ComCoast

Workpackage 3: Development of Alternative Overtopping-Resistant Sea Defences

Phase 3: Wave Overtopping Erosion Tests at Groningen Sea Dyke

Interreg North Sea Region

Workpackage 3: Development of Alternative Overtopping-Resistant Sea Defences

Phase 3: Wave Overtopping Erosion Tests at Groningen Sea Dyke

This report has been prepared by a consortium of Royal Haskoning and Infram

The ComCoast project is carried out in co-operation with ten partners:
- Rijkswaterstaat (NL - leading partner)
- Province of Zeeland (NL)
- Province of Groningen (NL)
- University of Oldenburg (D)
- Environmental Agency (UK)
- Ministry of the Flemish Community (B)
- Danish Coastal Authority (DK)
- Municipality of Hulst (NL)
- Waterboard Zeeuwse Eilanden (NL)
- Waterboard Zeeuws Vlaanderen (NL)

This report is an initiative of the ComCoast project, co-financed by the EU-Interreg IIIb North Sea Programme.

Workpackage 3: Development of Alternative Overtopping-Resistant Sea Defences

Phase 3:
Wave Overtopping Erosion Tests at Groningen Sea Dyke

Final Report

Acknowledgement:
This report is a deliverable of WP 3 " Smart Grass Reinforcement "
Part of the work was co-financed by the Ministry of Transport, Public Works and Water Management under the SBW-programme (Sterkte & Belasting Waterkeringen)
The work is performed by a consortium of Royal Haskoning and Infram
The report is written/edited by G.J. Akkerman, K.A.J. van Gerven, H.A. Schaap and J.W. van der Meer.

PREFACE

Mission Statement of ComCoast

MISSION OF COMCOAST (= COMbined functions in COASTal defence zones)
ComCoast is a European project which develops and demonstrates innovative solutions for flood protection in coastal areas.

ComCoast creates and applies new methodologies to evaluate multifunctional flood defence zones from an economical and social point of view. A more gradual transition from sea to land creates benefits for a wider coastal community and environment whilst offering economically and socially sound options. The aim of ComCoast is to explore the spatial potentials for coastal defence strategies for current and future sites in the North Sea Interreg IIIb region.

ComCoast Goals:
- developing innovative technical flood defence solutions to incorporate the environment and the people and to guarantee the required safety level;
- improving and applying stakeholder engagement strategies with emphasis on public participation;
- applying best practice multifunctional flood management solutions to the ComCoast pilot sites;
- sharing knowledge across the Interreg IIIb North Sea region.

ComCoast Solutions:
Depending on the regional demands, ComCoast develops tailor-made solutions:
- to cope with the future increase of wave overtopping of the embankments;
- to improve the wave breaking effect of the fore shore e.g. by using recharge schemes;
- to create salty wetland conditions with tidal exchange in the primary sea defence using culvert constructions or by realigning the coastal defence system;
- to cope with the increasing salt intrusion
- to influence policy, planning and people
- to gain public support of multifunctional zones.

ComCoast runs from April 1, 2004 to December 31, 2007. The European Union Community Initiative Programme Interreg IIIB North Sea Region and the project partners jointly finance the project costs of 5,8 million.

Information

Information on the ComCoast project can be obtained through the Project Management, located at the Rijkswaterstaat in the Netherlands.

Address
Rijkswaterstaat DWW
Postbus 5044
2600 GA Delft
The Netherlands

info@comcoast.org
www.comcoast.org

Project leader
Frans Hamer
Tel +31 15 251 8518

Project Communication
Marjolein Lippe, CUR
Tel +31 182 540 650

SUMMARY

Scope of the research

In the present report field erosion tests of the inner slope of a sea dyke in the province of Groningen (near Delfzijl) are described for the situation of severe wave overtopping. Three types of tests have been performed: tests at the present grass cover, tests at a reinforced grass cover and tests at a section of bare clay. At the reinforced grass cover section a provisional Smart Grass Reinforcement (SGR) system was installed in May 2006 (Royal Haskoning & Infram, 2005). The test sections were 4 m wide and extended over about 16 m along the inner slope, down from the dyke crest. It should be remarked that the erosion tests focused on the inner slope from the crest to some distance above the level of the service road. Below the service road, the berm slope was artificially strengthened with riprap as to avoid any damage on beforehand.

The tests have been carried out under the framework of ComCoast, Work Package 3 (WP3). Additional measurements have been commissioned by the SBW program (Sterkte & Belastingen Waterkeringen or in English: Strength & Loads Water Defences), a research program for improvement of knowledge of present defences. These measurements included: measurement of flow velocities and water depths, infiltration tests, determination of the soil shear strength and an additional erosion test at a bare clay section.

