Overtopping Tests on Lime Treated Clay in the Hedwigepolder

Overall report

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Summary

Since 2017 the topic of the use of lime treated clay in Dutch dike reinforcement has been an object of study. After literature research, also of international knowledge and experience, and laboratory research on the behavior of Dutch clays mixed with lime, the next step was to scale up and perform true scale experiments. In the Hedwigepolder three test sections were built. In each section an object was placed to simulate obstacles (e.g. housing) on the inner slope of a dike. Wave overtopping tests were performed with and without obstacles on the inner slope of the existing dike with a lime treated cover layer to test the erosion resistance of that cover layer.

All the wave overtopping tests showed similar results. At the start of the tests a certain amount of erosion took place. However, this was only loose, not compacted, material on top of the slope. After removal of the loose material the erosion slowed down. Part of the slope was eroded further but at a slow rate, and part of the slope did not erode at all as well compacted and erosion resistant material was exposed. Because the lime treated clay was compacted in horizontal layers a staircased erosion profile was formed. The hydraulic load was increased from a wave overtopping discharge of 10 l/s/m’ to 30 l/s/m’ to 50 l/s/m’ to a maximum of 190 l/s/m’ simulated given a significant wave height of 1 m. Although explainable differences in erosion resistance were observed, the cover layer of at least 40 cm thick did not fail at any point during the experiments.

At a few points there was extra erosion (around the object in section 1A, near the crest in section 1A and in the lower part and berm of section 2). After analysis and consultation with Lhoist (responsible for the construction of the test sections) it could be concluded that in all three of these cases the compaction of the lime treated cover layer had been insufficient (due to water flowing out of the lower part of the slope (section 2), due to bad weather conditions (the crest in section 1) and lack of experience and difficulty operating with compaction around the first object that was installed.

Based on these findings it can be concluded that:

- Lime treated clay (if constructed and compacted properly) is very erosion resistant. Even with the maximum load of 190 l/s/m’ little or no erosion was observed.
- This large erosion resistance was also observed around the obstacles, where concentration of the hydraulic load (water flowing down the slope) was heavier than elsewhere on the slope.
- With these findings the applicability of the technique to allow for higher overtopping discharges in dike design has been convincingly shown. Lower required crest heights provide an excellent business case as this saves costs, needed space for dike reinforcement and can save houses and help solve bottlenecks where space is scarce.
- The quality of clay used to form the lime treated cover layer does not seem to have an influence; the lesser quality clay proved to be as erosion resistant as the good (EK1) type of clay. This means that locally available clay can be used instead of excavating and transporting clay from elsewhere. That provides advantages in cost and environmental impact.

The technique of mixing clay with lime to enhance the erosion resistance seems very promising. The recommendations following the overtopping tests is that it should be considered to make the next steps towards practical application in actual dike reinforcement projects.

Points to be considered are:
- Upscaling the technique from constructing a test section towards construction of substantial lengths; the quality of the outcome is strongly connected to the quality of the soil and treatment studies as well as the quality of the mixing and compaction process.
- Ways to check the quality of the outcome; at this stage it should be considered that overtopping tests are very reliable to check this, but it is not feasible to perform these tests at the required scale over the entire length of a dike reconstruction.

Two extra tests were performed with an intentionally damaged cover layer:
- One test in section 2B. The lime treated cover layer was removed over an area of 2,5 m^2. The purpose was to observe the washing out of core material (sand). This test with an overtopping discharge of 30 l/s/m' lasted 1,5 hours. In this time a substantial amount of sand was washed out. The cover layer of lime treated clay was undermined, but did not yet fail. Cover layer failure does not immediately lead to dike failure.
- One test in section 3A. A hole of 20 cm to 40 cm deep was dug, ending in a vertical cliff. This was intended to represent a transition construction in lime treated clay, with a cover layer that has failed (leaving this hole behind). This was tested during 1,5 hours and after that with 25 waves (full load). No erosion was observed.

Both of these tests were only ‘observation tests’. The effect of residual strength after failure of the top layer and the suitability of lime treated clay in transition constructions were not quantified by modelling.

Finally, the working of lime treatment has not been investigated for other applications, such as:
- Prevention of damage of dikes from animal burrowing.
- Application on the outside slope of the dike, in the wave runup zone or even in the wave impact zone.
- Due to the low permeability of well compacted lime treated clay it can also have a role in preventing overtopping water from infiltrating on the inner slope. A new and compacted clay layer on a dike has about the same permeability as a lime treated and compacted clay layer. However, in time clay forms structures, crumbles and becomes more permeable. For that reason it is assumed in current practice that overtopping water infiltrates into the dike, thus having a substantial negative effect on the sliding stability of the inner slope. It is expected that lime treated does not form structures, does not crumble and keeps the same permeability through its lifetime. That would have a significant effect on the way geotechnical failure of the inner slope is modelled. This positive effect of a lime treated cover layer on geotechnical stability of both superficial and large slip planes has not yet been quantified.

Especially the effect on animal burrowing seems to have a large interest from several parties. These applications need to be further investigated, but seem promising.

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1 Introduction

1.1 Short history of the investigation

In 2017 the question of the potential of lime treatment of clay for use in Dutch dike reinforcements was raised by two separate parties: Rijkswaterstaat (Corporate Innovation Programme) and Lhoist (a lime production company from Belgium).

This has resulted in several studies, each study provided a step in the development of the concept (see Figure).

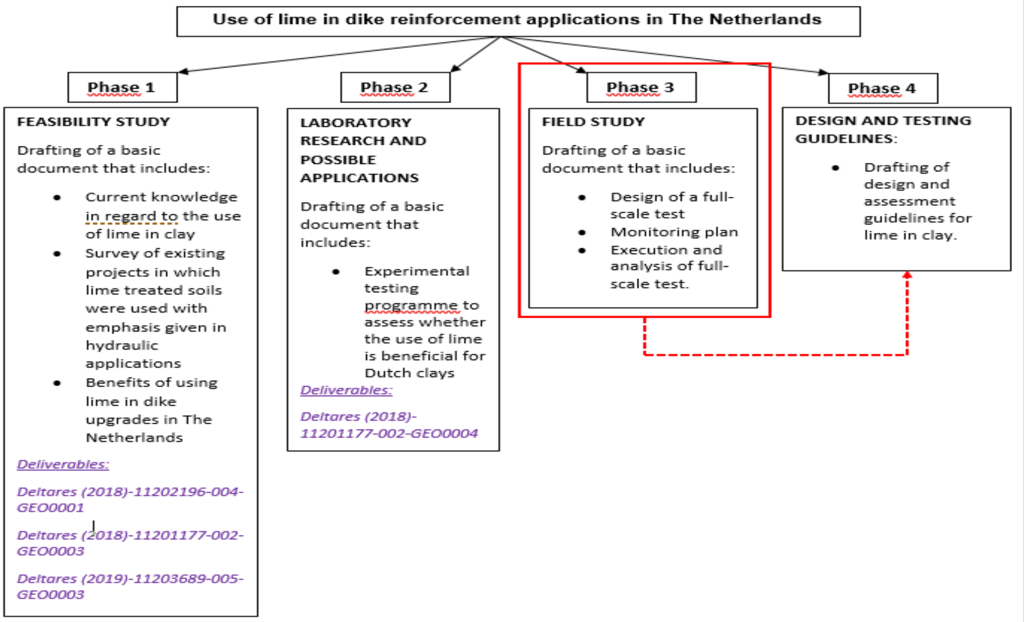

Figure 1-1 Four phases in the development of lime treatment technique in dike reinforcement.

After the feasibility study and laboratory research into the practical use for lime treatment in dike reinforcements had been concluded with favourable results, Rijkswaterstaat and Lhoist have joined forces and prepared a proposal together with Deltares for the HWBP KIA program (Knowledge and Innovation Agenda of the Hoogwaterbeschermingsprogramma). In the midst of 2021 this proposal was adopted by Waterschap Limburg and incorporated in the POV DGG (Project Overstijgende Verkenning Dijkversterking met Gebiedseigen Grond). Thus, the logical next steps in the development of the concept of lime treatment of clay in dikes was funded by the Dutch dike renovation program (HWBP).

At that time it had already been decided that the investigation would focus on the use of lime treated clay on the inner slope of dikes, with the main function of enhancing the erosion resistance of dike covers around objects, i.e. houses. There is a tendency to design dikes with a higher overtopping discharge than historically has been the case. This means that a lower crest height for a dike reinforcement can be accepted. This is a logical step in view of research on the erosion resistance of grass covers, which shows that these higher overtopping discharges can be resisted by a good grass cover. However, around objects there sometimes is no sufficient grass cover, and hydraulic loads are concentrated because the object acts as an obstacle and the water has to flow around the object. In that case, enhancing the erosion resistance with lime treatment could be the answer.

In the meantime the Prosper/Hedwigepolder project of Polders2C's was well underway. The Polder2C's project is an initiative of the Flemish Belgian Hydraulic Research (FHR) and the Dutch STOWA. Because of deepening of the Western Scheldt for ship transport nature compensation is required and the dikes around the Prosper and Hedwigepolder will lose their function as primary dikes, rendering the polder as nature reserves. This is a way to compensate for loss of ecological values due to the dredging activities. Because the dikes lose their function they become available for experiments, especially experiments where the dike is allowed to fail. This is a rare opportunity.

Because it was discussed that small scale experiments on erosion of lime treated soil could possibly lead to false insights (that is because the question is whether the relevant erosion mechanisms on dike-scale can be simulated on a small scale) the need for true scale experiments became apparent. The possibility of performing true scale wave overtopping experiments in the Hedwigepolder was granted by the Polder2C's project.

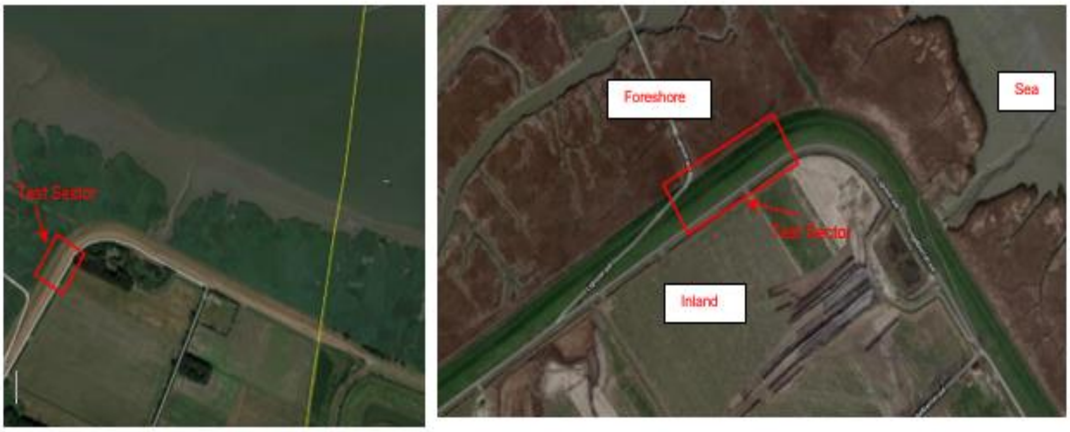

Figure 1-2 Location of the test site in the Hedwigepolder.

In November 2021 the test sections were constructed by Lhoist. Lhoist provided both the know how, the lime and the costs of the contractor. The lay-out of the test sections and the objects within the test sections were provided by Deltares.

The test sections were left alone during the months December 2021 and January and the first half of February 2022. The lime treated material needs a certain curing time during which the reactions between lime and clay take place and the material gains strength. In the second half of February and the first week of March 2022 the wave overtopping tests were performed by Infram, on direction of Rijkswaterstaat and Deltares. Measurements of the resulting erosion were done in cooperation with the University of Liège.

1.2 Purpose and goals of the prototype testing

The stabilization of cohesive soils by lime has been of interest for many years. The lime stabilization technique has proven to be an effective technique in improving the engineering properties of cohesive soils and has been widely used in the construction of embankments and in repair, construction and maintenance work in civil engineering. The use of lime treatment technique in the construction of hydraulic structures in Europe has attained growing attention the last years. Full scale dike experiments performed recently at the Research and

Experimentation Centre (CER) in France, highlighted a series of benefits of this method in enhancing the mechanical behavior and the overflow resistance of the soils used in the construction of dikes.

Lime treatment with clay can also be seen as an innovation in dike upgrades in the Netherlands. In 2018 a literature study has been performed for Rijkswaterstaat Corporate Innovation Program regarding the use of lime in clay in hydraulic structures and, more specific, Dutch dikes (RWS CIP 2018 investigation, Deltares 2018). The outcome of the study was that in three main areas application of lime in clay seemed promising, that is:

- The erosion resistance of clay can be enhanced by adding lime; this is true for both the inner slope (overtopping water) as the outer slope (direct wave impact and wave run up) of a dike.
- The (peak) strength of clay can be enlarged.
- The processability of too wet clay (or even dredge slurry) can be improved. This means that construction time of a dike renovation can be shortened substantially.

To evaluate whether the lime treatment technique is effective when applied to typical Dutch clays, an experimental testing program has also been performed in 2018 in the Lhoist and Deltares laboratories. This program involved testing on samples treated with lime and on untreated samples of a sandy clay taken from Warmenhuizen in the Netherlands. The test results demonstrated the effectiveness of this technique in improving certain soil parameters that are of interest in dike construction.

Lime treated clay can have different applications in dike upgrades. During discussion of the findings from the literature survey and the experimental campaign performed in 2018, it was decided that as a starting point emphasis should be given in investigating the use of treatment technique in one application.

The common agreement was that the most promising application would be to focus on the benefits of lime treatment on the external erosion resistance of the inner slope of the dikes during overtopping.

This application is of particular importance as by considering the newest climate scenarios it is to be expected that in large parts of the river Delta area the dikes will be subjected to higher water levels and higher waves leading to more severe overtopping events. Particular attention should be given to the erosion resistance around 'obstacles'[1] such as buildings and houses. Around such obstacles, typically, no sufficient grass cover will grow whereas overflowing water must seek a way around, causing flow concentration, turbulence and extra hydraulic load leading to extra erosion. If the erodibility of the soil in the inner slope and around structures is proven to be improved via lime-treatment, then a higher overtopping discharge can be allowed. This can result in cost savings in dike reinforcements as the required crest height of the dike can be lower.

1.3 Scope of work

The main target of the testing program was summarized as:

The applicability of lime treated soil for use in the inner slope of a dike for increasing the erosion resistance against overtopping compared to a good grass cover is to be investigated. This scope includes research on the use of lesser quality clay with lime.

[1] The correct term for the Dutch 'Niet-waterkerende objecten (NWO's) is 'encroachment'. In this report the term 'obstacle' is used, as these objects form an obstacle for the overtopping water flow.

The use of the lime-treatment technique in improving the erosion resistance of the inner slope of the dikes shall be evaluated for the case of different applications. The following applications were foreseen:

Scenario A: The first application to be investigated is that of erosion around obstacles (buildings) in the lower (downstream) river area. The purpose of this scenario is to test/demonstrate that the lime treatment can add to a solution around obstacles were grass erosion resistance is insufficient or where a good grass cover cannot be sustained.

Scenario B: The same conditions (downstream river area) can be tested for the inner slope of the dike without obstacles. The purpose of this scenario is to test/demonstrate that lime treated clay can be used where a good grass cover is not strong enough, thus allowing for a lower crest level to be sufficient.

Scenario C: In this scenario the good quality clay is substituted for a clay of lesser quality. The purpose is to test/demonstrate that lime treated clay of a lesser quality can serve as well to enhance the erosion resistance, as the enhancement is largely due to the chemical processes of the lime treatment and to a lesser degree dependent on the clay characteristics. If this can be demonstrated than it is possible to use local available clay instead of having to gain and transport good quality clay to the site. This has both financial and environmental benefits.

Scenario D: The applicability of the lime-treatment technique is to be tested for sea dikes (high waves) in the context of marine environments. For these dikes the sensitivity of crest height to wave overtopping is large. To ensure a substantial lower amount of wave overtopping the crest height has to be considerably raised. A higher allowable overtopping rate will result in savings in material and space. In the testing program the emphasis will be on higher hydraulic loads as a characteristic for sea dikes. However, when application in marine conditions is considered further research is necessary on saline conditions.

Scenario E: Applicability of the lime-treatment technique in transition zones within the slope (i.e. transition zone between dike's inner slope and asphalt road). Transitions usually form weak points in the revetment as they enhance (turbulent) hydraulic loads and hinder the development of a well rooted grass cover. The purpose of this test is to examine whether lime treatment can provide support to transition zones to withstand large amounts of water overflow.

As mentioned in Section 1.2, the effectiveness of lime – treatment technique in terms of external erosion will be evaluated on the basis of comparison of the behavior of lime-treated and non-treated cover layer tested under the same hydraulic loading conditions. Inherent to this statement is that the erosion resistance against overtopping of the inner slope of the dike must also be assessed for the condition of a good quality clay layer with a good grass cover. Available results from wave overtopping test on grass covers can be used in the comparison with the overtopping tests on lime treated clay cover. Furthermore, the grass cover on the Dutch and on the Belgian side of the border is to be tested in the Hedwigepolder in a separate project. Consequently, there is enough data to compare the performance of lime treated clay with a grass cover, so additional tests with a grass cover in the Hedwigepolder as part of the current project are not necessary.

The testing program can be considered a success when the enhanced erosion resistance of lime treated clay cover proves to be larger than the erosion resistance of a good grass cover for one or more of the abovementioned scenarios.

1.4 Partners in this project

The possibility to perform tests on real dikes in the Prosper and Hedwigepolder is an initiative of the Flemish Hydraulic Research (FHR) and STOWA and is subsidized in the Polders2C's project by European funds. Both the Hedwige and the Prosper polder are destined to become intertidal nature reserves. This means that the dikes around these polders will lose their function, but before they do, they are available for experiments. This provides a unique possibility to perform tests that can cause damage to the dikes which normally would not be acceptable. That this is now possible is mainly due to the efforts of the FHR and STOWA.

The actual funding of experiments that wish to use this opportunity has to be provided by the parties that plan tests in this context. In the case of the overtopping tests on lime treated inner slopes of dikes the funding is provided for by two parties.

The construction of the test sections (the lime treated slopes in three test sections) was provided by Lhoist, which is a lime producing company based in Belgium. Lhoist has started collaboration with Deltares in 2018 on developing knowledge on the use of lime treatment in dike reinforcement. This started in 2018 with a literature survey on the use of lime combined with Dutch (organic) clays, followed by experimental research on these clays in 2018/2019. Lhoist provides a large amount of knowledge on the treatment of clay with lime, and has developed procedures on dosing, mixing, testing, applying the material and densification on slopes that serves the purpose of providing a high-quality result. This is paramount for the project. As Lhoist has an interest in positive results of the testing, the company has agreed to provide the contractor's costs for the construction of the test sections and provide their knowledge on an 'in kind' basis.

The funding for the tests with the overflow simulator and the following trajectory towards design rules and implementation in the dike instrumentarium (BOI) has been provided for by HWBP KIA. The HWBP is the cooperation between Rijkswaterstaat and the Dutch waterboards that join forces to realize the dike reinforcement program that is currently in progress in the Netherlands. The abbreviation KIA stands for 'Kennis en innovatie agenda' (Knowledge and Innovation Agenda), which funds research that contributes to the goals of the program to be able to perform the dike reinforcements better, faster and cheaper.

