation procedure, utilizing a resin capable of exercising high pressure during its expansion.
The result of the vertical compression tests with free lateral expansion under eudiometric conditions are shown here. Through opportune variation of the resin volume weight the results show how the resistance to compression increases quickly with the volume weight. As far as the initial module of elasticity E is concerned, the tests permitted identification of the field of values between 15 Mpa and 80 MPa, comparing them with E modulus characteristics of loose grains. This means that the average rigidity of the mass does not undergo significant variations, but remains homogeneous throughout the volume treated, with no abnormal re-distribution of tensions applied. The expansion pressure was expressed in terms of the pressure necessary to block any upward movement of the piston. The expansion pressure values between 0,2 MPa and 10 MPa were measured in the field of volume weights examined (0,5 kN/m^3 and 10 kN/m^3). The solid volume weight of the resin and its degree of volumetric expansion, measured upon the termination of the process, are both a function of the pressure value. The Uretek consolidation procedure develops its action vertically through a succession of low pressure injections, performed underneath the level of the foundation, of a resin that has significant and rapid capabilities of expansion. The pressure increases the level of confinement that the resin itself is subject to during the intervention; the more the resin is

confined during the course of treatment, the greater its consolidating action will be. The model is based on the following hypothesis (Yu and Houlsby 1991):

- An un-limited three-dimensional medium is considered, made up of a homogeneous , isotropic, expanding, elastic and perfectly plastic terrain
- The terrain contains a single cylindrical or spherical cavity
- The initial radius of the cavity is a_0 and the cavity is subject to an initial hydrostatic pressure of p_0
- The internal pressure p of the cavity is gradually increased, making the eventual dynamic effects negligible
- Expansion of the cavity is monitored, adding the contributions deriving from an analysis of large deformations in the plasticized region and from a solution of small deformations in the elastic regions

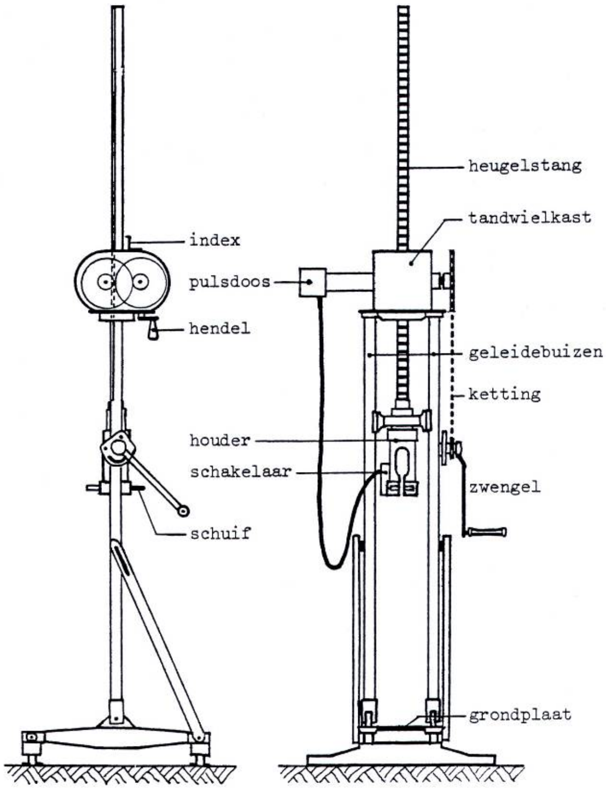
Due to the evident problems of invasiveness that traditional intervention underneath the foundation would have involved it was decided to proceed with the injection of high expansion pressure resins. The pre-existing structures of the building, such as the altars, floors, choir and machinery make any intervention that would cross the wall structure and terrain at a considerable depth impossible, as it would damage parts of the Church that have considerable historical value. The operation involves low-pressure injection into the terrain of high expansion pressure resins, obtained by mixing components which, due to their chemical reactions, provoke the consolidation by exercising a pressure of up to 10 MPa on the host terrain, within a maximum time of “6 – 10” from mixing. The liquid resin injected at medium and low pressure, given its characteristics, expands where it encounters the least resistance from the terrain and thanks to its great volumetric increase; it compacts and consolidates the foundation terrain, amalgamating with it to constitute agglomerate with characteristics of high resistance and cross tensions. The total quantity of resin injected amounted to approximately 1750 kg, an equivalent of approximately 14 kg per meter of column-type treatment. Considering the total volume of the substratum involved in the intervention, which amounts to approximately 150 m^3, the so called filling index can be determined: $\eta = \frac{1750}{150} = 11,7 kg/m^3$

The crack measuring equipment is electrical, with a centesimal potentiometer that has a field of +/-25mm, to obtain precision measurements of variations between two points on the wall. During the phases of drilling and injection of the resins, the time interval between readings was lowered to 10 minutes in order to record the evolution of moments during the progress of the works with greater precision. The purpose of the monitoring in this case is to record eventual trends or processes of sinking underway over an extended period of time, and in the short term as well, during the performance of works or settlement of the structure. In fact, as it was noted during the initial phase of monitoring, all of the cracks are very sensitive to daily temperature changes, undergoing distance variations within the space of one day on the order of 3 tenths of a millimeter for temperature changes of about 6°C between the day and the night. At some moment the period of interval of time between readings was diminished from 30 to 10 minutes in order to have a more precise numerical evaluation in relation to the time of perforation and injection into each borehole. The significant variations, relative to the period in which the works were performed, takes place independently of the temperature and are a function of the effects of consolidation of the foundation terrain.

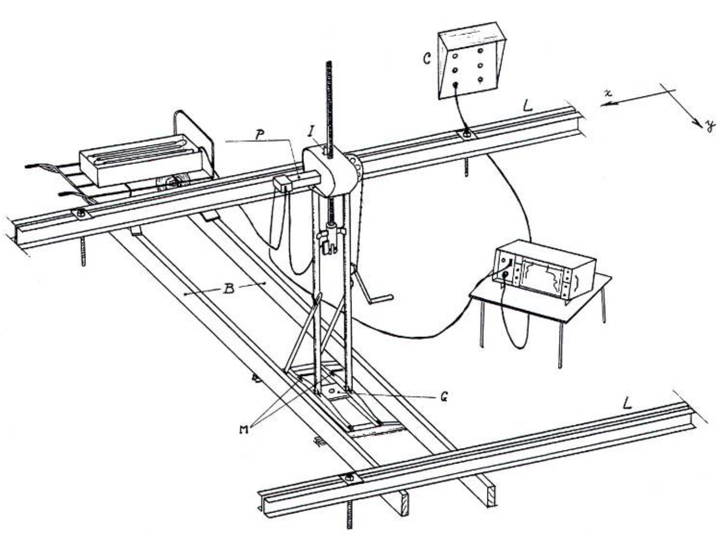
Conclusion:
The use of this technology to consolidate the foundation terrain permitted homogeneous improvement of the geotechnical characteristics of the ground underlying the structure. The maintenance of a rigid medium of mass comparable with the elastic modulus characteristics of loose terrains allows us to avoid important re-distribution of tensions in deeper layers of terrain. The intervention was completed in seven working day’s time, and involved the realization of perforations, which were made manually with the rotary percussion equipment, which did not worsen the pre-existent lesions in any way. The characteristics of the material injected are decidedly greater resistance to compression than the load induced by the structure on the foundation terrain and high expansion pressure, which permitted improvement of the general state of density of the foundation terrain.
The resin injected is an ecologically compatible material, which respects the rigid norms in force on this subject.

Appendix 5 Hand Sounding Equipment

Front and side view of the hand sounding equipment



Total overview of the hand sounding construction. On the test site in Wolvega were the steel profiles [L] absent. In stead of the steel profiles there were ground anchors and the second time there was a crane who delivered the contra weight.



B: Wooden beams to guide the uplift forces to the soil.
C: Electricity, Wolvega a generator.
G: Guidance hole of the steel rods.
I: Cogwheel
L: Steel beams anchored to the soil, Wolvega soil anchors
M: Stability feet of the hand sounding machine
P: Pulse box, information circuit.

Appendix 6 Pagani

The average measured efficiency (73%) allows normalizing the measured N_{20} values simplifying the correlations with other in situ measurements and mainly with N_{SPT}

The efficiency of the DPSH penetrometer has been measured at two different sites in soils ranging between sand and gravel and unsaturated silty clay. This preliminary experience has shown that the ratio between the N_{SPT} value and normalized $N_{20\ (DPSH\ Pagani)}$ is about 1.5 for gravelly soils, and increases to more than 4 for unsaturated silty clayey soils.

The DPSH penetrometer produced and marketed by "PAGANI GEOTECHNICAL EQUIPMENT" (PGE) can carry out continuous dynamic penetrometric tests (DP) both according to the methods foreseen by the International and European regulations, and by the Italian rules.

The International and European procedures are in practice identical; the Italian one differs in the hammer weight (73,5 Kg instead of 63,5) and in the cone shape (conical instead of cylindrical-conical); but in Italy the use of the protecting coating against frictions along the rods is compulsory, while in the other mentioned procedures such an activity is only suggested and, as alternative, a mud injection in the annular space between the rods and the hole wall is allowed, having a lubricating function.

From the mid nineties even in Italy the European procedure is more frequently adopted.

With the PGE penetrometer it is easy to insert a set of steel coating tubes whose outside diameter does not exceed that of the penetrometric cone. The insertion of the coatings requires an extra time of about 60 % in comparison with the time required to infix only the penetrometric rods with the cone.

In the Italian procedure each coating cut down size is inserted in at the end of penetration of each penetrometric rod; the lateral friction corresponding to the length of a single rod just infixed is considered as unimportant.

The efficiency measures of the beating device have been carried out by the company ISMES, appointed by PGE, equipping a standard rod by applying 4 strain gage electrically connected to form a Wheatstone bridge; the rod surface has been previously smoothed by turning and put into the furnace to eliminate all residual tensions due to the steel working.

The position of the loading cell was included between +1.00 and 0.00 m over the field level.

The energy E_a (kgm), transmitted to the rods has then been calculated by ISMES, for each hammer stroke, through the following expression:

$$E_a = K \int_0^{2l/c} f(t)\,dt$$

Where:

E_a = calculated energy (kgm)
K = constant depending on the area of the equipped rod, on the E module and on the steel density
l = distance between the measure sections and the rod base
c = rate of sound propagation into the rods (m / s)
f(t) = strength measured in the rods connected to the measure section (kg)

The efficiency of the beating device, expressed in percentage is:

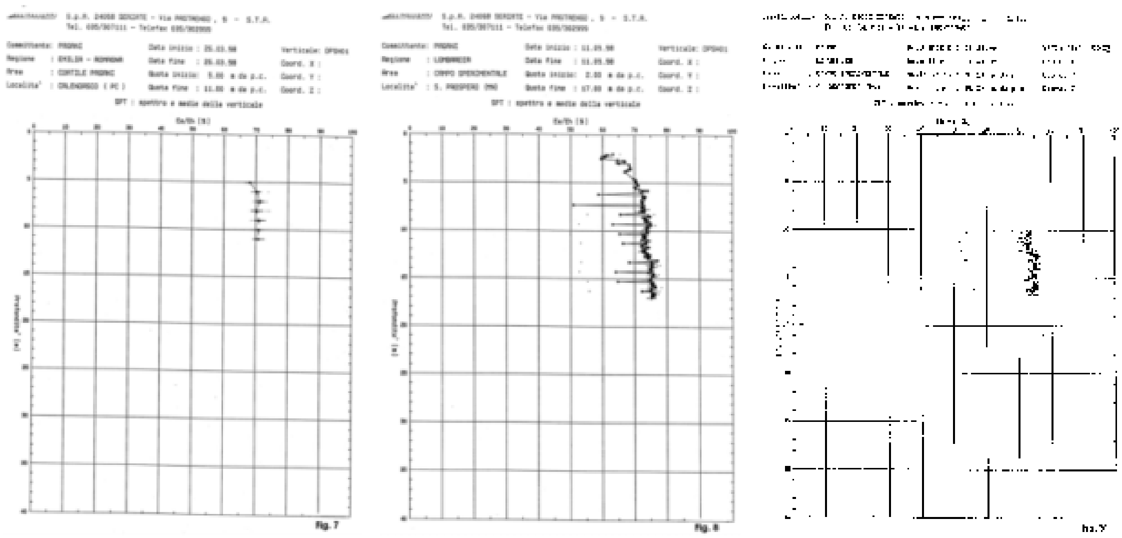
$\eta = E_a / E_h$

Where:

The potential energy: **$E_h = m * H$ (kgm)**
Being:
m = the hammer mass (kg)
H = the falling height of the mass (m)

EFFICIENCY MEASURES

Figures 7, 8, 9 are chartered the values of efficiency measured at different depths in the Santimento site (Figure 7) and in S. Prospero one (Figures. 8, 9).



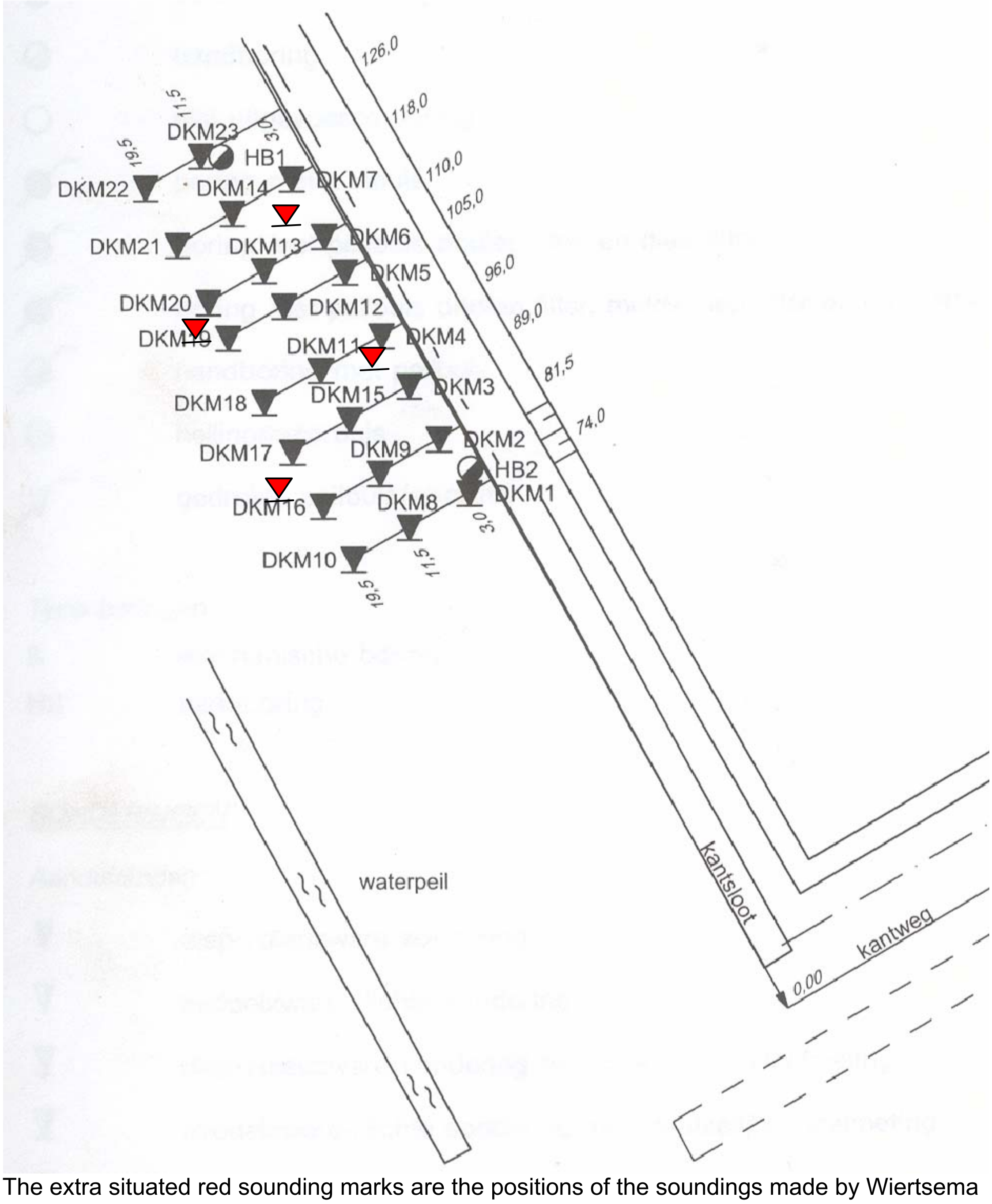
You can note that variations mainly concern the more superficial measures and that the average value of the efficiency slightly increases with depth:

- 72 % at 5 m
- 73 % at 10 m
- 74 % at 15 m.

In calculation of **$N_{20\,(60\%)}$** value **an average value of 73 % has been used.**

Also for SPT tests executed by ISMES at the time the efficiency of the beating device has been measured, obtaining an average value equal to 55¸ 60 %.

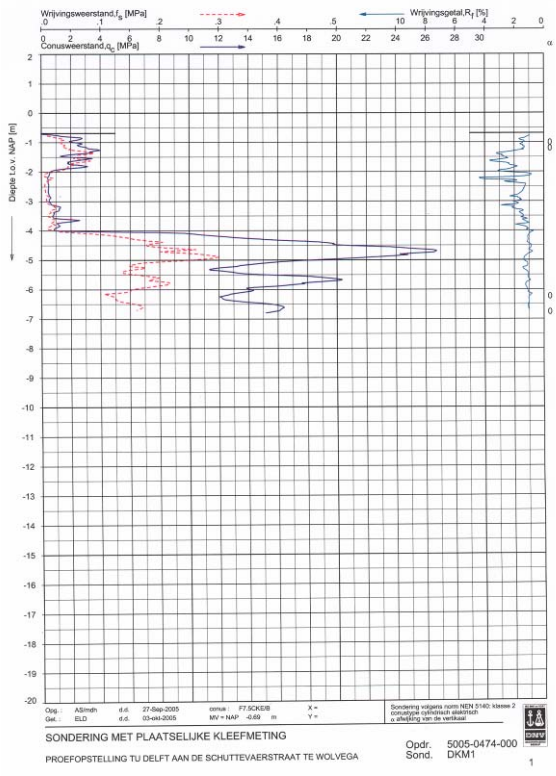
Appendix 7 CPT's Fugro

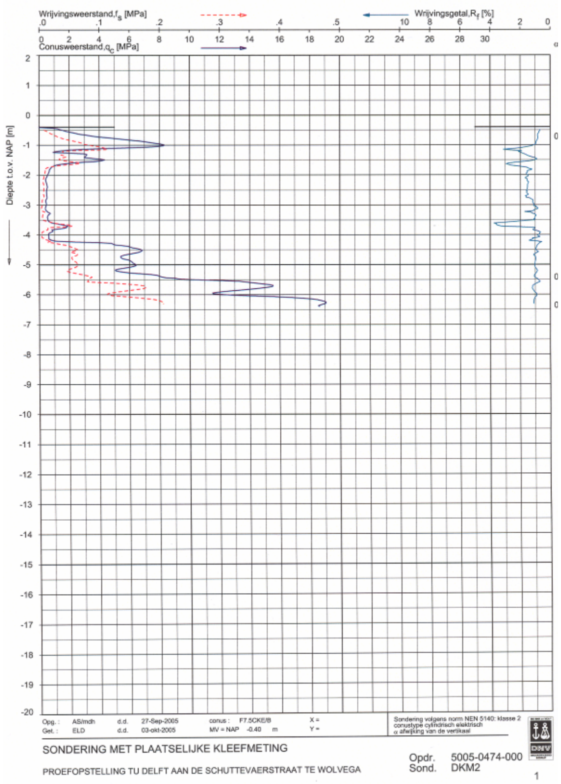


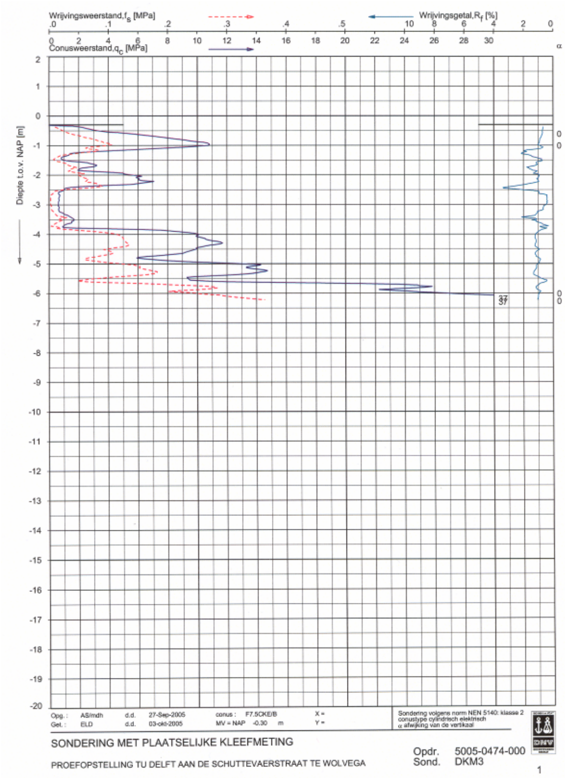
The extra situated red sounding marks are the positions of the soundings made by Wiertsema and Partners. The positions are:

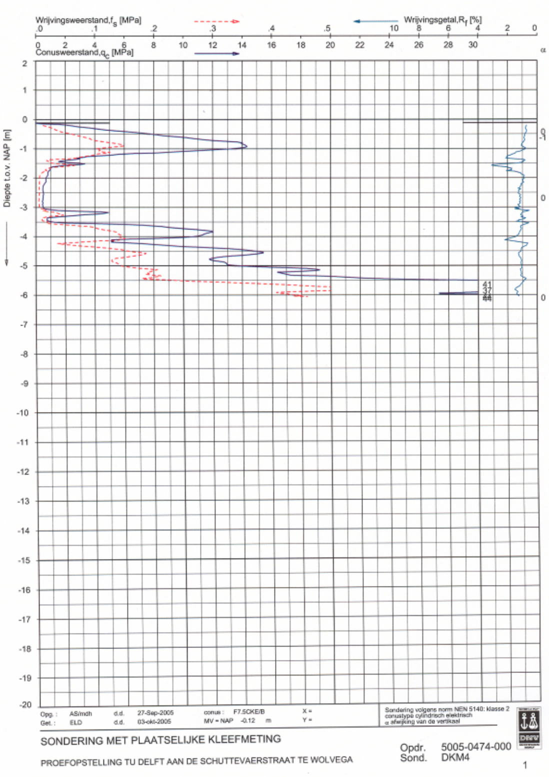
- 82 m from the side of the street and 21 m from the side of the ditch.
- 94 m from the side of the street and 5 m from the side of the ditch.
- 106 m from the side of the street and 21 m from the side of the ditch.
- 115 m from the side of the street and 5 m from the side of the ditch.

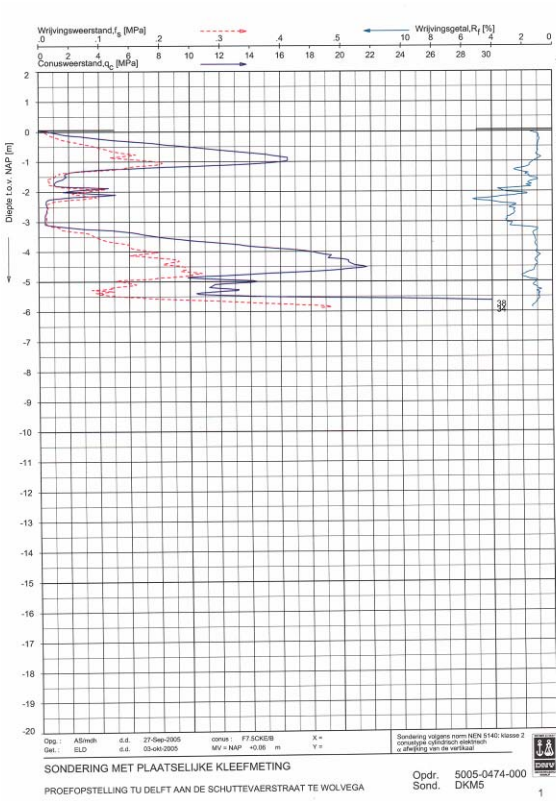
These soundings were the result of the first soil investigation. The position of the HB1 mark is the position of the observation well inserted by Fugro.