The present report is an overall report, presenting a full overview of all test activities, as far as conducted under the responsibility of the consortium. At a later stage, the SBW measurements will be analysed further within the scope of the SBW program. Where applicable, however, some preliminary results are included in this report. For further information, the present report addresses background reports that were written within the framework of ComCoast or SBW. Special reference is made here to the report on the wave overtopping simulator (Infram & Royal Haskoning, 2007), a summary of which has been presented in the present report in Chapter 4.

Parties involved

The test activities have been prepared, organised, co-ordinated and executed under the responsibility of the consortium of Royal Haskoning and Infram, from which Royal Haskoning was the leading party. Many other individuals and parties, both governmental and market parties, contributed to the tests as well, especially to mention:

- Ministry of Transport, Public Works and Water Management and CUR as contracting partners;
- The Water Board Hunze and Aa's as the principal supporting partner;
- Province of Groningen and Municipality of Delfzijl;
- Groningen Seaports was involved in providing permission to access their terrain near the site (parking and information cabin).
- Co-operating partners (as subcontractors within the consortium): Flevo Green Support for the installation of the SGR, assisted by Queens Grass; Huesker Synthetic for co-operative engineering and delivery of the selected Geogrid (free of charge).
- Specialized institutions that contributed to the tests: GeoDelft, Delft Hydraulics and Alterra, for specialized inputs in geotechnical research, measurements and grass research respectively.
- Contractor's for the measuring cabin and information cabin (Bussman BV), the water circulation system and accessories (Buitenkamp BV), professional video-

equipment (Provision BV), scaffolding (BIS Industrial Services) and security services from the Security Guard of Corus.
- A student of the Technical College Leeuwarden (Kathinka Schaap) and a student from the Van Hall Larenstein Institute at Leeuwarden (Ronald Rense), who assisted in the execution of the tests. In addition, a student from the Delft University of Technology (Gijs Bosman) assisted during the measurements and provided information to visitors at the test site. In addition, in his thesis, Gijs Bosman analysed previous research and developed new wave overtopping formulae.

Contents of this report, authors and acknowledgement
After a review of the test set up (Chapter 2), the report deals with a brief description of the Smart Grass Revetment (SGR), which has been described more extensively in the consortium report (Royal Haskoning & Infram, 2006). A brief description of the wave overtopping simulator has been presented in Chapter 4. This Chapters refers to the detailed report on the wave overtopping simulator (Infram & Royal Haskoning, 2007). In Chapter 5 the in-situ grass cover, substrate and clay determination is presented, from which an impression on the strength of the grass/clay cover at the test site was obtained. The infiltration tests that have been carried out prior to the erosion tests, have been summarized in Chapter 6. Chapter 7 summarizes and discusses the velocity and flow depth measurements of the wave overtopping tongue and the subsequent analysis of earlier research and new overtopping formulae by Gijs Bosman. This information has been treated in-depth in the report on the wave overtopping simulator (Infram & Royal Haskoning, 2007). Subsequently, the methodology of the erosion measurements is briefly dealt with in Chapter 8. In Chapter 9 the actual erosion results are described for the unreinforced test section, the results for the reinforced test section in Chapter 10 and the results for the bare clay section in Chapter 11. A preliminary analysis of the results , placed against the background of other investigations and experience and prediction models, is given in Chapter 12, where after communication and publicity issues are briefly addressed in Chapter 13. Finally, conclusions and recommendations are given in Chapter 14.

The main author of this report is Gert Jan Akkerman, assisted by Koen van Gerven and Kathinka Schaap. Jentsje van der Meer contributed to Chapters 4 and 7 (being summaries of the content of the separate report written by him ((Infram & Royal Haskoning, 2007)), as well as to Section 5.3. In addition, we gratefully acknowledge Ronald van Etten for his contributions to Chapter 5 and 6, as well as the supplements and comments of the Project Team of the Ministry of Transport, Public Works and Water Management and CUR, the Expert Team and the Feedback Group. Moreover, the remarks of Jan Willem Seijffert are highly appreciated.

Special thanks are due to the facilitator of CUR, Joop Koenis, and to the project leader of ComCoast on behalf of the Ministry of Transport, Public Works and Water Management: Patrizia Bernardini and the project leader of SBW on behalf of the same Ministry: Gijs Hoffmans. There support was of great value for the project, which is highly appreciated by the consortium.

Results
It should be noted that the maximum overtopping rate was over 30 l/s/m (time-averaged overtopping rate): in the tests ‘50’ l/s/m has been mentioned, but this overtopping rate may

not be fully representative. Hence, where applicable, the highest overtopping rate has been indicated as ’50‘ l/s/m.

The tests described in this report are unique in its kind, as never before the stability of the inner slope of a real dyke has been tested at true 1:1 scale for wave overtopping. Both the unreinforced and the reinforced grass slope have been exposed to extremely severe wave overtopping, up to multiple storms of 30 l/s/m and more. As far as could be verified, the overtopping wave tongues have been reproduced successfully by the wave overtopping simulator. It should be noted here, that the measurements of velocities and water depths were not good enough as to verify this quantitatively; visual observations, e.g. of the front velocity of the wave tongues, however add to this qualitative impression of a good performance of the wave overtopping simulator.