In order to be approved for funding by the HWBP KIA it is necessary that a project and a waterboard are willing to act as initiator and leading partner for the work to be done. This is a guarantee that the investigations are recognized as useful and promising for application in actual dike reinforcements. The waterboard that has agreed to take this role is waterschap Limburg. The investigation has been adopted as a part of the POV DGG (Dike Renovation with Local Soils).

The acting consortium (steering committee) has consisted of Waterschap Limburg, Rijkswaterstaat (Corporate Innovation Program), Deltares and Lhoist. Rijkswaterstaat has identified lime treatment as a promising subject in 2017, has ordered a literature survey by Deltares in 2018 (dealing with the question if the lime treatment was promising in regard to application in Dutch dikes) and has followed up on this idea in 2019 and 2020 with the funding of the continuing making of plans towards the large scale experiments. Deltares acts as a knowledge hub between application in dike reinforcement, knowledge on (dike) material and how to treat and handle this with lime, which knowledge is provided for by Lhoist. Deltares also has a large say in the testing program which is required to make the next step in the innovation development of lime treatment as an erosion resistance measure.

For the testing use will be made of the wave overtopping simulator that has also frequently been used to test the strength of grass covers. The apparatus is owned by Rijkswaterstaat, but in practice the handling and operating of the simulator is done by Infram-Hydren. They provide the necessary know how and experience on performing the tests, as they have done all previous testing of grass covers with the overtopping simulator. For the handling of the water flowing over the slope the overflow simulator is operated by software from van der Meer Consulting, which is designed to give a replication of a distribution of wave overtopping volumes during a real storm event, given a certain wave height, wave period and overtopping discharge.

1.5 Reading guide

The current report summarizes the activities that have led up to the experiments, the preparations, the performance of the overtopping tests, the findings of the experiments and the conclusions and next steps that result from the experiments:

- Preliminary design of the test sections (Chapter 2).
- Construction of the test sections (Chapter 3).
- Sampling and triaxial tests on the lime in clay cover after construction and following moments in time (Chapter 4).
- Testing program for the overtopping tests (Chapter 5).
- Test results (Chapter 6).
- Erosion measurements (Chapter 7).
- Interpretation of test results (Chapter 8).
- Follow up and recommendations (Chapter 9).

2 Preliminary design of the test sections

2.1 Introduction

The plan for the test sections was originally thought of in 2018. Following the earlier investigations the effectiveness of strengthening (Dutch) clay in the laboratory by adding lime was considered proven, but the next step was to determine and test the most feasible application for lime treated soils in dike reinforcement. The consent was to focus the tests on the erosion of an inner slope of a dike around objects (e.g. housing). In the design for dike reinforcements a larger accepted overtopping discharge leads to a lower required crest height and consequently a smaller required dike footprint. This can be economically favourable. Nowadays, more and more dike reinforcements are designed for overtopping rates of 5 l/s/m' or 10 l/s/m' instead of 0,1 l/s/m' as was common practice in the past decades. This originates from the research on the erosion resistance of grass covers. A grass cover can withstand these higher overtopping rates. However, around objects a grass cover can often not be sustained, whereas hydraulic loads caused by the overtopping water have to flow downwards around the object, resulting in concentration of water flow and possibly extra turbulence (and thus extra erosion capacity) around the object. The main purpose of testing of the lime treated material for erosion resistance was to study the suitability for enhancing the erosion resistance of clay around objects. A positive outcome would be favourable for dike design, because a possible bottleneck of a lesser erosion resistance around objects could be solved, allowing for lower dike crests (higher acceptable overtopping discharge). Having to realize a lower crest height for a dike reinforcement presents an attractive business case.

A second line of investigation was to examine the possibility to enhance the quality of local soils by adding lime to it. Clay that is currently used in dike reinforcement traditionally is of erosion category 1. Often local soils do not comply. That means that (expensive) erosion category 1 clay has to be won from elsewhere and then transported to the site. If the local soil could be upgraded and made suitable for use in dikes this saves costs and is more sustainable.

2.2 Location of the test site

As part of the Sigma plan to reinforce flood control of the Scheldt and its tributaries, a number of major projects are being carried out, including the Hedwige Prosper Project. This consists of restoration to a natural state of two existing polders (Hedwigepolder and Prosperpolder). Figure 2-1 shows the area involved for the Hedwige Prosper Project and the location of the two polders to be restored to their natural state.

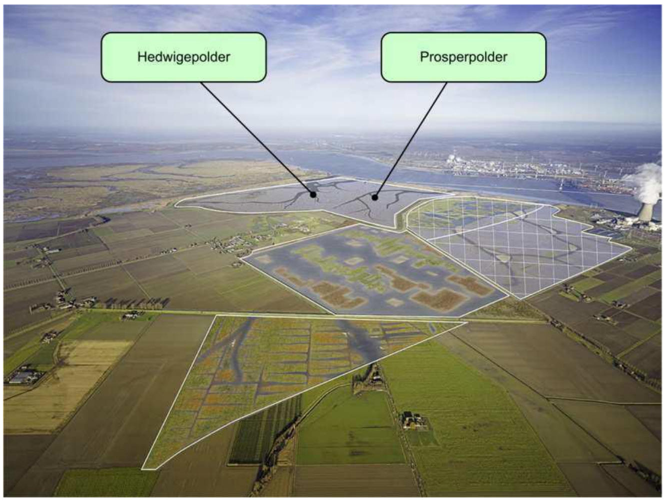

Figure 2-1. Sigma Plan development project in Hedwigepolder – Prosperpolder.

A section on the Hedwigepolder dike was reserved for the full-scale experiments discussed in this report.
The location of the experimental area where treated soils were applied on top of the existing Hedwigepolder dike is shown in Figure 2-2 (a). An aerial photograph of the test location is shown in Figure 2-2(b).

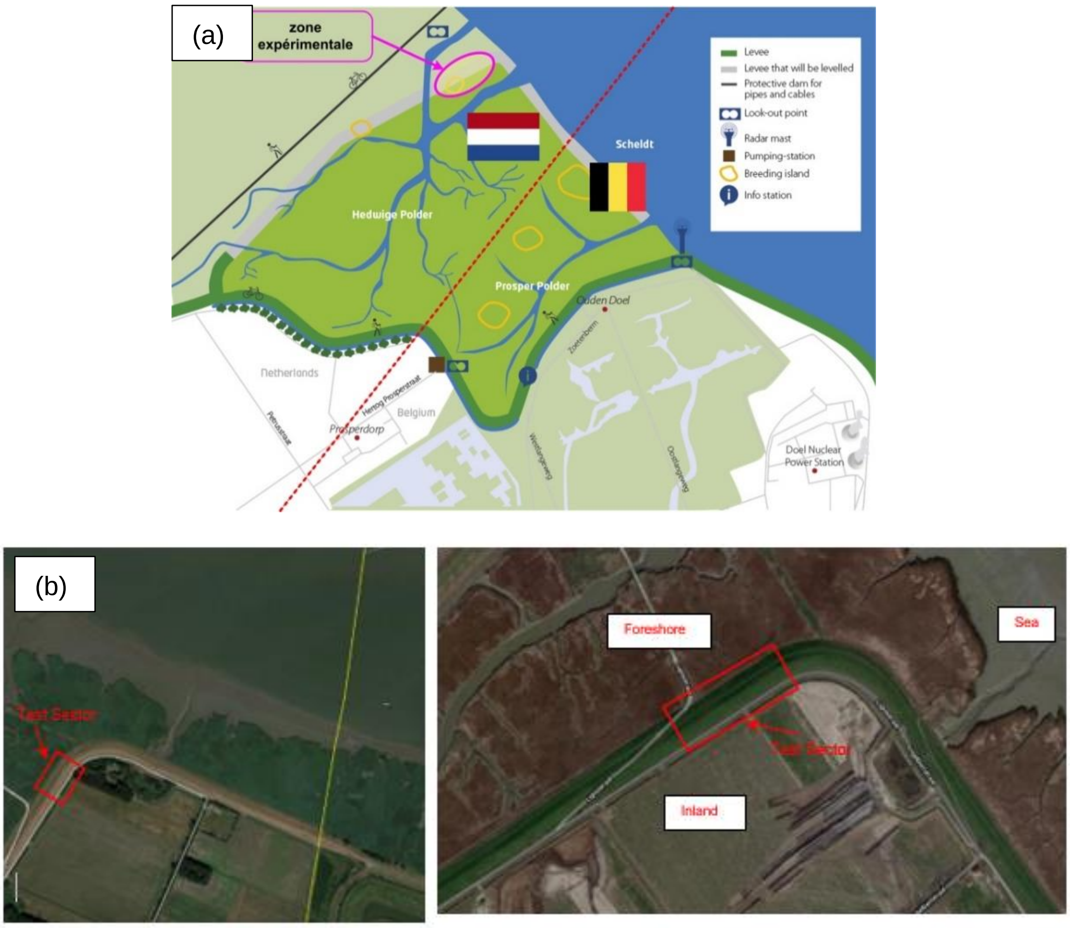

Figure 2-2. (a) Location of the experimental area and (b) aerial photograph of the test section in the Hedwigepolder (~ 120 m wide).

2.3 Soil Profile at the test location

Figure 2-3 shows the location of a cross-section representative of the section of the Hedwigepolder dike that is to be tested for the purposes of this project. This preliminary research was done before the final location was determined. Therefor this section and the final test section are different.

The field data, coming from a previous investigation campaign [SGS (2016), Report Nr: 21545251.V4], consist of 6 borings as indicated in Figure 2-3 (Boring ID: BS103, BD10-WZ, BD10, BD10-LZ, BI132, BI133). No cone penetration data are available. Table 2-1 shows the soil layers and relevant depths as identified in each boring. As evident in Table 2-1, borings run up to a limited depth. It is therefore not possible to have the complete in-depth soil schematization within the investigated area. Based on the available data it can be concluded that - in general - the soil profile consists of a clay top layer underneath of which a sand layer is present. The thickness of the top clay layer is approximately 4.30, 1.6 and 0.70 m at foreshore, crest and hinterland respectively. The exact thickness of the sand layer per dike location remains unknown. It should be pointed out that the cross-section in Figure 2-3 differs from the testing cross-section shown in Figure 2-2. The soil schematization is consistent in the area and therefore the soil profile shown in Table 2-1 is representative of the soil profile encountered at the actual testing section.

Figure 2-3. Location of cross- section (left side figure) and available borings (right side figure).

Boring ID	Location with respect to dike body	Soil schematization		
			Top NAP m	Bottom NAP m
BS103	Foreshore	CLAY	3.15	-1.15
		SAND	-1.15	-1.45
BD10-WZ	Foreshore slope	SAND	5.93	4.93
BD10	Crest	CLAY	9.4	7.85
		SAND	7.85	1.4

BD10-LZ	Inland slope/Toe	Top NAP m, Bottom NAP m CLAY 3.66 3.06 SAND 3.06 2.66
BI132	Inland slope/Toe	Top NAP m, Bottom NAP m SAND 2.26 1.76 CLAY 1.76 -0.14
BI133	Inland	Top NAP m, Bottom NAP m CLAY 1.35 0.65 SAND 0.65 0.15

Table 2-1. Soil schematization based on available borings within the investigated cross-section at Hedwigepolder.

2.4 Geometry of the test location

The elevation profile at the selected cross section is drawn from data provided in the AHN website (Actueel Hoogtebestand Nederland), see Figure 2-4. Based on this profile together with the soil schematization data in Section 2.3 the cross-section at the test side is formed as given in Figure *2-5*.

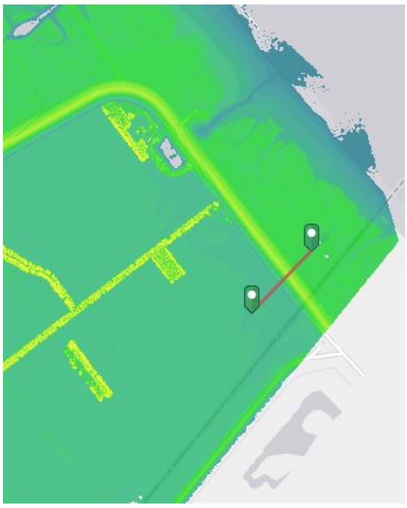

Figure 2-4. Terrain model derived from AHN; with red line the selected cross-section.

The inclination of the inner slope is approximately 1:2.6. With a crest level of NAP + 9.6 m, a hinterland/berm at NAP +3.8 m and an innerland at NAP +1.5 m the dike is quite high and steep. The ditch on the land side has a maximum depth of 1.75 m and it is 5 m wide. As mentioned in Section 2.2, a road is present in the land side between dike and toe ditch.

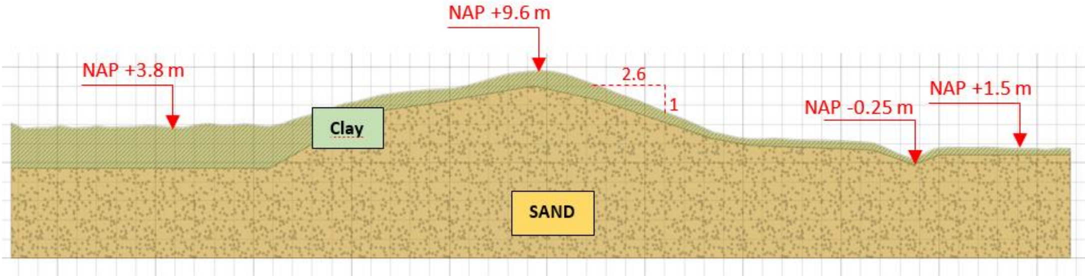

Figure 2-5. Hedwigepolder test cross – section.

2.5 Design of testing sections

2.5.1 Division in testing sections

The effectiveness of lime-treatment was originally to be assessed for three types of materials:
A. Good quality river clay.
B. Poor quality river clay.
C. Sea (salty) clay.

Erosion tests on good quality river clay
In current practice a good quality clay (category 1, 'erosiebestendige klei') is used for the dike cover material. It can be disputed whether or not this quality clay is absolutely required, as in addition, this clay layer is covered with grass to increase the resistance against external erosion. Normally, the grass cover is responsible for the erosion resistance, and the clay layer does not come into consideration for this purpose. Traditionally it is considered good practice to use only erosion resistant clay in dikes however.
Close to obstacles or objects, a grass cover might not be sustainable and the hydraulic load is relatively high, so the grass and the soil layer around the object may not resist erosion. In such a context, the use of good quality clay treated with lime is promising if it proves to provide enough erosion resistance. In addition, regardless of the presence of obstacles, an improved erosion resistance response of the lime-treated good quality clay will demonstrate that such material can be used as revetment material where a good grass cover is not enough.

Erosion tests on poor quality river clay
Testing with poor quality river clay clay which in current practice would not be accepted for use in Dutch dikes (category 'weinig erosiebestendig' or even 'ongeschikt') is of similar interest. The advantage of this is that in the case that this material provides sufficient erosion resistance, locally available soils can be lime treated and directly used as cover layers in the inner slope of the dikes. The use of local materials has cost advantages and it is more sustainable as the need to transport suitable – good quality - clay is no longer there.

Erosion tests on sea (salty) clay
A successful application of lime-treatment using sea (salty) clay can expand the use of this technique for strengthening the downstream erosion resistance of sea dikes which are more prone to crest height rises to maintain low water overtopping discharges in view of future sea level rise and more severe storms due to climate change.

Based on the different types of testing materials it was decided to construct three main test sections:

- Section 1: testing material good (river) clay.
- Section 2: testing material lesser quality (river) clay.
- Section 3: testing material sea (salty) clay.

2.5.2 Test layout

For designing the test sections the following aspects should be considered:

- The operational width of the wave overtopping simulator is 4m. The width of each test section should therefore not be less than this length.
- To avoid interference of the testing conditions among adjacent test sections, a transition zone is considered between the test sections. The width of this zone is 2.5 m and is left out of testing.
- It is desirable to design the test sections in a way that if needed additional tests or tests in duplo can be performed.

A draft showing the layout of each test section is illustrated in Figure 2-6. The layout of each testing section allows per testing material the performance of two experiments with the wave overtopping simulator. For these two experiments, the object is positioned in a mirrored way. With this arrangement, if needed a testing section of 4 m remains free from obstacles/influence by obstacles and can be used in testing in case the presence of obstacles is not desired.

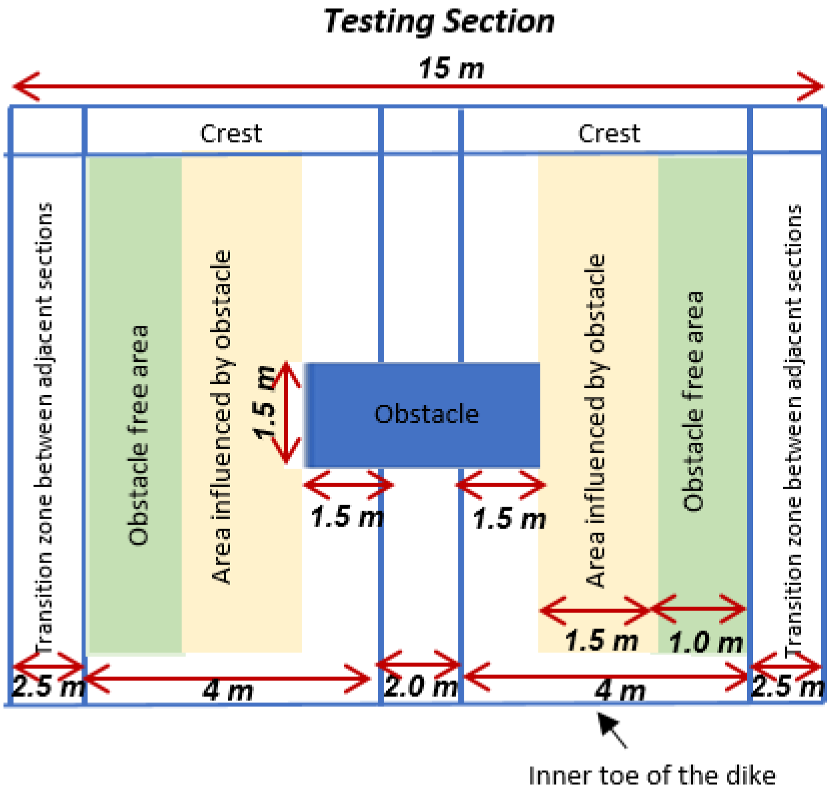

Figure 2-6. Sketch of test section per soil type.

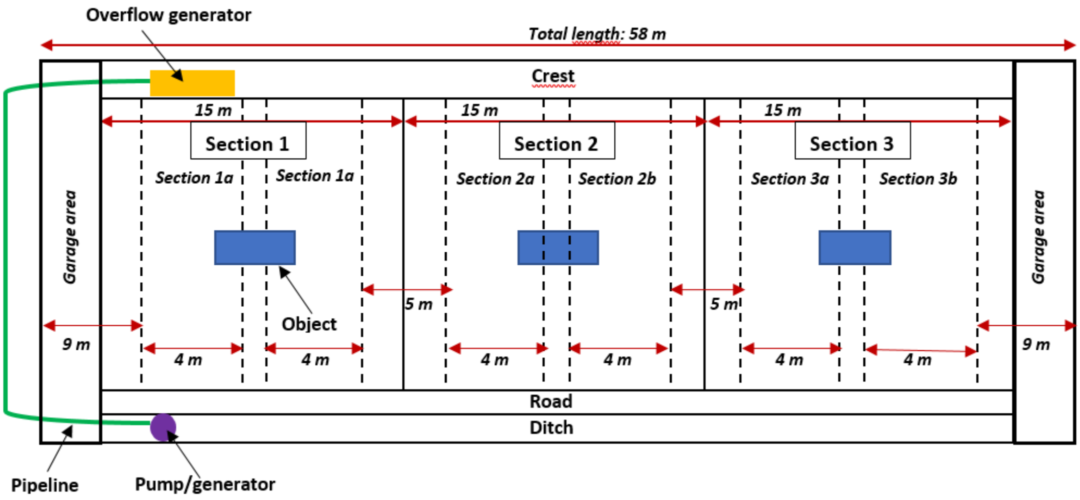

Figure 2-7 Lay-out of the three test sections.