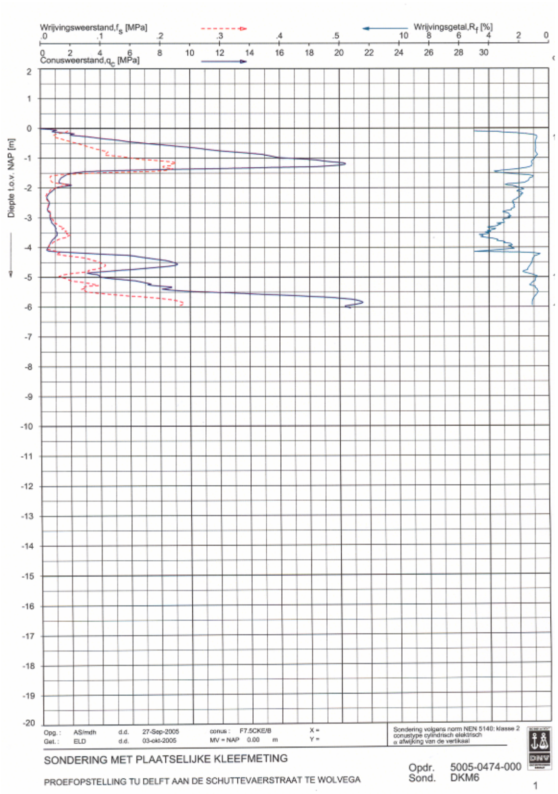

Wrijvingsweerstand,f_s [MPa]
Wrijvingsgetal,R_f [%]
Conusweerstand,q_c [MPa]
Diepte t.o.v. NAP [m]
SONDERING MET PLAATSELIJKE KLEEFMETING
PROEFOPSTELLING TU DELFT AAN DE SCHUTTEVAERSTRAAT TE WOLVEGA
Opdr. 5005-0474-000
Sond. DKM1

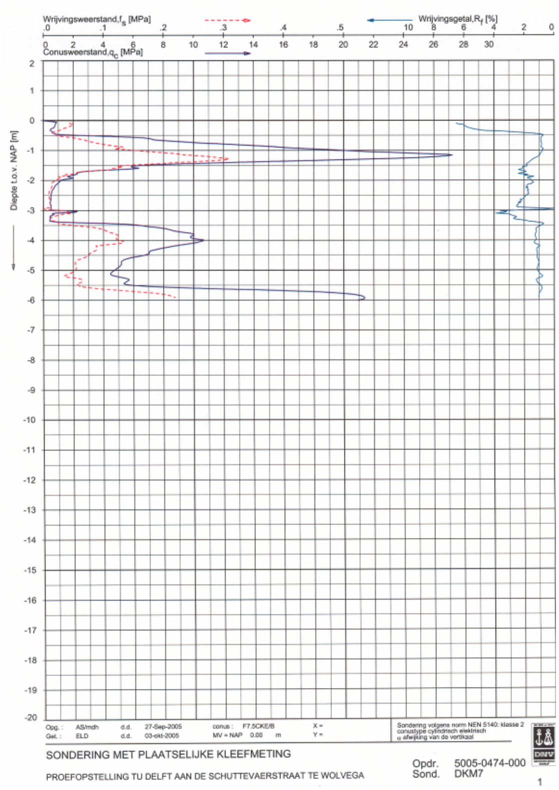

Wrijvingsweerstand,f_s [MPa]
Wrijvingsgetal,R_f [%]
Conusweerstand,q_c [MPa]
Diepte t.o.v. NAP [m]
SONDERING MET PLAATSELIJKE KLEEFMETING
PROEFOPSTELLING TU DELFT AAN DE SCHUTTEVAERSTRAAT TE WOLVEGA
Opdr. 5005-0474-000
Sond. DKM2

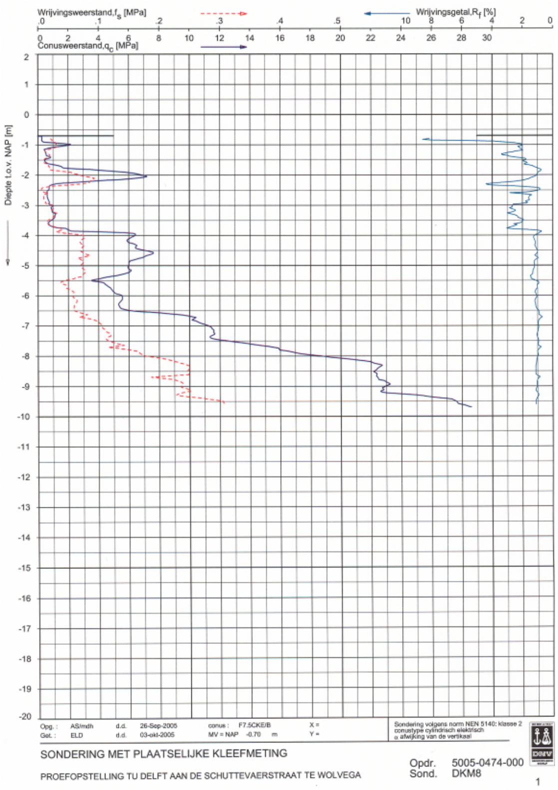

Wrijvingsweerstand,f_s [MPa]
Wrijvingsgetal,R_f [%]
Conusweerstand,q_c [MPa]
Diepte t.o.v. NAP [m]
SONDERING MET PLAATSELIJKE KLEEFMETING
PROEFOPSTELLING TU DELFT AAN DE SCHUTTEVAERSTRAAT TE WOLVEGA
Opdr. 5005-0474-000
Sond. DKM3

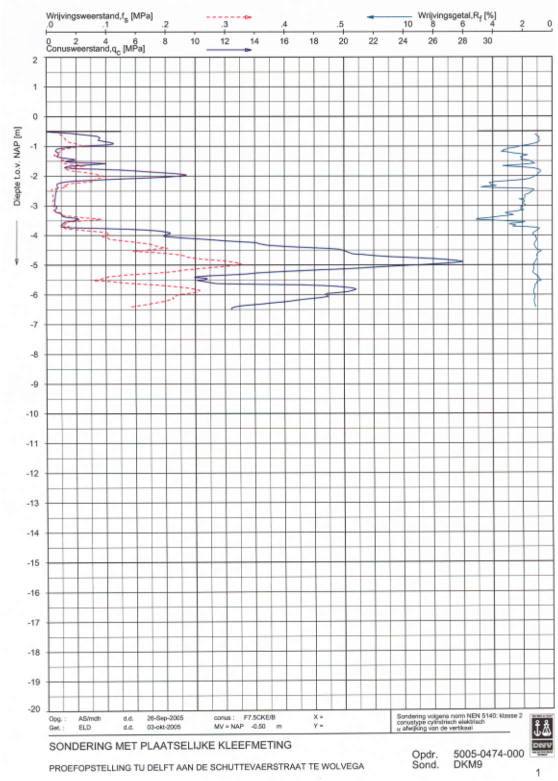

Wrijvingsweerstand,f_s [MPa]
Wrijvingsgetal,R_f [%]
Conusweerstand,q_c [MPa]
Diepte t.o.v. NAP [m]
SONDERING MET PLAATSELIJKE KLEEFMETING
PROEFOPSTELLING TU DELFT AAN DE SCHUTTEVAERSTRAAT TE WOLVEGA
Opdr. 5005-0474-000
Sond. DKM4

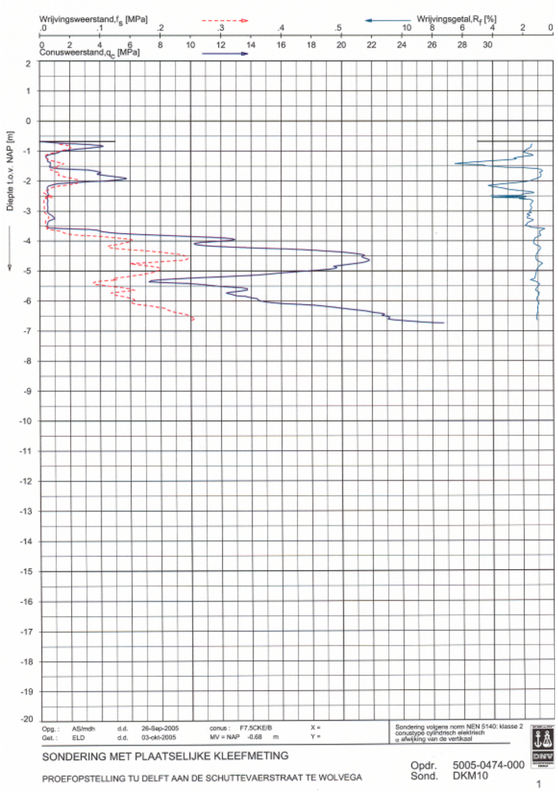

Wrijvingsweerstand,f_s [MPa]
Wrijvingsgetal,R_f [%]
Conusweerstand,q_c [MPa]
Diepte t.o.v. NAP [m]
SONDERING MET PLAATSELIJKE KLEEFMETING
PROEFOPSTELLING TU DELFT AAN DE SCHUTTEVAERSTRAAT TE WOLVEGA
Opdr. 5005-0474-000
Sond. DKM5

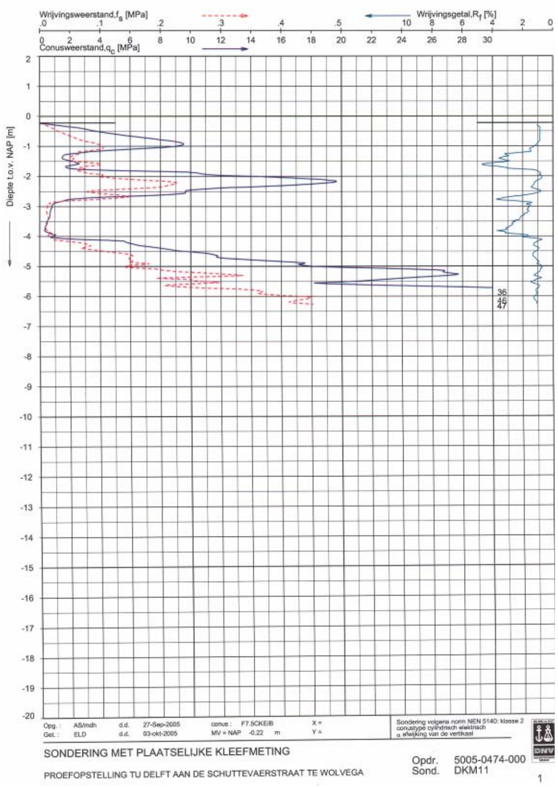

Wrijvingsweerstand,f_s [MPa]
Wrijvingsgetal,R_f [%]
Conusweerstand,q_c [MPa]
Diepte t.o.v. NAP [m]
SONDERING MET PLAATSELIJKE KLEEFMETING
PROEFOPSTELLING TU DELFT AAN DE SCHUTTEVAERSTRAAT TE WOLVEGA
Opdr. 5005-0474-000
Sond. DKM6

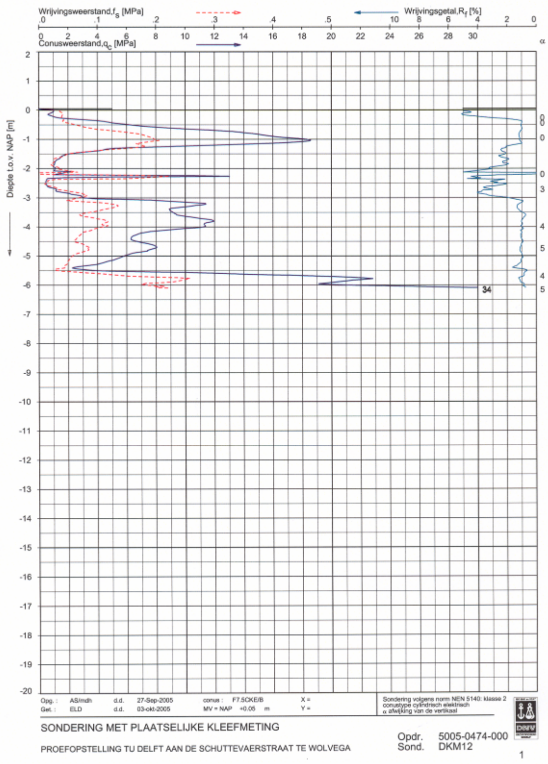

Wrijvingsweerstand,f_s [MPa]
Wrijvingsgetal,R_f [%]
Conusweerstand,q_c [MPa]
Diepte t.o.v. NAP [m]
SONDERING MET PLAATSELIJKE KLEEFMETING
PROEFOPSTELLING TU DELFT AAN DE SCHUTTEVAERSTRAAT TE WOLVEGA
Opdr. 5005-0474-000
Sond. DKM7

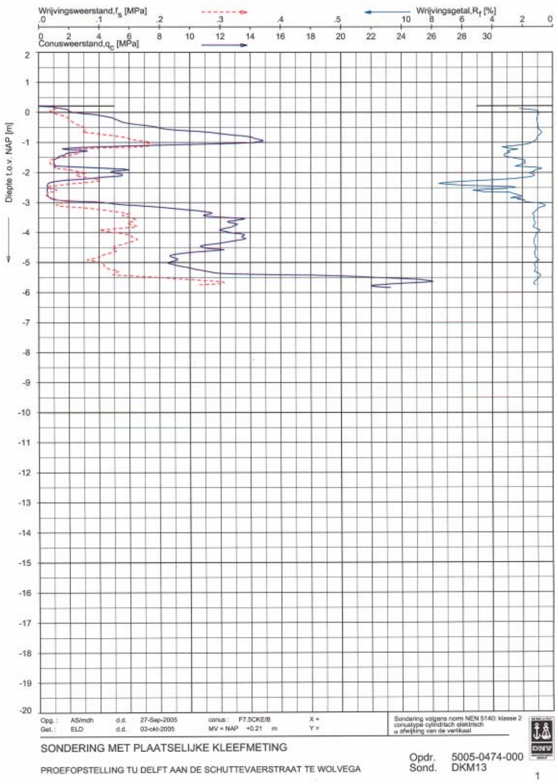

Wrijvingsweerstand,f_s [MPa]
Wrijvingsgetal,R_f [%]
Conusweerstand,q_c [MPa]
Diepte t.o.v. NAP [m]
SONDERING MET PLAATSELIJKE KLEEFMETING
PROEFOPSTELLING TU DELFT AAN DE SCHUTTEVAERSTRAAT TE WOLVEGA
Opdr. 5005-0474-000
Sond. DKM8


Wrijvingsweerstand, f_s [MPa]
Wrijvingsgetal, R_f [%]
Conusweerstand, q_c [MPa]
Diepte t.o.v. NAP [m]
SONDERING MET PLAATSELIJKE KLEEFMETING
PROEFOPSTELLING TU DELFT AAN DE SCHUTTEVAERSTRAAT TE WOLVEGA
Opdr. 5005-0474-000
Sond. DKM9

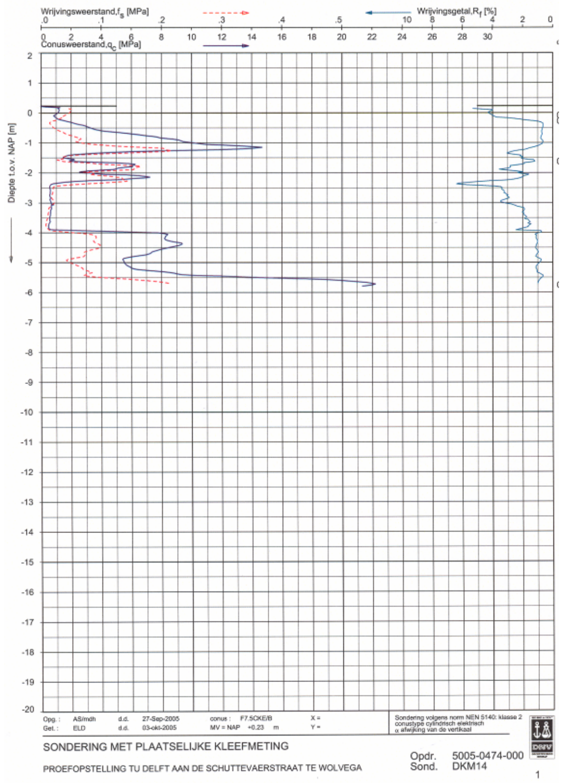

Wrijvingsweerstand, f_s [MPa]
Wrijvingsgetal, R_f [%]
Conusweerstand, q_c [MPa]
Diepte t.o.v. NAP [m]
SONDERING MET PLAATSELIJKE KLEEFMETING
PROEFOPSTELLING TU DELFT AAN DE SCHUTTEVAERSTRAAT TE WOLVEGA
Opdr. 5005-0474-000
Sond. DKM10


Wrijvingsweerstand, f_s [MPa]
Wrijvingsgetal, R_f [%]
Conusweerstand, q_c [MPa]
Diepte t.o.v. NAP [m]
SONDERING MET PLAATSELIJKE KLEEFMETING
PROEFOPSTELLING TU DELFT AAN DE SCHUTTEVAERSTRAAT TE WOLVEGA
Opdr. 5005-0474-000
Sond. DKM11

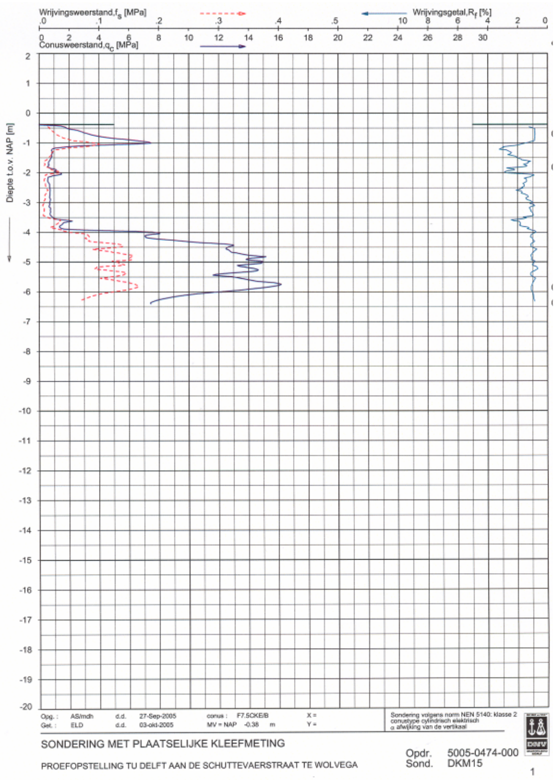

Wrijvingsweerstand, f_s [MPa]
Wrijvingsgetal, R_f [%]
Conusweerstand, q_c [MPa]
Diepte t.o.v. NAP [m]
SONDERING MET PLAATSELIJKE KLEEFMETING
PROEFOPSTELLING TU DELFT AAN DE SCHUTTEVAERSTRAAT TE WOLVEGA
Opdr. 5005-0474-000
Sond. DKM12



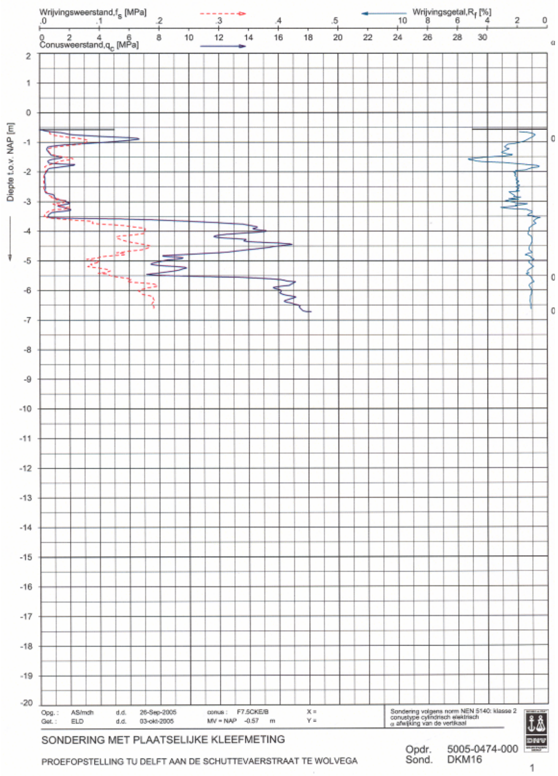
Wrijvingsweerstand,f_s [MPa]
Wrijvingsgetal,R_f [%]
Conusweerstand,q_c [MPa]
Diepte t.o.v. NAP [m]
SONDERING MET PLAATSELIJKE KLEEFMETING
PROEFOPSTELLING TU DELFT AAN DE SCHUTTEVAERSTRAAT TE WOLVEGA
Opdr. 5005-0474-000
Sond. DKM13



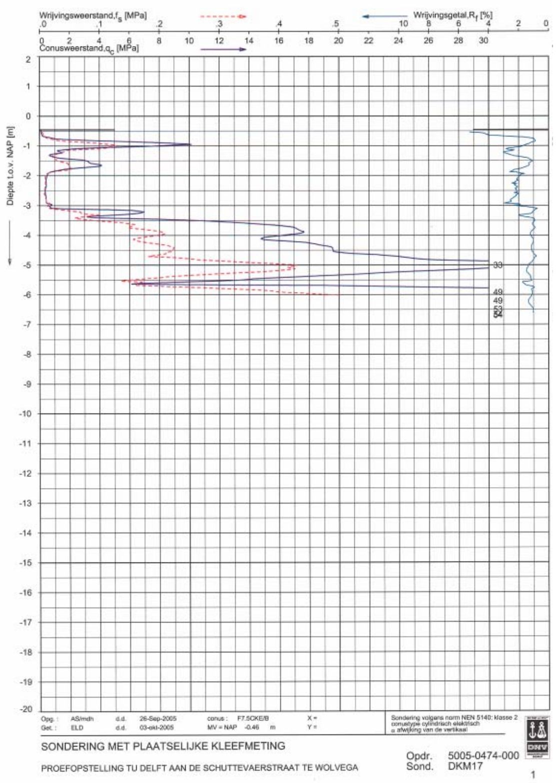
Wrijvingsweerstand,f_s [MPa]
Wrijvingsgetal,R_f [%]
Conusweerstand,q_c [MPa]
Diepte t.o.v. NAP [m]
SONDERING MET PLAATSELIJKE KLEEFMETING
PROEFOPSTELLING TU DELFT AAN DE SCHUTTEVAERSTRAAT TE WOLVEGA
Opdr. 5005-0474-000
Sond. DKM14

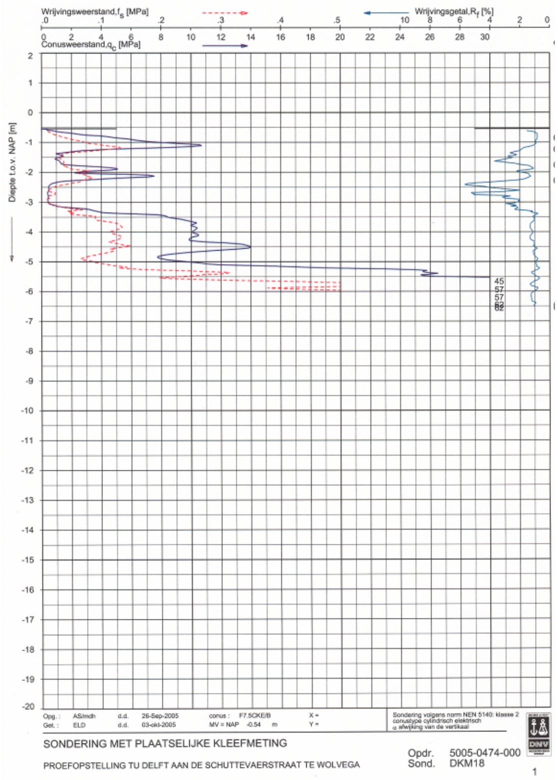


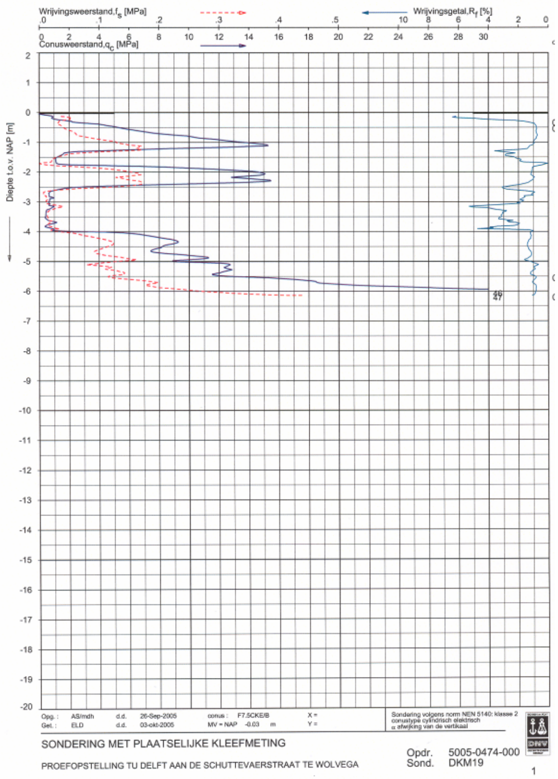
Wrijvingsweerstand,f_s [MPa]
Wrijvingsgetal,R_f [%]
Conusweerstand,q_c [MPa]
Diepte t.o.v. NAP [m]
SONDERING MET PLAATSELIJKE KLEEFMETING
PROEFOPSTELLING TU DELFT AAN DE SCHUTTEVAERSTRAAT TE WOLVEGA
Opdr. 5005-0474-000
Sond. DKM15

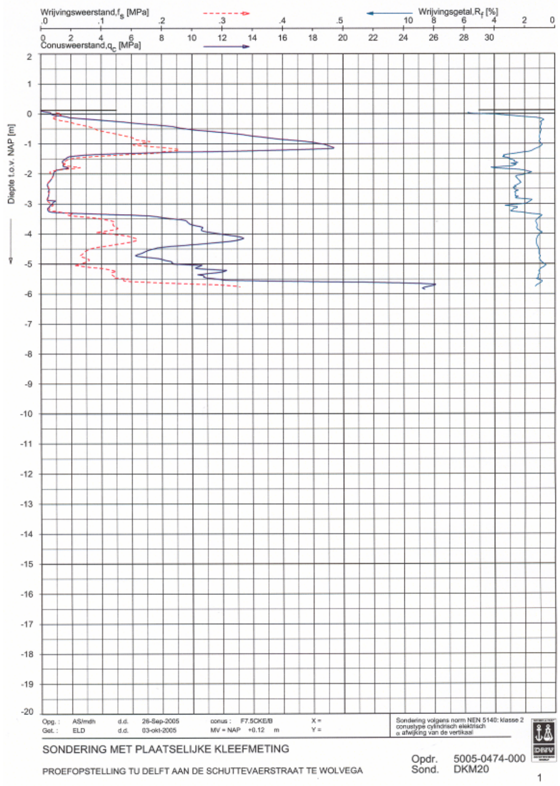


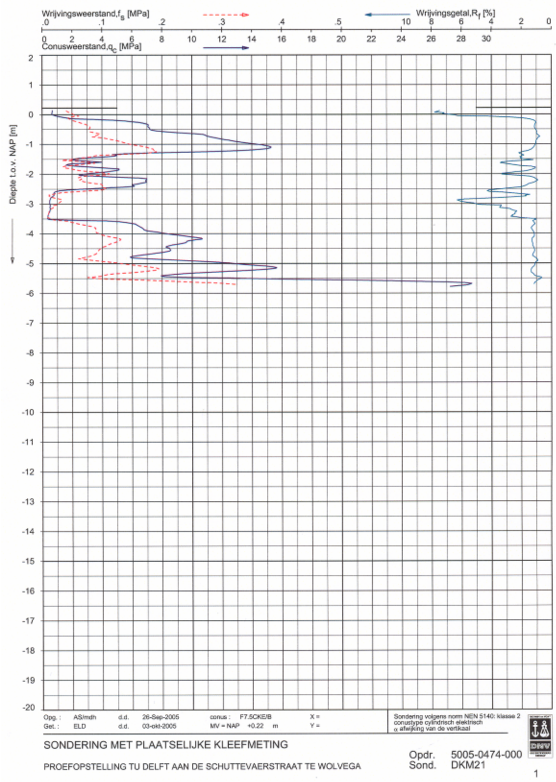
Wrijvingsweerstand,f_s [MPa]
Wrijvingsgetal,R_f [%]
Conusweerstand,q_c [MPa]
Diepte t.o.v. NAP [m]
SONDERING MET PLAATSELIJKE KLEEFMETING
PROEFOPSTELLING TU DELFT AAN DE SCHUTTEVAERSTRAAT TE WOLVEGA
Opdr. 5005-0474-000
Sond. DKM16