The main conclusion is that the present grass cover, in spite of the ‘poor’ overall score according to the VTV (= safety assessment code in the Netherlands), proved to be able to withstand the full series of storms up to ’50’ l/s/m. The same applies to the reinforced grass cover (SGR), in spite of the rather poor grass coverage and a similar ‘poor’ overall score according to the VTV for the grass and subsoil. It was only after introducing artificial damage that ongoing erosion could be observed. At the natural grass cover, grass sods downstream of the bare spots gradually disappeared and distinct gullies were formed that progressed rather quickly downstream the slope, see Figure 9-16. With the SGR progressing erosion was much less: limited removal of grass could be observed downstream of the largest bare spot, but gully formation in the subsoil did not occur. Downstream of a bare spot of 0.4 m * 0.4 m, the SGR could even prevent any further progressing erosion.
This showed that the anticipated strengthening effect of the SGR, see Section 3.1, proved to function very well.

The protection of the clay layer at bare spots by the SGR is in strong contrast to the relative high erosion sensitivity of a bare clay layer without the SGR, as could be observed at the bare clay section. At this section heavy erosion occurred at an overtopping rate of 10 l/s/m. This overtopping rate is considerable lower than applied at the grass section (’50’ l/s/m, for which no erosion occurred). The clay erosion showed a cliff type incision in the slope, that progressed in time towards the crest of the dyke. This type of erosion in bare clay slopes could be anticipated and is indicated in literature as ‘head cut erosion’. In spite of the much stronger erosion sensitivity of the bare clay slope as compared to the grass slope, the absolute overtopping rate was 10 fold of what is usually applied at grass slopes, i.e. 10 l/s/m in stead of 1 l/s/m. This means that the clay layer at the Delfzijl dyke still has a considerable residual strength. However, it should be noted here that this dyke section had no sand core, so erosion took place in massive clay only.

Infiltration measurements showed a rather quick pressure build-up in the clay layer up to 1.2 m below the surface and a relatively slow decay. An explanation may be found from observation of the clay during the bare clay tests: the clay core was crisscrossed by numerous worm holes and small fissures, which caused the clay to be rather permeable, in spite of the rather good clay quality. This could also be observed during the start of the tests, during which the first series of overtopping waves did not even reach the toe of the slope.

Unfortunately the instruments that recorded velocity and flow depth of the wave overtopping tongues did not work properly under the simulated conditions. It appeared that it is very difficult to carry out proper measurements in the extremely turbulent and aerated overtopping flow. Hence, the EMS and thin wire gauge instruments that have been deployed need further improvement in future, or alternative instruments should be looked for. The measured front velocities between two instruments gave a good agreement with anticipated velocities. Visual analysis of the video recordings sustained this agreement further. Hence, in spite of the poor measurements, the conclusion is justified that the simulator performed well and was in agreement with the expectations. Analysis by Gijs Bosman of earlier research on wave overtopping flow velocities and flow depths (Bosman, 2007) resulted in new prediction formulae. These formulae can be used to set-up the wave overtopping simulator more accurately for further testing in future.

Evaluation and recommendations

A basic conclusion is that the natural grass cover proved to be so strong that limit-state loading could not be attained. This is remarkable as regards the moderate grass coverage rate and the poor overall score with the VTV on the grass/substrate. Hence it may be concluded that the presence of grass sods is of the utmost importance in preventing the erosion of the clay. This presence of grass sods is even sufficient when small bare spots are present, e.g. of 1 decimetre. This conclusion followed from the tests with artificial initial damage, as was as by the rather poor grass coverage prior to the tests. However, the tests with initial damage also showed that larger bare spots, e.g. 1m * 1m, may lead to progressing erosion; downstream of these bare spots strong gully formation could be observed.
When the grass layer is completely absent, the stability of the bare clay substrate proved to be much less and strong erosion occurred in the clay at overtopping rates much less than at the grass covered slope. Nevertheless the (massive) clay layer still showed a considerable residual strength.

The outcome of the tests with the SGR are very encouraging: progressing erosion with the SGR was much less, which can be explained from the additional anchoring of the grass roots with the Geogrid and from the protective function of the Geogrid against erosion of the under laying substrate.
The situation of artificial damage may well match real situations at sea dykes, at which some erosion may occur anyhow (sheep, fences, burrowing animals). In that case the presence of the SGR may be decisive for the stability of the dyke.

As limit-state conditions could not be observed with the present wave overtopping simulator, questions remain about the quantitative limit-state behaviour of the SGR as compared to the unreinforced grass cover. From the observations, the SGR gives high hopes which seems to be justifiable when extrapolating the observations. Further verification is desirable and needed when applied at large scale.