2.5.3 Preliminary testing plan

The basic idea was to start testing at a level where grass covers do not (necessarily) give reliable protection. That is to say that a logical starting point would be 10 l/s/m' overtopping rate around obstacles. A further step would be to prove that lime treated cover layers can withstand a substantial higher hydraulic load, which would provide an extra security that could be used to introduce the necessary margin for error and uncertainties for a design method. Basically it was envisaged that the logical next load steps would be ¼, ½ and full capacity of the wave overtopping simulator.

An important aspect of the testing plan was that the entire experiment was designed to study erosion of the inner slope under wave overtopping conditions. That means that other possible failure mechanisms must be excluded. The failure mechanism where the newly constructed cover layer would slide over the existing slope had to be prevented. This led to a staircased profile, where the grass cover would be stripped from the existing dike, and a staircased profile was to be dug into the existing dike. The new cover layer was then to be constructed on this stair case profile. This method of working is not uncommon in Dutch dike reinforcements.

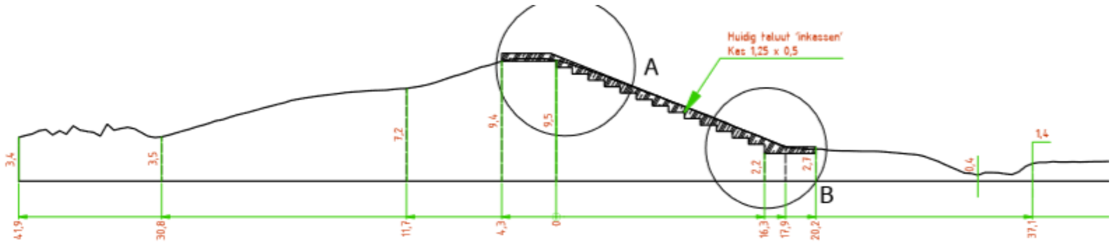

Figure 2-8 Design of staircased profile for test sections.

2.6 Wave overtopping tests

2.6.1 Wave overtopping apparatus

The wave overtopping simulator has been developed to test the crest and inner slopes of dikes on resistance against erosion. In the last few years numerous tests on grass covers with the overtopping simulator have been performed.

Figure 2-9. The wave overtopping simulator.

The simulator, in essence, is a box containing a large amount of water, and a valve that opens up the lid in a way that is designed to simulate an overtopping wave (see Figure 2-9). Although the principle is simple, it is an advanced apparatus in the sense that is operated by software that has been designed to simulate actual storm (represented by an x number of waves with exact characteristics of overtopping events). These characteristics are mainly determined by wave height, wave period and overtopping discharge. In a period of (usually) 6 hours an entire wave overtopping event during a design storm can be simulated. A maximum wave overtopping discharge of 6 hours is roughly believed to be representative for the erosive load of an actual storm event with an increasing overtopping discharge, a short peak and decrease of the overtopping discharge.

The wave simulator is owned by Rijkswaterstaat. The idea and the development of the apparatus and software stems from Van der Meer Consulting. The apparatus is operated by Infram-Hydren, who have done all the grass tests with the apparatus.

2.6.2 Trial programme

As explained, there are three test sections. For each of these test sections it was foreseen that (at the most) one week of testing would be required.

In that week the same sequence of tests could be done:

- One day to install the apparatus.
- First testing with 10 l/s/m’ overtopping.
- Second test at ¼ of the maximum load.
- Third test at ½ of the maximum load.
- Final test at the maximum load.
- Dismantling and moving the apparatus to the next test section.

To be able to decide quickly if the testing program needed to be adjusted once the results of the first tests become available required that beforehand several scenarios have been thought through. The design of the layout of the test sections allows for several (extra) variations to be tested and or adjustments to the test program. For instance:

- If the first tests show little or no erosion the load can be heightened by moving the test strip of 4 meters width such that the obstacle is placed more in the middle. It can also be considered a possibility to start the next test section with the second or highest load (skipping the first test with the lowest load), thus creating the possibility of extra tests to be added within budget.
- If the first tests show more erosion than expected the obstacle can be placed more to the side (lessening the load around the object) or the overtopping rates for the following tests can be adjusted.
- If there are doubts regarding the tests and the outcomes the test sections allow for duplicate tests.
- It is foreseen that samples from the three test sections are taken and tested in the laboratory to have an idea of for instance the shear strength or bearing capacity. This could lead to the conclusion that the three test sections are expected to behave very similar of quite different from each other. This could be a consideration to make changes to the testing plan.
- If there is little or no damage during the testing an extra test with an induced damage can be added.
- It is possible to perform a test on a test strip of 4 meters wide without an obstacle. It is also possible to add an extra transition structure (for instance a wooden beam in the slope below the obstacle) to see if that results in extra turbulence and extra erosion.

2.7 Instrumentation and monitoring

The overtopping tests were designed to study erosion of the lime treated cover layer. It was envisaged that the result would to a large extent depend on the quality of the cover layer that was constructed. This has been extensively monitored by Lhoist during and after construction.

An additional objective of this monitoring is to provide information on the variation of the mechanical properties of the lime-treated cover clay layer. An important aspect in evaluating the effectiveness of lime treatment is not only in demonstrating a high average (erosion) strength, but also in showing that the in-situ distribution of the soil properties in dikes is smaller than that of natural materials. It was expected that lime-treatment would result in a uniform material with less variation in the soil characteristics. The monitoring was to provide a first order insight on the spreading of the parameters concerning erosion resistance.

The evaluation of the erosion phenomena after testing has been measured. An extensive monitoring program including measurements of water pressures within the cover layer was considered to be too detailed considering the project objective.

3 Construction of the test sections

3.1 Introduction

Lhoist has been responsible for financing and construction of the test sections. This was a major contribution to the realisation of the entire project. Throughout the project there has been consultation between Lhoist, Rijkswaterstaat and Deltares concerning construction issues. This collaboration has contributed largely to the success of the project.

It must be mentioned that the knowledge and know how of Lhoist on constructing projects with lime treated materials has been crucial in this project. Special issues concerned the use of lime treated material in hydraulic structures, such as a dike, and specific Dutch demands and regulations for materials to be used in Dutch dike reconstruction.

In writing this report extensive use has been made of Lhoist reports (lit. [8] and [9]).

3.2 Tender process

The process of soliciting and selecting a contractor has been done by Lhoist. Important to notice is that the soliciting process was started by asking contractors about the proper equipment and know how to perform the required work. It was also a prerequirement that the contractor had experience in reconstruction of Dutch dikes. Six contractors, or consortia of contractors, applied. Although the work with lime treatment on dikes in the Netherlands is innovative, the knowledge and equipment is available for the Dutch market.

3.3 Soil types

Part of the work for the contractor was to provide suitable soils for the testing sections. This proved somewhat difficult. One clay type of suitable properties for use in Dutch dikes was found externally and brought to the site. Two other clays were retrieved from different places in the Hedwigepolder. The first analysis showed a distinct difference between these two. Later classification tests showed that in fact the two clays from the Hedwigepolder showed more or less the same properties.

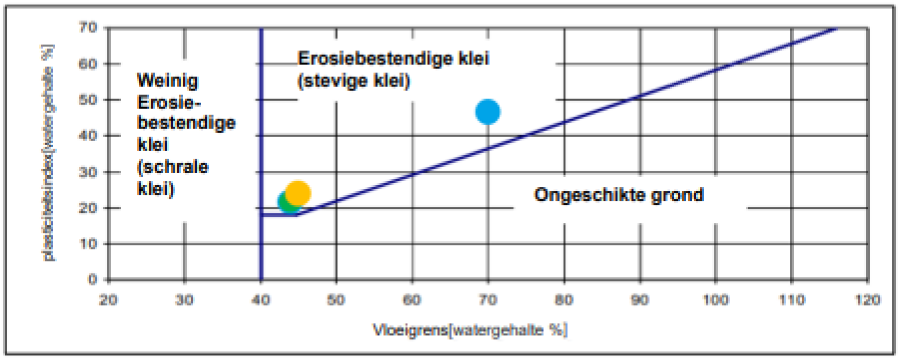

Figure 3-1 Dutch diagram for soil classication for use in dikes. The three types of clays used are also shown (good quality blue dot, yellow dot poor quality clay A and green dot poor quality B) (figure courtesy Lhoist).

After testing it was clear that the samples from the Hedwigepolder were well within limits of salt content. It was decided that the clay types ‘poor quality A’ and ‘poor quality B’ were to be used to compare results (first to see whether or not results would reproduce, but also to have the possibility to compare all results with poor clay with the results for good quality clay). That meant that the original idea of using salt clay (sea clay) was abandoned. The clay types that are referred to as ‘poor quality clay’ actually (just) comply with the demands for ‘Erosiebestendige klei’. These clays are representative for local clays that have not been specifically selected for use in Dutch dikes (EK1). Both ‘poor quality clay A’ and ‘poor quality clay B’ were taken from the Hedwigepolder. The clay that is called ‘good quality clay’ has been specially selected by the contractor and is suitable without doubt.

3.4 Treatment of soils, construction of test sections, placement of objects

To have a solid foundation for operations a production platform was built near the test sections. The production platform consisted of a densified sand body. The production platform and the three stockpiled clays are shown in the figure below.

Figure 3-2 Production platform and stockpiled clay (Figure courtesy of Lhoist).

On the platform all operations took place. Next to the platform a mobile laboratory was present to check on soil properties such as moisture content and density.

Based on samples taken from the clays to be used several choices had to be made:
- The required percentage of lime per type of clay (typically 1 to 2 % above the so-called Lime Fixation Point. Below the Lime Fixation Point the lime reacts with substances in the clay that have little or no effect on the strength. The gain in strength is dependant on the so-called pozzolanic reaction, and that takes place only above the lime fixation point). Based on the findings the lime dosage was chosen to be 5 % for the good quality clay and 4 % for both poor quality clays. Lime dosage is expressed as percentage of weight of dry soil. Important to add is that it was chosen to add + 1% extra lime on both the good and the lesser quality clay. Because the organic content was 2 – 5 % in both types of clay, for the benefit of the test it was decided to add extra lime. To be specific: good quality clay 5 % lime (LMO of 4 % + 1 %), Poor quality clay A 4 % lime (LMO of 2,5 % + 1,5 %), Poor quality clay B 4 % (LMO of 2 % + 2%).
- The optimal moisture content for densification. As an example the effect of water content on density is shown in the next figure.

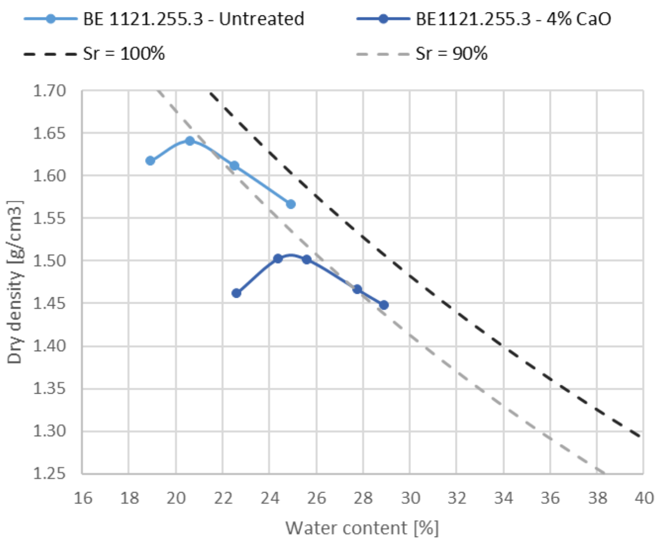

Figure 3-3 Dry density curves for "Poor quality clay B" natural soil and "Poor quality clay B" soil treated with 4% Proviacal® DD lime. (Figure courtesy of Lhoist).

3.5 Execution

The proposed construction methodology incorporates the following stages which are visualized in figures 3.4 to 3.7.

- Stage 1; Excavation of the steps in the existing structure and the first part of the berm (Figure 3-4).It was found that after stripping the grass and excatvating the staircase profile the sandy core of the dike was reached.
- Stage 2; Implementation of the first part of the berm in 2 layers and implementation of 17 layers of backfill (Figure 3-5).
- Stage 3; Recutting of the embankment and stocking of soil extracted from the recutting ((Figure 3.6).
- Stage 4: Placing the object, refilling and compacting the soil around the object (Figure 3.7).

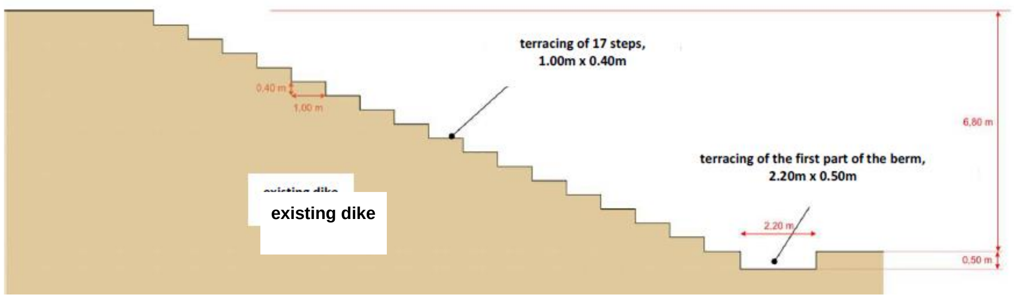

Figure 3-4.Construction methodology: stage 1: constructing the existing earthenwork steps.

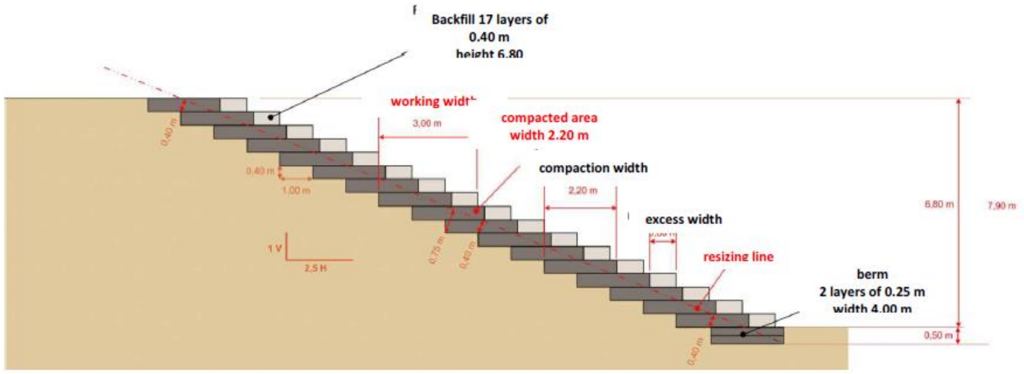

Figure 3-5. Construction methodology: stage 2: implementation of the first part of the berm and 17 layers of backfill.

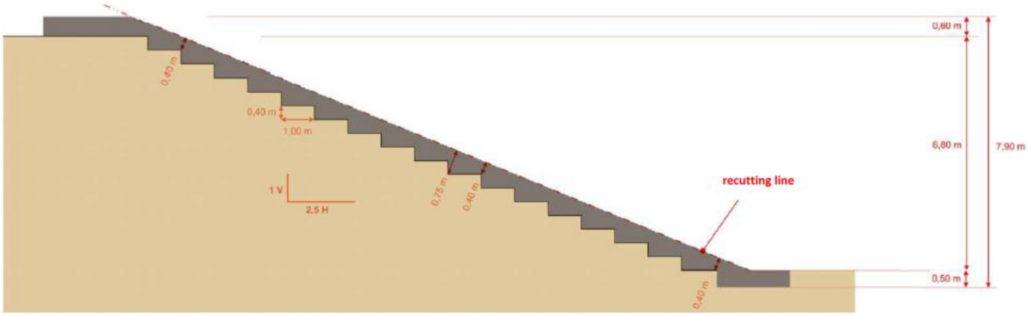

Figure 3-6. Construction methodology; stage 3: recutting of the embankment.

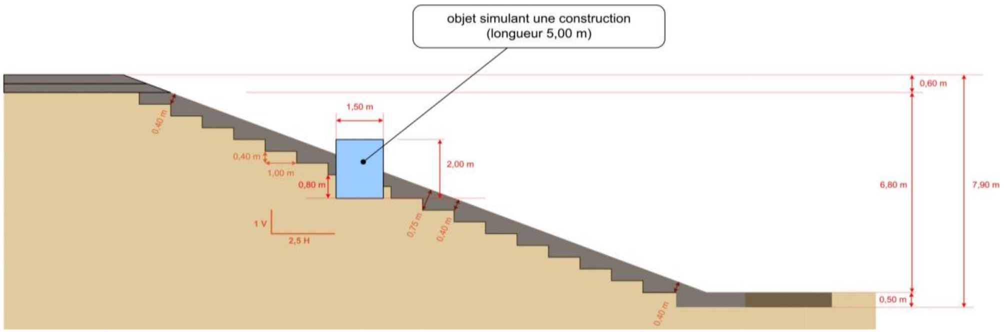

Figure 3-7 Construction methodology; Digging a hole, placing the object and densification around the object.

It should be pointed out that in the proposed methodology:

- The treated soils are applied in successive layers in the longitudinal direction with prior terracing of steps in the existing structure. Each layer corresponding to each step will be laid and compacted with an excess width of 0.80 m (not compacted) on the embankment side.
- Each step is terraced over a height of 0.40 m compacted in two layers of 20 cm after compaction. 17 steps are needed to reach the crest height or 17 layers of 0.40 m.
- Each layer is applied over a width of 3.00 m at the same altimetric level of each step or at a higher level, to take into account the counter-expansion coefficient (compaction that is the result of settling). Compaction is carried out with a vibrating padfoot compactor with a compaction width of approximately 2.20 m, leaving a safety zone at the edge of the embankment of 0.80 m wide. This 0.80 m zone will not be compacted.

- Recutting of the embankment was done by shovel from the bottom of the embankment. After recutting the slope, the second part of the berm and the dike crest are covered with the treated soils from the recut

3.6 Construction of test pad

The entire process of construction was very thorough. Before the material was applied on the dike the result of the operations was tried out on a test pad, or construction platform of 40 m by 40 m. The construction platform consisted of a densified flat sand bed in the polder, on which all preparation operations could take place. The following points were examined:

- Verification of actual soil treatment conditions, and monitoring of the results obtained in terms of:
  - Moisture content of the natural soil before treatment.
  - Thickness of the natural soil layer before treatment.
  - Dry density of the natural soil before treatment.
  - Amount of water if irrigation is needed.
  - Binding agent mixture (mass per unit area).
  - Milling of the soil binding agent.
  - Mixer speed.
  - Mixing depth.
  - Moisture content of the treated soil.
- Verification of actual conditions under which work is done with treated soil, including compaction, and monitoring of the results obtained in terms of:
  - Provisioning methodology for leveling and adjustment.
  - Thickness of the layers used to reach the compacted thickness.
  - Moisture content before compaction.
  - Compaction effort (compactor speed, number of passes).
  - Layer thickness after compaction.
  - Density obtained after compaction.
  - Peak resistance depending on the depth after compaction.