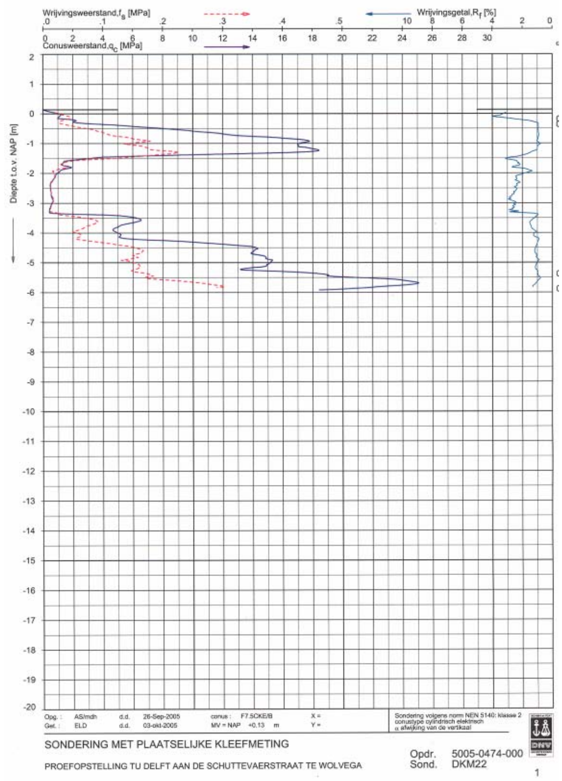

Wrijvingsweerstand,f_s [MPa]
Wrijvingsgetal,R_f [%]
Conusweerstand,q_c [MPa]
Diepte t.o.v. NAP [m]
SONDERING MET PLAATSELIJKE KLEEFMETING
PROEFOPSTELLING TU DELFT AAN DE SCHUTTEVAERSTRAAT TE WOLVEGA
Opdr. 5005-0474-000
Sond. DKM17

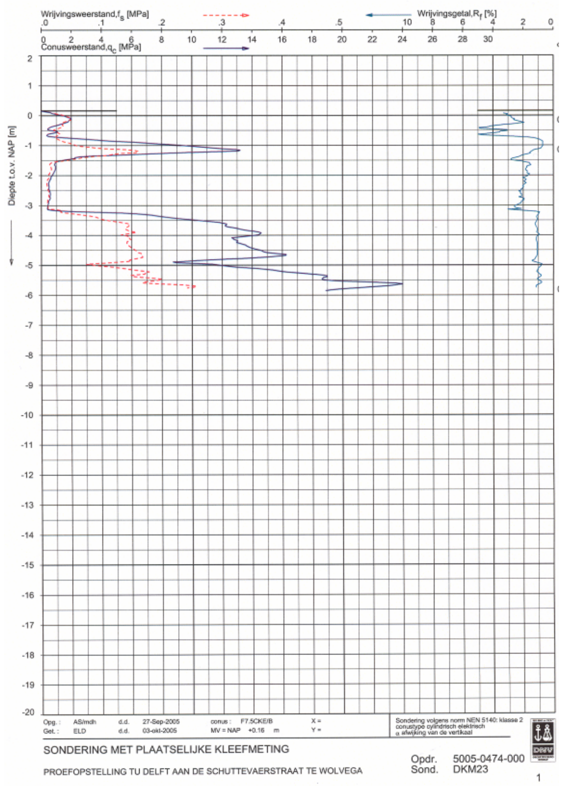

Wrijvingsweerstand,f_s [MPa]
Wrijvingsgetal,R_f [%]
Conusweerstand,q_c [MPa]
Diepte t.o.v. NAP [m]
SONDERING MET PLAATSELIJKE KLEEFMETING
PROEFOPSTELLING TU DELFT AAN DE SCHUTTEVAERSTRAAT TE WOLVEGA
Opdr. 5005-0474-000
Sond. DKM18


Wrijvingsweerstand,f_s [MPa]
Wrijvingsgetal,R_f [%]
Conusweerstand,q_c [MPa]
Diepte t.o.v. NAP [m]
SONDERING MET PLAATSELIJKE KLEEFMETING
PROEFOPSTELLING TU DELFT AAN DE SCHUTTEVAERSTRAAT TE WOLVEGA
Opdr. 5005-0474-000
Sond. DKM19

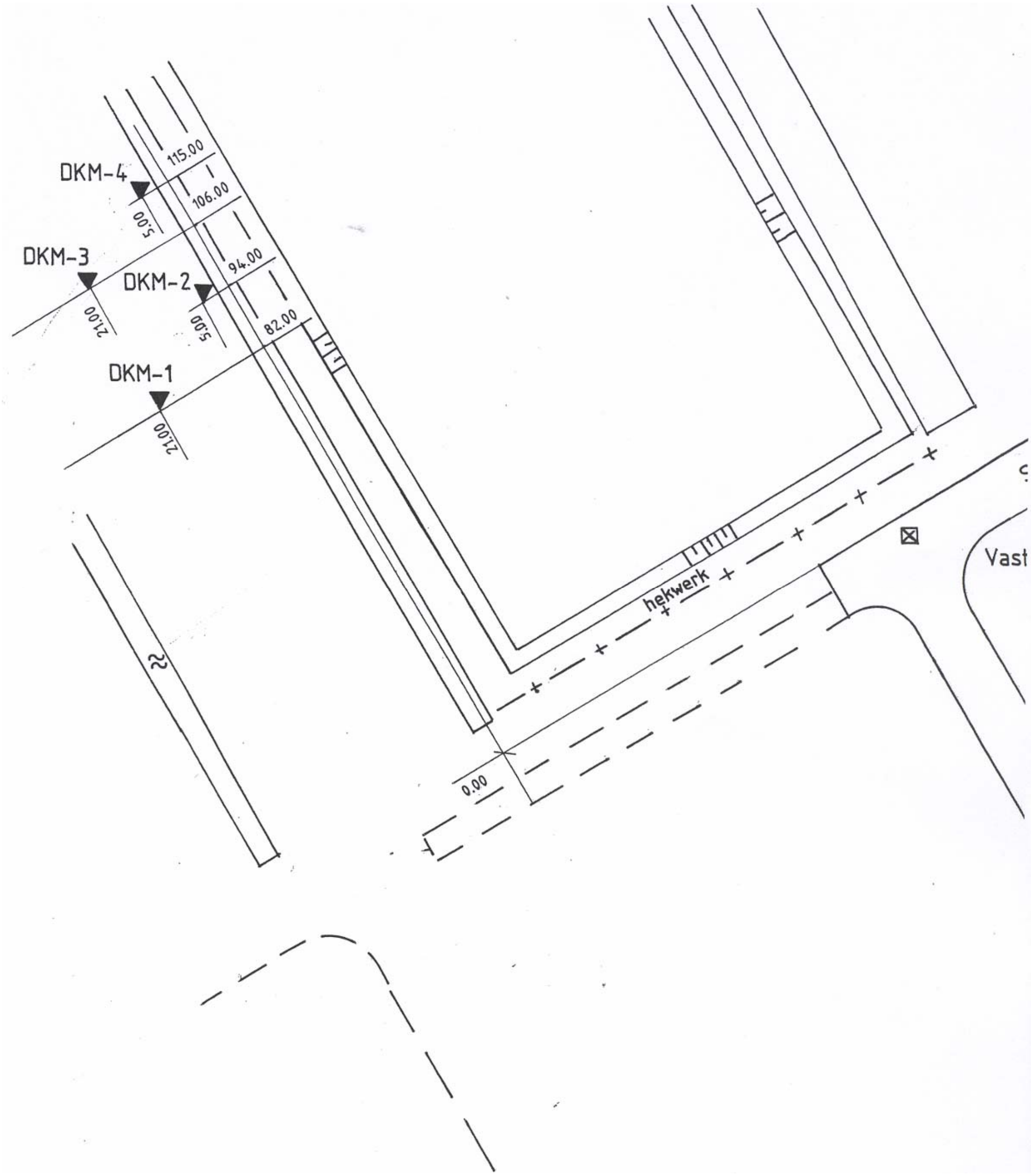

Wrijvingsweerstand,f_s [MPa]
Wrijvingsgetal,R_f [%]
Conusweerstand,q_c [MPa]
Diepte t.o.v. NAP [m]
SONDERING MET PLAATSELIJKE KLEEFMETING
PROEFOPSTELLING TU DELFT AAN DE SCHUTTEVAERSTRAAT TE WOLVEGA
Opdr. 5005-0474-000
Sond. DKM20


Wrijvingsweerstand,f_s [MPa]
Wrijvingsgetal,R_f [%]
Conusweerstand,q_c [MPa]
Diepte t.o.v. NAP [m]
SONDERING MET PLAATSELIJKE KLEEFMETING
PROEFOPSTELLING TU DELFT AAN DE SCHUTTEVAERSTRAAT TE WOLVEGA
Opdr. 5005-0474-000
Sond. DKM21


Wrijvingsweerstand,f_s [MPa]
Wrijvingsgetal,R_f [%]
Conusweerstand,q_c [MPa]
Diepte t.o.v. NAP [m]
SONDERING MET PLAATSELIJKE KLEEFMETING
PROEFOPSTELLING TU DELFT AAN DE SCHUTTEVAERSTRAAT TE WOLVEGA
Opdr. 5005-0474-000
Sond. DKM22

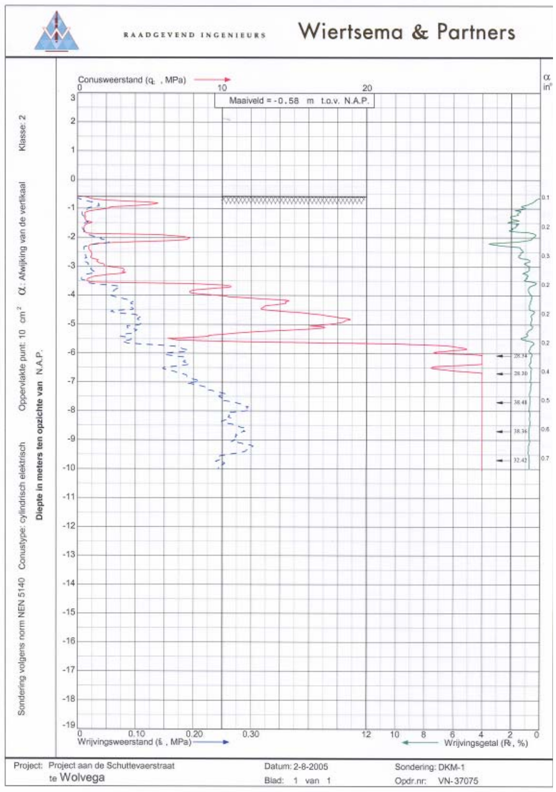

Wrijvingsweerstand,f_s [MPa]
Wrijvingsgetal,R_f [%]
Conusweerstand,q_c [MPa]
Diepte t.o.v. NAP [m]
SONDERING MET PLAATSELIJKE KLEEFMETING
PROEFOPSTELLING TU DELFT AAN DE SCHUTTEVAERSTRAAT TE WOLVEGA
Opdr. 5005-0474-000
Sond. DKM23

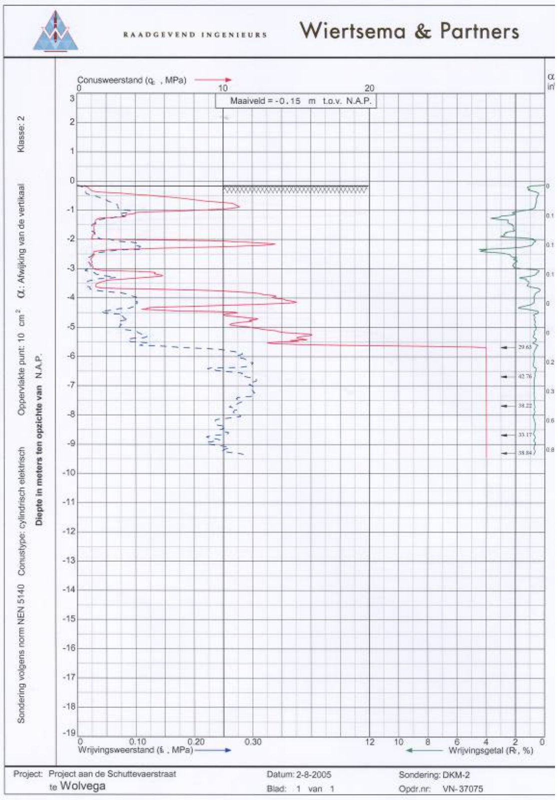
Appendix 8 CPT's Wiertsema and Partners

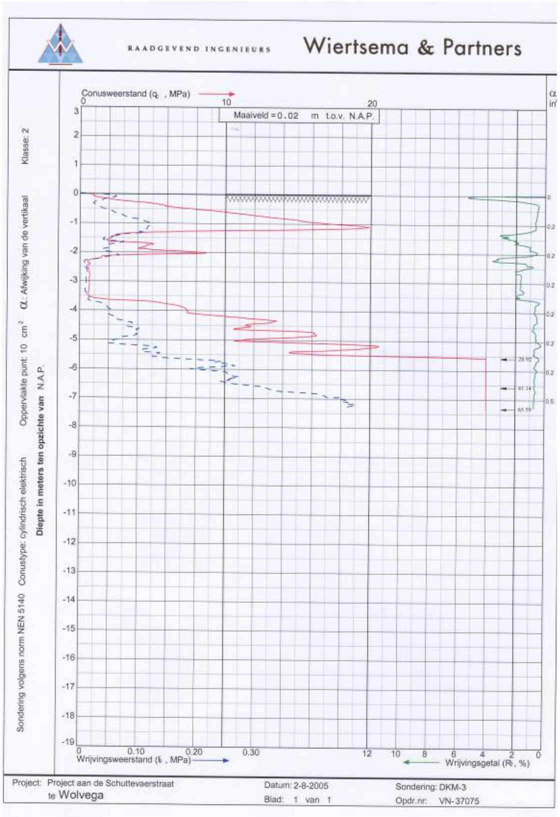


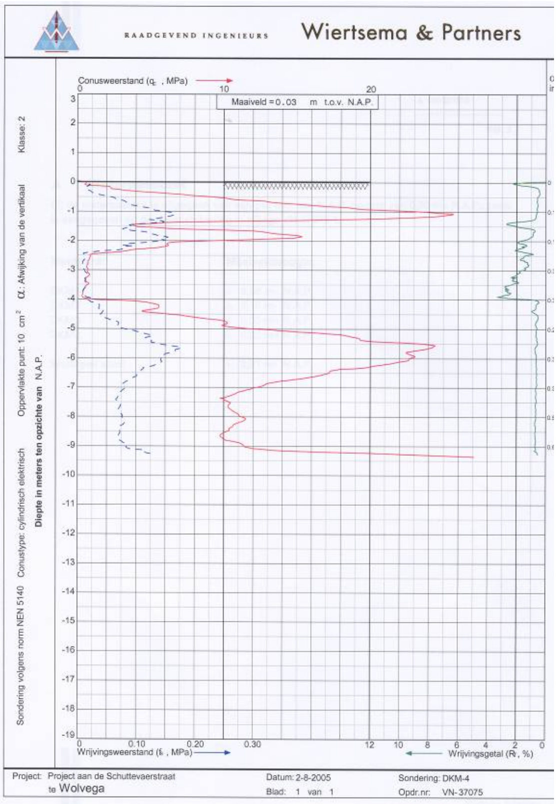
The positions of the first four CPT's on the test site at the Schuttevaerstraat in Wolvega

Four soundings executed by Wiertsema en Partners





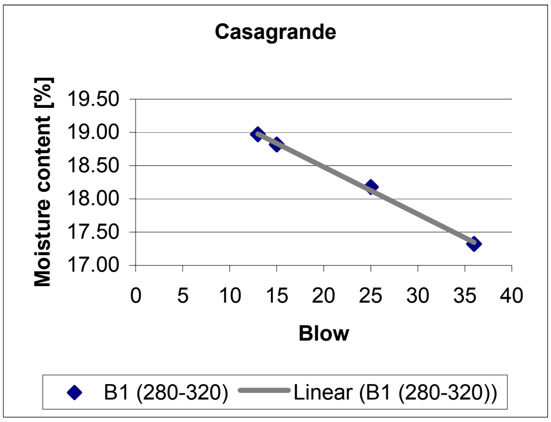


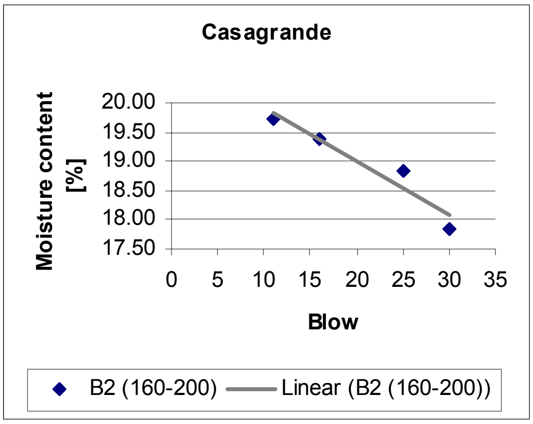


Appendix 9 Laboratory Tests

Sample	**Cup**	**Empty**	**Soil**	**Oven**	
2B	Group J	24.45	37.28	35.91	
160-200	Blank	40.82	52.45	51.25	
1B	I1	47.62	56.82	55.74	
280-320	K2	34.44	46.84	45.39	
Casagrande					**Number of taps**
2B	B2	34.37	49.24	46.79	11
160-200	I2	35.35	43.66	42.31	16
	WA	34.19	46.37	44.44	25
	B3	41.34	58.7	56.07	30
1B	PL1	62.9	84.16	80.77	13
280-320	Group J 1	34.44	51.3	48.63	15
	G9	50.07	70.05	67.1	36
	B1	35.06	52.29	49.64	25

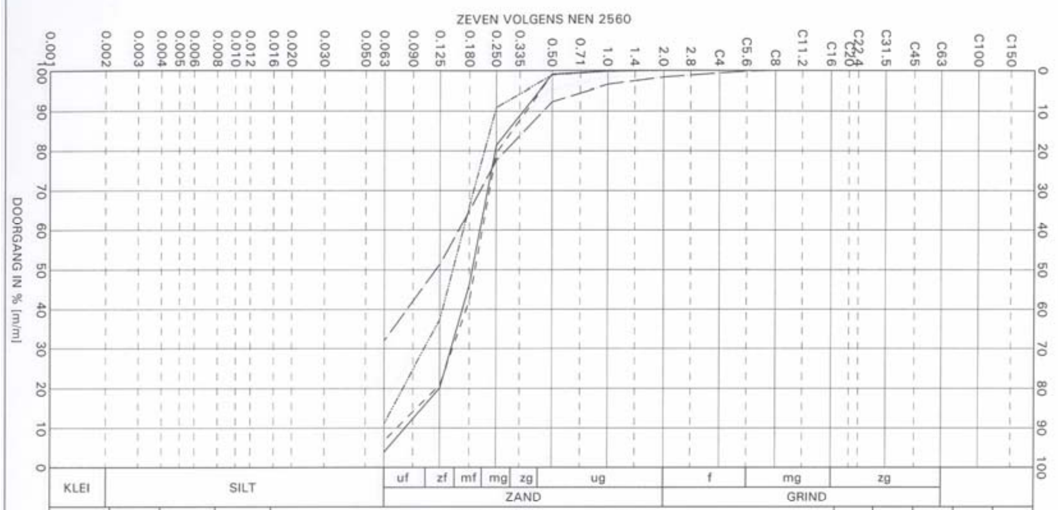
Wet	**Dry**	**Water**	**Percentage**	**True / False**			
12.83	11.46	1.37	11.95	TRUE	PL		
11.63	10.43	1.2	11.51		11.73		
9.2	8.12	1.08	13.30	TRUE			
12.4	10.95	1.45	13.24		13.27		
Wet	**Dry**	**Water**	**Percentage**		**LL**	**PL**	**PI**
14.87	12.42	2.45	19.73		19	12	7
8.31	6.96	1.35	19.40				
12.18	10.25	1.93	18.83				
17.36	14.73	2.63	17.85				
21.26	17.87	3.39	18.97		18	13	5
16.86	14.19	2.67	18.82				
19.98	17.03	2.95	17.32				
17.23	14.58	2.65	18.18				





Appendix 10 Sieve Curves

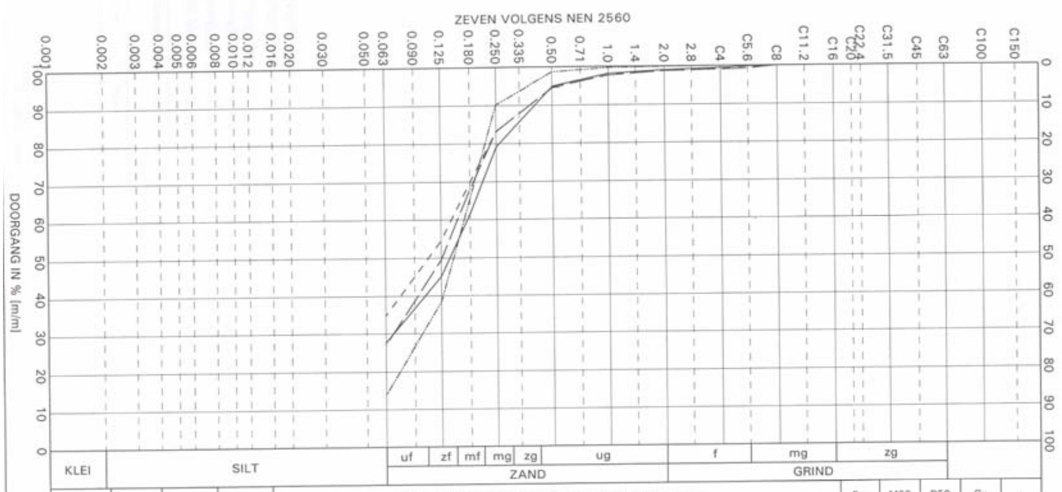
AHM PSD 2.13 /11:04:47/H:*.PSD Opg.:AWG dd:20-Okt-2005 Gec.: dd: 21-10-05



BORING NR.	MONSTER NR.	DIEPTE [m]	GRAFIEK	C100	C63	C31.5	C16	C8	C4	2.0	1.0	0.50	0.250	0.125	0.063	<.063	<.016	<.002	Fm / CuZND	M63 / M2000	D50 / Dm	Cu / Cc	U cijfer
B1	1	-1.10 / -1.20	———							0.0	0.2	0.8	18.6	79.9	96.1	3.9			1.00 / 2.2	190 / --	186 / 192	2.5 / 1.2	75
B1	2	-1.20 / -1.35	– – –							0.0	0.1	0.9	20.6	79.2	93.3	6.7			1.01 / 2.2	199 / --	193 / 196	2.8 / 1.4	81
B1	4	-2.00 / -2.40	—— –					0.0	0.7	1.7	3.4	7.7	22.4	48.6	68.0	32.0			.84 / 2.7	181 / --	119 / 158	0.8	169
B1	8	-3.75 / -4.00	-·-·-							0.0	0.2	1.1	9.1	62.6	89.0	11.0			.73 / 2.2	157 / --	147 / 149	1.0	109

Opm.: Diepte is in meters tov. maaiveld

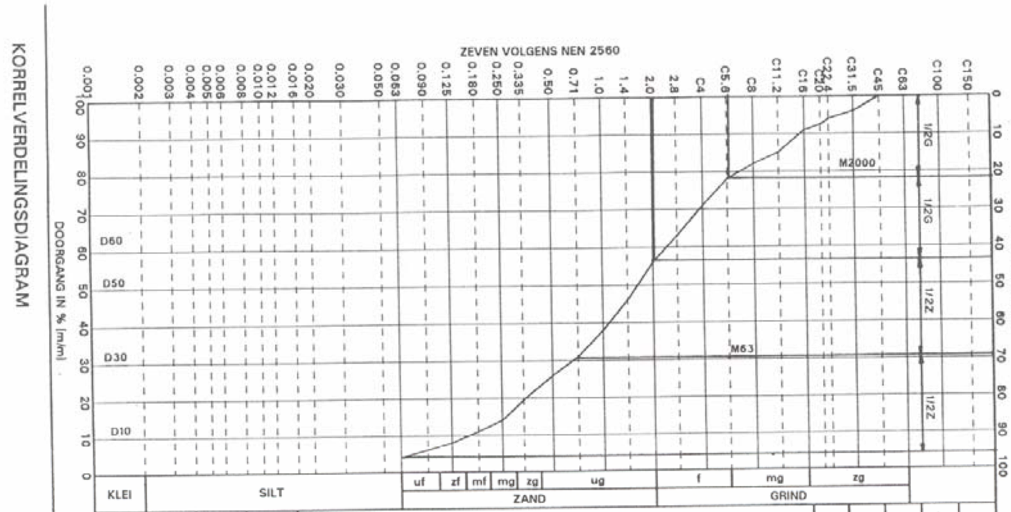
AHM PSD 2.13 /11:04:54/H:*.PSD Opg.:AWG dd:20-Okt-2005 Gec.: dd: 21-10-05



BORING NR.	MONSTER NR.	DIEPTE [m]	GRAFIEK	C100	C63	C31.5	C16	C8	C4	2.0	1.0	0.50	0.250	0.125	0.063	<.063	<.016	<.002	Fm / CuZND	M63 / M2000	D50 / Dm	Cu / Cc	U cijfer
B2	1	-0.80 / -1.20	———					0.0	0.5	1.0	1.9	5.1	20.9	54.9	72.4	27.6			.84 / 2.6	186 / --	139 / 160	0.9	154
B2	3	-1.75 / -2.00	– – –					0.0	0.7	1.1	2.3	5.5	16.9	45.5	65.2	34.8			.72 / 2.5	168 / --	107 / 137	0.8	181
B2	6	-2.95 / -3.20	—— –					0.0	1.0	1.3	2.3	5.4	16.9	50.5	73.1	26.9			.77 / 2.4	162 / --	126 / 148	0.8	156
B2	7	-3.45 / -3.60	-·-·-						0.0	0.1	0.3	1.3	9.9	61.6	86.1	13.9			.73 / 2.2	160 / --	146 / 148	1.0	117