As failure by surface erosion could not be reached with the present wave overtopping simulator, it is recommended to investigate the possibility to increase to the size of the wave overtopping simulator such that more than ‘50’ l/s/m can be produced for future tests.
In addition, it is recommended to develop instruments for adequate measurement of flow velocity and flow depth, that can cope well with the extremely turbulent and aerated flow.

A recommendation is to continue adequate limit-state testing for surface erosion, with and without a SGR, with a large wave overtopping simulator. Partly such investigation can be

carried out at present coastal dykes, partly at a special location that allows the dyke to be eroded to a large extent. At present, in the Netherlands such a continuation of tests is foreseen within the SBW program (Sterkte & Belasting Waterkeringen), This program envisages testing in the years to come at various dyke locations in the Netherlands as well as at a special location in Groningen within the 'Calibration Dyke Program' (in Dutch: 'IJkdijk').
It should be noted here that the better surface erosion failure can be coped with, other failure mechanisms may become more normative such as slip-failure and internal erosion (which could not be tested at this dyke). We think the SGR may give a major contribution to mitigating such failure mechanism as well, provided that the reinforcement is properly placed. We would advise to verify this with further tests as well.

Finally we would recommend to develop an improved SGR by further exploration of feasible (= economical) installation methods in coherence with feasible geosynthetics in the case that dykes are not to be reconstructed. A major challenge of this exploration is that the grass cover remains intact as much as possible, as to allow full recovery of the grass before the next storm season. For 'new work' during reconstruction, installation will be rather simple and straight-forward then. This makes a SGR a feasible means already for reduction of the reconstruction works, which will be highly economical and will increase the resiliency of the dyke.

CONTENTS

1 INTRODUCTION

1.1 Goal of the wave overtopping erosion tests

Wave overtopping erosion tests cannot be scaled down well in physical scale models. Hence, tests have to be carried out in a large wave tank or in a real situation. The latter is to be preferred, as the condition of the dyke remains undisturbed and the costs for constructing an ‘undisturbed’ dyke section in a wave tank are excessive. The present report deals with erosion tests at a real sea dyke in the province of Groningen of the Netherlands, These tests are unique in its kind, as never before such tests have been carried out thus far. The wave overtopping could be realized by a special device that was developed within the present research framework: the wave overtopping simulator.

The aim of ComCoast is to develop wide coastal zones, for which overtopping-resistant sea dykes may apply, see Section 1.2. A prerequisite for such dykes is to reinforce the crest and inner slope of the dyke to such an extent that severe wave overtopping can be allowed as an alternative to continuous increase of the crest level of the dyke. Prior to the actual erosion tests in 2007, a Smart Grass Reinforcement (SGR) had been developed by the consortium and applied to the dyke test section in 2006 (Royal Haskoning & Infram, 2006).
The major ComCoast-formulated goal of the tests is to check the performance of the SGR, as compared to the natural grassed slope.

In addition, the SBW program, see section 1.3, focuses on the actual strength of present defences, i.e. the unreinforced grassed slopes. Moreover, tests have been carried out at a bare clay section. The tests carried out for this program aim at a better understanding of the stability behaviour of grass covers and the clay substrate. Hence, additional measurements have been assigned: flow velocities, water depths, infiltration tests, and additional geotechnical and grass surveys.

The tests have been prepared and carried out under the guidance, supervision and responsibility of a consortium of two parties, Royal Haskoning and Infram, from which Royal Haskoning was the leading party. These tests were commissioned by CUR, on behalf of Ministry of Transport, Public Works and Water Management (ComCoast program and SBW program).

1.2 Background information on the ComCoast program

Wave overtopping is a phenomenon that is anticipated to increase strongly on a global scale in coastal areas due to sea level rise. As a response, crest levels of primary defences need to be raised at huge expenditure and (generally) with increased risks. As an alternative, dykes can be strengthened as to accommodate for increased wave overtopping. The European ComCoast project seeks for introducing wide coastal defence zones in stead of single-line defences alone (e.g. sea dykes) thus improving the safety of the defences.
Within Work Package 3 (WP3) of this project, research is being undertaken on the feasibility of overtopping-resistant dykes. Royal Haskoning and Infram developed the winning reinforcement concept. This concept exists of a spefically selected geosynthetic (Geogrid) that reinforces the crest and inner slope of the defences. Especially innovative is that it can be applied at existing dykes with minor disturbance of the grass cover

(Royal Haskoning & Infram, 2005). This concept is denoted 'Smart Grass Reinforcement' (SGR) and has been awarded to Royal Haskoning and Infram for testing at the sea dyke early 2007. Prior to this, the actual placement of the SGR at the sea dyke took place in May 2006 (Royal Haskoning & Infram, 2006). More background information can be found at the ComCoast website: www.comcoast.org, from which relevant reports can been downloaded free of charge.