Depending on the observations made on the test pad, and the results of the checks carried out, the actual treatment and earthwork methods will be confirmed or modified where necessary.

3.7 Quality control and monitoring procedures

Quality control has been an important part of the entire construction process. It is not possible to go into detail of all measurement and results. A detailed description is given in lit. [8]. Throughout the earthwork operations, the following factors were monitored on an ongoing basis:

- Conditions for placing natural soil onto the stockpile:
  - Shoveling over the entire height of the stock (visual monitoring).
  - Stirring, if necessary (visual monitoring).
- Work Area for Treatment (Sequential):
  - Thickness of natural soil conveyed onto the treatment platform prior to compaction (using surveyor data or on-board guidance system).
  - Moisture content of the natural soil prior to compaction (random sampling and rapid measurement by microwave drying).
  - Humidification of the natural soil if needed (visual monitoring, grading equipment, bin weighing if necessary).
  - Surveyor data of the soil layer before treatment.
  - Binding agent mixture (bin weighing if necessary).
  - Amount of binding agent spread (spreader weight scale tickets).
  - Covering of the spreading strips (visual monitoring).
  - Mixing depth (visual monitoring, measurement with a soil auger).
  - Mixing speed (visual monitoring, timing).
  - Covering of mixing bands (visual monitoring).

- Moisture content of the treated soil (random sampling and rapid measurement by microwave drying).
- Milling of the treated soil (visual monitoring, random checks with sieving if necessary).
- Thickness of the placed treated soil (surveyor data and visual monitoring).

• Work Site for Earthwork:
  - Interval between the end of treatment and compaction (time elapsed time).
  - Thickness leveled soil prior to compaction (using surveyor data or on-board guidance system).
  - Moisture content prior to compaction (random sampling and rapid measurement by microwave drying).
  - Compaction methods: number of passes, speed, sweep-plane (visual monitoring, timing, tachograph).

The moisture content was measured on the experimental pads immediately after leveling is completed, and prior to compaction. This was done by taking in situ soil samples, the moisture content of which was determined by weighing and immediate microwave drying (or after weighing and packing in a leakproof bag by laboratory drying). At least 2 samples per layer were taken per pad, distributed over the surface of the layer.

The company surveyor will check the thickness of each layer, by measuring levels or using an on-board guidance system. If a survey carried out by the surveyor, the measurements will be taken at the same locations (same X and Y coordinates in the $\pm$ 0.20 m plane) for each layer on a grid to be defined by the surveyor, in order to determine the average thickness of each compacted layer. The level measurements will be taken at the top of the imprints left by the vibrating padfoot compactor.

The densification level was also monitored with in-place density and peak resistance measurements with a dynamic variable energy penetrometer. The results were analyzed with reference to the peak resistances corresponding to the typical case concerned (soil class - hydrological status) and by comparison with the peak resistances obtained on the compaction test pad. These measurements were carried out after the first two layers have been applied, at mid-fill, and on the last two layers applied.

A site logbook was kept and updated on a daily basis:
• Weather conditions (temperature, rainfall, wind).
• The schedule for the various jobs (start time, end time, metrics; cubatures):
  - Conveyance of natural soil onto the treatment platform.
  - Preparation of the natural soil before treatment.
  - Irrigation.
  - Spreading.
  - Mixing.
  - Movement of treated soil.
  - Shell earthwork.
  - Compaction.
• Surfaces and volumes applied layer by layer.
• observations noted during routine monitoring (visual observations).
• Incidents encountered.
• List of checks carried out (type, location), with all associated information and data.

Figures 3-8 and 3-9 show the final result of the test sections after completion.

Figure 3-8 Completed test sections with objects. From left to right: Section 1 with Poor quality clay A, Section 2 with Poor quality clay B and Section 3 with Good quality clay.

Figure 3-9 Side view of completed test sections.

3.8 Panda tests after construction and after three months

As part of the quality control of the work for the test sections 14 so called PANDA tests have been performed (among many other things) (lit. [9]). These PANDA tests have been repeated at the request of Deltares during the overtopping tests three months later. This provides the best idea of the influence of curing of the lime treated soil in this three month period.

The principle of a light dynamic penetrometer with variable impact energy, type PANDA, consists of driving a fixed CPT point (base surface 2 cm² with an apex angle of 60°) into the ground, attached to a series of metal rods (with surface area 2 cm²) with a variable speed using a hammer with a normalized weight. For each blow on the hammering head, sensors measure the value of the driving and the hit speed at the time of impact, which is used to determine the energy supplied.

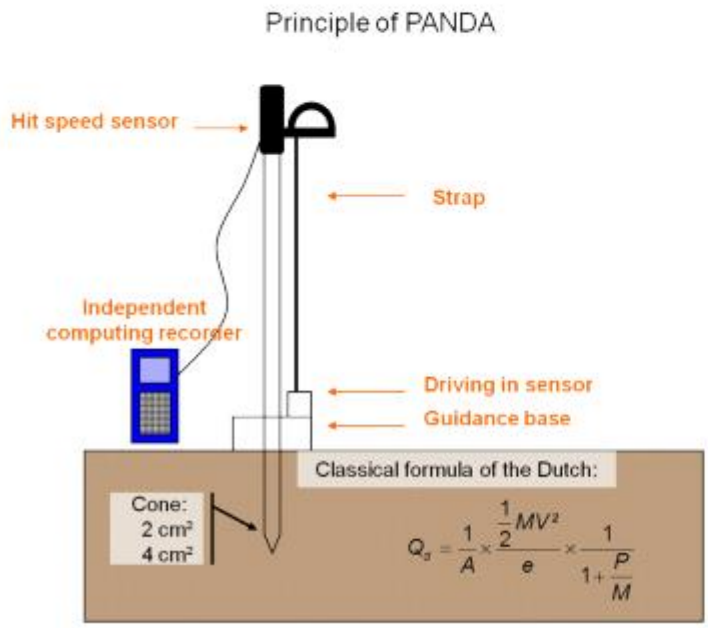

Figure 3-10 Principle of the PANDA test for determining soil density (Figure copied from [9]).

The compaction control using the PANDA light dynamic penetrometer is carried out in accordance with the French standard NF P 94-105. Depending on the desired quality of compaction, the thickness of the compaction layers, the type of the material used and their moisture content, two lines can be defined, a reference line (green line) and a rejection line (red line). These are visually presented on the graph. The control of the compaction then consists in comparing the obtained measured values with these reference and rejection lines.

The PANDA results are of interest because the test have been performed after completion of the work, and have been repeated three months later while the overtopping tests were taking place. It is interesting to compare these results, as the strength of the lime treated clay develops over time.

As an example the results are shown for Poor quality A clay, Section 1. These results are exemplary for the other sections as well.

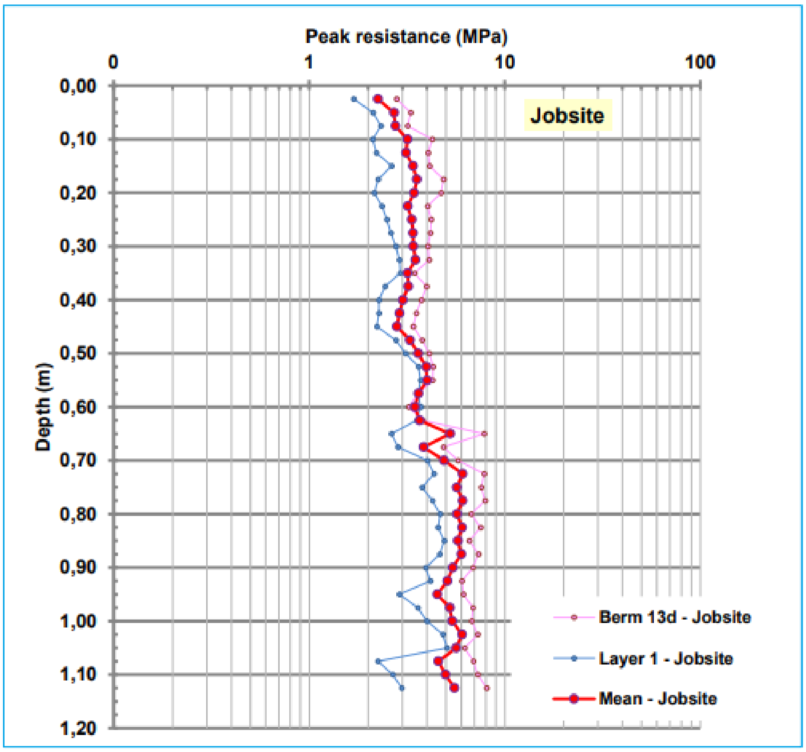

Figure 3-11 Results of PANDA test in November 2021(Figure courtesy of Lhoist).

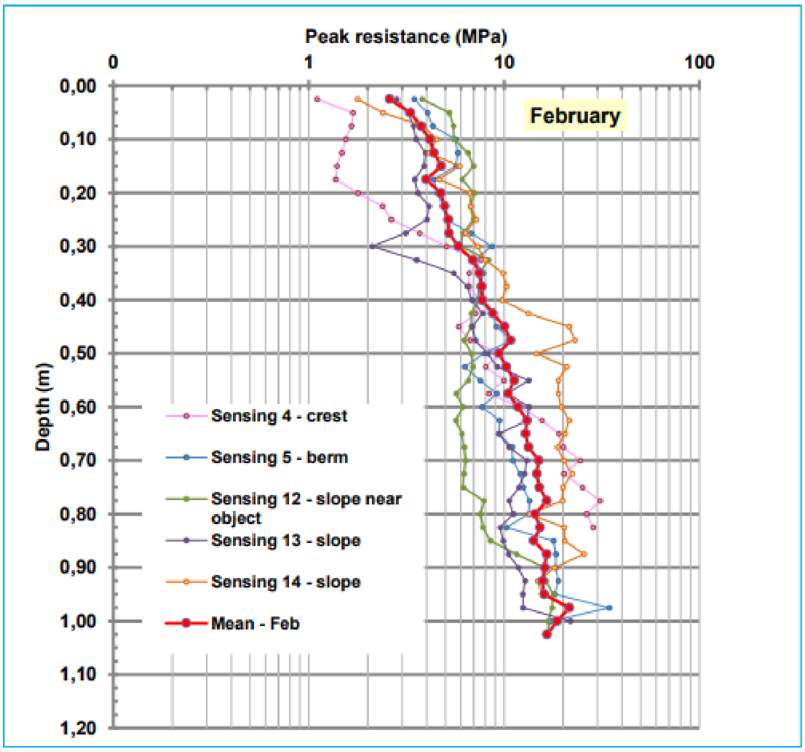

Figure 3-12 Result of PANDA test in February 2022 (Figure courtesy of Lhoist).

From these figures it can be concluded that the lime treated material has gained in strength considerably, even though conditions have been far from optimal (too wet, too cold).

4 Sampling and triaxial tests on the lime treated clay cover

4.1 Introduction

The information in this chapter has been derived from the report Laboratory tests on the lime treated cover clay layer by Deltares (lit. [6]). More detailed information can be found in that report.

At three moments Deltares has taken samples of the cover layer of the three different test sections in the Hedwigepolder:

- November 30 2021 (round A, about 11 tot 16 days after construction of the test sections).
- January 18 2022 (round B, about 60 to 65 days after construction).
- February 10 2022 (round C, about 83 to 87 days after construction).

The general idea was to take samples and perform laboratory tests on the samples to follow the development of strength in time.

4.2 Summary of the findings

It is known that lime treatment needs curing time. Strength develops in time as the chemical reactions that cause the increase in strength are not instantaneous.
In literature many figures like the figure below demonstrate this development of strength in time.

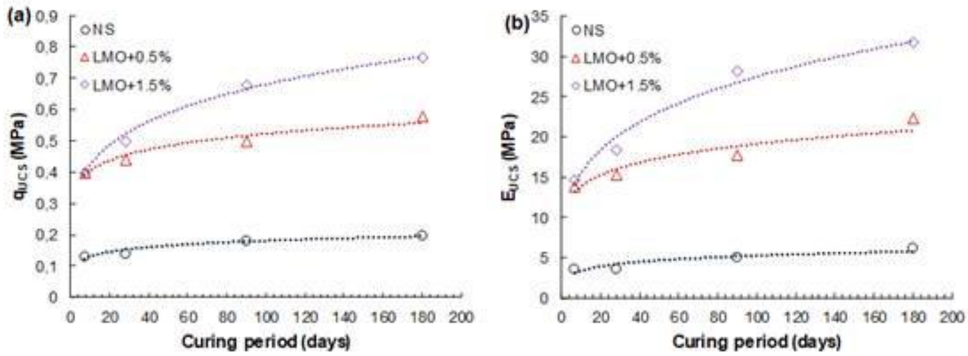

Fig. 3. Development of: (a) unconfined compressive strength, q_{UCS}, and (b) unconfined compressive elastic modulus, E_{UCS}, with curing period for untreated and lime-treated Warmenhuizen clay samples

Figure 4-1 Expected development of strength in time (lit. [2]).

Deltares has taken a first round of samples shortly after construction of the three test sections. The samples were taken in a traditional way, by pushing a sampler into the slope, thus extracting samples from the slope. The material was reported to be brittle, and it was difficult to take samples of sufficient size, as many samples broke. However, there was enough material left from the borings to take samples from each test section and test those in the laboratory.

In the laboratory triaxial tests were performed on the samples from different sections. In figure 4-2 the results from samples of Section1, 11 to 16 days after construction, are compared to one sample of the untreated natural clay. Compared to natural soil it was demonstrated that in a few days there was a substantial gain in strength for the lime treated samples.

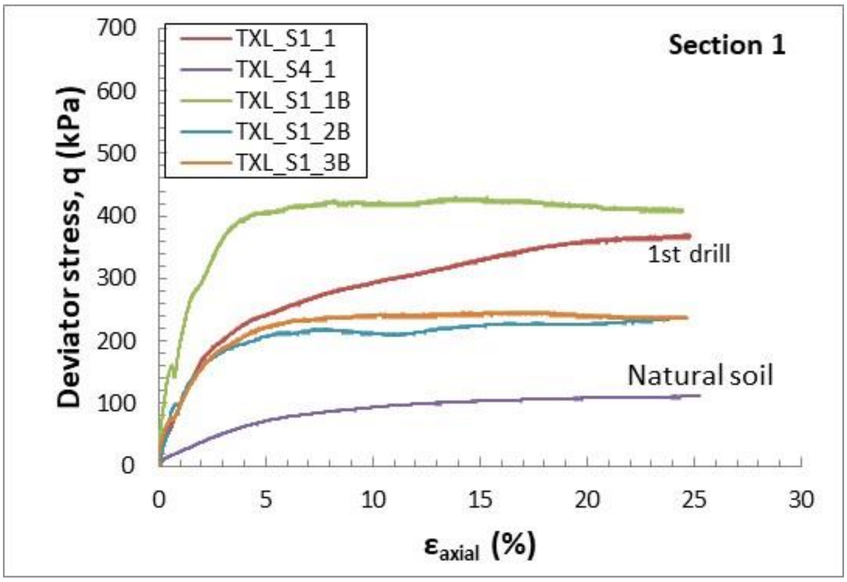

Figure 4-2 Results from the first round of testing.

In January, 60 to 65 days after construction, a second round of samples was taken. It was anticipated that the strength would have developed further, and that traditional sampling could be difficult. For that reason it was decided to use a drilling machine to take samples.

Figure 4-3 Use of drilling machine for taking samples in round B and C.

Again the material was reported to be brittle and broke easily. Triaxial tests were performed on the samples of Round B. Contrary to expectations it was found that 5 out of 6 samples showed lower (peak) strength.

Besides the factor time, the main difference between the two rounds of testing was the method the samples were taken. Because the material was brittle it was difficult to derive samples of sufficient size, which raised the question whether or not the samples could be declared representative for the actual material on the slope in the Hedwigepolder. Samples were sent to the technical university of Delft to make scans of the interior of the samples (X-Ray micro-computed tomography (µCT) scans).

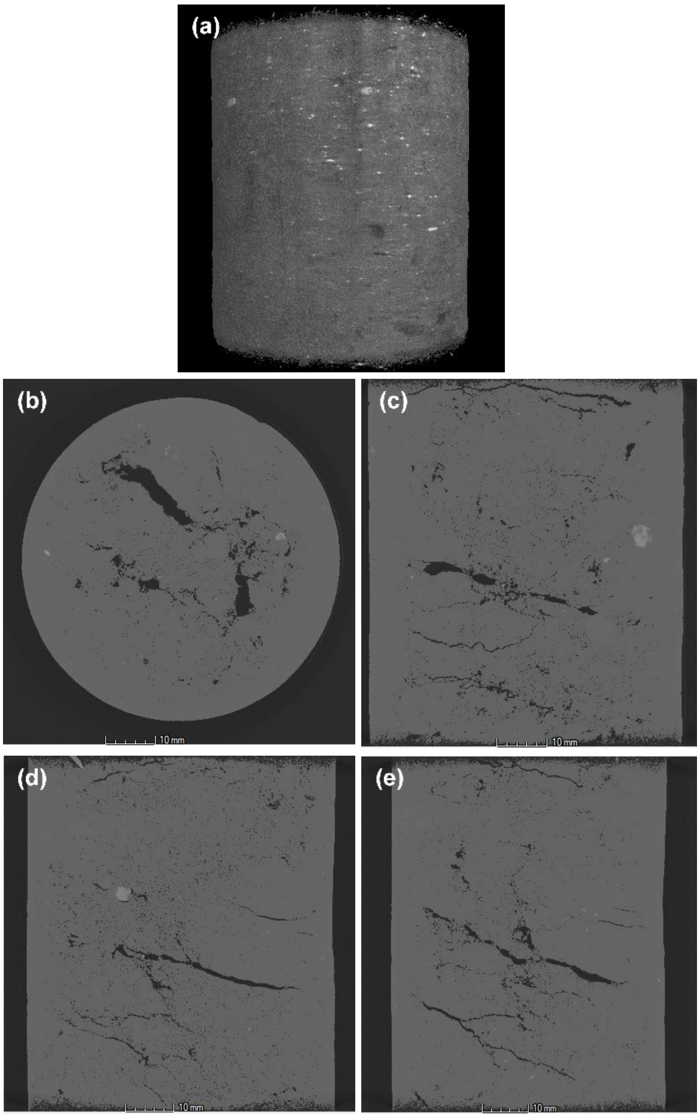

Figure 4-4 X-Ray µCT scans on samples TXL_S1_1C: (a) 3D reconstruction image of the surface of the clay sample, (b) horizontal and (c), (d), (e) vertical sections of the sample.

The samples showed many defects. Apparently the method of taking samples does not provide trustworthy results. The alternative explanation, that the deficiencies are already present in the material in the slope prior to sampling, is considered less probable. By communication of Lhoist past experiences in France also lead to serious doubts about the integrity of the specimens microstructure after the coring operations.

A third round of sampling, again using a drilling machine, was taken in February 2022, 83 to 87 days after construction. Based on the earlier findings it was decided not to perform triaxial tests on these samples.