Opm.: Diepte is in meters tov. maaiveld

VERKLARING PARAMETERS UIT KORRELVERDELING



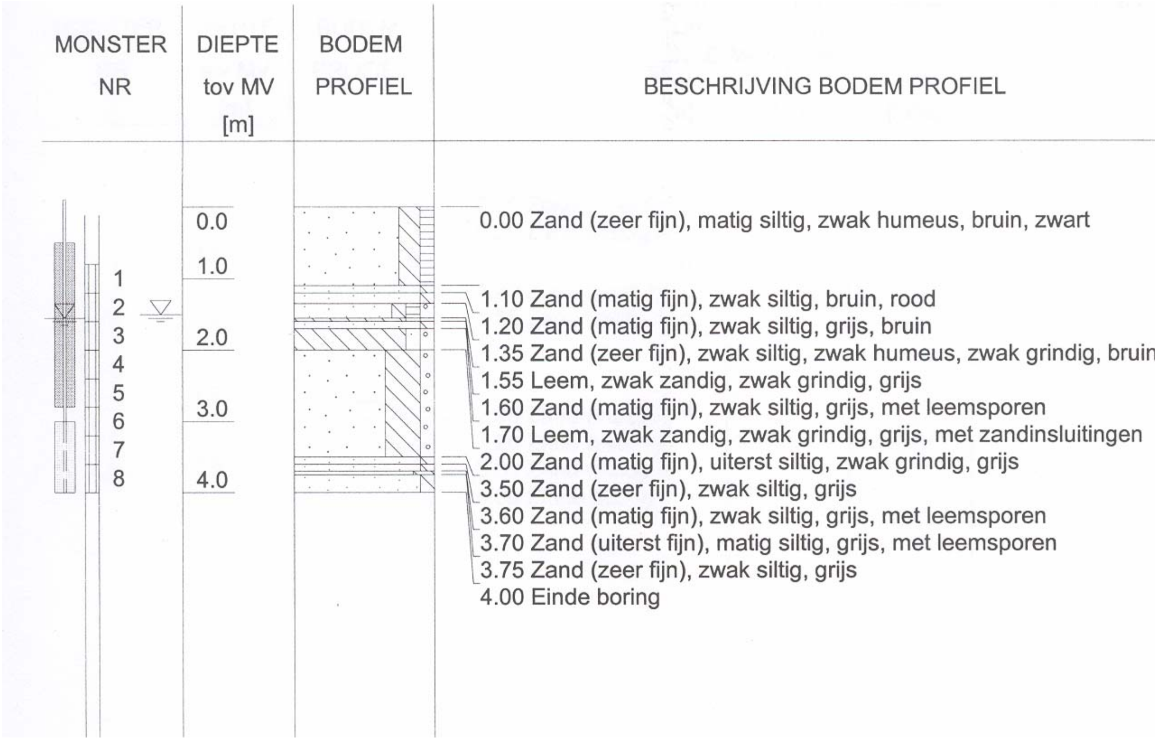
BORING NR.	MONSTER NR.	DIEPTE [m]	GRAFIEK	TOTALE ZEEFREST IN MASSA % T.O.V. DROGE STOF OP ZEEF															Fm / CuZND	M63 / M2000	D50 / Dm	Cu / Cc	U cijfer
				C100	C63	C31.5	C16	C8	C4	2.0	1.0	0.50	0.250	0.125	0.063	<.063	<.016	<.002					
1	1		——			4.0	9.0	18.0	29.0	44.0	62.0	74.0	86.0	92.0	96.0	4.0			4.18 / 6.0	662 / 5.6	1615 / 3543	15.2 / 1.1	30

Fm (fijnheidsgetal)	:	som van de massapercentages op de zeven: C63, C31.5, C16, C8, C4, 2mm, 1mm, 500 μm, 250 μm en 125 μm, gedeeld door 100.
M63 (zandmediaan)	:	gemiddelde korrelgrootte van de zandfractie in μm, waarbij 63 μm staat voor de ondergrens en 2 mm voor de bovengrens.
M2000 (grindmediaan)	:	gemiddelde korrelgrootte van de grindfractie in mm, waarbij 2mm staat voor de ondergrens en 63 mm voor de bovengrens.
D50	:	de gemiddelde korrelgrootte van al het materiaal in μm.
Dm	:	de som van de zeefdoorgang in μm, per massapercentage in stappen van 10 (10 t/m 90%), gedeeld door 9.
Cu (gelijkmatigheids coëfficiënt	:	D60/D10 is het quotiënt van de afmetingen van de denkbeeldige zeefopeningen, waardoor 60% en 10% van al het materiaal doorgaat.
CuZND (gelijkmatigheids coëfficiënt van materiaal >63 μm / < 2 mm	:	D60/D10 is het quotiënt van de afmetingen van de denkbeeldige zeefopeningen, waardoor 60% en 10% van het materiaal tussen 63 μm en 2mm doorgaat.
Cc (krommingscoëfficiënt)	:	[$D30^2$/(D60 x D10)] is het quotiënt van het kwadraat van de denkbeeldige zeefopeningen, waardoor 30% van al het materiaal doorgaat en het product van de denkbeeldige zeefopeningen, waardoor 60% en 10% van al het materiaal doorgaat.
U-cijfer	:	specifiek oppervlak zandfractie, berekend als:

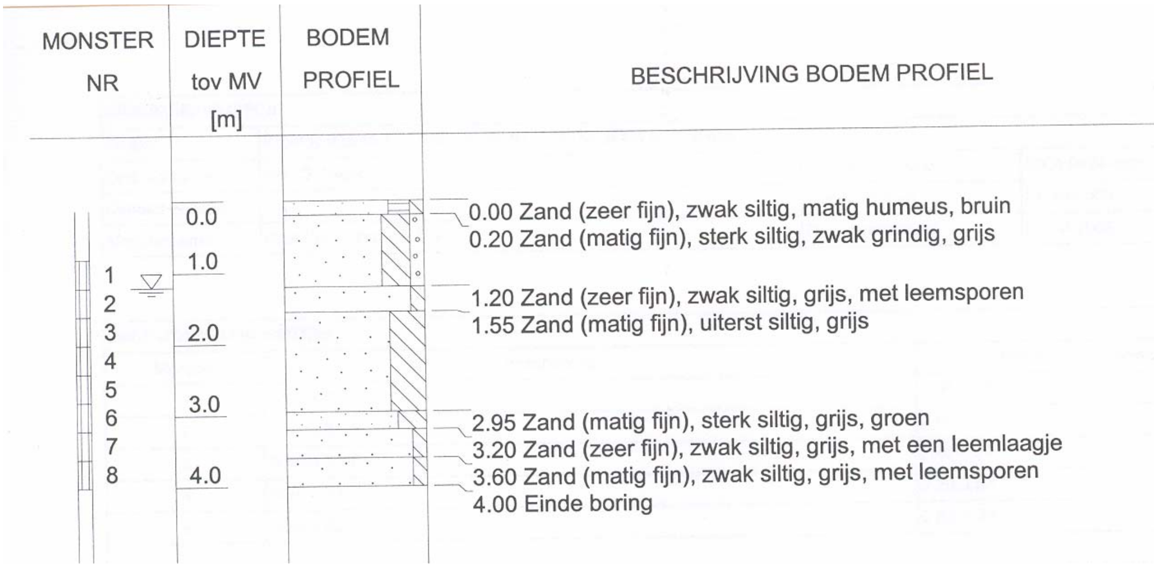
$$\frac{\Sigma_n: (m_1 \times u_1) + (m_2 \times u_2) \ldots (m_n \times u_n)}{\text{massa zandfractie}}$$

waarin: m_1, m_2, etc. = massa subfractie
u_1, u_2, etc. = specifiek oppervlak subfractie

Appendix 11 Bore Stats



Bore stat 1



Bore stat 2

Appendix 12 Settlement Prediction

50 kPa

Depth top	Depth Bottom	Center of Layer	Sigma top	Sigma bottom	Gamma	Sigma	Sigma water
-0.60	-1.00	-0.80	0.00	18.00	18.00	14.40	0.00
-1.00	-2.20	-1.60	18.00	42.00	20.00	30.00	6.00
-2.20	-3.50	-2.85	42.00	67.00	19.00	54.35	18.5
-3.50	-10.00	-6.75	67.00	197.00	20.00	132.00	57.5

Effective = p0	Delta p	C	z (m)	te (second)	te (days)
14.40	37.50	1000	0.000512836	8.00E-04	9.25926E-09
24.00	18.75	1000	0.000692778	7.20E-03	8.33333E-08
35.85	10.53	87	0.00386168	8.45E+05	9.780092593
74.50	4.44	1000	0.000376644	2.11E-01	2.44502E-06

0,1 day (1%)	Secundair	1dag (10%)	Secundair
0.0000051	-	0.0000513	-
0.0000069	-	0.0000693	-
0.0000335	0.0000051	0.0003347	5.14891E-05
0.0000038	-	0.0000377	-
0.0000493	0.0000051	0.0004929	5.14891E-05
Totaal	0.0000544	Totaal	0.0005444

Strip Width	Load	C'p	C's	reloading factor
0.6	50	25	650	4
C's/c'p	Factor	Primairy	Secundair	
26	30	0.866666667	0.133333333	

Total	**0.0054**	m

Cv sand	1.00E+02
Cv loam	1.00E-06

100 kPa

Depth top	Depth bottom	Centre layer	Sigma top	Sigma bottom	Gamma	Sigma	Sigma water
-0.6	-1.0	-0.8	0.0	18.0	18.0	14.4	0
-1.0	-2.2	-1.6	18.0	42.0	20.0	30.0	6
-2.2	-3.5	-2.9	42.0	67.0	19.0	54.4	18.5
-3.5	-10.0	-6.8	67.0	197.0	20.0	132.0	57.5

Effective = p0	Delta p	C	z (m)	te (second)	te (days)
14.4	75	1000	0.000730357	0.0008	9.25926E-09
24	37.5	1000	0.00112918	0.0072	8.33333E-08
35.85	21.05263158	86.66666667	0.00692997	845000	9.780092593
74.5	8.888888889	1000	0.000732654	0.21125	2.44502E-06

0,1 dag (1%)	Secundair	1dag (10%)	Secundair
0.0000073	-	0.0000730	-
0.0000113	-	0.0001129	-
0.0000601	0.0000092	0.0006006	0.0000924
0.0000073	-	0.0000733	-
0.0000860	0.0000092	0.0008598	0.0000924
Total	0.0000952	Total	0.000952216

Strip Width	Load	C'p	C's	Recompressionfactor
0.6	100	25	650	4
C's/c'p	Factor	Primair	Secundair	
26	30	0.866666667	0.133333333	

Total	**0.009522161** m

Cv zand	1.00E+02
Cv leem	1.00E-06

50 + 50 kPa

Depth top	Depth bottom	Centre layer	Sigma top	Sigma bottom	Gamma	Sigma	Sigma water
-0.60	-1.00	-0.80	0.00	18.00	18.00	14.40	0.00
-1.00	-2.20	-1.60	18.00	42.00	20.00	30.00	6.00
-2.20	-3.50	-2.85	42.00	67.00	19.00	54.35	18.50
-3.50	-10.00	-6.75	67.00	197.00	20.00	132.00	57.50

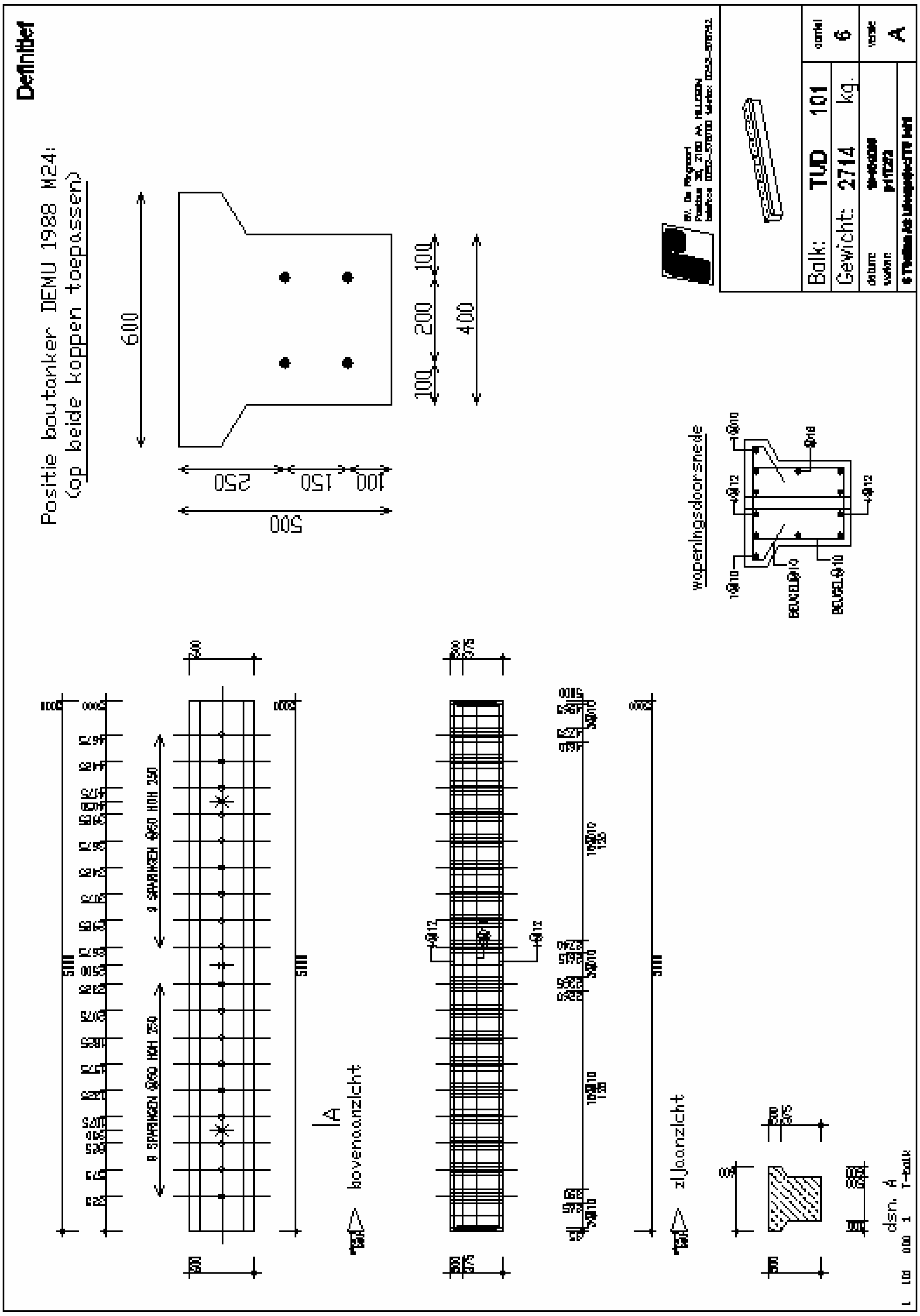
Effective = p0	Effective=p1	Delta p	C	z (m) p0	z (m) p1	te (second)	te (days)
14.40	51.90	37.50	1000.00	0.0005	0.0001	8.00E-04	9.25926E-09
24.00	42.75	18.75	1000.00	0.0007	0.0002	7.20E-03	8.33333E-08
35.85	46.38	10.53	86.67	0.0039	0.0015	8.45E+05	9.780092593
74.50	78.94	4.44	1000.00	0.0004	0.0002	2.11E-01	2.44502E-06

Strip Width	Load	C'p	C's	Recompression factor	
0.6	50	25	650	4	

Total 0	0.0054	m
Total 1	0.0020	m
Total	**0.0075**	

Cv sand	1.00E+02
Cv loam	1.00E-06

Appendix 13 Detailed Schematic



Appendix 14 Calculations for the Test Facilities

Steel Profile
The steel profiles have an own weight of 930 N/m
Maximum load is (20 x 11.919 + 7,5 x 1.635 + 3,8 x 930)/2 = 127.088 N.

The load on the middle profile is the biggest. The profile in the middle has a half of the total amount.

- $E_{steel} = 2{,}1 \cdot 10^5 \, N/mm^2$
- $I_y = 14919 \cdot 10^4 mm$
- $l_{eff} = 3000mm$
- $F = 127.088N$
- $q = \frac{2 \cdot F}{l_{eff}} = 84,73 \, N/mm$
- $A_{strip} = 400 \cdot 260 = 104.000mm^2$
- $\sigma_{strip} = \frac{F}{A_{strip}} = 1,22 \, N/mm^2$
- $\phi = \frac{q \cdot l_{eff}^3}{24 \cdot EI} = 3,04 \cdot 10^{-3} rad$
- $u = \frac{5}{384} \cdot \frac{q \cdot l_{eff}^4}{EI} = 2{,}85mm$

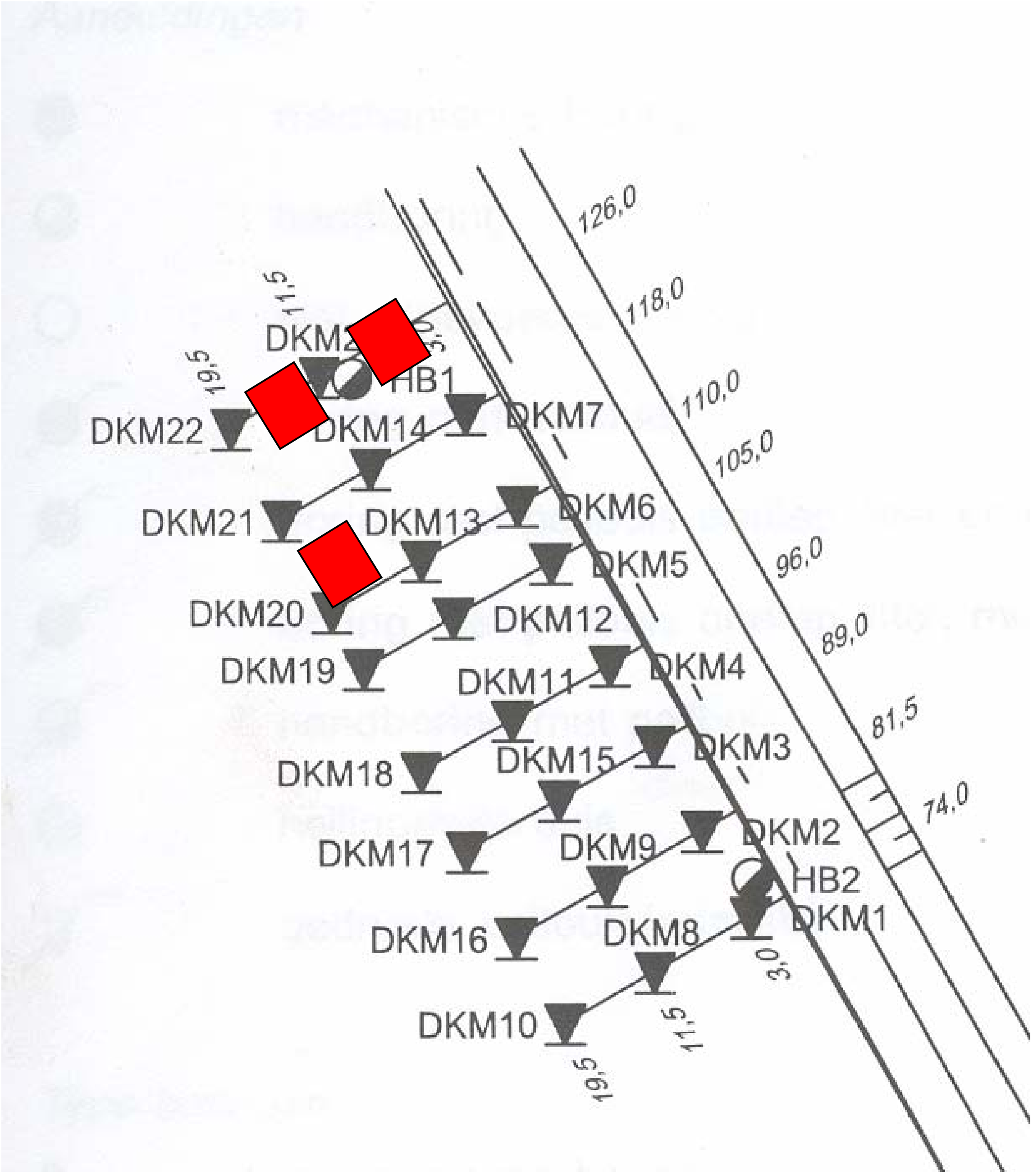
Dragline Floor Plate

Maximum load 5/8 x (40 x 11.919 + 15 x 1.635) = 313.303 N.

Contact area between dragline floor plate and steel profile 260 x 3000 = 780.000 mm^2.

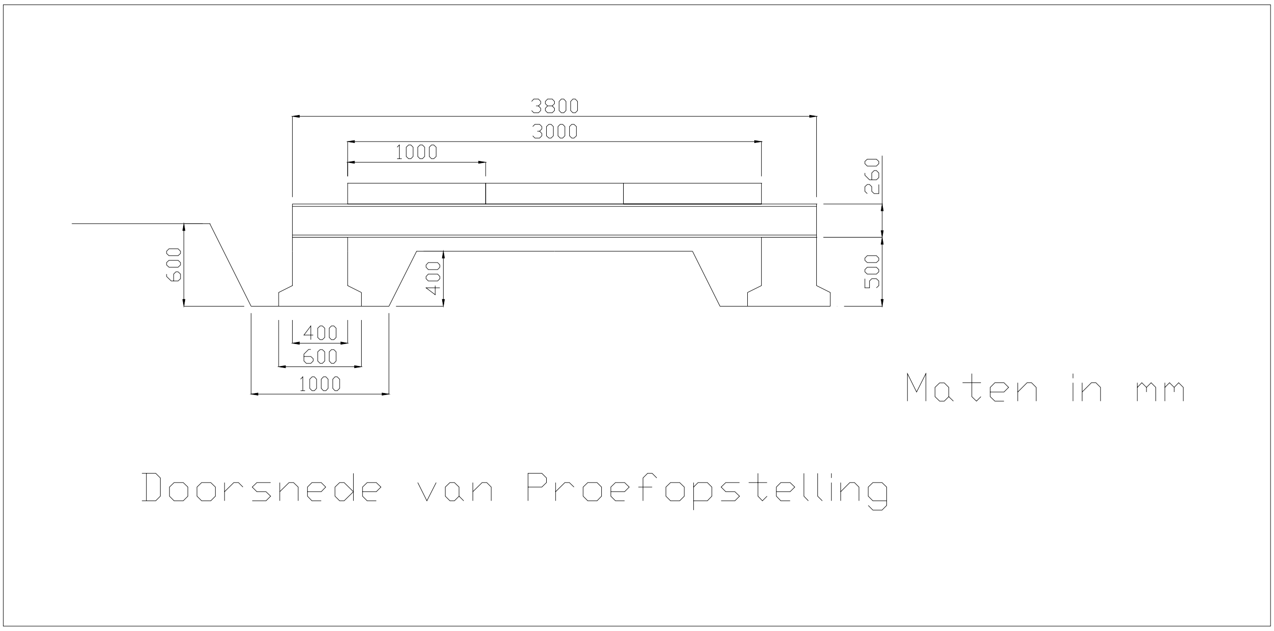
- Dragline Floor Plate: $1000kg$
- $DFP = \frac{(1000 \cdot 9{,}81)}{6} = 1635 \, N/m^2$
- $A_{HE\,260B} = 260 \cdot 3000 = 780.000mm^2$
- $c = 1{,}0$ but for steel it has a value of 0,04 so extremely safe
- $\sigma = \frac{F}{A_{HE\,260B}} = \frac{313.303}{780.000} = 0,40 \, N/mm^2$