- Wave overtopping and strength of inner slopes (SBW, 2007) (in Dutch)

The first report deals with the installation of the grass reinforcement (SGR). The second report with the overtopping erosion tests (present report). The third report deals with all issues related to the wave overtopping simulator. The fourth report deals with predictions of stability of grass and clay layers at dykes.

The necessity of performing real-scale dyke tests, is caused by the impossibility to scale down properly the grass and clay resistive properties. As an alternative, large scale flume investigations might be considered in which a 'real dyke' section is reproduced. The grass cover and clayey subsoil, however, will easily be disturbed and reproduction will be very costly. Moreover, real waves may simply be too large, i.e. well over 1.5 m, to be reproduced well in the wave flume.

1.3 Background information on the SBW program

While ComCoast focuses on strengthening the dykes as to provide overtopping-resistant sea dykes, the SBW Program ('Sterkte & Belastingen Waterkeringen') concentrates on improvement of reliable overtopping criteria for present sea dykes, i.e. without a reinforcement. This program is basically a research program, encompassing in-depth studies, analysis and measuring campaigns. The SBW Program will continue for some more years after now and aims at improving the VTV2011 (safety assessment code for dykes in 2011). The present tests at the sea dyke in Groningen have taken advantage from SBW, by granting to the consortium measurement of flow velocities and water depths and additional testing, such as the tests at the bare clay section, infiltration tests and additional geotechnical and grass surveys.

1.4 Background information on present wave overtopping standards

Present wave overtopping standards have been based on tests and experiences. Experience from the flooding disaster in the Netherlands in 1953 showed that deterioration of the inner slopes of the sea dykes in the province of Zeeland was one of the main causes of breaching of the dykes. Partly this may have been due to slip failure by over-saturation and partly by surface erosion by overtopping waves. Since then, dykes have been raised and inner slopes have been flattened from about 1:2 or steeper to 1:3.
Early research in the Netherlands used the 2% overtopping run-up value as the design value for overtopping. This design value was already mentioned in wave flume research at Delft Hydraulics before World War II.
After the flooding of 1953, the Delta Commission translated wave run-up and wave overtopping in a qualitative way, which was further elaborated in the 80's. For most of the prevailing conditions along the Dutch sea coast, the 2% wave run-up corresponds roughly to 1 l/s/m. For river dykes the 2% wave run-up corresponds to a much smaller overtopping discharge, e.g. 0.1 l/s/m. Hence the overtopping discharge has been taken as the primary criterion for all dykes in the Netherlands. Only in those cases that the

grass and clay is of good quality, 10 l/s/m was considered acceptable. Overtopping rates higher than 10 l/s/m were considered as conflicting with the water retaining function of the dyke (personal communication of Jan Willem Seijffert)
An European state-of-the art of wave overtopping is presented in the Assessment Manual of EurOtop, that is due this summer (www.overtopping-manual.com).

Reviewing the above, most of the dykes in the Netherlands are designed at an overtopping rate of 1 l/s/m or less. The present tests at the Groningen sea dyke focused on overtopping rates up to ‘50’ l/s/m. For this situation, the maximum overtopping volumes are approximately 3.5 m^3/m for each of the highest waves. Comparing this with the 1 l/s/m criterion, the wave overtopping is increased by more than a factor 30.

1.5 Communication and publicity

Adequate communication and publicity was considered an essential part of the present project. To enable this, communication officers of Ministry of Transport, Public Works and Water Management were involved in facilitating and organising communication by communication protocols, assistance to the ‘opening event’ of the dyke tests, information services and in streamlining publicity. The consortium contributed to the communication activities as well. Thanks are due a.o. to Hanneke Derksen of Ministry of Transport, Public Works and Water Management, being the leading communication officer. In Chapter 13 communication and publicity issues are addressed in more detail.

1.6 Assignment

The proposal for the tests have been issued 27th April 2006 (identification 9R9112.A0/L0006/GJA/SEP/Nijm), and awarded by CUR 5th May 2006 by letter with identification C136A_OB_06_15390, assignment number 3640.
To cover part of the unforeseen preparatory activities, a additional proposal was issued 26th October 2006 (identification 9R9112.B0/L0005/401070/SEP/Nijm), and awarded by CUR 2nd November 2006, assignment number 3653.
During the final preparations, the approximate estimate of the first contract proved to be insufficient for coverage of the costs of third parties, especially due to the external assistance of the Water Board. To cover these costs, and the costs of additional tests assignment was obtained by CUR by letter C136A_OB_07_21203 dated 7th May 2007, assignment number 3655.
Finally, SBW participated in this research by facilitating additional measurements and analyses to be carried out. Our proposal for this work, issued 2nd February 2007 (identification 9R9112.B0/L0010/401070/SEP/Nijm) was awarded by CUR by letter with identification C136A_OB_07_20685, assignment number 3656.