4.3 Conclusions

A laboratory testing program has been performed at the Deltares geotechnical laboratory on samples treated with lime taken from Hedwigepolder testing site in Zeeuws Vlaanderen (the Netherlands). In summary the triaxial test results have shown that:

- The response of the samples to triaxial shearing does not show consistent material improvement or deterioration with time. In terms of undrained shear strength, almost all of the lime-treated samples have higher strengths than the natural soil sample, however, the measured strength in the current series of triaxial tests cannot be linked to curing period or type of clay used for treatment.
- The lime-treated material is dry, hard and brittle. It easily breaks into pieces during trimming in the laboratory. X-Ray tomography scans of the samples before testing reveal the presence of fractures that were not observable by the naked eye, a finding the repercussions of which should be considered in the interpretation of the experimental results.
- The triaxial test results in combination with the X-Ray scans lead to the conclusion that the samples could have been fractured during sampling. In this case, what is measured in the laboratory is not representative of the in-situ material properties, but it is rather a soil response dominated by the structure of the material after sampling.
- When soils in the field are treated with lime, it is recommended to assess their properties using in-situ measurement techniques. Due to the fragile nature of the lime-treated material, sampling should be avoided as it can lead to material disturbance, poor quality samples and misleading correlations between the in-situ and laboratory measured soil properties.

Apart from the triaxial tests the salt content of the samples was determined. That was well within limits for the demands for use of the clay in Dutch dikes. The permeability of the lime-treated samples is in the order of 10^{-7} to 10^{-9} m/s. These permeability values are low, and they are indicating a rather impermeable material.

5 Testing program for the overtopping tests

5.1 Introduction

To make lime treated clay an interesting alternative for grass covers (on regular slopes) or clay covers (around obstacles where no good grass cover can develop) the starting point was that the lime treated layer would have to be able to withstand an overtopping discharge of at least 10 l/s/m'. If succesful higher overtopping discharges could be tested, in steps, to the maximum capacity of the wave overtopping simulator.

Van der Meer Consulting was asked to make a proposal for the basic testing program. This resulted in a memo (Mogelijke proefcondities Kalk in klei, Van der Meer Consulting, memo vdm20503.3122.1, 3 januari 2022) that has formed the basis for the testing program.

The memo and the basic testing program was discussed with van der Meer Consulting, Infram, Rijkswaterstaat and Deltares. Scenarios were developed in the case the actual behavior during the testing would differ from expected. However, in practice the basic testing program as proposed was followed.

Figure 5-1 The wave overtopping simulator (WOS) on site at the Hedwigepolder.

5.2 The basic test program with increasing wave loads

The text of this paragraph has been largely derived from the memo by van der Meer Consulting.

It was assumed that erosion of the treated clay cover layer was of a different type than that of a grass cover. That is meant to say that the erosion is not expected to be of the type 'cumulative overload', but is more or less independent from repeated loads. If that assumption is correct, that means that it is not necessary to simulate entire storms (for instance 6 hours duration), but that in 2 hours of simulation of a storm condition the result would be clear.

The load sequence is based on a few principles:

- Start with the supposed weakest cover with the lowest conditions. If there is no erosion then it is an option to skip that step for supposed better covers.
- If little or no erosion occurs in the first 2 hours make a few big steps towards higher loads. If erosion is substantial it can be deemed necessary to prolong the test.
- As a basic program consisting of four conditions for each cover layer has been thought of, each step more or less doubles the load of overtopping water of the previous step, and in four steps the maximum capacity of the wave overtopping simulator is reached. The distribution of overtopping wave volumes is given in the next Figure.

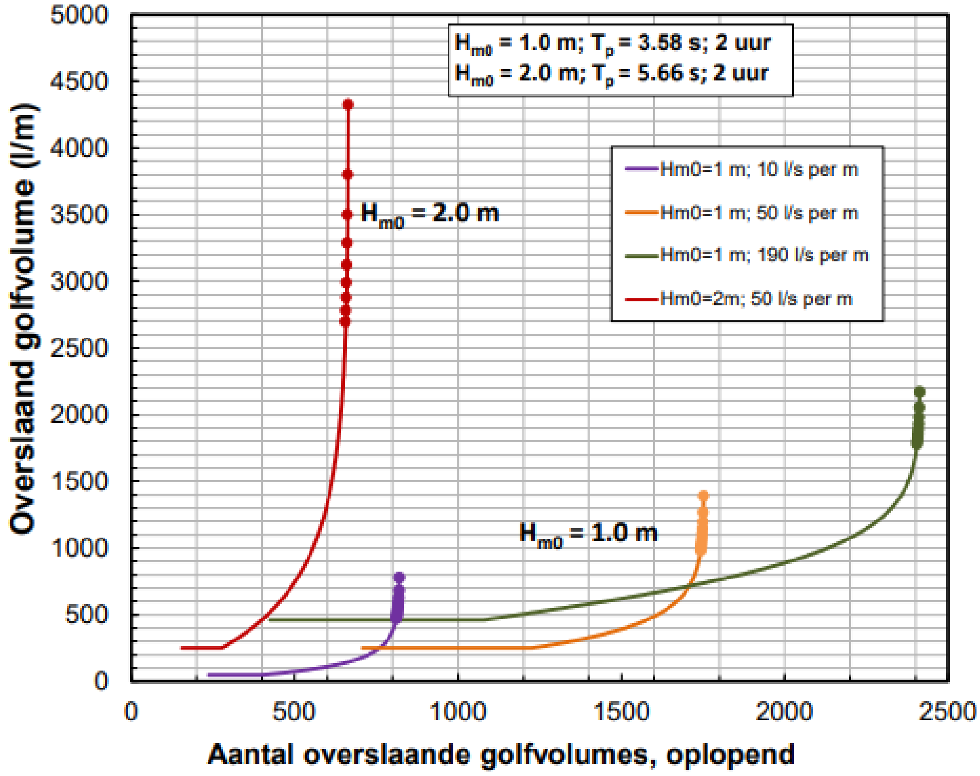

Figure 5-2 Proposed sequence of ascending wave loads (Figure courtesy van der Meer Consulting).

The tested conditions are:

- River dikes: H_{m0} = 1 m; T_p = 3.58 s; wave steepness s_{op} = 0.05; overtopping discharge q = 10 l/s per m; duration 2 hours.
- River dikes: H_{m0} = 1 m; Tp = 3.58 s; wave steepness s_{op} = 0.05; overtopping discharge q = 50 l/s per m; duration 2 hours.
- River dikes: H_{m0} = 1 m; Tp = 3.58 s; wave steepness s_{op} = 0.05; overtopping discharge q = 190 l/s per m; duration 2 hours.
- Sea dikes: H_{m0} = 2 m; Tp = 5.66 s; wave steepness s_{op} = 0.04; overtopping discharge q = 50 l/s per m; duration 2 hours.

Each test can be performed in real time, except for the test with an overtopping discharge of 190 l/s/m’. At this condition the waterline is about equal to the crest height, and a higher overtopping rate would mean that also overflowing conditions would occur. This cannot be simulated with the current wave overtopping simulator. With a maximum pump capacity of 90 l/s/m’ the test has to be slowed down. A simulation of 2 hours storm takes about 5 hours in reality.

With this basic program it was estimated that an entire sequence of testing, including intermediate stops to measure erosion would take 2 testing days for one test section. Taking into account that moving the wave overtopping simulator to a next section would also take one day it was estimated that a minimum of 4 and a maximum of 5 test sections could be tested.

It should be noted that these test conditions are very severe, and represent conditions that in current practice have an extreme low probability of occurring during the lifetime of a dike reconstruction. It represents conditions where a grass cover has a high probability of failure and is meant to study the failure strength of lime treated soil for these conditions.

5.3 Testing of subsequent sections

To make reference to the test sections see the next figure:

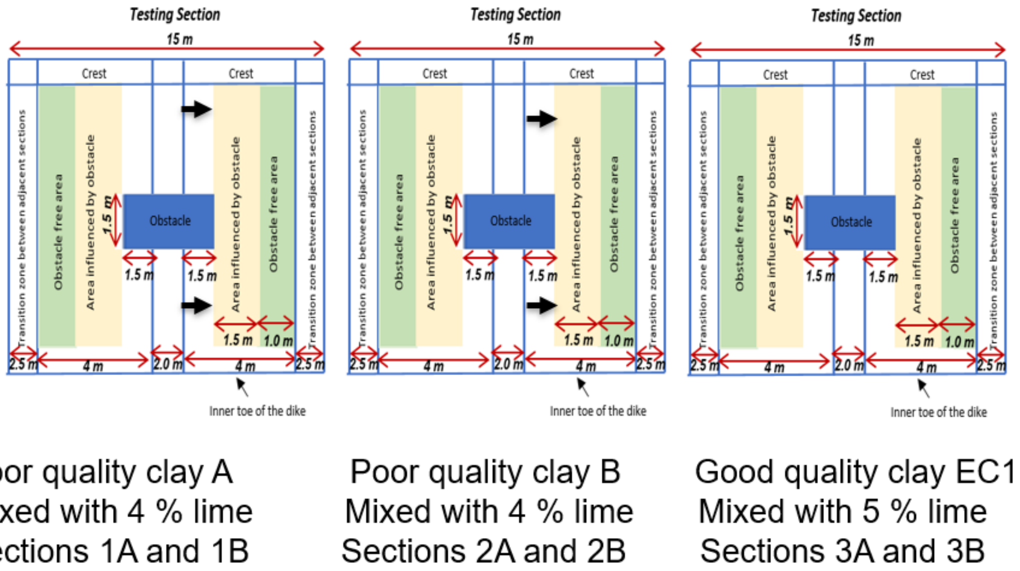

Figure 5-3 Reference to the different test sections. Note that tests in sections 1B and 2B were performed without object (2,5 m wide instead of 4 meters) and test section 1A, 2A and 3A are with object (test section of 4 m wide).

The testing sequence was designed to make the movement from one section to the next as easy and swift as possible: (section 1A → section 1B → Section 2A → Section 2B → Section 3A). The purpose of the tests was:

- Section 1A: testing of poor quality clay with lime treatment: reference test with object.
- Section 1B: testing of poor quality clay with lime treatment, without object.
- Section 2A: Repetition of test 1A to see if results reproduce.
- Section 2B: testing of poor quality clay with regular waves; the idea was that with ascending (regular) wave loads the erosion parameters could be derived. Test conditions are summarized in Table 5-1.

- Section 3A: testing of good quality clay with lime treatment; compare good quality with poor quality clay.

Figure 5-4 The wave overtopping simulator at work.

Proef	*N* [-]	*V* [l/m]
1	200	100
2	200	250
3	100	500
4	100	1.000
5	50	2.000
6	25	3.000
7	20	3.400

Table 5-1 The regular wave tests (N = number of waves, V is overtopping volume per metre).

6 Test results

6.1 Introduction

The overtopping tests were performed by Infram Hydren BV. Large parts of the information in this chapter has been derived from the Factual Report Praktijkproeven Kalk in klei of Infram Hydren (lit. [7], in Dutch).

The overtopping waves were simulated with the Wave Overtopping Simulator (WOS, see Figure 6-1). In the huge yellow container the water is stored. The waves are simulated by hydraulic valves that release the water. The valves can be operated very exact to simulate the correct volume of overtopping water. The overtopping volumes are determined and established in steering files, so that real life storm events are reproduced as close as possible.

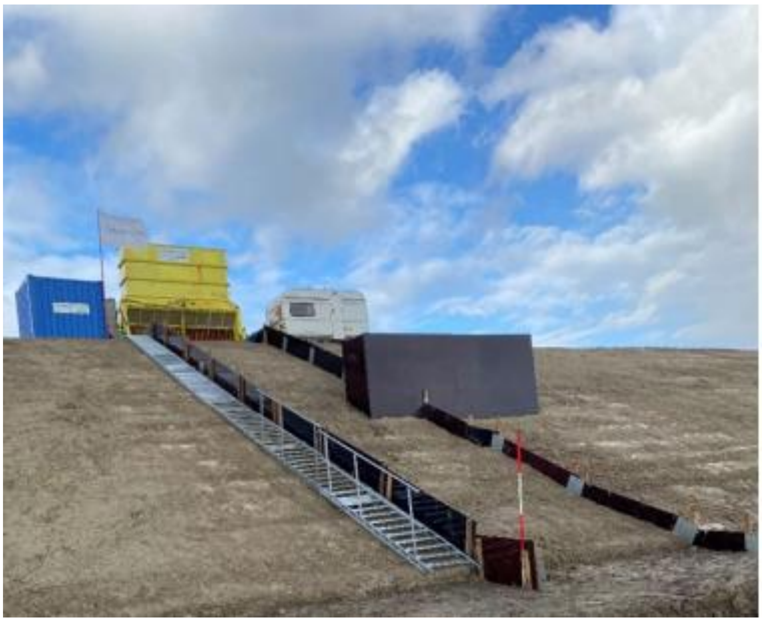

Figure 6-1 The Wave Overtopping Simulator in Section 1A.

In this chapter the events as observed in Test Section 1A (the first sequence of overtopping tests) are described in some more detail. The events in the other test section are described more briefly, to prevent parts of text repeating.

6.2 Test section 1A

6.2.1 Test 1m10lsm (2 storm hours)

With the start of this test it was directly visible that lumps of clay were eroded. The staircased profile became visible during the intervals of this test (each 0,5 hour testing was paused to take erosion measurements). On each step of this staircase the prints of the compaction machine became more visible. Around the object there was erosion, both directly around the upstream side of the obstacle as at a distance of about 50 cm next to the obstacle (the border between originally deposited and compacted material and the material around the obstacle that was placed and compacted after placement of the obstacle).

As a result of erosion (taken the form of staircase steps) the wooden boards were undermined. A substantial (and growing) amount of leakage of water flowing from the test section took place.

Erosion was most severe during the first hour of testing. During the second hour of testing the situation seemed to stabilize, so it was determined to proceed with the next step.

6.2.2 Test 1m30lsm (0,5 storm hour)

Because the amount of erosion seemed larger than anticipated an extra test step was introduced. However, little extra erosion was observed, and testing continued with 50 l/s/m' overtopping.

6.2.3 Test 1m50lsm (2 storm hours)

The staircased profile became more prominently visible. This means some extra erosion took place, especially during the first hour of testing. Around the object the erosion profile did not develop very much. Near the crest an erosion hole of about 1,5 m width and 10 cm depth developed. This erosion hole near the crest kept developing slowly during this test and the following steps.

Figure 6-2 Erosion near the crest after 50 l/s/m'.

6.2.4 Test 1m190lsm (2 storm hours)

The staircased profile became more prominently visible. The erosion around the object was deepened from approximately 20 cm deep to 50 cm deep, also forming a kind of staircase step next to the obstacle. Below the obstacle there was also erosion visible, which did not show after the earlier stages of the test (50 l/s/m' and less).

Figure 6-3 Erosion also below the obstacle.

6.2.5 Test 2 m 50lsm

Before starting the test with 2 m waves hydraulic measurements were performed to determine the flow velocity of the overtopping waves on the slope. This consisted of an ascending series of regular wave overtopping volumes to measure the wave propagation velocity on the slope. With the biggest wave (overtopping volume of 3000 l/m') of this sequence the object was pushed from the slope and ended on the berm below.

Figure 6-4 The failed obstacle.

6.3 Test section 1B

In test section 1B the same wave overtopping sequence as in section 1A was repeated, but without object. The wooden boards were placed next to the object, but the width of the test section was reduced to 2,5 m. The treated and densified area did not extend further.

The most distinctive difference in behaviour was not the erosion on the slope (that was very comparable to the observations in section 1A) but the behaviour next to the obstacle. Although there was no obstacle present in the test section, there was a zone next to the object that was excavated to be able to place the object and densified separately after bringing back the lime treated clay. There was almost no erosion directly next to the obstacle, but on the transition next to the obstacle an erosion trench was formed.

Figure 6-5 Test section 1B and erosion trench next to the obstacle after the test with 10 l/s/m'.

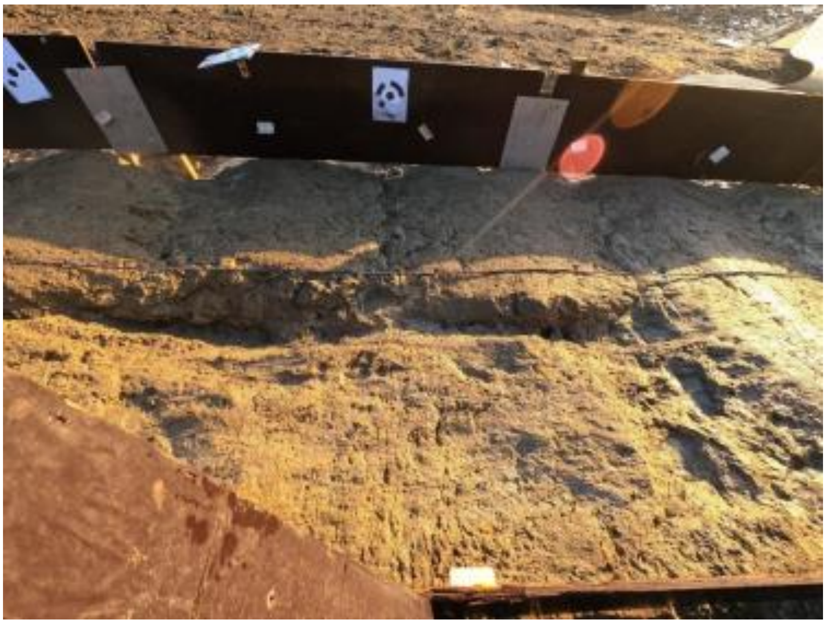

Figure 6-6 The erosion trench after the last test (2m 50 lsm).

6.4 Test section 2A

Test section 2A was a copy of test section 1A (clay of poor quality with an object). This was done to see whether or not the results from test 1A would reproduce.

To prevent the object from failing two large iron bars were placed behind the object into the soil. This was also done with the object in section 3A.

In comparison to section 1A there was little or no erosion around the object, and there was little or no erosion near the crest. Only after the tests with H_{m0}=1 m and 190 l/s/m and H_{m0}= 2m and 50 l/s/m there was some erosion around the object.

Figure 6-7 Erosion around the object after the final test (2m 50 lsm).

6.5 Test section 2B

In test section 2B the situation without object was tested with a series of regular waves. The purpose was to see if erosion would increase with waves of increasing volume. The width of the testing section was 2,5 m.

Before the start of the testing there already a substantial amount of erosion on the downward side of the object. This was caused by leaking water during testing of section 2A. Also on the berm and at the toe of the slope there was substantial erosion.

Figure 6-8 Before the start of testing in section 2B.

During the actual testing erosion on the upper part of the slope developed as expected with a staircased profile as observed in the tests with irregular waves. Erosion on the slope below the object was substantial, especially near the toe and the berm. The cover layer of at least 40 cm thick was not broken anywhere on the slope. It was very clear from the test that the lower part of section 2B was the weakest of all test sections. That had nothing to do with a difference in hydraulic load between the regular waves and the irregular waves, as the erosion was already apparent at the lower discharge volumes. The difference had to do with the construction of this test section (see also chapter 8.5).

6.6 Additional test on section 2B

After the test in section 2B a hole was dug in the cover layer of lime treated clay, exposing the sandy core of the dike to the overflowing water.

Figure 6-9 The situation at the start of the extra test: the original stair case is exposed to wave action (picture courtesy of Infram).

The test was started with half an hour of waves corresponding with an overtopping discharge of 10 l/s/m', which gave limited erosion of the stairs. After that the test was continued 1 hour with an overtopping discharge of 30 l/s/m'. After that the entire upper step was eroded and the step beyond that was seriously undermined (see picture).

Figure 6-10 One step has been eroded and the next step is seriously undermined (picture courtesy of Infram).

After this the test was stopped.

6.7 Test section 3A

The test in section 3A was done with object and the same sequence of waves as in sections 1A and 3A. The behaviour was very much comparable with section 2A: a distinct stair cased profile was eroded during the tests, and only at the largest wave loads (2m 50 l/s/m') there was a little erosion around the object. The berm showed a bit more erosion, but the cover layer was at no point eroded through the cover layer of lime treated clay.