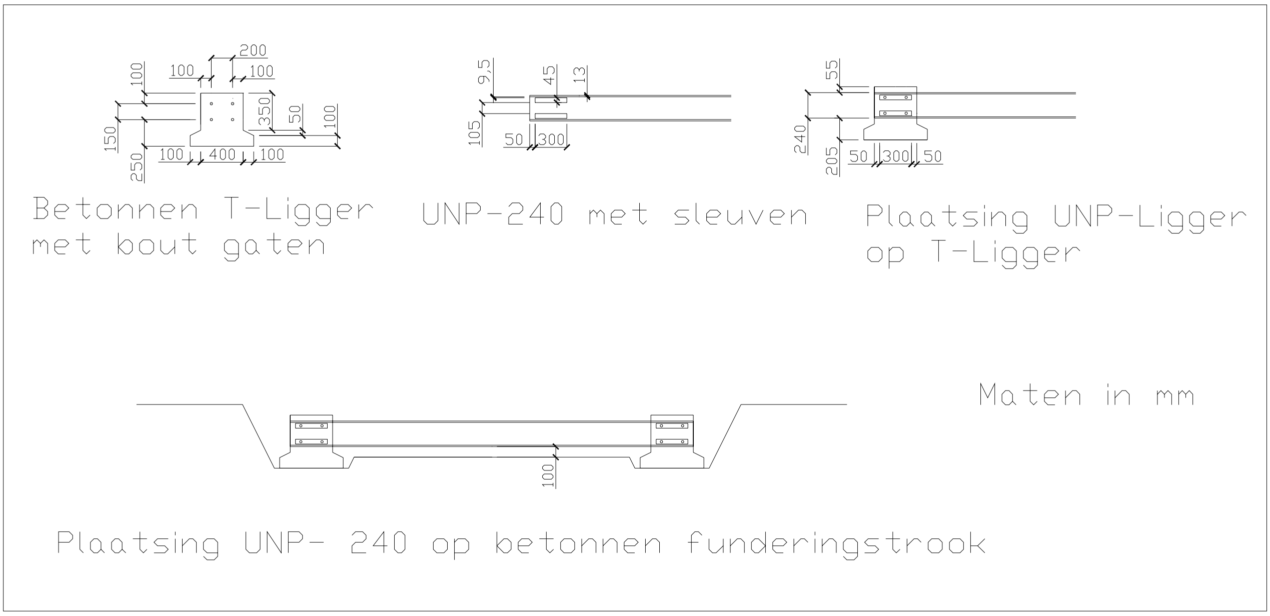
Appendix 15 Test Facility Positions

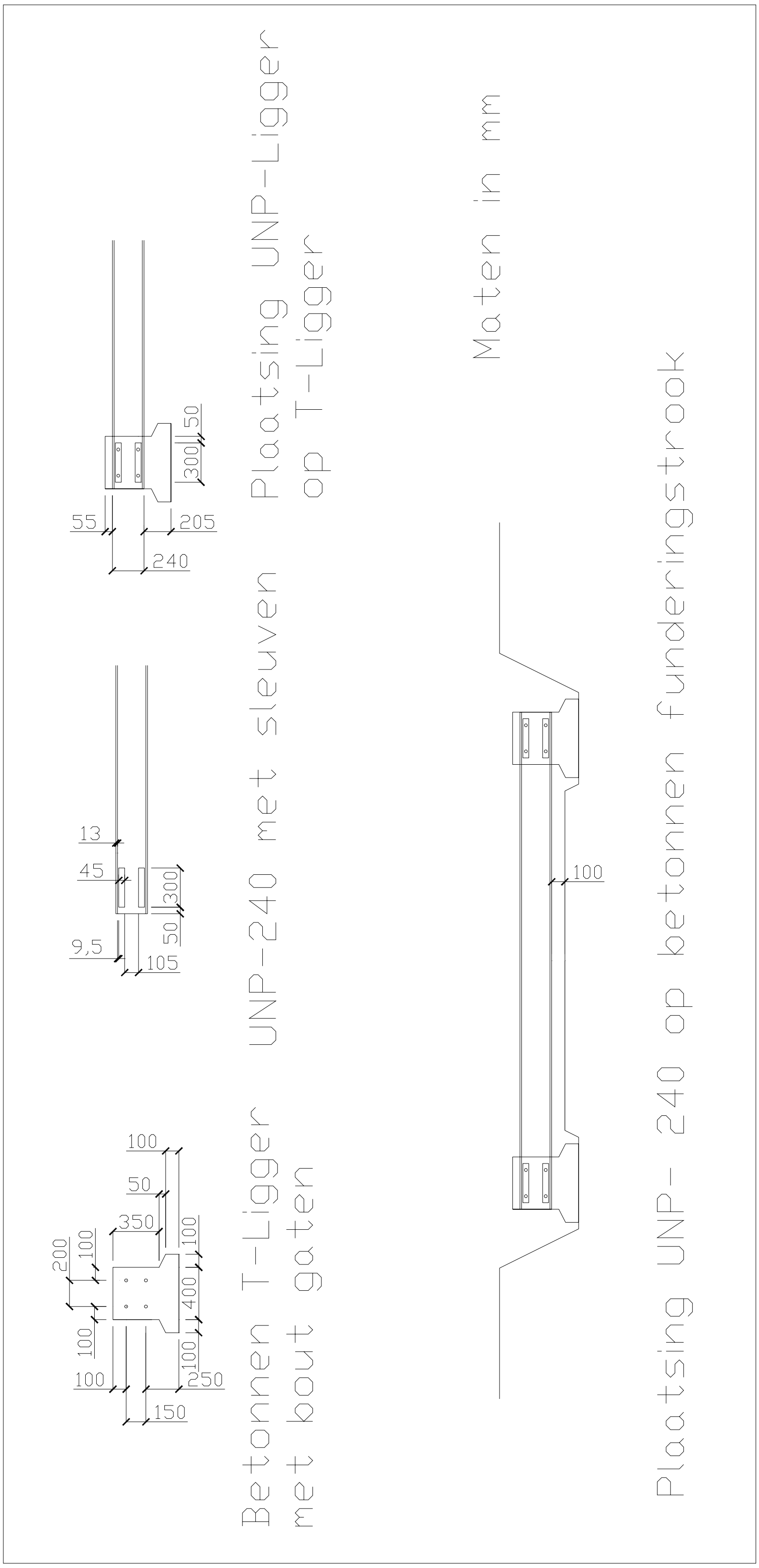


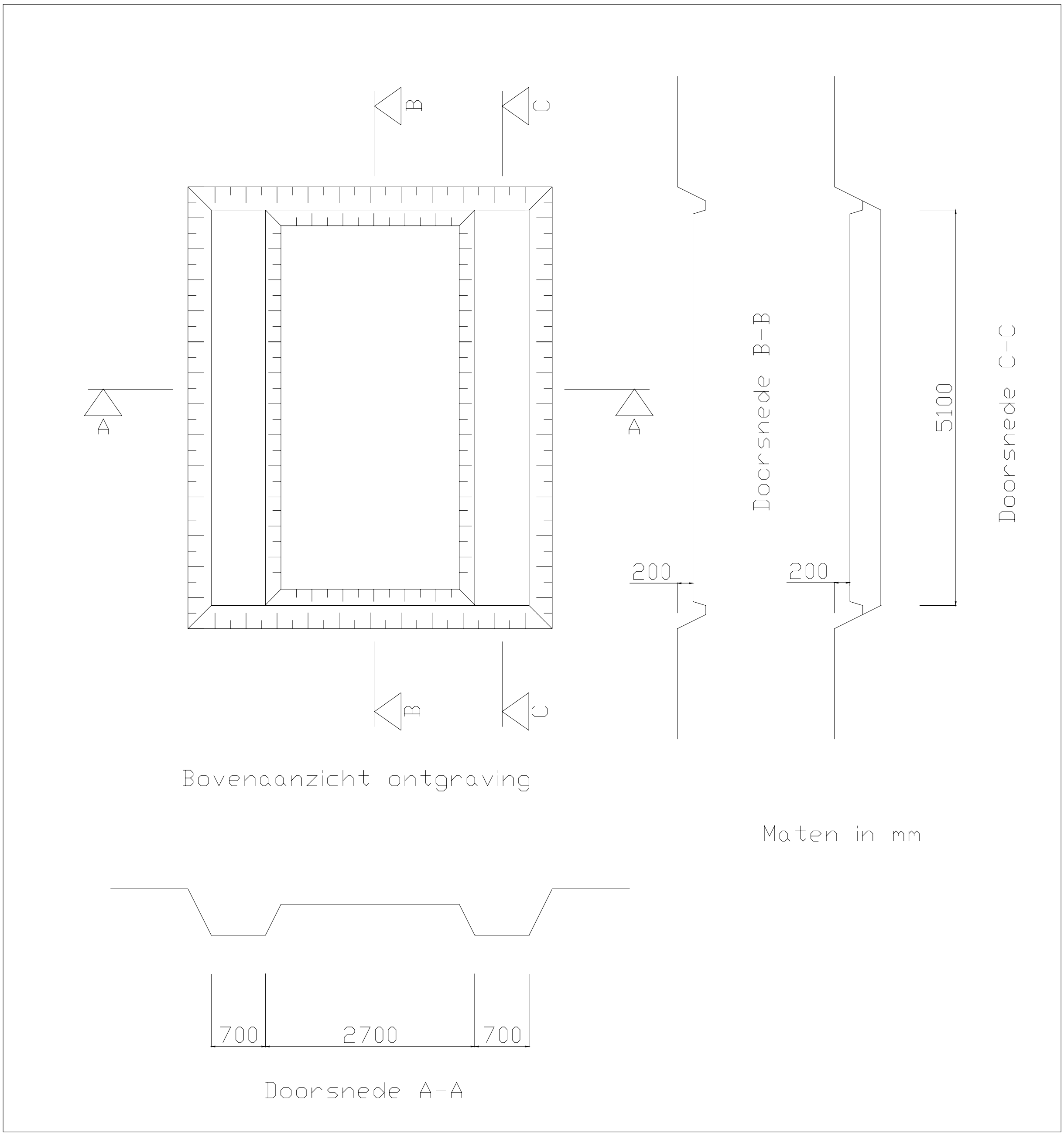
The red squares are the positions of the Test Facilities

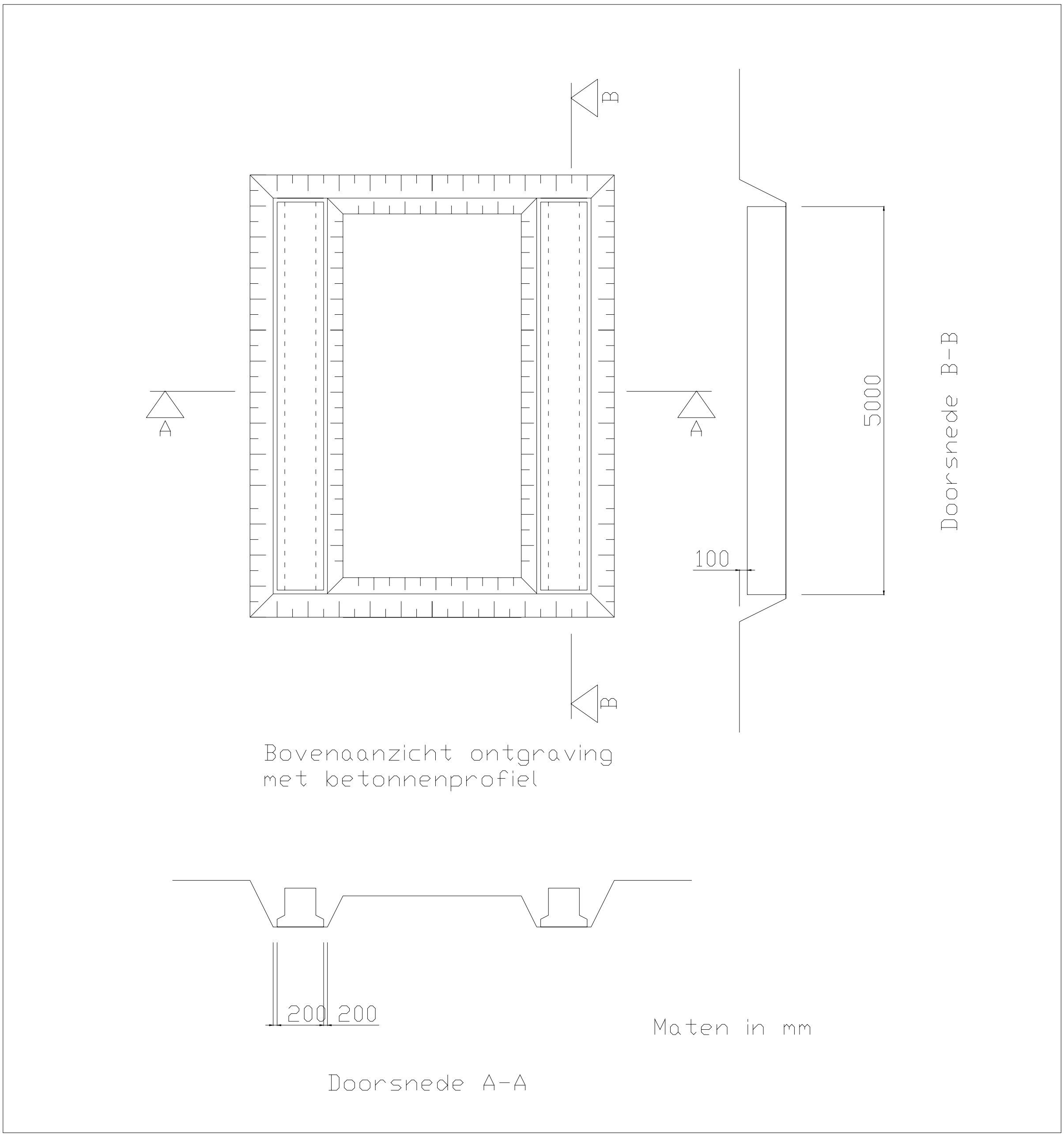
Appendix 16 Schematic of the Test Facilities

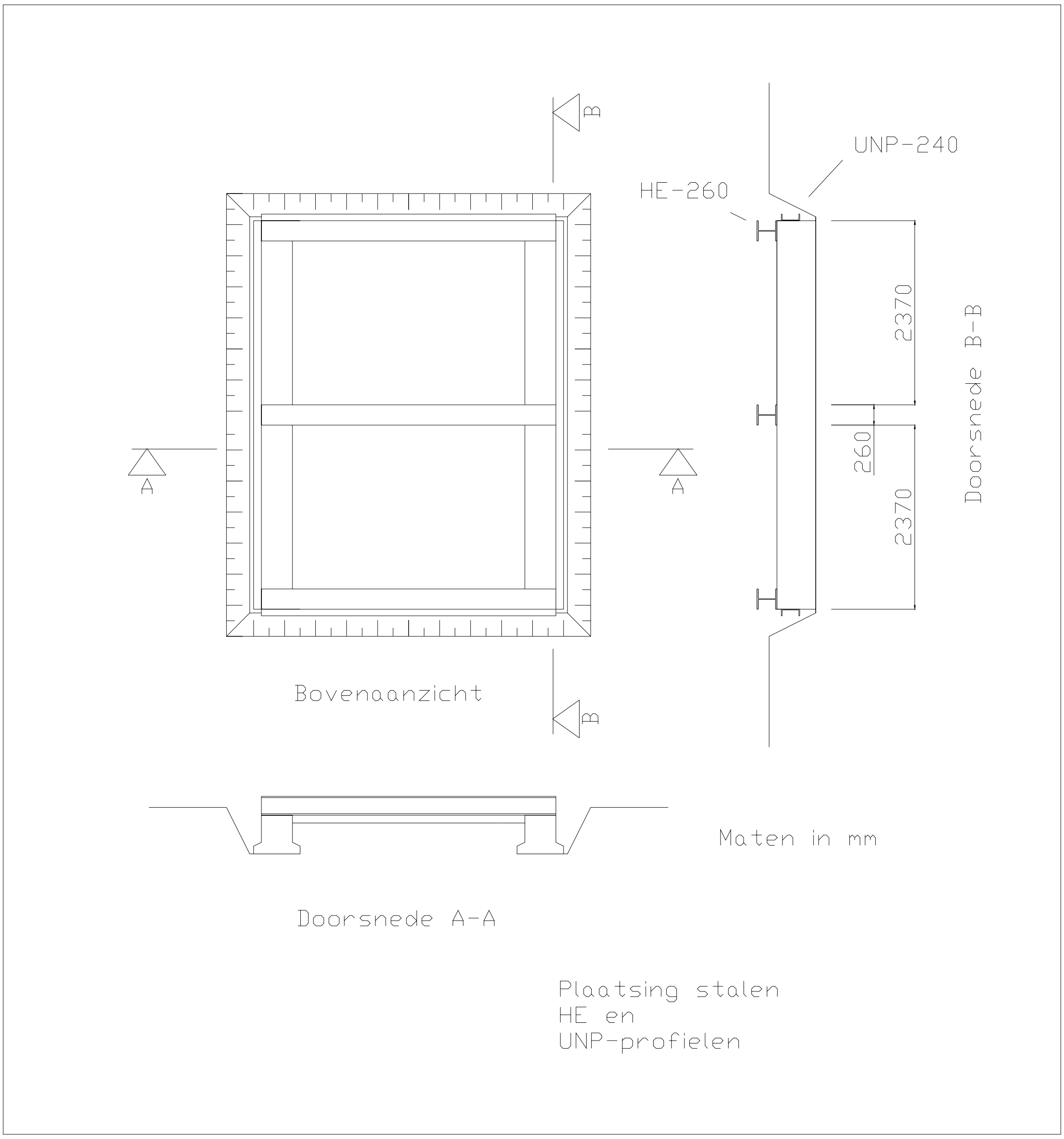


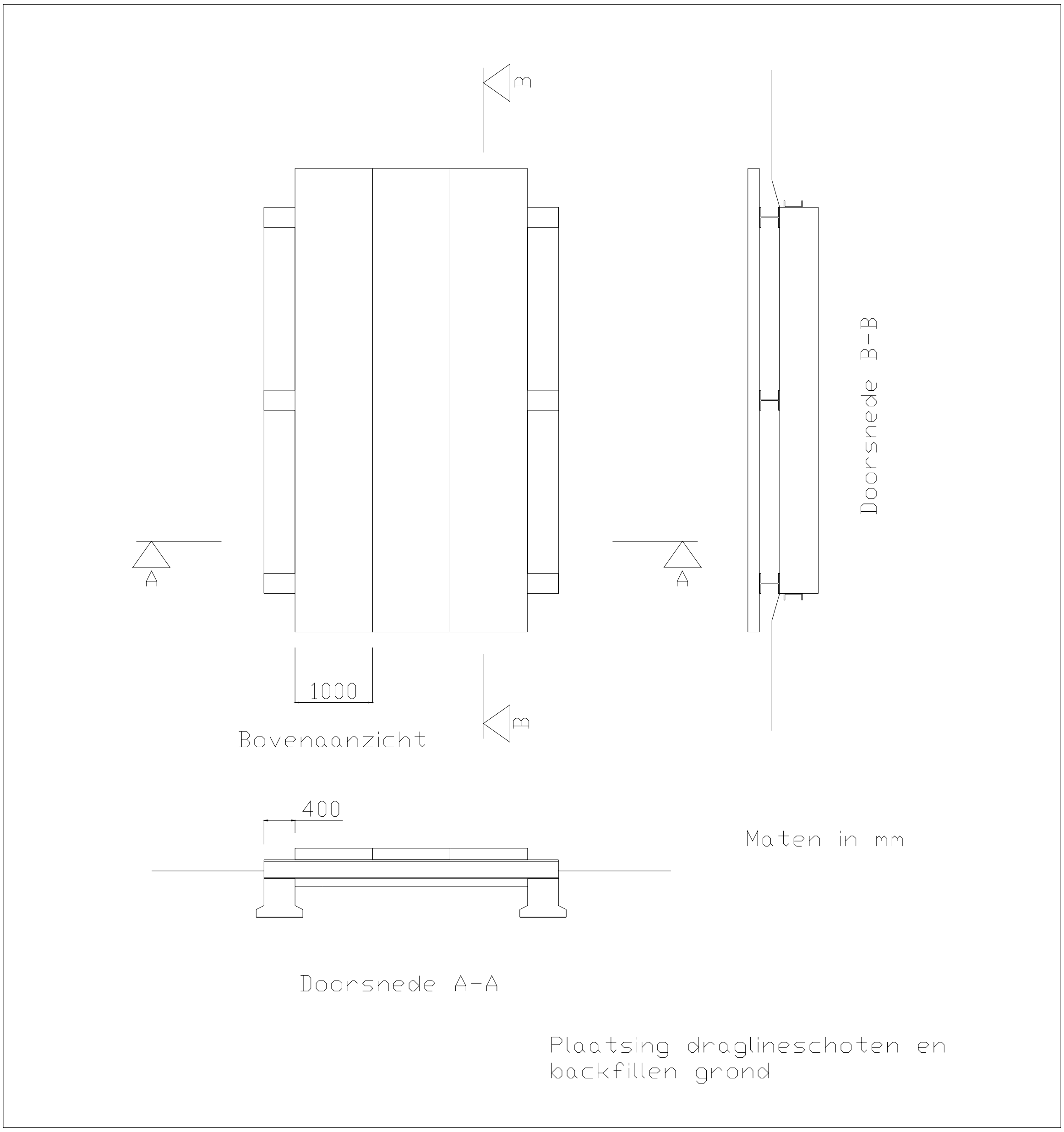


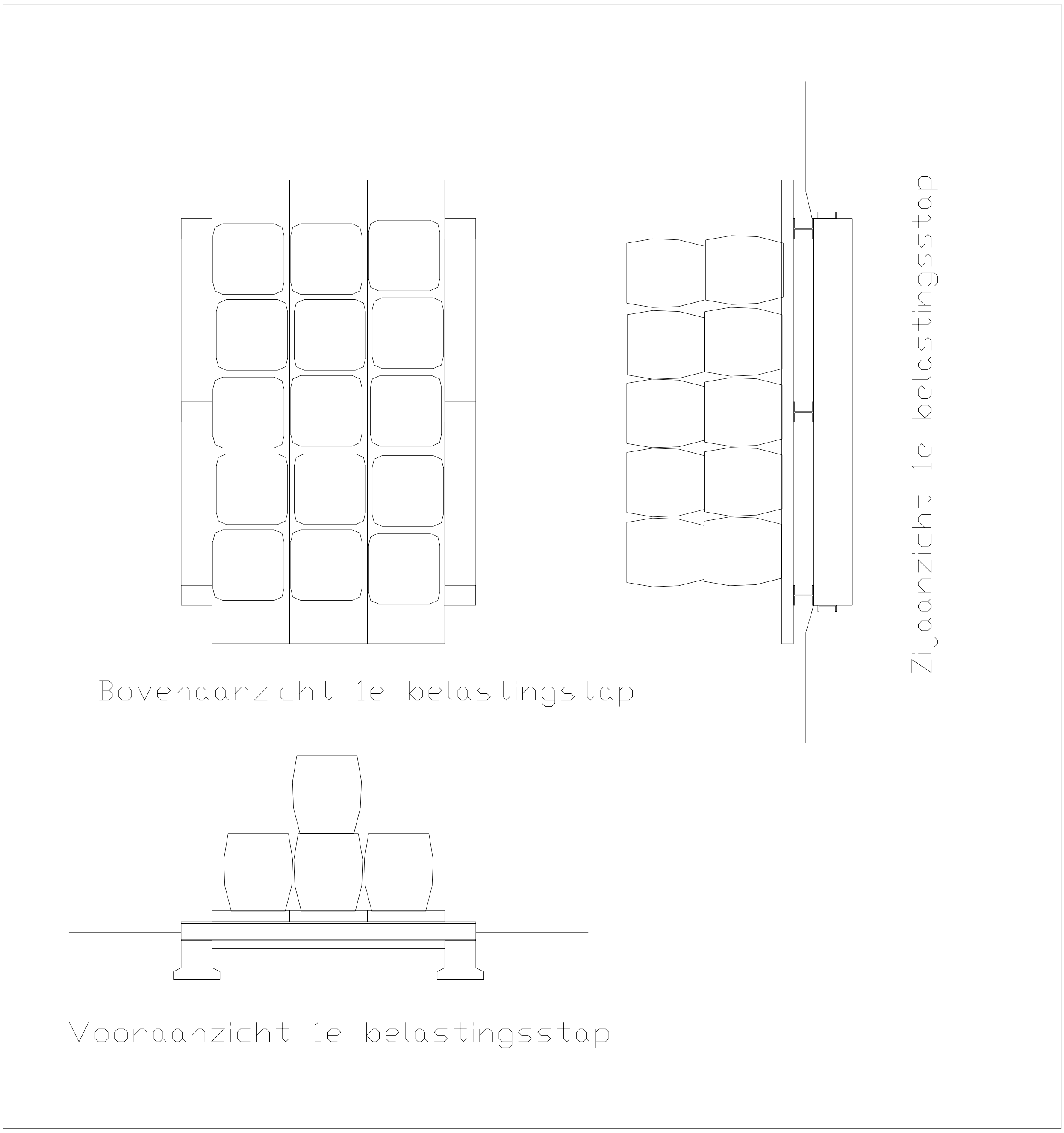

Betonnen T-Ligger
met bout gaten
UNP-240 met sleuven
Plaatsing UNP-Ligger
op T-Ligger
Maten in mm
Plaatsing UNP- 240 op betonnen funderingstrook

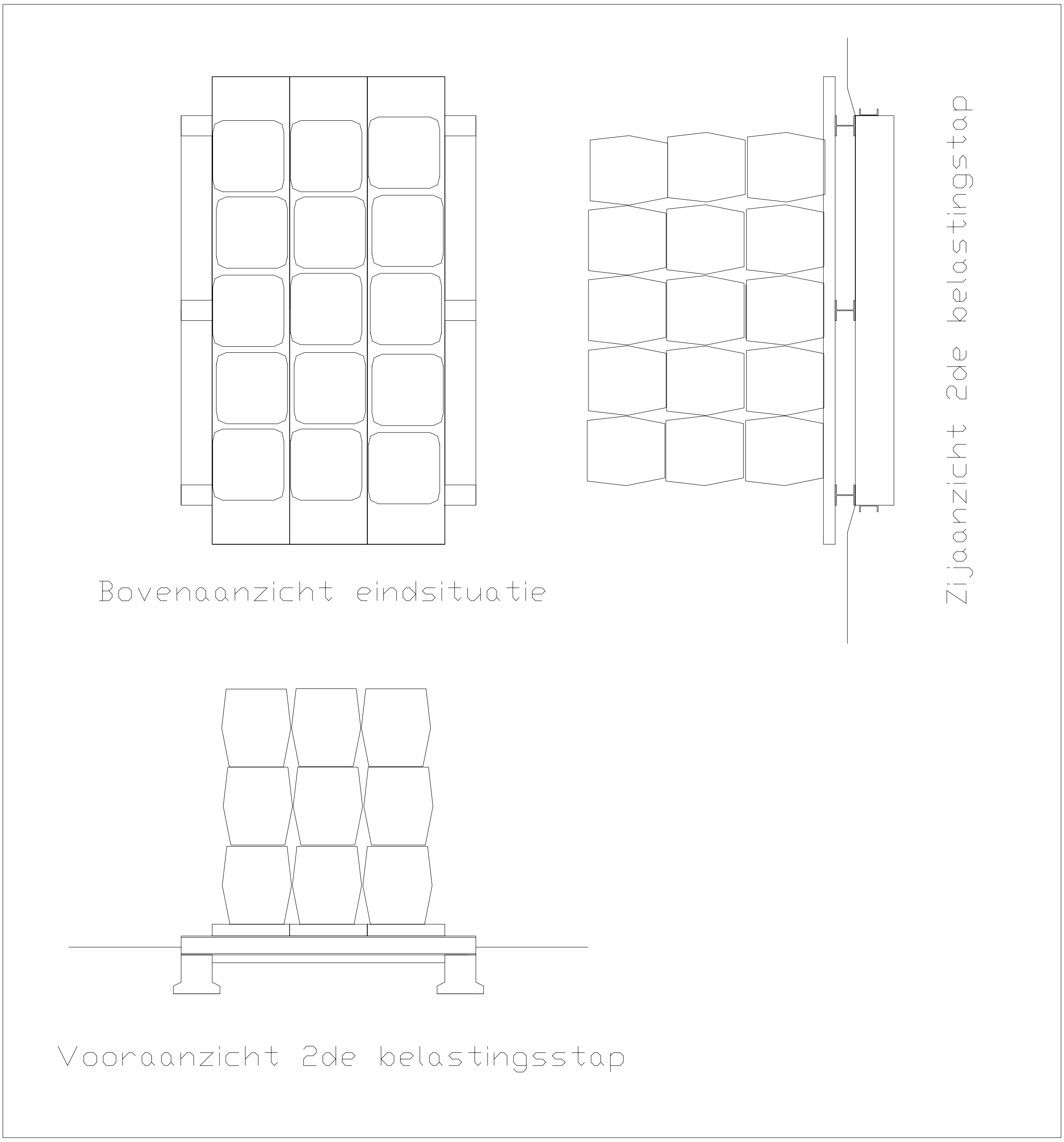

B
C
A
A
B
C
Doorsnede B-B
Doorsnede C-C
5100
200
200
Bovenaanzicht ontgraving
Maten in mm
700
2700
700
Doorsnede A-A


B
A
A
5000
Doorsnede B-B
100
B
Bovenaanzicht ontgraving
met betonnenprofiel
200 200
Maten in mm
Doorsnede A-A


B
UNP-240
HE-260
2370
260
2370
Doorsnede B-B
A
A
Bovenaanzicht
B
Maten in mm
Doorsnede A-A
Plaatsing stalen
HE en
UNP-profielen