1.7 Contents of the report

After the introduction in Chapter 1, a review of the test set-up is given in Chapter 2: preparations, the test arrangement, the measuring protocol and an outline of the test-program. Chapter 3 gives a brief impression of the Smart Grass Reinforcement (SGR) installation in 2006. In Chapter 4 the wave overtopping simulator is briefly presented. Chapter 5 deals with the in-situ grass and subsoil determination: overall subsoil investigation (Fugro, GeoDelft), grass cover determination (Alterra), root intertwinement with the SGR and shear strength determination of the substrate (GeoDelft).

The methodology and results of the infiltration tests (GeoDelft) are dealt with in Chapter 6. The water depth and velocity measurements of the overtopping waves have been described in Chapter 7. In Chapter 8, the methodology of the erosion observations is briefly dealt with. The actual erosion observations are described in Chapter 9 (unreinforced test section), Chapter 10 (reinforced test section) and Chapter 11 (bare clay section). The stability results are preliminary analyzed against the background of earlier investigations and experience and prediction models in Chapter 12.
In Chapter 13 communication and publicity issues are briefly dealt with.
Conclusions and recommendations are presented in Chapter 14.

2 TEST SETUP

2.1 Preparations

Preparations for the tests took place at an early stage of the project: selection of the test site, initial assessment of the grass cover strength and preliminary soil investigation. Where relevant to this report, the major outcomes are summarized hereafter.

A major effort was the installation of the Smart Grass Reinforcement (SGR) at the sea dyke in May 2006, providing a primary testing section of 4 m wide and a backup section of 4 m wide (Royal Haskoning and Infram, 2006). Due to the provisional character of the installation, the SGR was placed at a practical distance below the grass cover surface, The installation is illustrated briefly in Chapter 3. Adjacent to the test sections, a small monitoring section had been installed as well.

During the preparations for the test setup, advanced insight added in optimising the setup, such as the desirability for adding a professional lighting installation to the test site and the placement of a measuring cabin on top of two containers, as to provide a high observation above the inner slope of the dyke.

Another important asset that required early design, was the development and calibration of the wave overtopping simulator, see Chapter 4. This work had been done from spring 2006 onwards. The first step was to design and develop a prototype version of 1 m wide (Infram & Royal Haskoning, 2007). After successful calibration, the 4 m wide wave overtopping simulator was constructed.

Related to the wave overtopping simulator, a water circulation system was designed. However, the initial design with a subdivided circulation system with an intermediate storage tank, was abandoned in favour of a heavy-duty fully controllable pump system. The electricity generator that was required for the advanced pump system also provided electricity to the auxiliary units.

At 20 December 2006, trial testing had been carried out at the sea dyke to check the performance of the wave overtopping simulator, the water circulation system and the methodology for observation of erosion. These trial tests delivered useful information to be used for the final preparations for the actual tests and lead to e.g. adaptation of the footing structure of the simulator, adaptation of the water circulation system and insight in the optimum location of the observation point above the inner slope.

The timing of the actual tests was chosen such that the tests would start at the end of February 2007 and would proceed throughout March 2007. This period was critical as regards the preceding winter time period at which the grass condition is at its weakest.

2.2 Test arrangement

The test arrangement is described in this section, apart from the wave overtopping simulator which is described in Chapter 4.
Items mentioned are: location, schematic set-up and test lay-out, water circulation system and measuring protocol and outline of the test program. The methodology for the erosion measurements is treated separately in Chapter 8.

The location near the industrial area of Delfzijl is shown in Figure 2-1. The harbour jetty in front of the sea dyke reduces the wave attack. Hence, the height of the sea dyke at the test location allows for some erosion (the crest is actually higher than according to standards). The test arrangement is schematically shown in Figure 2-2.