6.8 Additional test on section 3A

After the last test in section 3A an additional test was performed by digging a cliff in the slope (see Figure). At the downstream end the cliff was a maximum of 40 cm deep. This could represent a failed transition (the hole) between slope covers that was strengthened by using lime treatment.The part above the cliff was supposed to be a failed cover layer, and the part below the cliff was to represent the transition construction that consisted of lime treated clay.

Figure 6-11 A Vertical cliff was dug into the slope (picture courtesy of Infram).

A 1,5 hour storm was simulated with overtopping rates of 1m 10 l/s/m', 1m 30 l/s/m' and 2m 50 l/s/m' . No ongoing erosion was observed. After that 25 full loads (3400 liter of water per meter) were released. Still no erosion occurred.

Figure 6-12 Situation after the final test (picture courtesy of Infram).

7 Erosion measurements

7.1 Introduction

Erosion has been measured in two different ways. The University of Louvain used a for this situation experimental technique of photogrammetry to provide very detailed images of the entire surface. Infram used a standard measuring beacon, providing a back up measurement in case the photogrammetry would give unsatisfactory results.

7.2 Findings photogrammetry (University of Louvain)

On the sideboards and object specific marks were placed (see picture below, the marks are the A4 sized white papers). The exact location of all the markers was measured with a measuring beacon (Infram).

Figure 7-1 Markers in test section.

After approximately each half hour during the test sequence the wave overtopping machine was stopped and measurements were taken. A measurement was done by a GoPro camera mounted on a large stick. The GoPro camera took time-lapse pictures with an interval of 1 second. By manoeuvring the camera under different angles the aim of the pictures was that at least two of the markers on the sideboard were visible on the picture. If that is the case, then the exact location of the camera is known, and the elevation of the surface can be calculated very accurately.

Figure 7-2 Measurement of the surface elevation using a GoPro camera mounted on a stick.

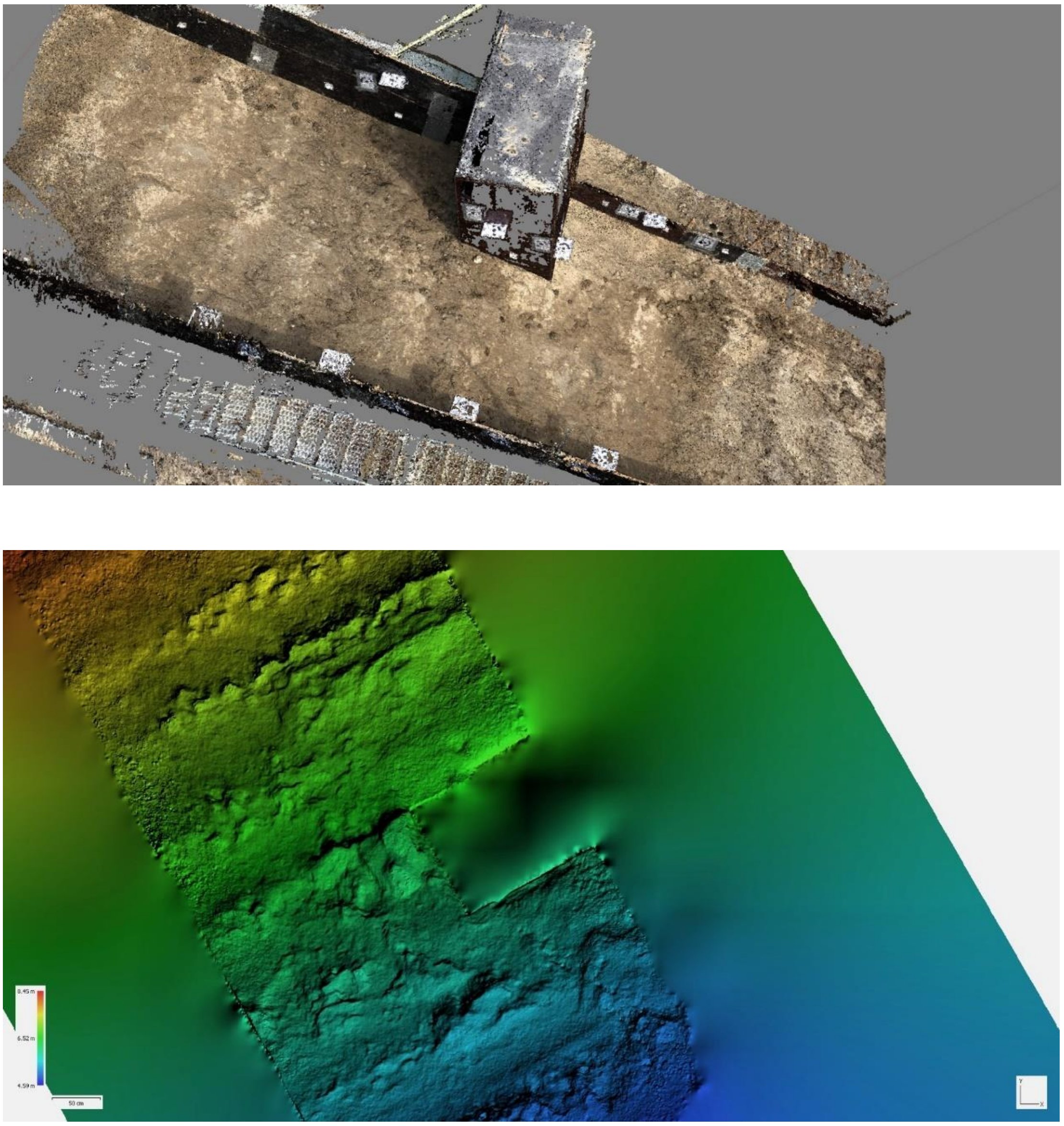

Figure 7-3 The two pictures give an impression of the detailed information obtained with the photogrammetry analysis of the University of Louvain. The pictures were taken after the first test period in Section 1A with an overtopping discharge of 10 l/s/m'. Erosion is not very prominent, still all the lumps of clay and the staircased profile can be seen. Next to the object some erosion is forming. In the second figure the legend is not adapted to the fact that the picture is taken on a slope: red is high and blue is low. The figure is for illustrative purposes only.

7.3 Some figures from the photogrammetry surveys

Before the actual experiments (T0) and after each test condition (T1, T2, T3, T4 and T5) surveys were made with the GoPro cameras. See Table 7-1 for an overview.

Table 7-1: Phhotogrammetry processing intervals.

Ti	Condition	LTC1	LTC2	LTC3
T0	Non-clean section/ As constructed	Done	Done	Done
T1	1m10lsm- after 30 minutes	Done	Done	Done
T2	1m10lsm-after 2 hours	Done	Done	Done

T3	1m190lsm-after 30 minutes	Done	-	-
T4	1m190lsm-after 2 hours	Done	*To be done*	Done
T5	2m50lsm-after 2hours	NA	Done	Done

In the next figures the aerial view from test sections 1A, 2A and 3A are shown at T0 (before testing), T1 (after the first experiment) and T5 (after the last experiment in the section) are shown.

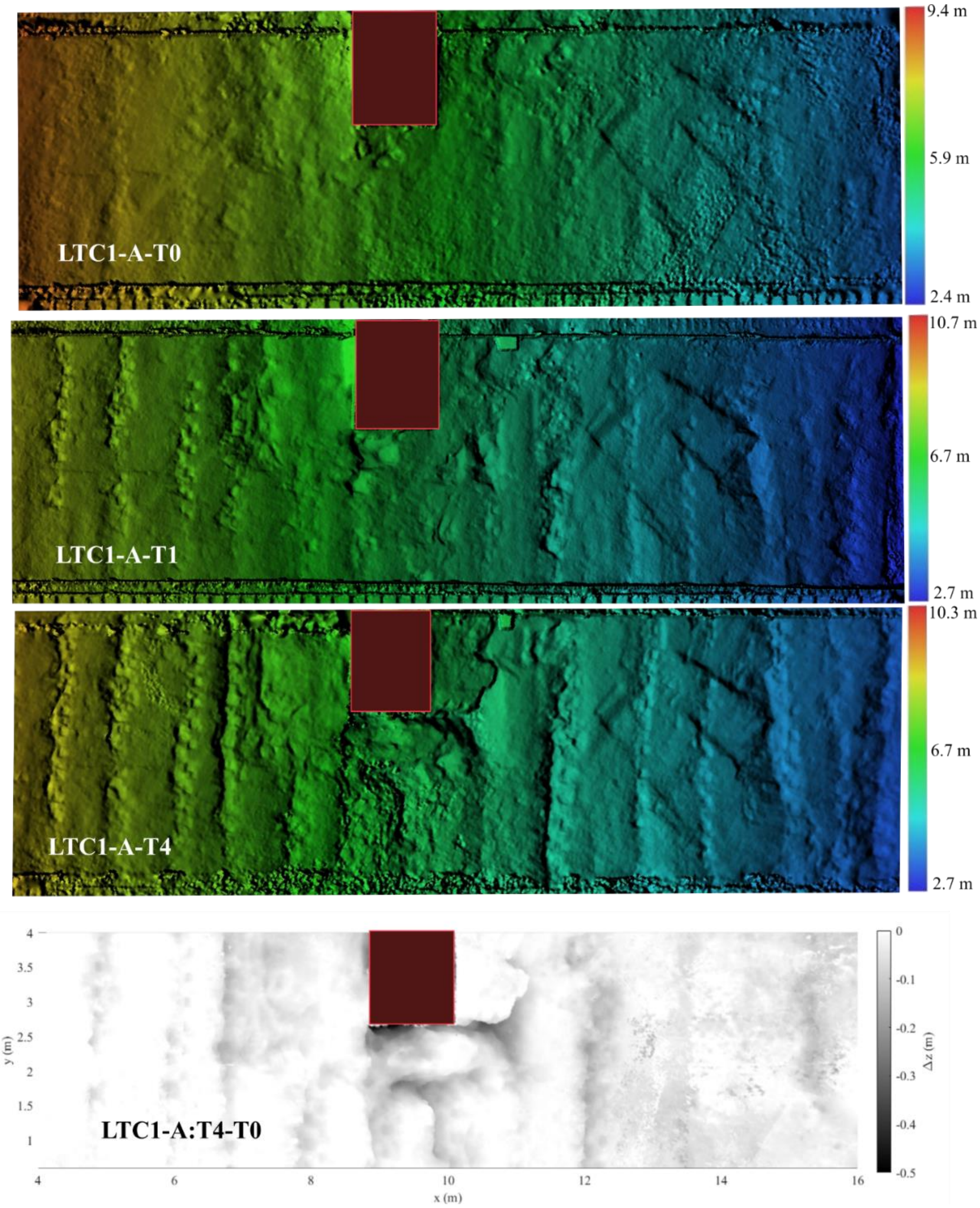

Figure 7-4 Results in test section 1A.

In Figure 7-4 the development of erosion into a staircase profile is clear. Also, around the object the most significant erosion is found, almost up to the layer thickness of 40 (minimum) to 70 cm (maximum).

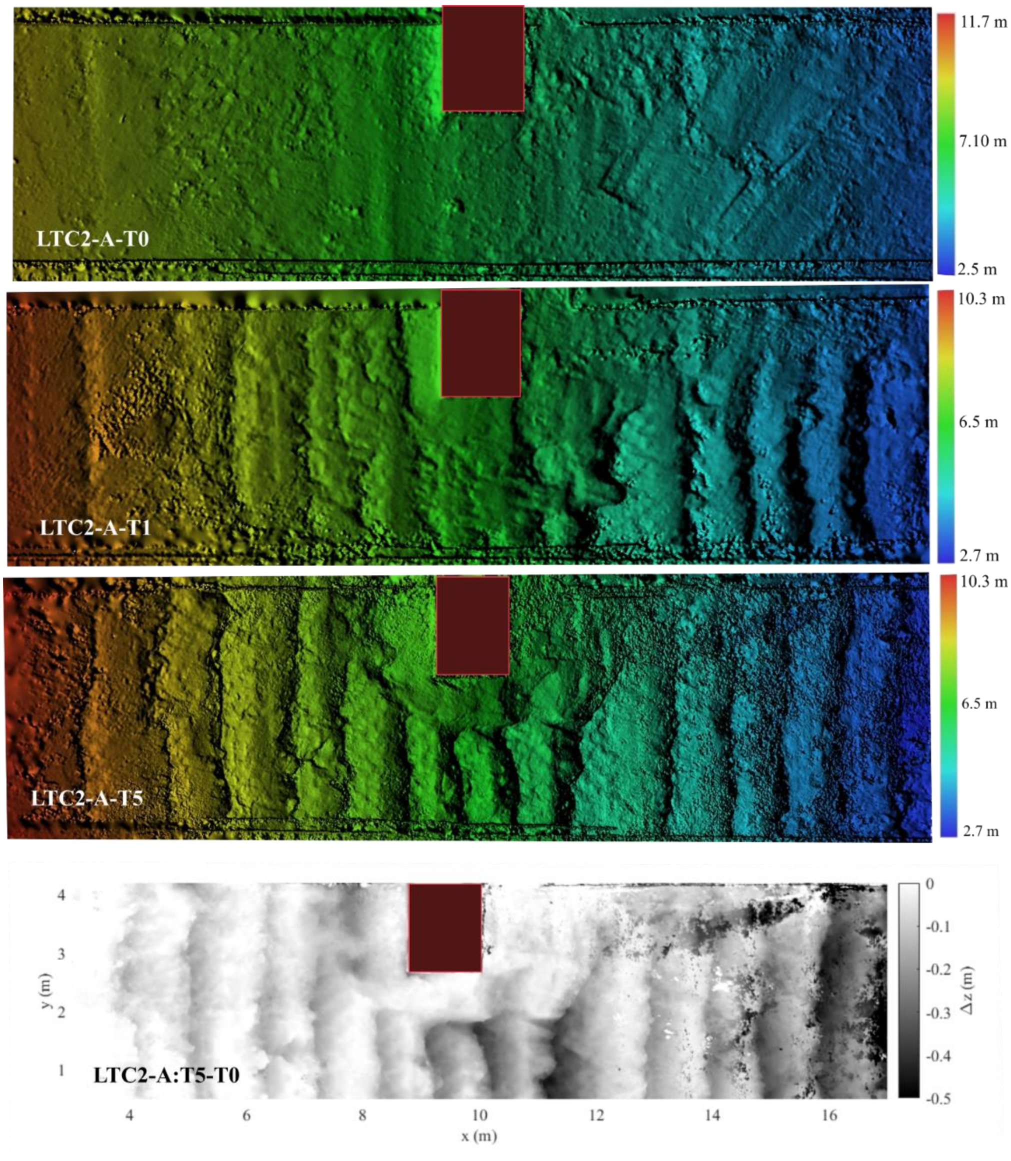

Figure 7-5 Results in test section 2A.

In Figure 7-5 it is evident that in contrast with Figure 7-4 almost no erosion has taken place around the object. There is, however, a gully and more erosion than elsewhere on the slope directly next to the carefully densified area around the object. The toe of the slope has also suffered more erosion than the slope.

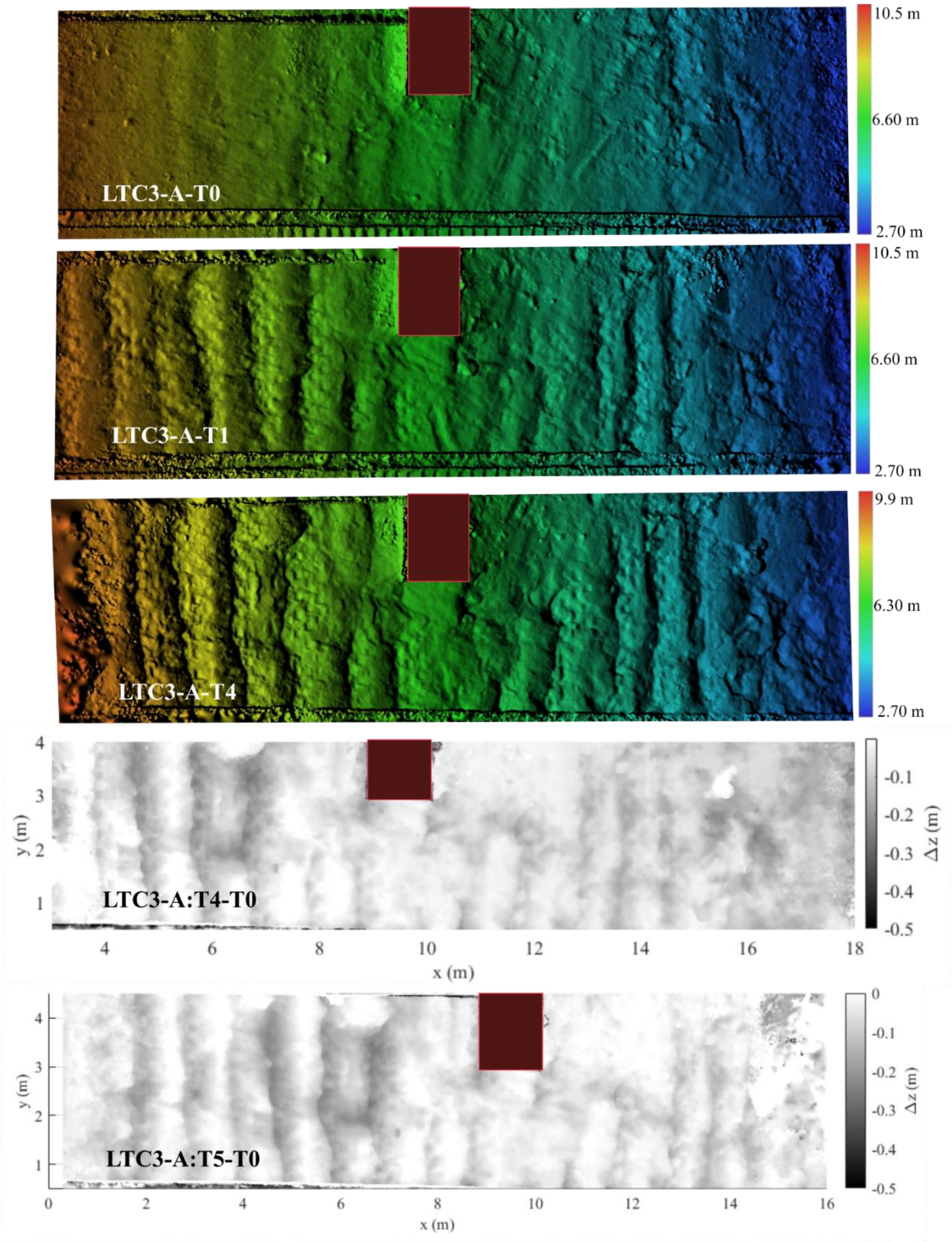

Figure 7-6 Results in test section 3A.

In Figure 7-6 (as is the case in the other test sections) a staircase profile forms, but in general the amount of erosion is less. This probably has something to do with the better quality of the clay used in this section. As will be discussed in paragraph 8.2 this has little to do with erosion of the lime treated material, but is a result of erosion in not treated material.