B
B
Doorsnede B-B
A
A
1000
Bovenaanzicht
400
Maten in mm
Doorsnede A-A
Plaatsing draglineschoten en
backfillen grond


Bovenaanzicht 1e belastingstap
Zijaanzicht 1e belastingsstap
Vooraanzicht 1e belastingsstap


Bovenaanzicht eindsituatie
Zijaanzicht 2de belastingstap
Vooraanzicht 2de belastingsstap

Appendix 17 Weight Determination

Load				
What	*Weight*	*Test Facility*	*Load*	
Foundation strips	15864	All Facilities	5288	Per Facility
Steel	4015	All Facilities	1338	Per Facility
Dragline Floor plates	4020	Long Term	4020	
	4260	Control	4260	
	4000	Short Term	4000	
Big bags	14480	Long Term 1		
	14880	Long Term 2	29360	
	14780	Control 1		
	14820	Control 2	29600	
	14840	Short Term 1		
	14760	Short Term 2	29600	

What	*Long term*	*Control*	*Short Term*
Foundation strips	5288	5288	5288
Steel	1338	1338	1338
Dragline Floor Plates	4020	4260	4000
Big bags first stage	29360	29600	29600
Big bags second stage	37667	37487	36987
Total (with facility)	77673	77973	77213
Total (without facility)	72385	72685	71925

Second Load Stage		
What	*Weight*	*Total per Facility*
Control first 10	15740	
Control second 10	15600	37487
Long Term first 10	15680	
Long Term second 10	15840	37667
Short Term first 10	15620	
Short Term second 10	15220	36987
Last four	18440	6147

Foundation pressure			
Length	5	m	
Width	0.6	m	
Amount	2	[-]	
	With Facility	*Without Facility*	*kPa (/1000)*
Long Term	126995	118349	118.3
Control	127486	118840	118.8
Short Term	126243	117597	117.6
Second load stage			
Total (tons)	35424	ton	
Total (kN)	347.50944	kN	
Area	6	m^2	
Foundation Pressure	57.91824	kPa	

Appendix 18 Test Facility Construction Pictures









AHLMANN













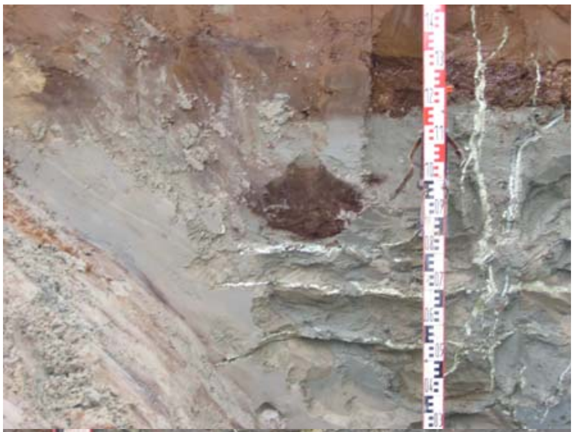


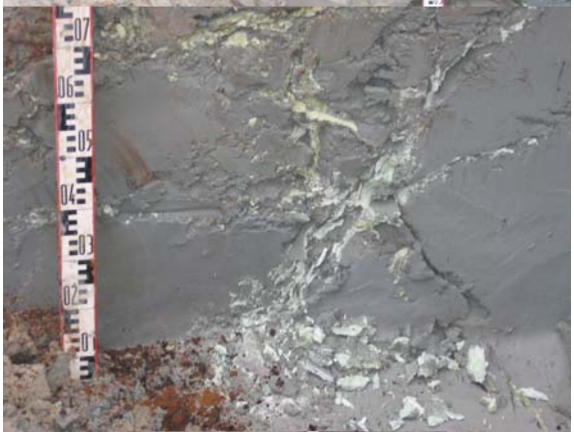
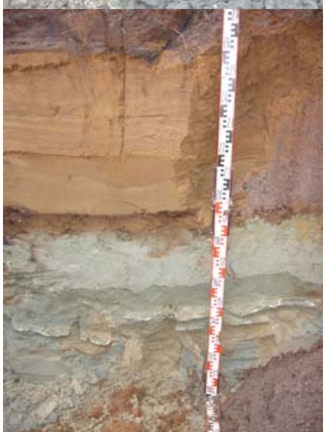
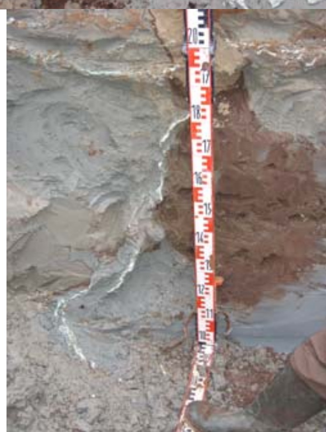




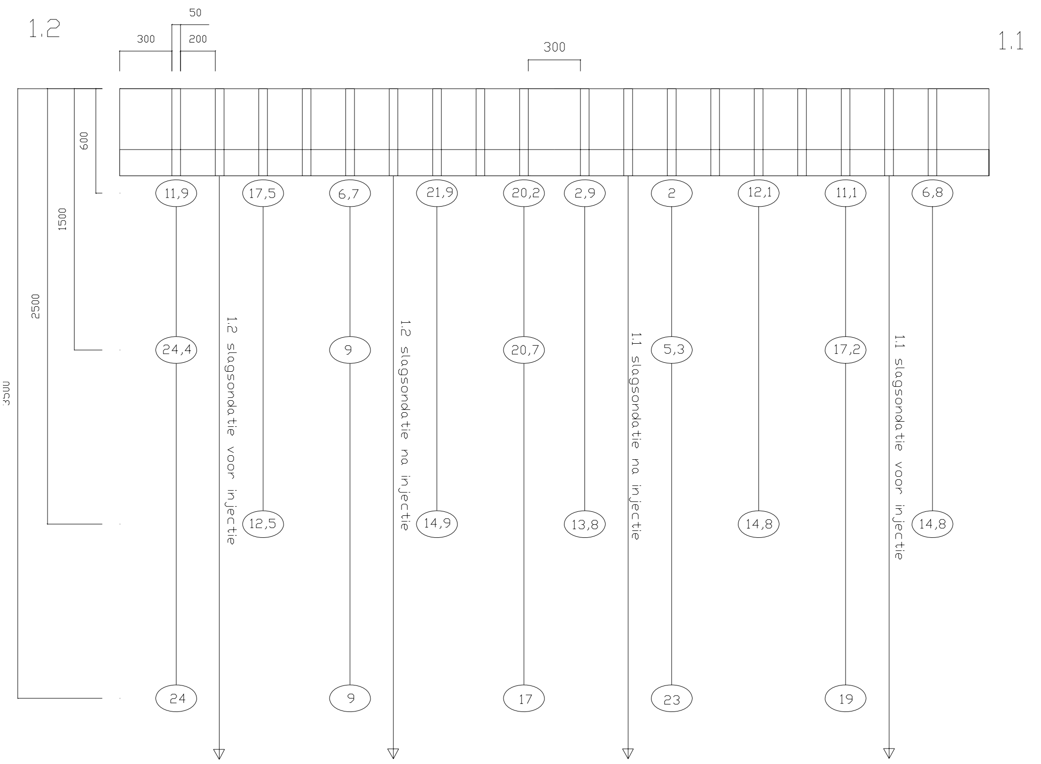


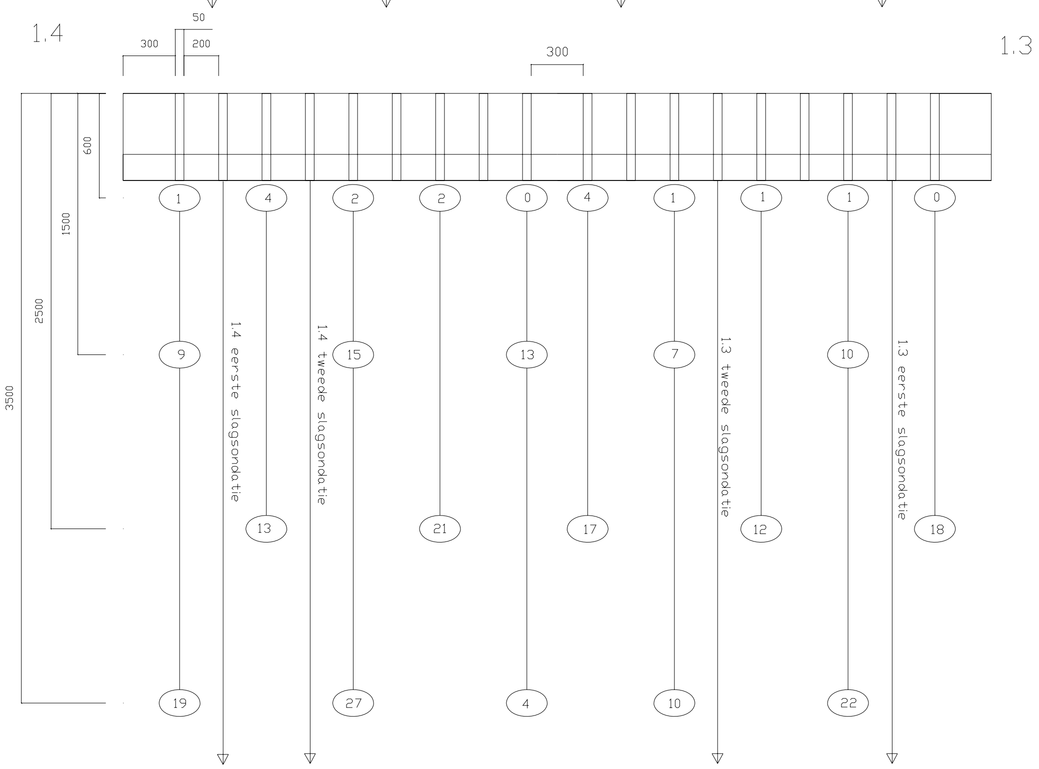


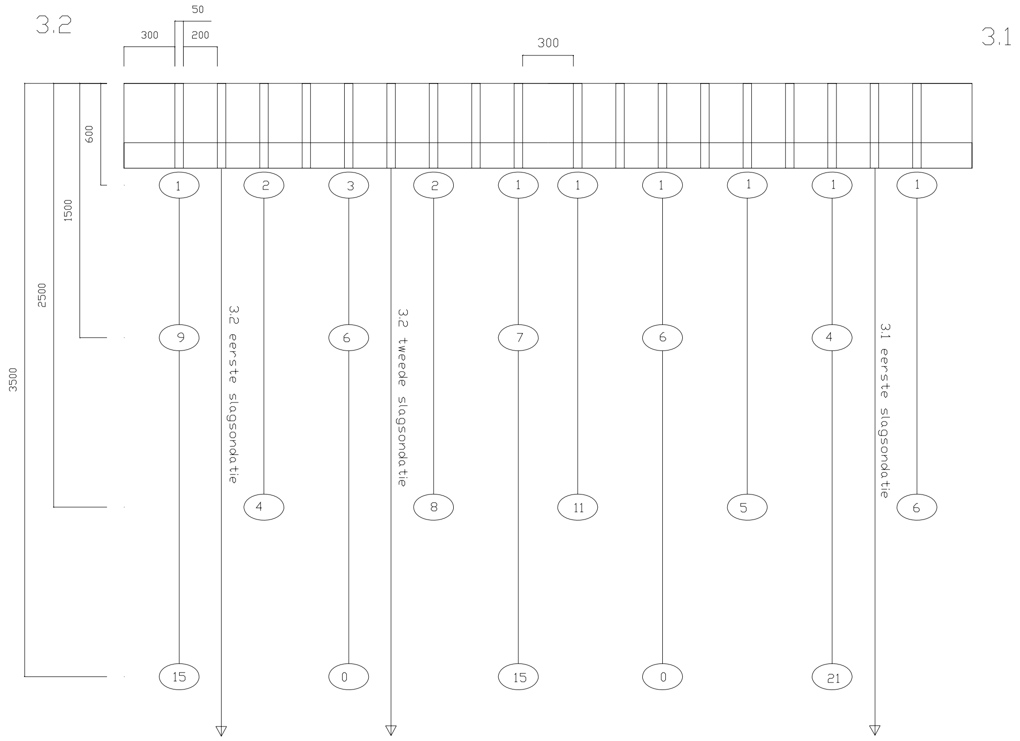


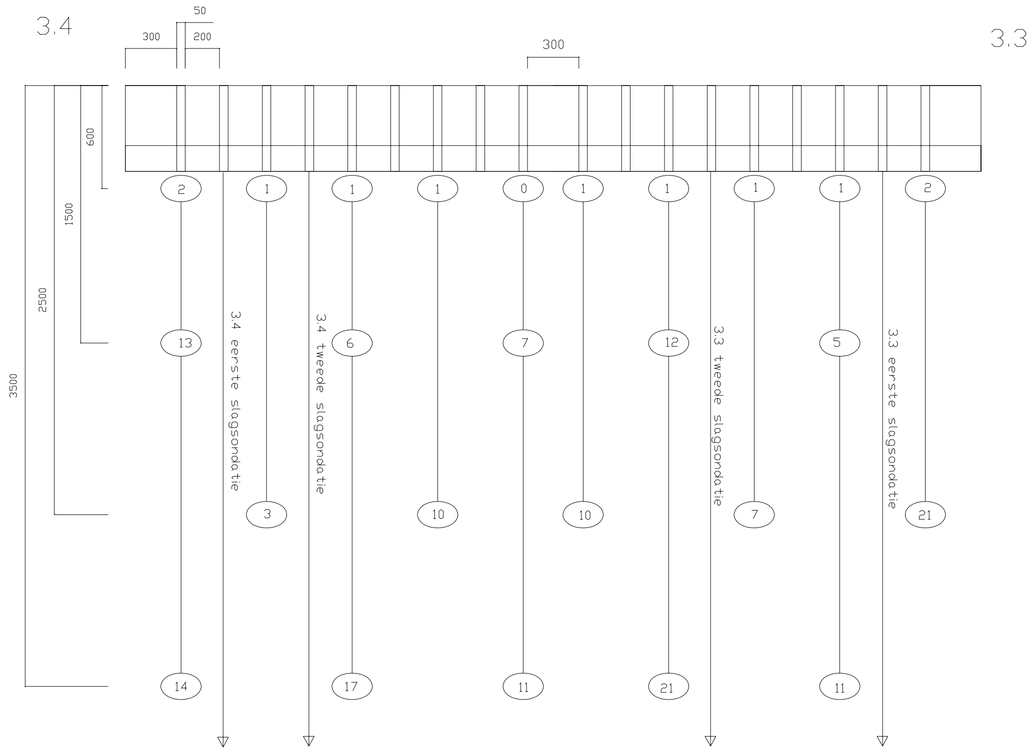




Appendix 19 Injection Schedule






3.2
3.1
50
300
200
300
600
1500
2500
3500
3.2 eerste slagsondatie
3.2 tweede slagsondatie
3.1 eerste slagsondatie


3.4
3.3
50
300
200
300
600
1500
2500
3500
3.4 eerste slagsondatie
3.4 tweede slagsondatie
3.3 tweede slagsondatie
3.3 eerste slagsondatie

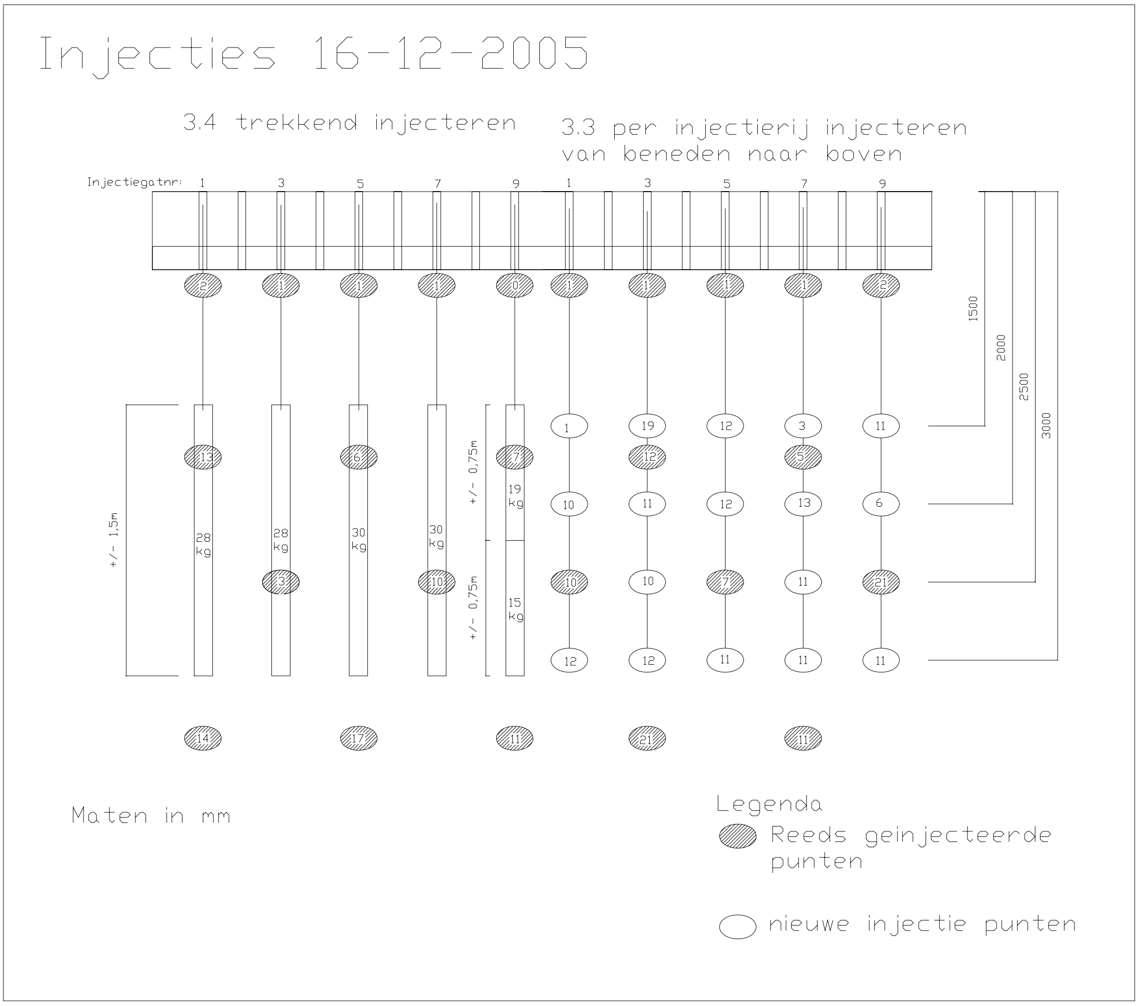
1.1-1.2	**1**	**3**	**5**	**7**	**9**	**10**	**12**	**14**	**16**	**18**		**sum**
-0.6	11.9	17.5	6.7	21.9	20.2	2.9	2	12.1	11.1	6.8		113.1
-1.7	24.4		9		20.7		5.3		17.2			76.6
-2.5		12.5		14.9		13.8		14.8		14.8		70.8
-3.5	24		9		17		23		19			92
											Total	***352.5***

1.3-1.4	**1**	**3**	**5**	**7**	**9**	**10**	**12**	**14**	**16**	**18**		**sum**
-0.6	1	4	2	2	0	4	1	1	1	0		16
-1.7	9		15		13		7		10			54
-2.5		13		21		17		12		18		81
-3.5	19		27		4		10		22			82
											Total	***233***

3.1-3.2	**1**	**3**	**5**	**7**	**9**	**10**	**12**	**14**	**16**	**18**		**sum**
-0.6	1	2	3	2	1	1	1	1	1	1		14
-1.7	9		6		7		6		4			32
-2.5		4		8		11		5		6		34
-3.5	15		0		15		0		21			51
											Total	***131***

3.3-3.4	**1**	**3**	**5**	**7**	**9**	**10**	**12**	**14**	**16**	**18**		**sum**
-0.6	2	1	1	1	0	1	1	1	1	2		11
-1.7	13		6		7		12		5			43
-2.5		3		10		10		7		21		51
-3.5	14		17		11		21		11			74
											Total	***179***

Appendix 20 Renewed Injection Schedule



Appendix 21 Control Measurement

The values of the Leveling spirit instrument with the marker for the level determination of the way point for the level measurements;

Plaats	Baak	Boven	Onder	Middel	Verschil	Afstand	Verschil baak	NAP
Put	1.560							0.800
Lantaren	1.688	1.823	1.550	1.687	0.273	27.30	-0.1280	0.672
	1.593	1.725	1.463	1.594	0.262	26.20		
Klinkers zwart	2.010	2.101	1.918	2.010	0.183	18.30	-0.4170	0.255
	1.848	1.976	1.720	1.848	0.256	25.60		
Midden hoop	1.995	2.143	1.852	1.998	0.291	29.10	-0.1470	0.108
	1.460	1.640	1.282	1.461	0.358	35.80		
Peilbuis Fugro	1.115	1.220	1.000	1.110	0.220	22.00	0.3450	0.453
	1.127	1.174	1.081	1.128	0.093	9.30		
Duiker	1.689	1.766	1.610	1.688	0.156	15.60	-0.5620	-0.109

The measured distance between the start- and end point is 209.20 meters.

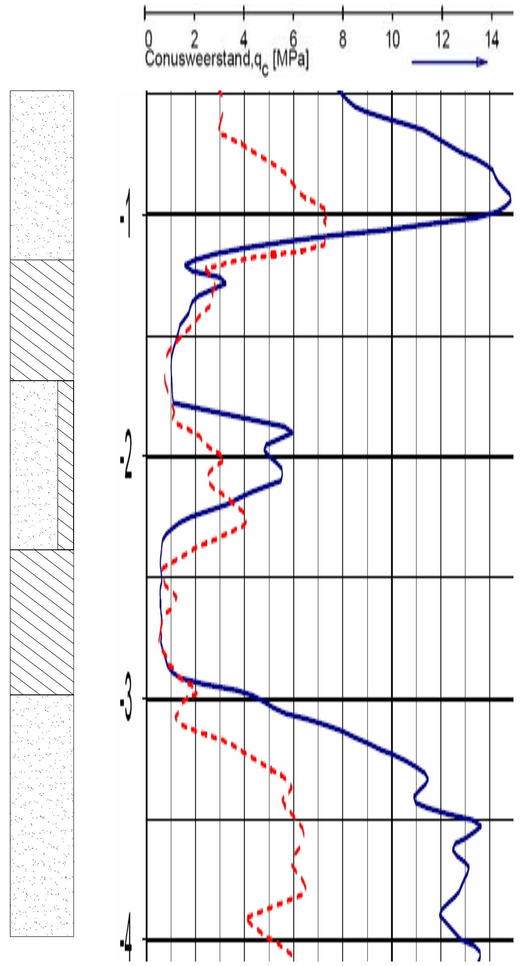
The values of the check for the determination of the way point;

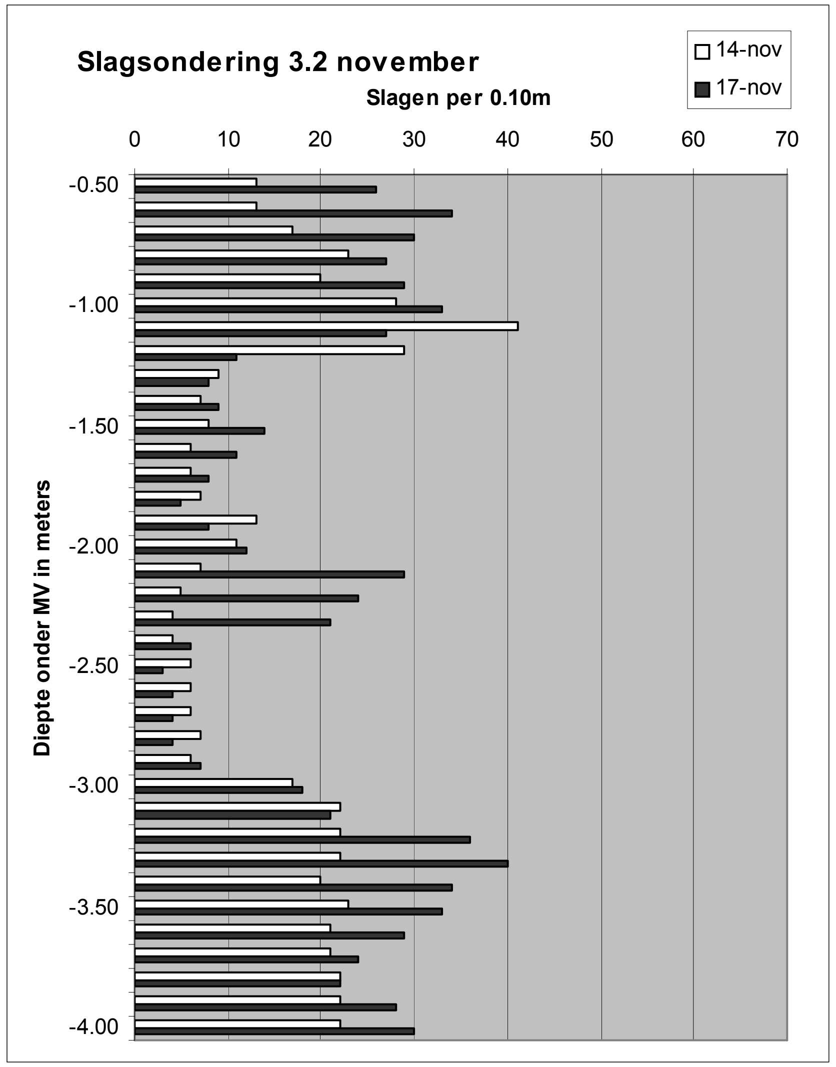
Plaats	Baak	Boven	Onder	Middel	Verschil	Afstand	Verschil baak	NAP
Duiker	1.743							-0.109
Midden hoop	1.477	1.714	1.235	1.475	0.479	47.90	0.266	0.157
	2.000	2.234	1.766	2.000	0.468	46.80		
Weg	1.365	1.580	1.150	1.365	0.430	43.00	0.635	0.792
	1.680	1.838	1.520	1.679	0.318	31.80		
Put	1.670	1.866	1.471	1.669	0.395	39.50	0.010	0.802

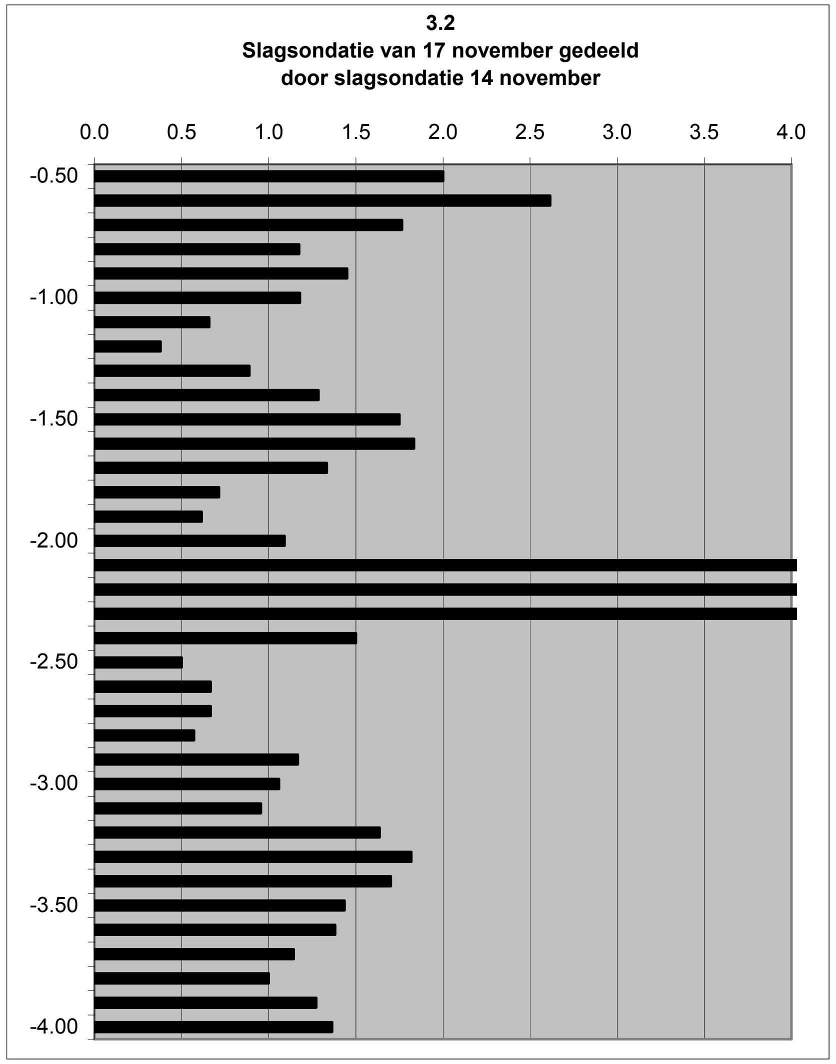
The measured distance between the start- and end point is 209.00 meters.