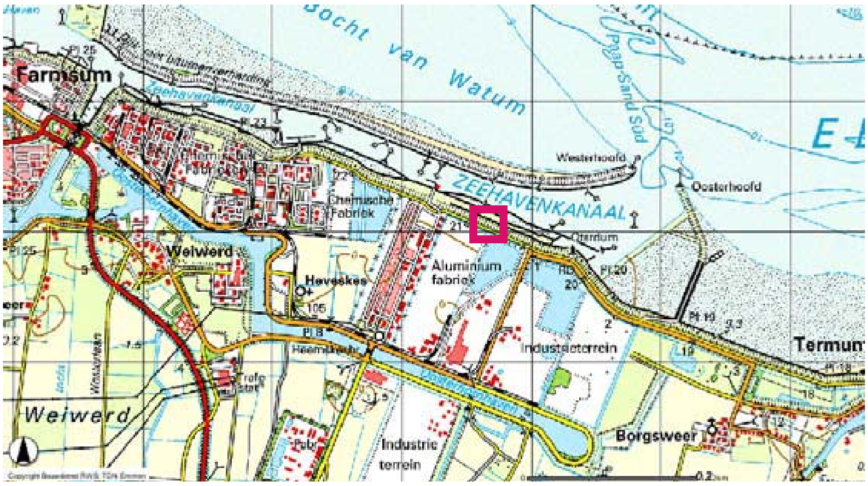


Figure 2-1: Location map of the test location

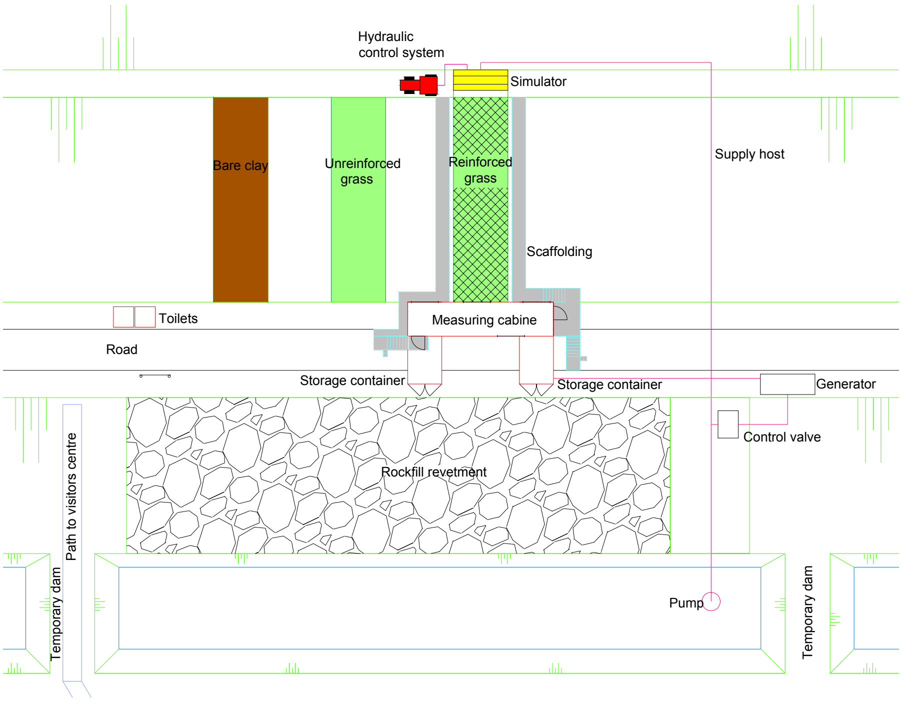


Figure 2-2: Schematic top view of the test arrangement

The reinforced back-up test section has not been indicated while this section was not used at all.

A photographic impression is shown in Figure 2-3.

Figure 2-3: Impression of the test set-up with the measuring cabin on top the two containers

The overtopping simulator was placed at the section to be tested and connected to a pumping system which derives its water from the toe ditch of the dyke. The measuring cabin was placed at the toe of the testing section on top of two containers that were placed on both sides of the section (as to provide a high observation point for the tests and at the same time allow the overtopping water to flow freely underneath the measuring cabin).
Side walls bordering the 4 m wide test sections were placed as well as a scaffolding type of staircases on both sides of the section.

First, tests had been carried out at the unreinforced grass section. After these tests, the whole test arrangement was moved to the reinforced grass section (grass cover with SGR). Finally, the whole test arrangement was moved to the bare clay section.

2.3 Water circulation system

Requirements for the water circulation system are directly related to the performance of the wave overtopping simulator. The requirements refer to the maximum capacity of the system and to the accuracy of measurement of the pumping discharges. As erosion did not occur at the grassed test section at the design maximum capacity (30 l/s/m) at a later stage a heavier pumping system was installed to provide for ‘50’ l/s/m.
Typical capacities required for the tests were:

0.1 l/s/m	1.44 m^3/hour;
1 l/s/m	14.4 m^3/hour;
10 l/s/m	144 m^3/hour;
20 l/s/m	288 m^3/hour;
30 l/s/m	432 m^3/hour;
‘50’ l//m	720 m^3/hour.

A first set-up of the system, rented from a specialized company: Buitenkamp, was tested during the trial tests in December 2006. The initially required pump capacity was 432 m^3/hour, sufficient for producing the 30 l/m/s overtopping at the required pumping height

of about 10 m. In general, the system proved to work well: the higher discharges could be adjusted effectively and stayed constant during testing. Hence, the concept with one pump proved to be feasible. The smaller discharges, however, could not be set accurately. For the final testing this regulation was improved, but the smallest discharge had still to be set roughly at the start of the tests and adjusted by hand, which was considered acceptable.
The heavier pumping system, that was arranged prior to the start of the ‘50’ l/s/m included a pump of 1000 m^3/hour and a heavier power generator. The frequency controller, however, collapsed after some time and was replaced. At a later stage also the pump engine failed and had to be replaced as well. For security reasons, however, the maximum pump capacity was set at 40 l/s/m (576 m^3/hour). As an additional measure, friction in the hose system was reduced by placing the out flowing hose directly into the box of the simulator in stead of connecting to the double supply pipe system of the simulator.
In order to reproduce the ‘50’ l/s/m condition, the storm duration was stretched from 6 to 7.5 hours. This implies that in principle the correct number of waves and volumes was reproduced, but with intermediate intervals 25 % longer, as the pump needed this extra time to fill up the simulator. Furthermore, the maximum capacity of the simulator was limited to the overtopping volumes of 3.5 m^3 per m.