7.4 Information from cross sections

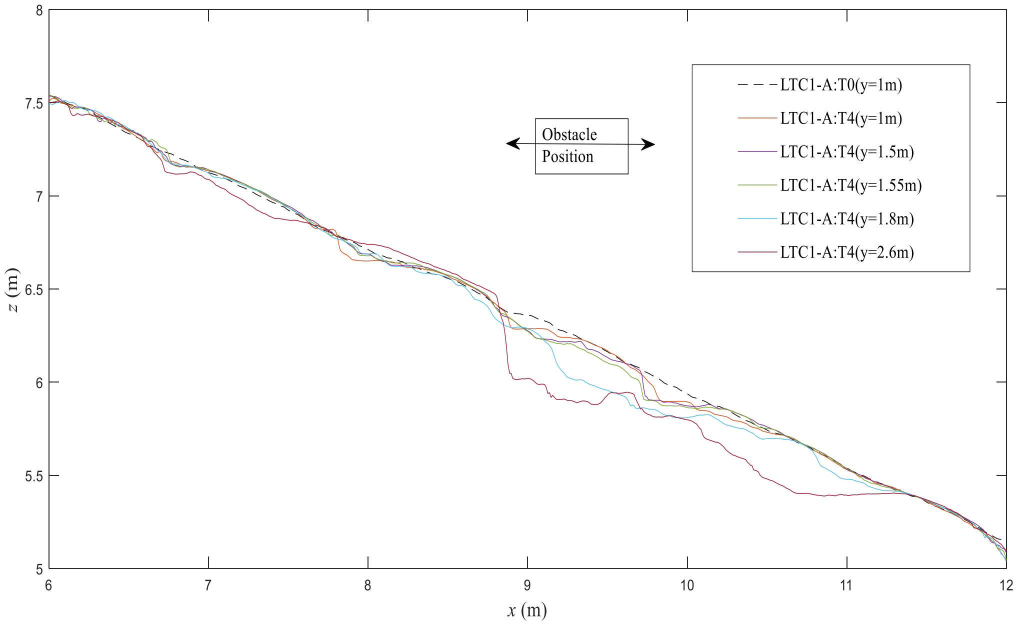

Figure 7-7: Final bed level profiles at different transversal distances (y) for LTC1.

In Figure 7-7 there is a considerable erosion in one trajectory (y=2,6 m), which is directly next to the obstacle. Also downstream from the obstacle erosion is visible. That results from the fact that the erosion gully next to the obstacle also attracts the most water.

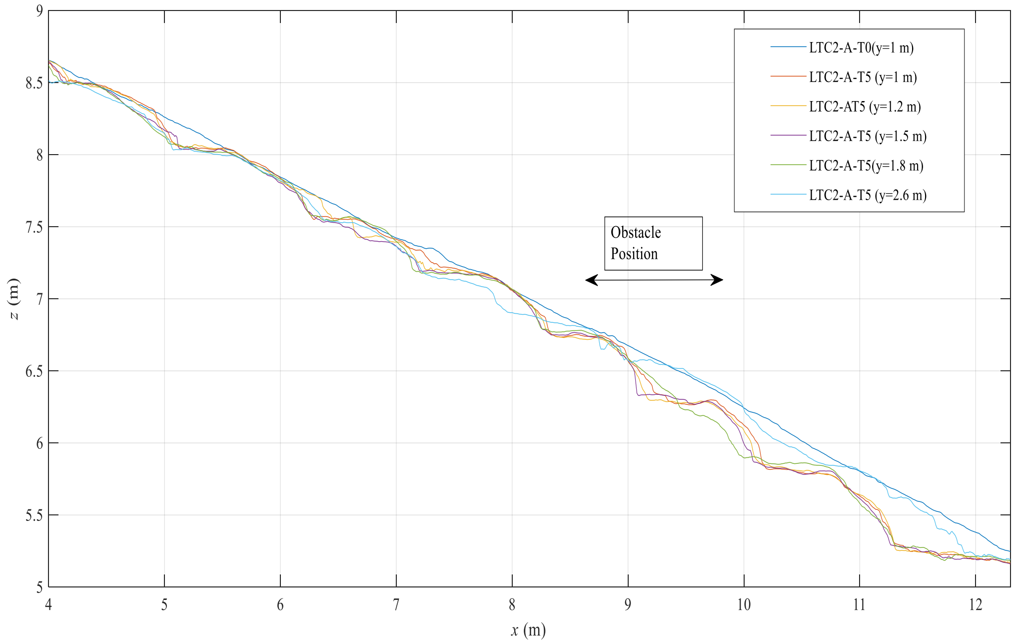

Figure 7-8 Final bed level profiles at different locations (y) along LTC2 slope.

In Figure 7-8 the final bed profiles are shown at different locations within test section 2A. In the area marked as "Obstacle Position" there is a substantial amount of erosion, but as y = 2,6 m is the position with the obstacle it is clear that there is little erosion near the obstacle and more erosion further away from the obstacle. This is also clear from the Figure below.

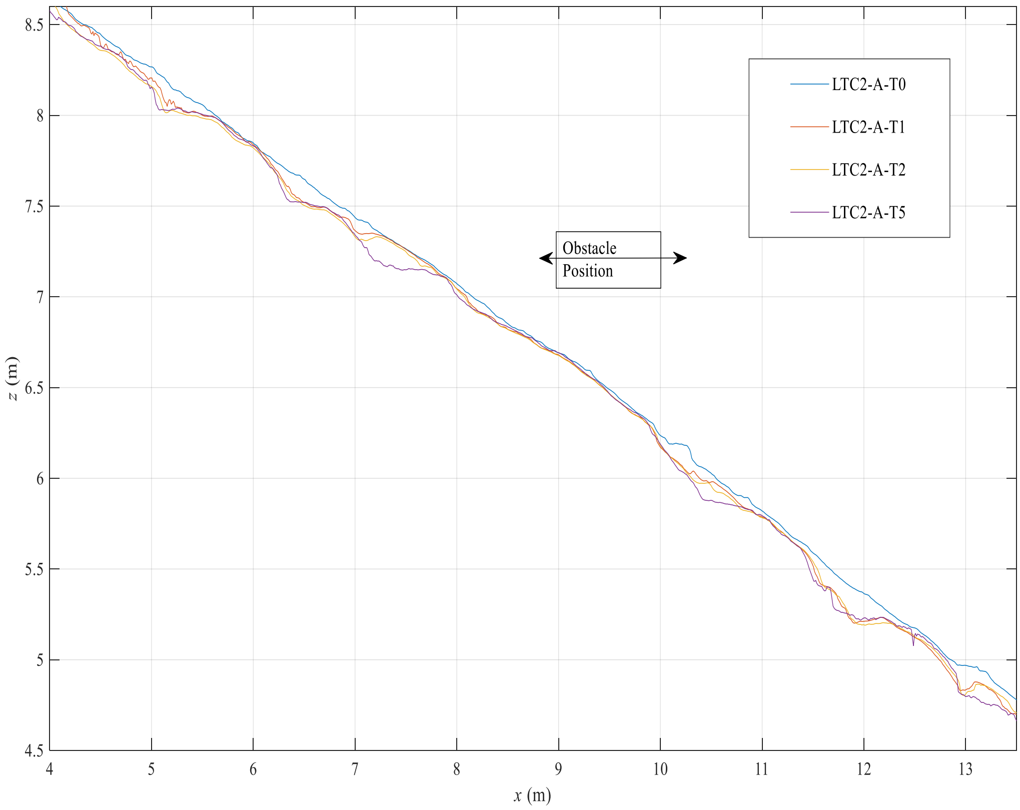

Figure 7-9: Temporal bed level evolution at y=2.3 m for LTC2-A.

Figure 7-9 shows the erosion in one cross section during time. This illustrates the development of the staircase profile but also shows that the total amount of erosion does not increase severely with increasing hydraulic load. Around the obstacle in Section 2A there is no erosion.

8 Interpretation of test results

8.1 Introduction

The results from the overtopping tests seem very promising. The well compacted lime treated clay has shown little or none erosion and has nowhere in the experiments come near to failure. There are however some observations which need further axplanation to better understand the behaviour of the lime treated cover layer, which in turn can help to enhance the application of the technique.

These observations are:
- Why does the staircase profile develop in all tests? The phenomenom that directly after the start of the tests parts of the slope erode almost instantly, while other parts do not seem to erode at all even at very high overtopping discharges was observed in all tests. This is further investigated in paragraph 8.2.
- Why did extra erosion around the obstacle take place in section 1A (as opposed to no erosion observed in section 2A and section 3A); see paragraph 8.3.
- Why did extra erosion take place in section 1A near the crest of the slope; see paragraph 8.4.
- Why did extra erosion take place on the lower part of the slope in section 2; see paragraph 8.5.
- Is there an influence of the quality of the original clay, see paragraph 8.6?
- What does the comparison between a grass cover, a clay cover and a lime treated clay cover look like; see paragraph 8.7.
- To what extent can the measurements and quality control predict the observed erosion (and thus be used to guarantee an erosion resistant cover layer); paragraph 8.8.

8.2 Development of staircase profile

In all tests it was observed that at the start of the overtopping tests part of the slope was eroded very easily. Also, after the initial erosion there was a substantial amount of leakage of water that flowed sideways under the wooden boards that were meant to keep the water in the test section. The leakage water was seen to erode the sections next to the test section. So part of the slope was eroded easily. After that the situation consolidated and further erosion was very limited.

From observations made in the Vidourle experiment (lit. 14) it is known that the first zero to 5 centimeters near the surface erode more easily due to weather impact and recarbonation of lime with air and water. The lime that recarbonates is not available anymore for the desired pozzolanic reaction which forms stable connections within the clay. However, as is shown in this paragraph, in the Hedwigepolder experiments the development of a staircase profile was not limited to a few centimeters depth.

The erosion pattern was coherent with the way the test sections were constructed. The test sections were constructed by first stripping the grass cover from the slope. Next, a staircased profile was dug, consisting of 17 steps of 40 cm high each. On each step of the stair two layers of 20 cm thick (after compaction) were laid. After construction the slope was profiled under a slope of approximately 1: 2.5. This is the starting point (see photo 8.1).

Figure 8-1 Situation in Section 1A before the start of the overtopping tests.

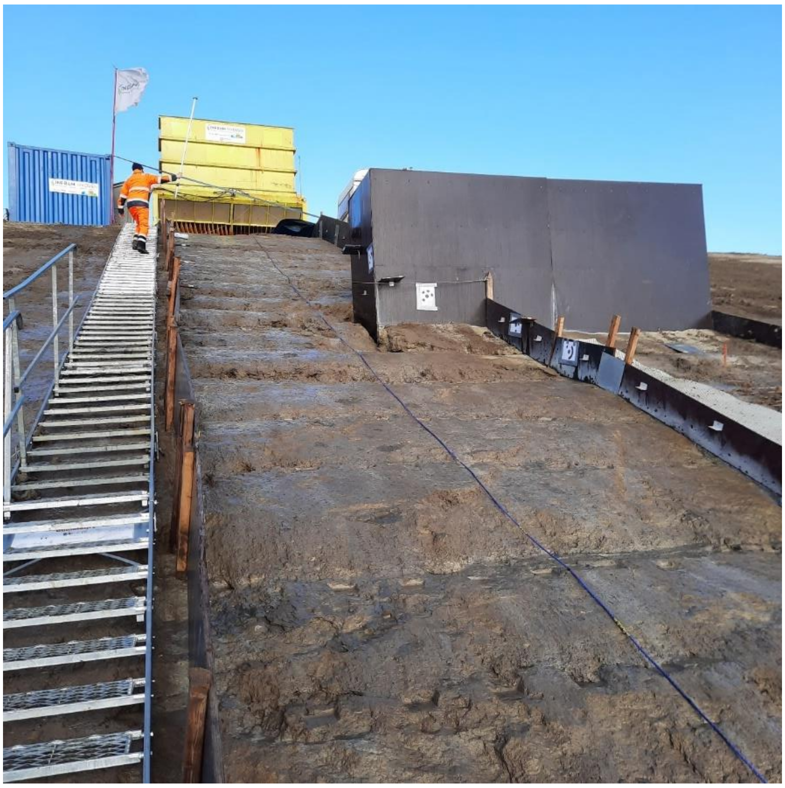

Figure 8-2 Situation in Section 1A after the test with 190 l/s/m'.

After the tests (see Figure 8-2) a new staircase profile has been formed, again with 17 steps. The top of each step is formed by the top of the compaction layer. The footprint of the compacting machine can distinctly be seen in each step (see Figure 8-3). The soil above this level has been eroded, but the soil beneath this level is not eroded even at the highest hydraulic loads.

Figure 8-3 Footprint of the compacting machine.

The observed behaviour means that the constructed cover layer was not homogenous. In Figure 8.4 the situation as observed on site is shown.

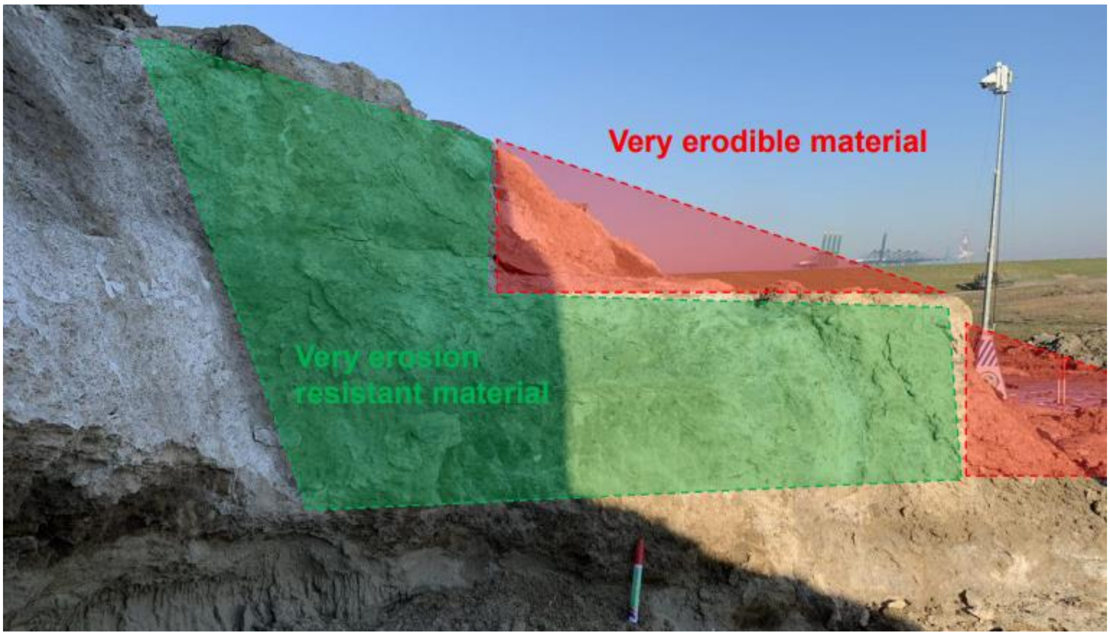

Figure 8-4 The division between very erosion resistant material and very erodible material as observed on site (courtesy Lhoist).

There are three possible explanations for the observed behaviour:

- The layers have been densified in layers of 20 thick (after compaction). One possibility is that there is a difference in compaction between top (very dense) and bottom (less dense) of each layer. Lhoist has performed PANDA tests (a measurement method of the density of the material by driving a rod into the slope and measuring of the required energy to do so) after construction of the test site and has repeated this during the overtopping tests 3 months later. From these tests a markedly strengthening of the material with time was visible, but in neither of the measurement rounds a stratification of the density over depth could be derived.
- After construction the slope has been reshaped into the desired slope of approximately 1: 2,5. The heavy compaction equipment can not operate on the edge of the slope because of the risk of the apparatus sliding down the slope. That means that a substantial part of the constructed slope has to be removed to ensure that all of the material of the slope consists of properly densified material. If the cutting line has been on the wrong spot that results in part of the slope consisting of not compacted material. This is shown in figure 8.5.
- The conditions for curing of the lime treated clay were not optimal. The temperature was low. It is a known fact that curing near the surface (contact with air) does not happen (or takes more time). So only the very well compacted top layer of each level of compaction has gained sufficient strength, but the lesser compacted levels did not reach the same level of strength near the surface. It is still possible that in time and with increasing depth this strength does develop. As the limetreated clay was tested after only two months of curing under unfavourable circumstances, and tested with severe loads and still did not fail this gives evidence of the potential for this construction method.

In the Vlassenbroekpolder a different construction method was applied (Ref. 14). The lime treated cover layer was placed in layers parallel with the slope. While this may solve some of the issues mentioned this also has some other issues, such as water pressure buildup near the toe of the slope. The pros and cons of the two construction methods should carefully be considered.

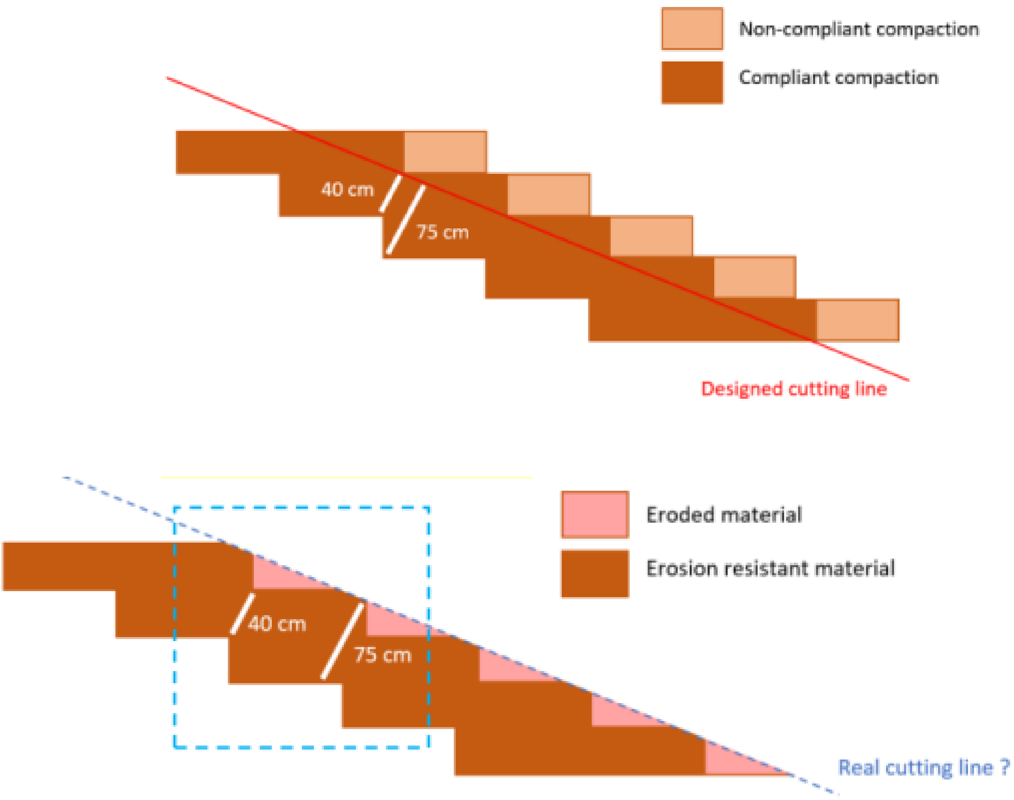

Figure 8-5 The difference between correct cutting line (above) and incorrect cutting line (below) (Courtesy Lhoist).

Lhoist returned on the site in June 2022 during the deconstruction of the test sections to investigate the thickness of the compacted layers. It was possible to clearly distinguish the treated clay. The dimensions appeared to coincide with the original design. Also the fact that the contractor used GPS to construct the final dimensions of the slope seems to point in this direction.