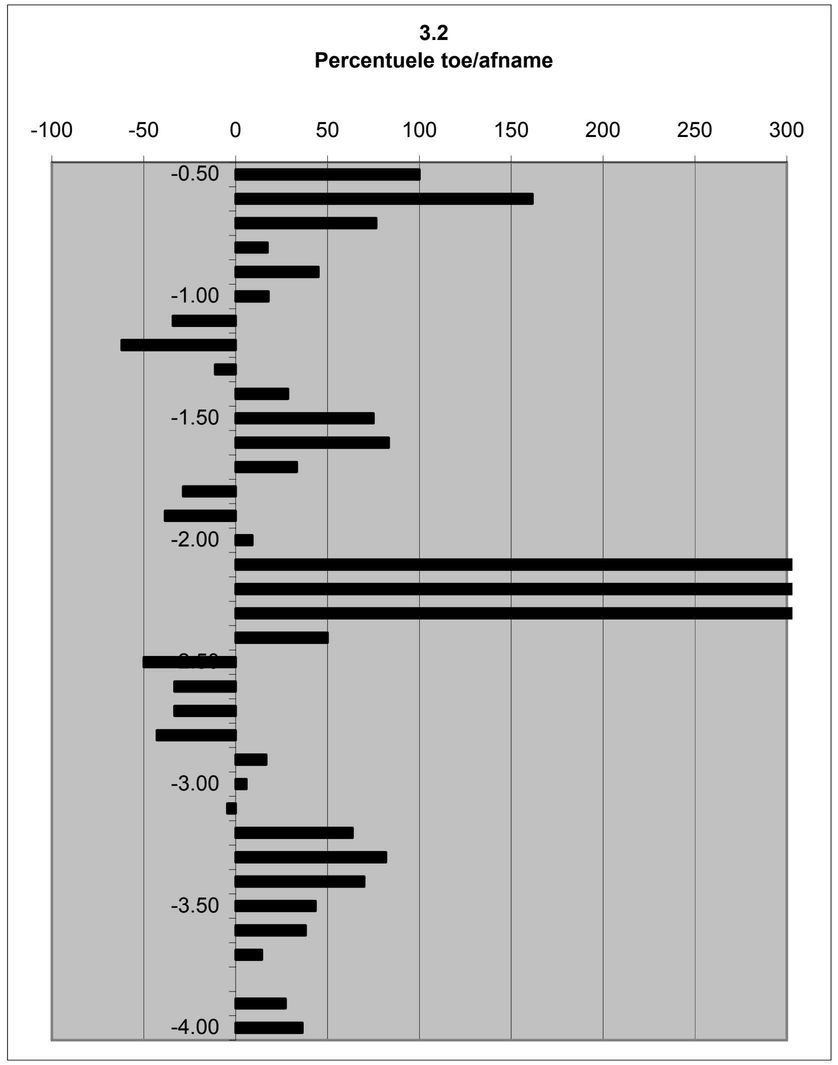
The difference of the leveling measurements is 2 mm and the difference of the distance measurements is 20 mm which is a reading error of 0.2 mm. Both differences are in the range of the error determination.

Appendix 22 Dynamic Soundings

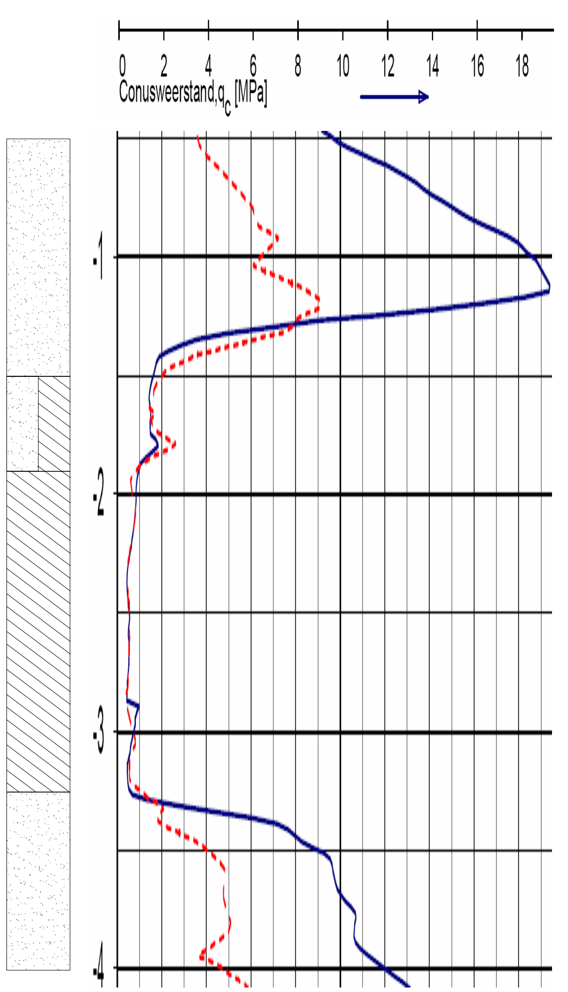

0 2 4 6 8 10 12 14
Conusweerstand,q_c [MPa]
-1
-2
-3
-4

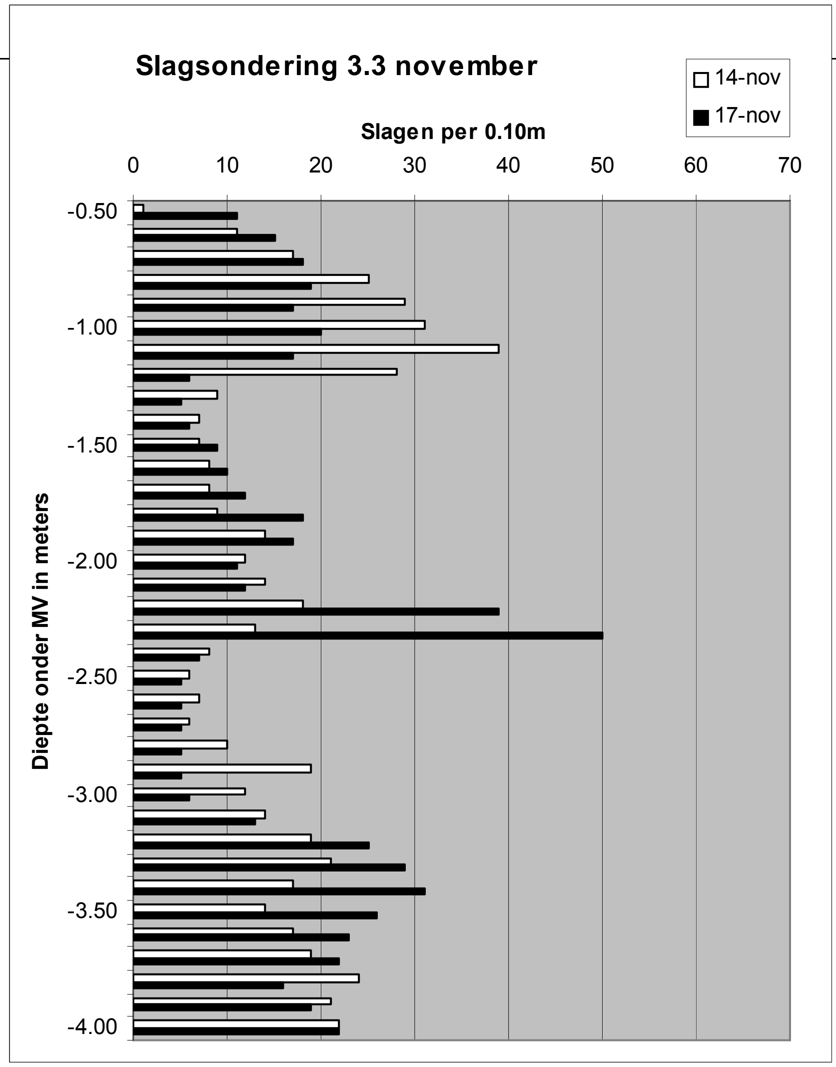

Slagsondering 3.2 november
Slagen per 0.10m
14-nov
17-nov
0 10 20 30 40 50 60 70
Diepte onder MV in meters
-0.50
-1.00
-1.50
-2.00
-2.50
-3.00
-3.50
-4.00

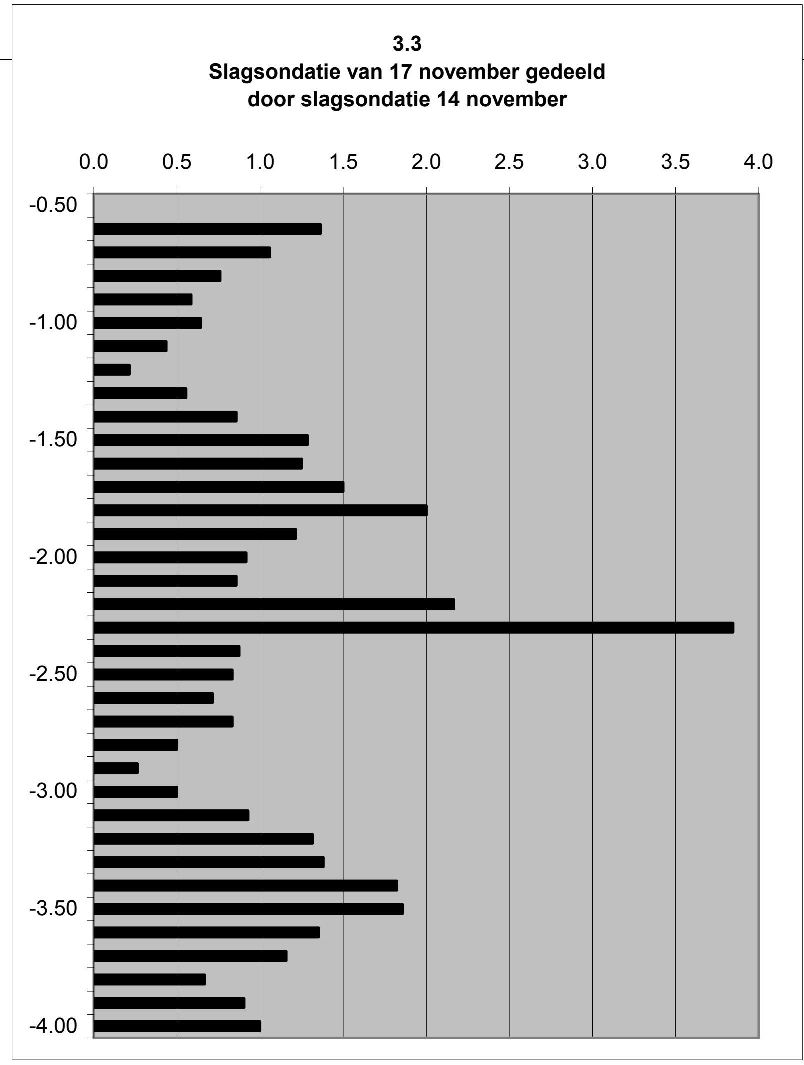

3.2
Slagsondatie van 17 november gedeeld
door slagsondatie 14 november
0.0 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0
-0.50
-1.00
-1.50
-2.00
-2.50
-3.00
-3.50
-4.00

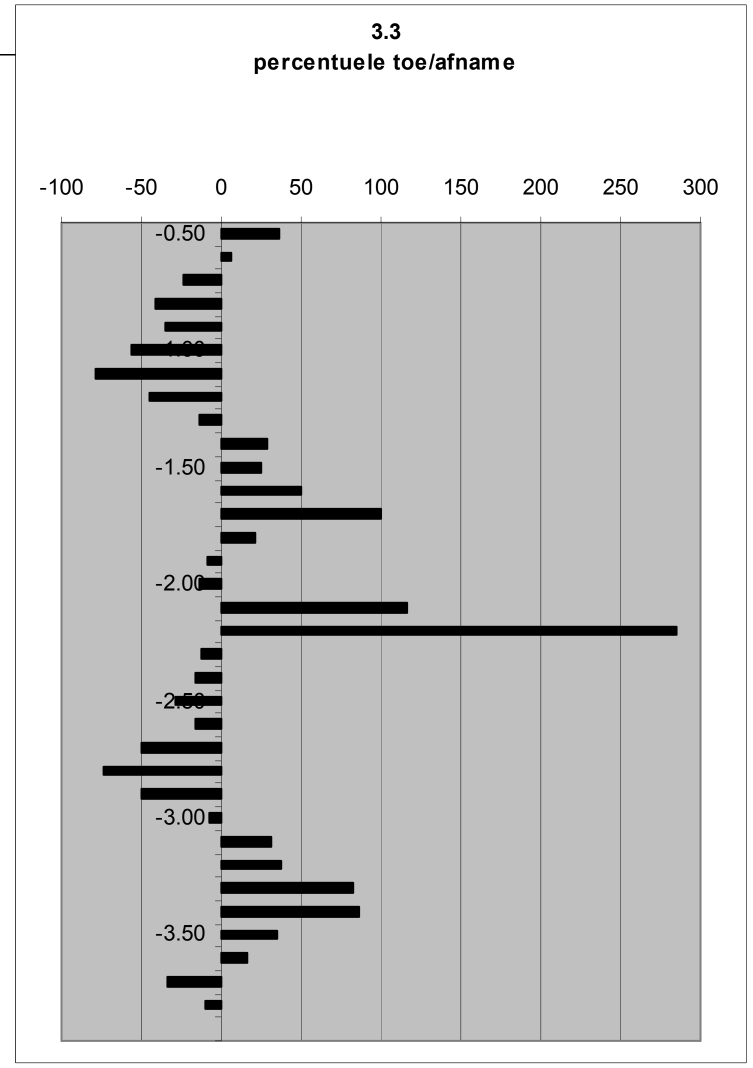

3.2
Percentuele toe/afname
-100 -50 0 50 100 150 200 250 300
-0.50
-1.00
-1.50
-2.00
-3.00
-3.50
-4.00

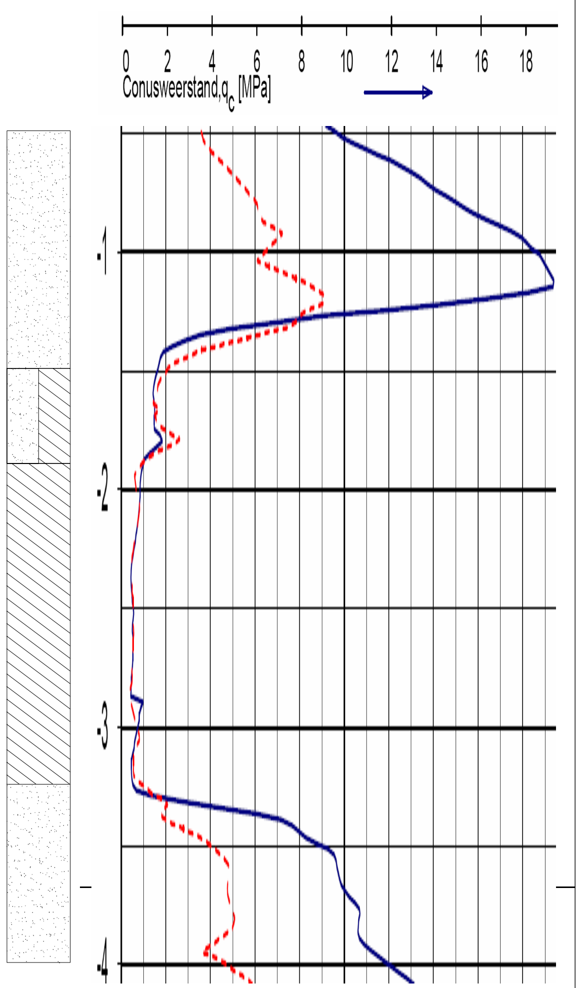
Uretek Deep Injection Method; Full Scale Test

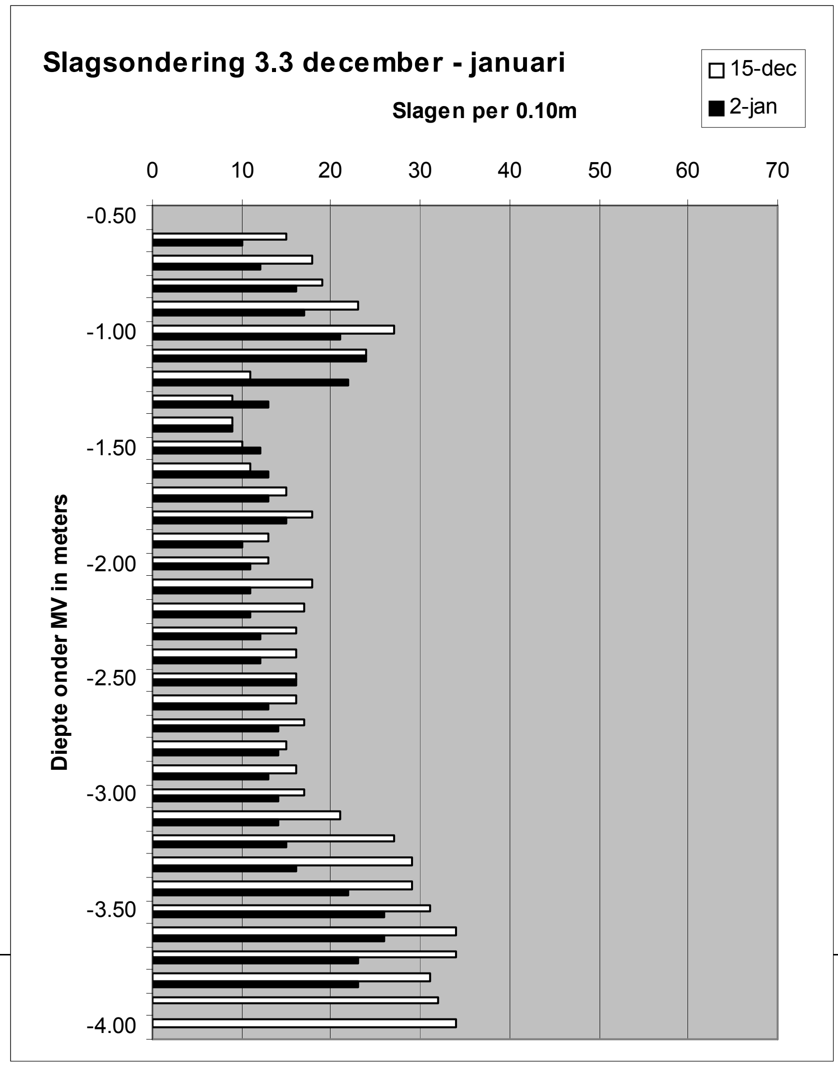


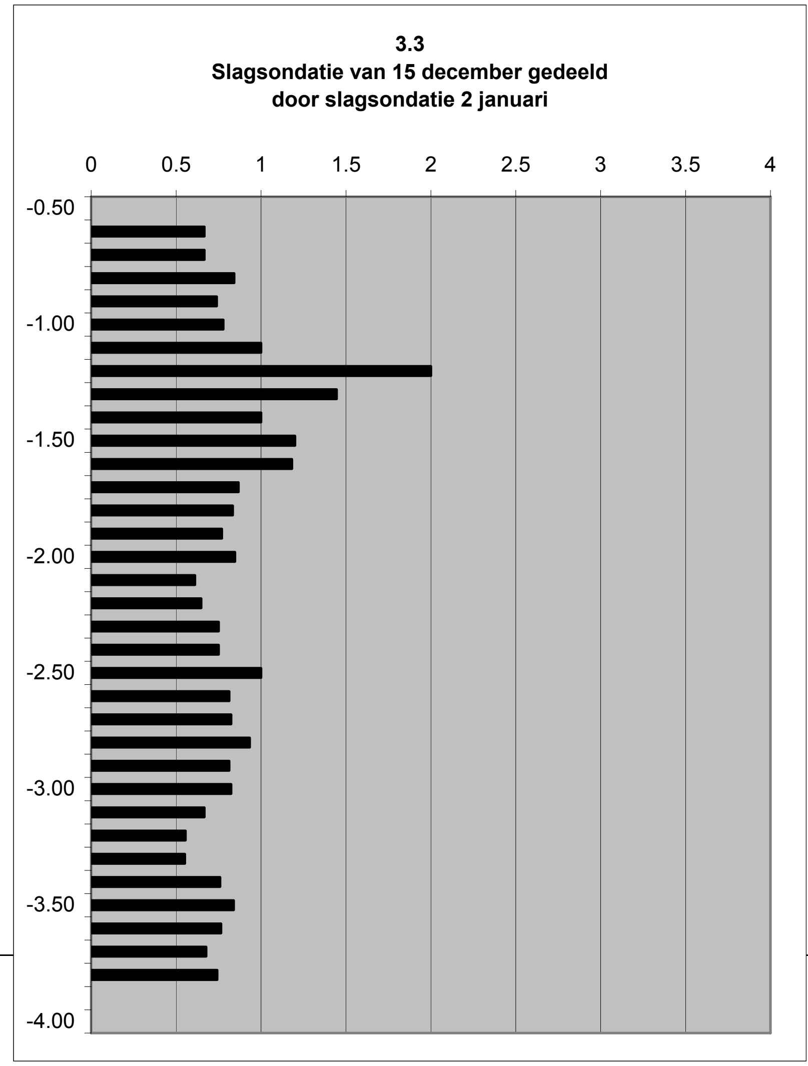


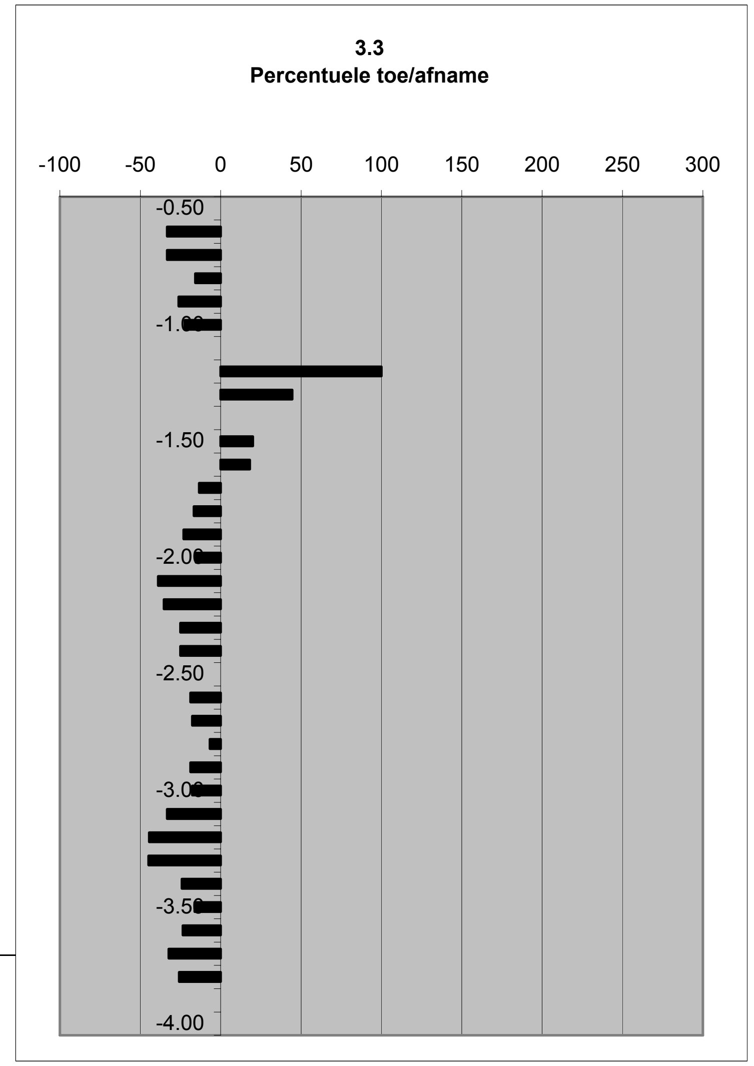


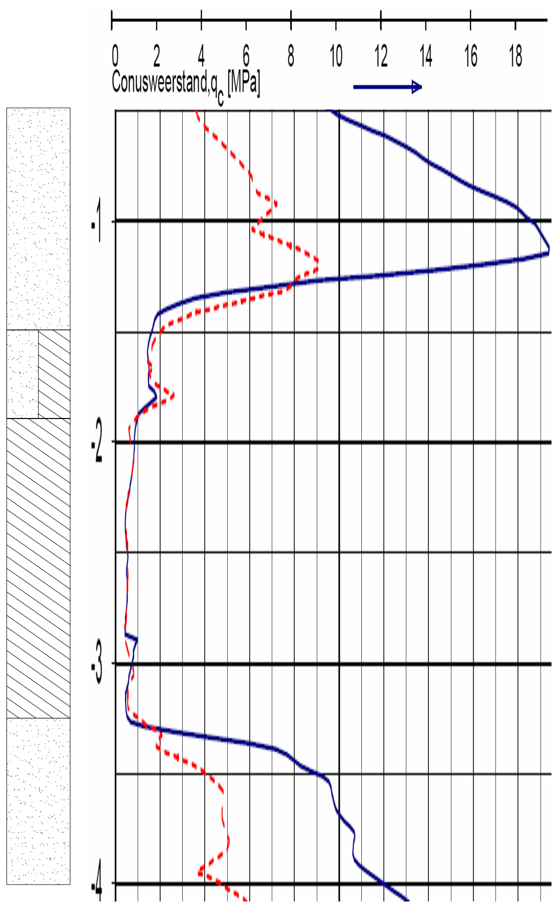


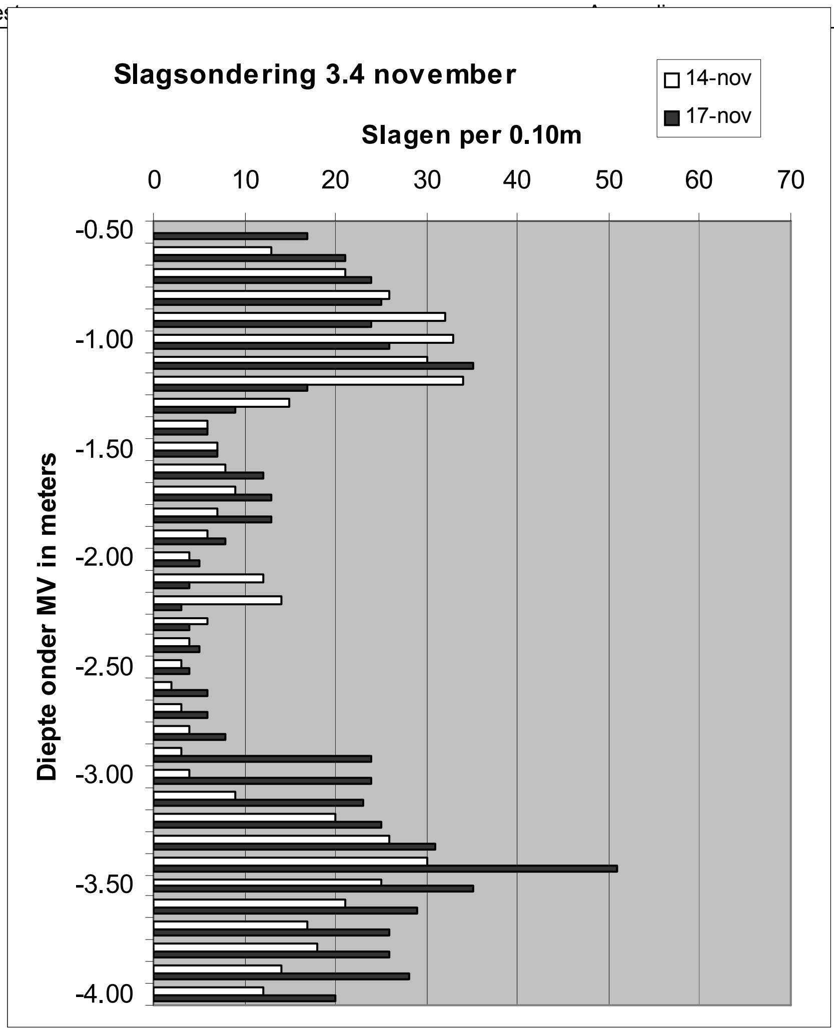


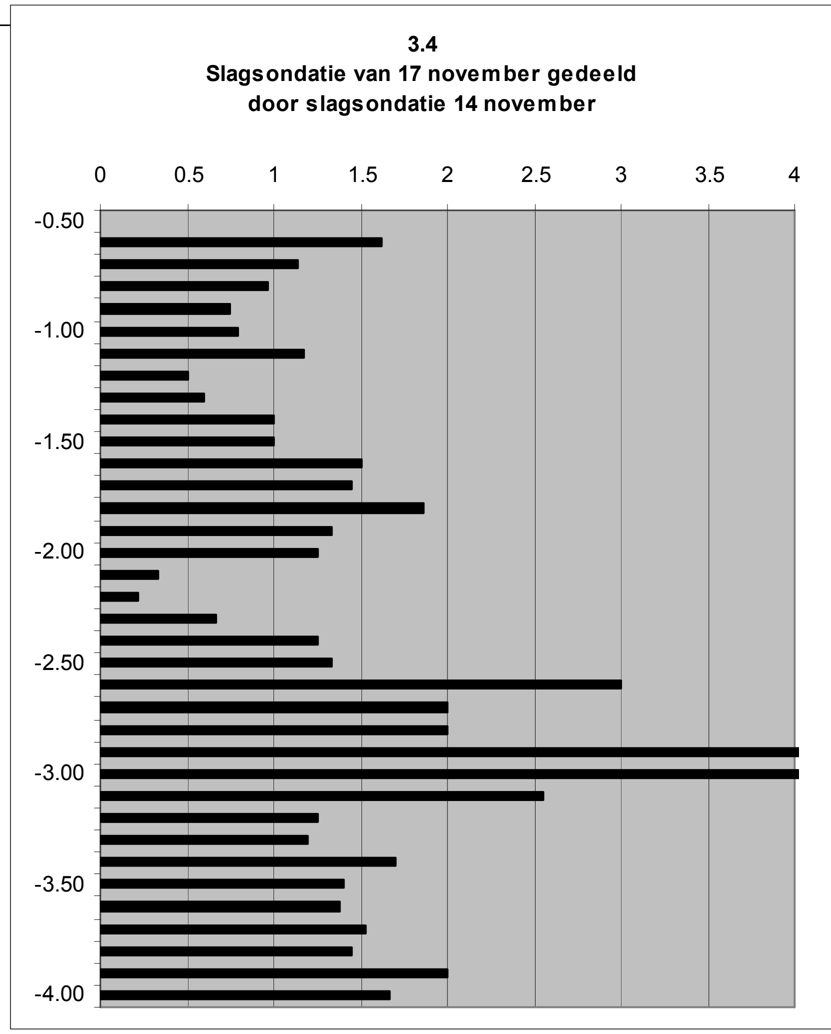


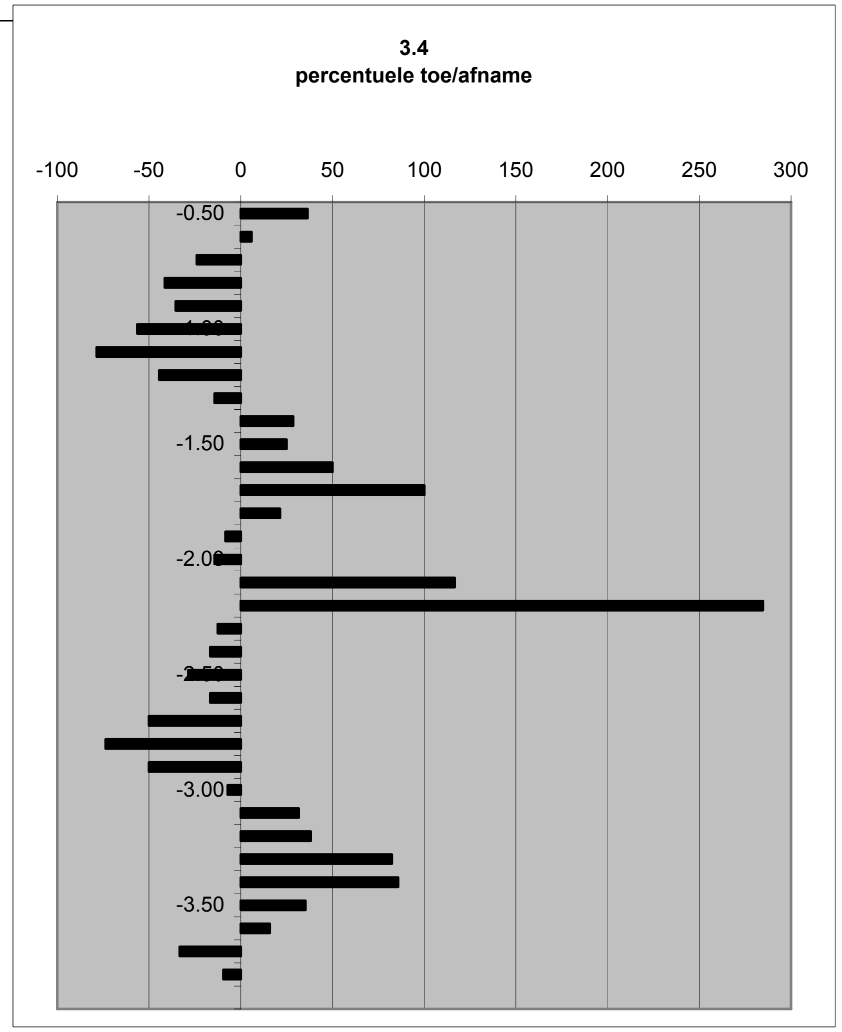


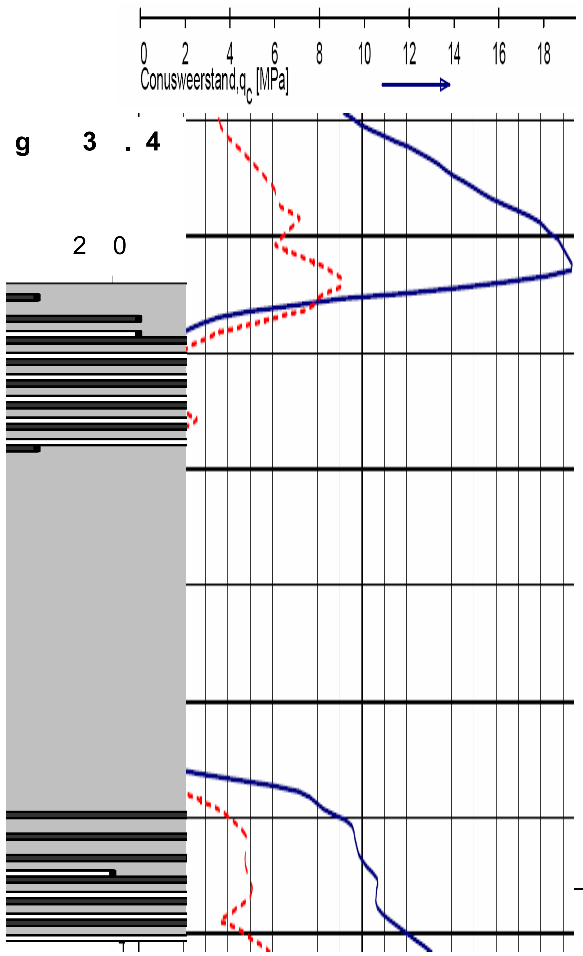


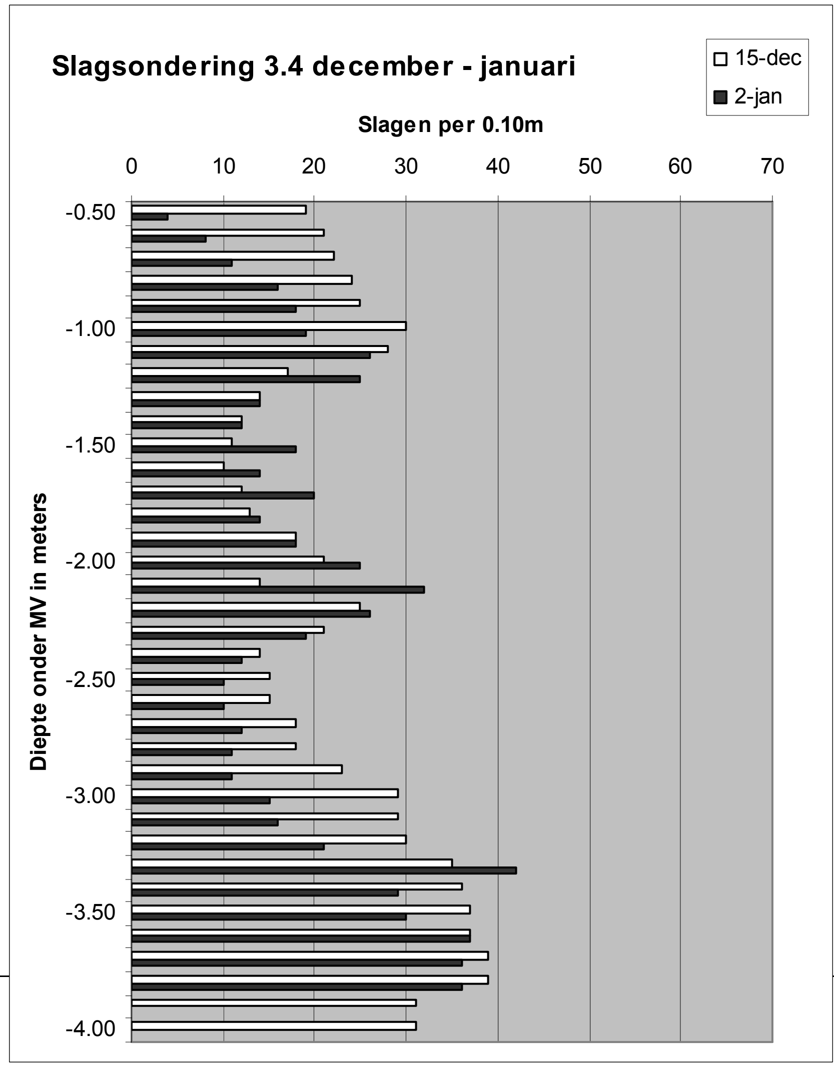

Conusweerstand,q_c [MPa]

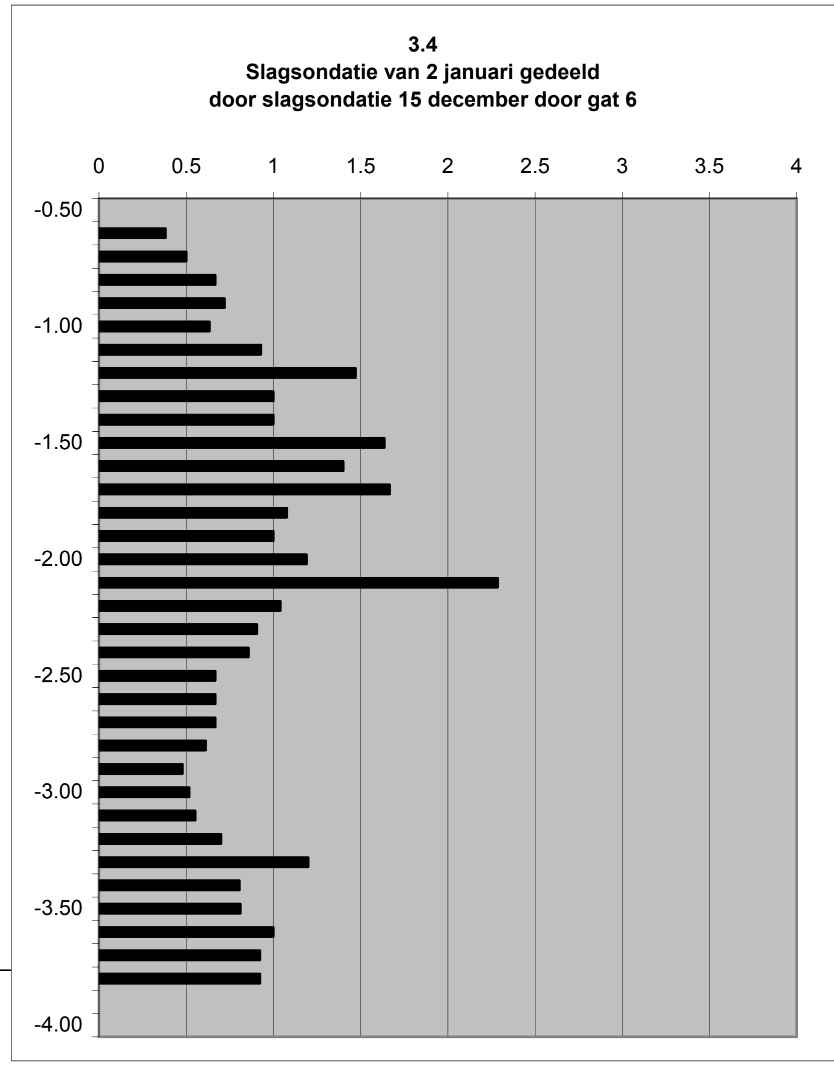

Slagsondering 3.4 november
14-nov
17-nov
Slagen per 0.10m
Diepte onder MV in meters

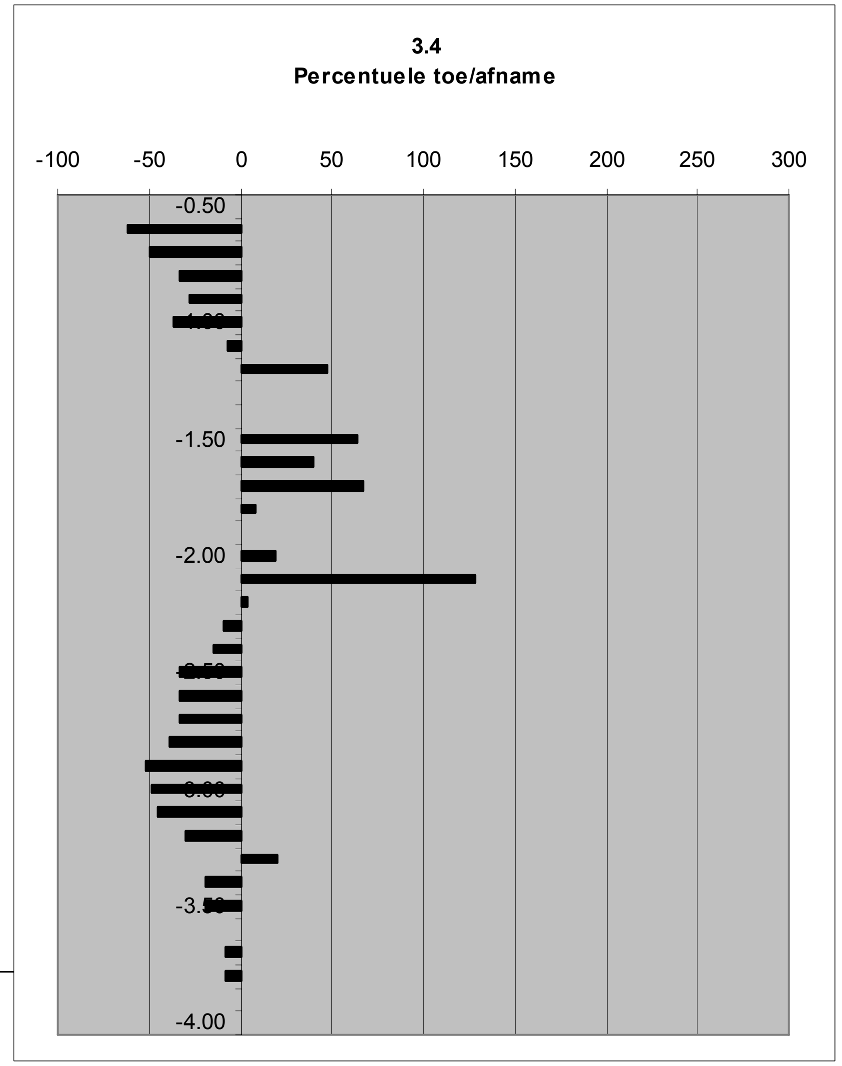

3.4
Slagsondatie van 17 november gedeeld
door slagsondatie 14 november


3.4
percentuele toe/afname


Conusweerstand,q_c [MPa]

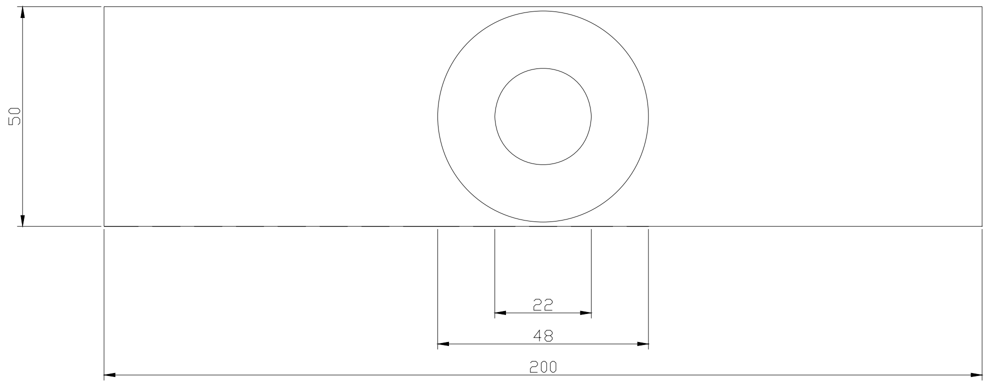

Slagsondering 3.4 december - januari
15-dec
2-jan
Slagen per 0.10m
Diepte onder MV in meters


3.4
Slagsondatie van 2 januari gedeeld
door slagsondatie 15 december door gat 6

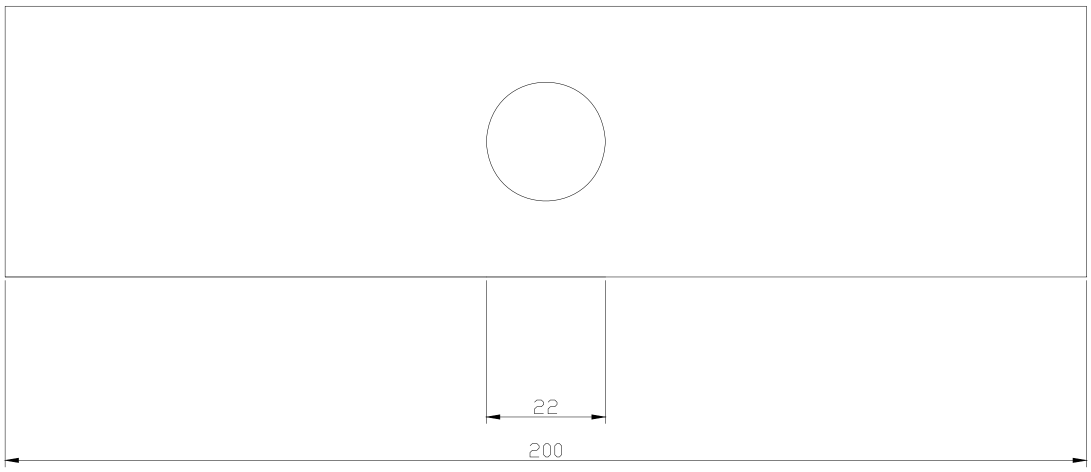

3.4
Percentuele toe/afname

Appendix 23 Marker Pot

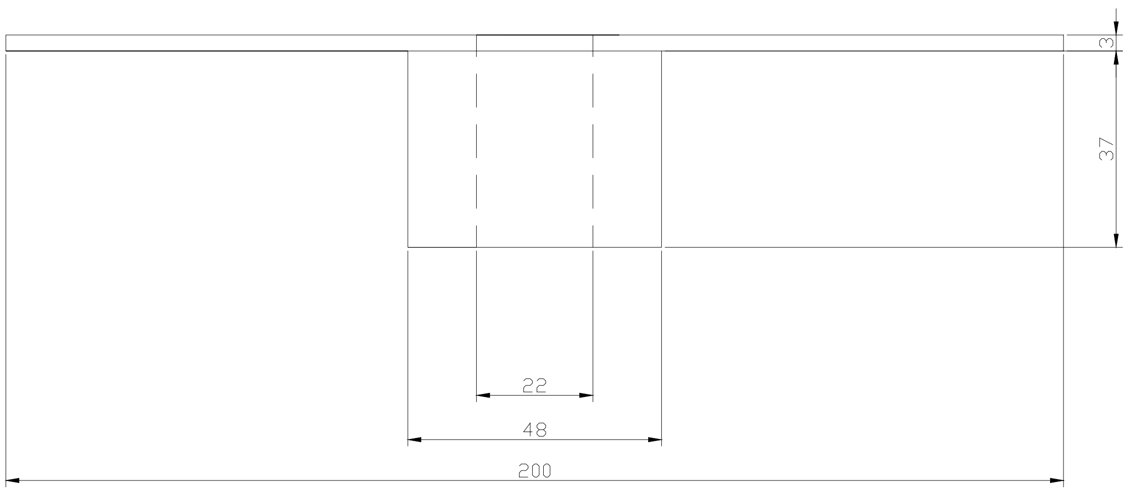
Bottom view



Top view



Side view



References

A. Verruijt — Grondmechanica, 5de druk
Delft, Delft University Press — 1999

James L. Hanson and Ruud J. Termaat — Soft Ground Technology
Virginia, American Society of Civil Engineers — 2000

James Warner — Practical Handbook of Grouting
Hoboken, New Jersey, John Wiley and Sons, Inc. — 2004

R. Felix, A.G. Jongmans and R. Miedema — Geologie, bodem en landschap Nederland
Wageningen, Landbouwuniversiteit Wageningen — 1995

A.F. van Weele — Moderne funderings technieken, 2de druk
Leiden, Spruyt, Van Mantgem & De Does BV — 1996

F.B.J. Barends — Theory of consolidation
Delft, Geodelft — 1992

R. te Grotenhuis — Fracture grouting in theory
Delft — 2004

R. Kleinlugtenbelt — Compensation Grouting Experiment
Delft — 2005

Wijnand J.M. Paans — The influence zone and Effectiveness of Fracture Grouting on Piled Foundations
Delft — 2002

A.F. van Tol — Foundation Engineering and Underground Construction, 4de druk
Delft — 2003

L.A.G. Wagemans — Infomap Algemene Constructieleer
Delft — 1998

A. Mulder and W. Verwaal — Soil Mechanics, test procedures
Delft — 2005

A.D. Visser — Elsevier's Dictionary of Soil Mechanics and Geotechnical Engineering
Amsterdam, Elsevier Publishing Company — 1965