It is the most plausible explanation that a) the very well densified material (directly below the compaction machine) does not erode, but b) material that is densified to a lesser degree and that is close to the surface does not cure well and remains erodible for a longer period of time. This division in erodible and none to less erodible material follows more or less the picture in figure 8.4. When constructing a cover layer it should be taken care of that there is always a very well compacted layer present or a sufficient layer thickness (see figure 8.5).

In conclusion: a learning point from this experience and the observed behaviour would be that a further research into the development of strength as a function of compaction, depth and time could further enhance our knowledge of the curing process and the resulting erosion resistance.

It remains a fact that even under unfavourable conditions for curing a very erosion resistant layer has been formed in all three test sections. If the fact is that the layers on the slope have been densified to a thickness of 20 cm than it seems inevitable that the conclusion is that the cutting line has not been correctly executed. The observed erosion (eroded material in Figure 8-5) corresponds to the lower part of the Figure.

8.3 Erosion around obstacle

In Figures 8.6, 8.7, 8.8 and 8.9 an impression of erosion around the objects is given.

Figure 8-6 Before testing.

Figure 8-7 After the first tests: a maximum erosion depth of about 20 cm next to the obstacle.

Figure 8-8 Shortly before the construction failed; the erosion has deepened to a maximum of (locally) 50 centimetres.

Figure 8-9 Shortly before the construction failed; the erosion has deepened to a maximum of (locally) 50 centimetres (Please observe that the layer thickness on the upward stream side of the object is more than 40 cm of thickness, which was the minimum thickness of the cover layer).

In communication with Lhoist it has been discussed what could be the cause for the fact that erosion was observed in test Section 1A around the object, while in other test section (Section 2A and 3A were the tests with objects) this was not the case. In test Sections 2A and 3A it appears that the soil around the object eroded less than the regular slope.

From the Lhoist report on the construction of the dike shell with lime treated clay there is not much reference. The results of densification have been checked especially around the object in Section 1 and in one point in section 2. There appears to be no decisive difference. The compaction was even slightly better in Section 1. The measurements of density used here are apparently not a safe way to predict erosion resistance. There are three remarks to be made on explaining the observed behaviour:

- The water content of the treated soil in section 1 was high. This can be a factor when compacting.
- The object in section 1 was the first object to be placed and there was no previous experience in densifying around the object. While compacting it was experienced that the slope with lime treated material was very slippery when wet. This made operating difficult, both for personnel and for equipment. Supervision was not as strict as it could have been, for instance in regard to the number of passes with the compaction rammer. It was difficult operating under the given circumstances. The experience gained in Section 1 was used to have better results and stricter supervision in sections 2 and 3. The eroded spot in section1A was a weak spot, likely caused by improper compaction due to difficult working conditions.

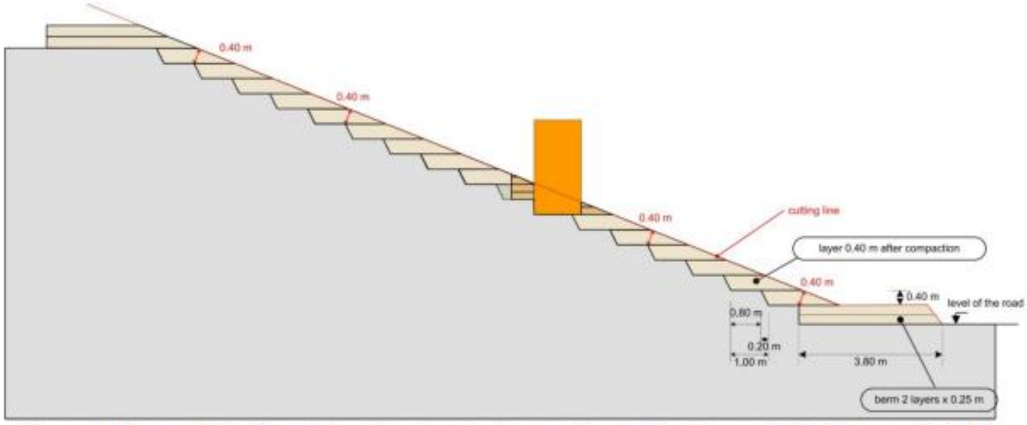

Diagram 107: layout of the object in the plot comprising "Poor quality clay A" soil treated with 4% Proviacal ®DD lime (cross-sectional view)

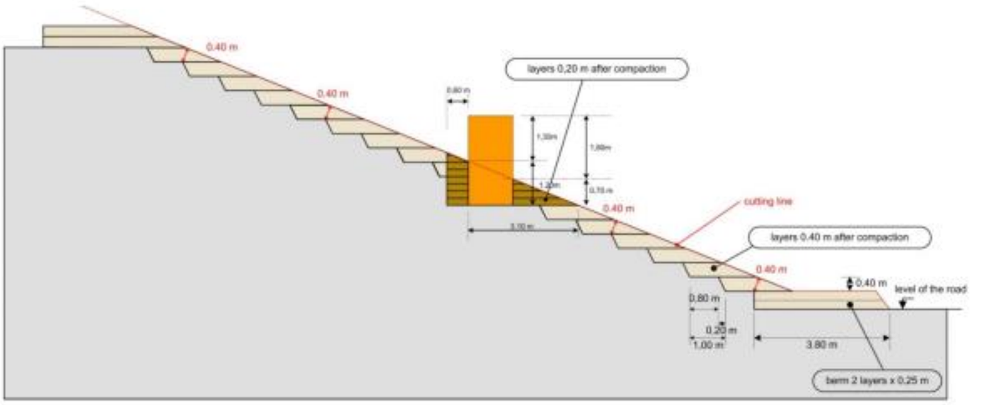

Diagram 108: layout of the objects in plots comprising "Poor quality clay B" soil treated with 4% Proviacal ®DD lime and in "Good quality clay" soil treated with 5% Proviacal ®DD lime (cross-sectional view)

Figure 8-10 Difference in depth of placing the objects (figures courtesy of Lhoist) The upper part of the picture represents test section1 and the lower part test sections 2 and 3.

- It was concluded that the object in section 1 was less deep into the slope. The object in section 2 and 3 was dug in 20 cm deeper, which gave more room for densification. It is also possible however, that the object in section 1 could move a bit more under waves and current forces than the objects in sections 2 and 3 as a consequence of the shallower foundation. As the erosion (and the heaviest attack from flowing water) is concentrated around the corner of the upper part of the object movement of the object itself can be an extra reason for erosion.

8.4 Erosion near the crest of the slope in Section 1A

Near the crest in section 1A it was observed that during the overtopping tests there was a slow but ongoing erosion visible. This was more prominent during the larger overtopping discharges. This was remarked as special, as the rest of the slope did not show this behaviour.

Figure 8-11 Erosion near the crest in section 1A.

According to the Lhoist report Construction of a dike shell in soils treated with Proviacal DD Lime this is explainable. A literal quote from this report reads: "*Depositing the second layer of the crest comprising "Poor quality clay A" soil plot treated with 4% Proviacal ®DD lime was* *completed on 12/06/2021 with soil from shell trimmings and soil stockpiles which had been left on the crest of the dike for a week and exposed to bad weather. On this date, the two vibrating compactors with padfoots were no longer located on the site, compaction had to be carried out using a small BOMAG BMP8500 vibrating compactor with padfoots, the only available machine. Due to the use of this small compactor, the thickness of the layer was reduced to 0.20 m following compaction. With this class PV2 compactor, according to the NF P 98-736 standard, 4 passes in dynamic mode were completed over the entire surface of the layer. The treated soil's water content was measured by taking 4 samples. The average value was 35.7% or WOPN + 10.7 % (or W/WOPN = 1.4), which led to the classification of this treated soil as "A2th" (very damp water class). According to the Technical Guide "Trench backfilling and roadway rehabilitation" [9], these water content conditions made compaction impossible*."

The observed erosion can be explained by the fact that no proper compaction has been obtained.

8.5 Erosion near the berm and the lower part of the slope in Section 2

The construction of the berm and the lower part in Section 2 (and also on part of section 3, although less distinctive) experienced a negative influence from groundwater flowing out of the slope. This has been described in detail in the Lhoist report on the construction of the test sections. Proper compaction proved to be very difficult due to a high water content in the subsoil, which made dynamic densification impossible, and even operating the heavy compaction equipment was troublesome. The problem was not that the lime treated material or the compaction equipment was different than elsewhere on the test location, but the subsoil was not suitable to work with. This was also the section that was constructed first, so there was no prior experience on this specific test location.

During the overtopping tests it was observed that more erosion than in other test sections was found in Section 2B and to a lesser degree in sections 2A and 3A.

Figure 8-12 Erosion in the lower part of Section 2B.

Figure 8-13 Erosion on the berm in section 2B.

In the photographs above it can be seen that in this section (section 2B) there is more erosion below the object. The wooden panels (sideboards) that were supported with wooden sticks that were driven into the slope were washed away. Due to the extra erosion the ground around the sticks was washed away and the panels were not stable any longer.

The first steps of the staircased profile consist of material which has not been properly densified during construction. Approximately from the third or fourth step this is better, which can be seen that the densified platforms do not erode. Below this point there is more erosion.

It can be learned from this experience that it is not advisable to construct the lime treated cover layer on a very soft subsoil or saturated subsoil. The soft subsoil should be removed first and replaced with steadier material before constructing and compacting the cover layer.

8.6 Influence of quality of original clay

Based on the findings of the three test sections (poor quality clay A in section 1, poor quality clay B in section 2 and good quality clay in section 3) it can not be concluded that the original clay has a large influence on the results of the testing. All three sections were tested with the same sequence of wave overtopping events (H_{m0} = 1m, overtopping discharges 10 l/s/m', 30 l/s/m', 50 l/s/m' and 190 l/s/m' and H_s = 2 m and overtopping discharge 50 l/s/m') with comparable results.

The total amount of material eroded from the slope does show a difference between the good quality clay and the lesser quality clay. The good quality clay shows less erosion. In view of the discussion in paragraph 8.2 it is likely that this is erosion of clay, and not erosion of the lime treated clays.

One of the objectives of the tests was to investigate whether or not a lesser quality clay can be used with a small addition of lime. The tests show that this is possible. There will be tests performed with an Erosion Function Apparatus (EFA) in a laboratory in Paris on both the good quality clay and the lesser quality clay mixed with lime. Due to the required long hardening period of 90 days the results are not included in this report (yet).

8.7 Comparison between lime treated clay cover, non treated clay and grass cover

There have been no tests on bare clay (non treated clay cover) and/or grass covers as part of the investigation of the erosion resistance of lime treated clay in the Hedwigepolder site.

The resistance of clay covers and grass covers have been the subject of other tests in the Polders2C's project. The lime treated cover convincingly outperforms the other two. In a section next to the lime treated test site tests were performed on sections that were stripped of the grass covers. At best the subsoil was bare clay, but it is more likely that the sandy core was soon reached by erosion, because the tests resulted in massive erosion holes.

A good grass cover can withstand a reasonable amount of wave overtopping. With a grass cover the resistance against damage of the grass cover is decreased by repeated loadings (cumulative overload approach). Even if a test with the overtopping simulator shows little erosion at higher overtopping rates (for instance 80 l/s/m'), the uncertainty about weak spots leads to a substantial lower design load (for instance 10 l/s/m').

Freshly constructed and well compacted clay covers can also withstand a certain (unknown) amount of wave overtopping. It is known however that clay develops soil structure over time. Dry/ wet conditions, frost/ thaw cycles and bacterial and animal activity form structures in clay that erode more easily. Clay that has been present on a dike for a long time show that lumps of centimeter size can be eroded.

The difference between well compacted lime treated clay and not compacted lime treated clay is shown in the following picture. It can be seen that lime treated material that has not been compacted shows considerable erosion, even though the hydraulic load is only from leakage water. Well compacted lime treated clay does not seem to erode even at the highest load the wave overtopping simulator can generate.

Cementation reactions strengthen the compacted soil and lime matrix. If the soil is not compacted too many voids will remain and the cementation will not be enough to bring extra resistance to the structure.

Figure 8-14 The difference between compacted treated clay and non compacted treated clay: the large erosion gaps are on the boundary between treated and densified material and material from the transition zone where no compaction took place. The erosion gullies are not the result of very large amounts of water flowing down the slope,but result from leakage water underneath the boarding of the test section.

8.8 Can the quality control determine the erosion resistance?

In chapter some light is shed upon the explanation of extra erosion that was observed during the wave overtopping tests. To find an explanation afterwards is a different thing from predicting a lesser performance on the basis of measurements on the constructed material or other forms of quality control. At least some remarks can be made on the basis of what we know and what we have seen during the construction and the experiments:

- The PANDA tests and the nuclear density measurements done shortly after construction do not necessarily reflect the erosion resistance as observed during the tests. For instance, the PANDA measurements in Section 1A next to the object were slightly better than that of section 1B , but the performance of section 1B was better. The nuclear density measurement is done from the surface and does not give information on layers that are better of less good compacted. The PANDA test gives more information on that, but the measurements do not show large differences in density over the depth. It does not seem plausible that weak spots can be discovered by these measurements shortly after construction.

- A comparison in PANDA measurements done shortly after construction and some three months later do show a distinct increase in strength. This can be used to see whether or not the curing effect has taken place or not and, if nothing else, gives some confidence that strength has built up.
- To find the weakest spot a very large number of these tests would have to be performed, or a different type of geophysical (non-destructive and quick) technique would have to be developed. Until then it is probably necessary (at least for the coming projects) to perform wave overtopping experiments with the wave overtopping simulator to get more experience and gain confidence in the innovative technique.
- The direct connection between quality of work and erosion resistance seems obvious. This gives rise to two observations:
  – Good pre-jobsite studies and strict and clean operations on the jobsite determine to a great extent the quality of the resulting construction. This is as important as quality control afterward.
  – If the construction is performed more often and on a larger scale it is probable that an experienced supervisor of the work will notice anomalies from the normal, and demand extra attention for these spots. Probably the best guarantee for high quality work is independent supervision.

9 Summary, conclusions and recommendations on follow up

9.1 Summary and conclusions

Since 2017 the topic of the use of lime treated clay in Dutch dike reinforcement has been an object of study. After literature research, also of international knowledge and experience, and laboratory research on the behavior of Dutch clays mixed with lime the next step was to scale up and perform experiments on a true scale. In the Hedwigepolder (made possible by the European Polder2C's project) three test sections were built. In each section an object was placed to simulate obstacles (e.g. housing) on the inner slope of a dike. Overtopping tests were performed with and without obstacles on the inner slope of the existing dike with a lime treated cover layer.

All the overtopping tests showed similar results. At the start of the overtopping tests a significant amount of erosion took place. However, this appeared to be only easy erodible material that was removed from the slope. After that the erosion slowed down. Part of the slope was eroded further but at a slow rate, and part of the slope did not erode at all as well compacted and erosion resistant layers were reached. In this way a staircased erosion profile was formed. The hydraulic load was increased from an overtopping discharge of 10 l/s/m' to 30 l/s/m' to 50 l/s/m' to a maximum of 190 l/s/m'. The cover layer of at least 40 cm thick did not fail at any point during the experiments.

At a few locations within the test sections there was extra erosion (around the object in section 1A, near the crest in section 1A and in the lower part and berm of section 2). After analysis and consultation with Lhoist (responsible for the construction of the test sections) it could be concluded that in all of these cases the compaction of the lime treated cover layer had been insufficient (due to water flowing out of the lower part of the slope (section 2), due to bad weather conditions (the crest in section 1) and insufficient compaction power (around the object).

Based on these findings it can be concluded that:

- Lime treated clay (if constructed and compacted properly) is very erosion resistant. Even with the maximum load the wave overtopping apparatus was able to produce little or no erosion was observed.
- This large erosion resistance was also observed around the obstacles, where concentration of the hydraulic load (water flowing down the slope) was heavier than elsewhere on the slope.
- With these findings the applicability of the technique to allow for higher overtopping discharges in dike design has been convincingly shown. Lower crest heights provide an excellent business case as this saves costs, needed space for dike reinforcement and can save houses and can help save bottlenecks where space is scarce.
- The quality of clay used to form the lime treated cover layer does not seem to have an influence; the lesser quality clay proved to be as erosion resistant as the good (EK1) type of clay. This means that locally available clay can be used instead of borrowing and transporting clay from elsewhere. That provides advantages in cost and sustainability.

The technique of mixing clay with lime to enhance the erosion resistance seems very promising. The recommendations following the overtopping tests is that it should be considered to make the next steps towards practical application in actual dike reinforcement projects.

Points to be considered are:
- Upscaling the technique from constructing a test section towards construction of substantial lengths; the quality of the outcome is strongly connected to the quality of the mixing and compaction process.
- Ways to control the quality of the outcome; at this stage it should be considered that overtopping tests are very reliable to check this, but it is not feasible to perform these tests at the required scale over the entire length of a dike reconstruction.

Two extra tests were performed giving qualitative information about the effect of initial damages in the lime treated clay layer:
- One test in section 2B. The lime treated cover layer was removed over an area of 2,5 m^2. The purpose was to observe the washing out of core material (sand). This test with an overtopping discharge of 30 l/s/m' lasted 1,5 hours. In this time a substantial amount of sand was washed out. The cover layer of lime treated clay was undermined, but did not fail. There is some strength left after the cover layer would have failed.
- One test in section 3A. A hole of 20 cm to 40 cm deep was dug, ending in a vertical cliff. This was intended to represent a transition construction in lime treated clay, with a cover layer that has failed (leaving this hole behind). This was tested during 1,5 hours and after that with 25 waves (full load). No erosion was observed.
- In the context of encroachments ('Niet-waterkerende objecten' in Dutch) this also indicates that even if the object itself fails there does not have to be progressive erosion around a hole in the slope. The same holds for transitions between cover layers on the slope. The extra test in section 3A is illustrative for this.

Both of these tests were 'observation tests'. The effect on residual strength after failure of the top layer and the suitability of lime treated clay in transition constructions were not quantified.

Finally, the working of lime treatment has not been investigated for other applications, such as:
- Prevention of damage of dikes from animal burrowing.
- Application on the outside slope of the dike, in the wave runup zone or even in the wave impact zone.
- Due to the low permeability of well compacted lime treated clay it can also have a role in preventing overtopping water on the inner slope. In current practice it is assumed that overtopping water infiltrates into the dike, thus having a negative effect on the sliding stability of the inner slope. This positive effect of a lime treated cover layer on macrostability has not yet been quantified. At the same time, an impermeable layer on the inner slope may be disadvantageous as well. It can be the cause of water pressure built up below this layer, and cause this layer to slide down. Both these aspects have to be taken into account.

Especially the effect on animal burrowing seems to have a large interest from several parties. These applications need to be further investigated, but seem promising.

9.2 Recommendations for follow up of the experiments

The experiments show great promise for the lime treatment technique. A next step will be to scale up the size of operations, meaning that a practical application in a prototype dike reinforcement seems logical. Scaling up reveals questions like:
- Can the process of construction be scaled up with conservation of quality?
- What issues are revealed by scaling up or by practical questions from waterboard and/or public?

Furthermore it should be decided whether or not the lime treatment technique is applicable for other uses. This requires further study.

As a follow up of this report a Handbook for the use of lime treatment in Dutch dike reinforcement is drafted. This Handbook is meant to give all the available information on lime treatment and it's uses in dike technology and is meant to enhance the practical use of it in hydraulic structures. It will address questions like the maximum wave overtopping can be allowed under which conditions, how the link with quality of the work and quality control can be made and maintenance issues that have not been discussed in the current report. Basically, the Handbook is drafted to facilitate the use in actual dike reinforcement projects of the promising innovative technique of mixing clay with lime.

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