An impression of the water circulation system is given in Figures 2-4 and 2-5.

Figure 2-4: Overall impression of pumping system

Figure 2-5: Impression of pumping system: control room (yellow box), power supply (red trailer) and hose system towards the simulator

2.4 Measuring protocol

A measuring protocol was drawn up for the preparations and execution of the tests. The measuring protocol deals with the test set-up, the organization of the tests, the facilities and the work in progress. Furthermore, the protocol describes the possible risks and corresponding mitigating measures. Special attention has been paid to quality assurance and safety procedures. A so called Question and Answer (Q & A) document was added to the protocol to deal with information to press and public.
A draft measuring protocol was reviewed by Delft Hydraulics and by GeoDelft and was finalized after incorporating the comments.

The protocol for execution of the tests describes the following subjects in more detail:

- time-management and test denotation
- recording procedure of the overall and detailed erosion by photo and film
- measuring procedure erosion patterns
- measuring procedure of the wave characteristics (velocity and wave thickness)
- teamwork and task management
- data processing
- dismantlement of test arrangement

The protocol has been continuously updated during the tests.

The actual measurements of velocity and flow depth are described in Chapter 7 and the measuring methodology for erosion of the inner slope in Chapter 8.

2.5 Outline of test program

2.5.1 Initial test program

The ComCoast overtopping tests were basically focused on observation of surface erosion behaviour of grass covers and under laying clay under severe wave overtopping.

The testing program started with testing the non-reinforced section, for which the loads was increased until the maximum allowable erosion was obtained or until the overtopping program was finished. The maximum allowable erosion was to be determined by the Waterboard Hunze en Aa’s (hereafter referred to as the Waterboard).

The initial test program was as follows:

- 6 hours storm with overtopping rate of 0,1 l/s/m;
- 6 hours storm with overtopping rate of 1 l/s/m;
- 6 hours storm with overtopping rate of 10 l/s/m;
- 6 hours storm with overtopping rate of 20 l/s/m;
- 6 hours storm with overtopping rate of 30 l/s/m.

For the 0,1 l/s/m, the number of overtopping waves is very limited: hence, this test was speeded up to 36 minutes (by accelerating the intermediate periods 10 times). The other tests were carried out in real-time. After each 2 hours the tests were stopped for a detailed survey of the erosion.

After finishing the tests at the non-reinforced section the reinforced section was tested by moving the whole test set-up to this section, including overtopping simulator, measuring cabin, measuring equipment, scaffolding staircase and side walls. At the new secion, a similar testing program was carried out.

2.5.2 Extension of the test program

As no major erosion was observed during the 30 lt/s/m tests at both the unreinforced and reinforced grass sections, the test program was extended by a ‘50’ lt/s/m test. Although the ‘50’ l/s/m possibly may not have been fully representative, the wave overtopping was considerably more severe than the 30 l/s/m condition, due to the larger portion of largest overtopping volumes.

Even after the ‘50’ l/s/m test, no major erosion occurred. Therefore it was decided to carry out the ‘50’ l/s/m test with initial damage. This damage involved the removal of the upper grass layer with horizontal dimensions 1*1 m, application of a deep 15 cm hole at a bared spot with horizontal dimensions 0.4*0.4 m and introduction of two small holes 0,1m * 0,1 m and a pole and two pickets that were placed into the test section, according Figures 2-6 and 2-7.

Figure 2-6: Introducing damage; removal of the upper grass layer with horizontal dimensions 1*1 m and placing a pole with a diameter of 0.07 m.

2.5.3 Additional tests on bare clay

An extra test on bare clay had been assigned by the SBW program. Therefore the grass sod (upper 20 cm) was fully removed. The aim of the test was to obtain a better insight in the behaviour of the total system of grass sod + clay under layer and the clay layer only. Under SBW, prediction models have been developed for the behaviour of the inner slope by wave overtopping. The results of the tests can be used to validate or modify these prediction models (outside of the cope of the present study). The results of the erosion tests on the bare clay layer are given in chapter 11.
The test on bare clay have been subjected to the following average overtopping discharges: 1 l/s/m (for this overtopping rate the intermediate periods have been accelerated 10 times), 5 l/s/m and 10 l/s/m.

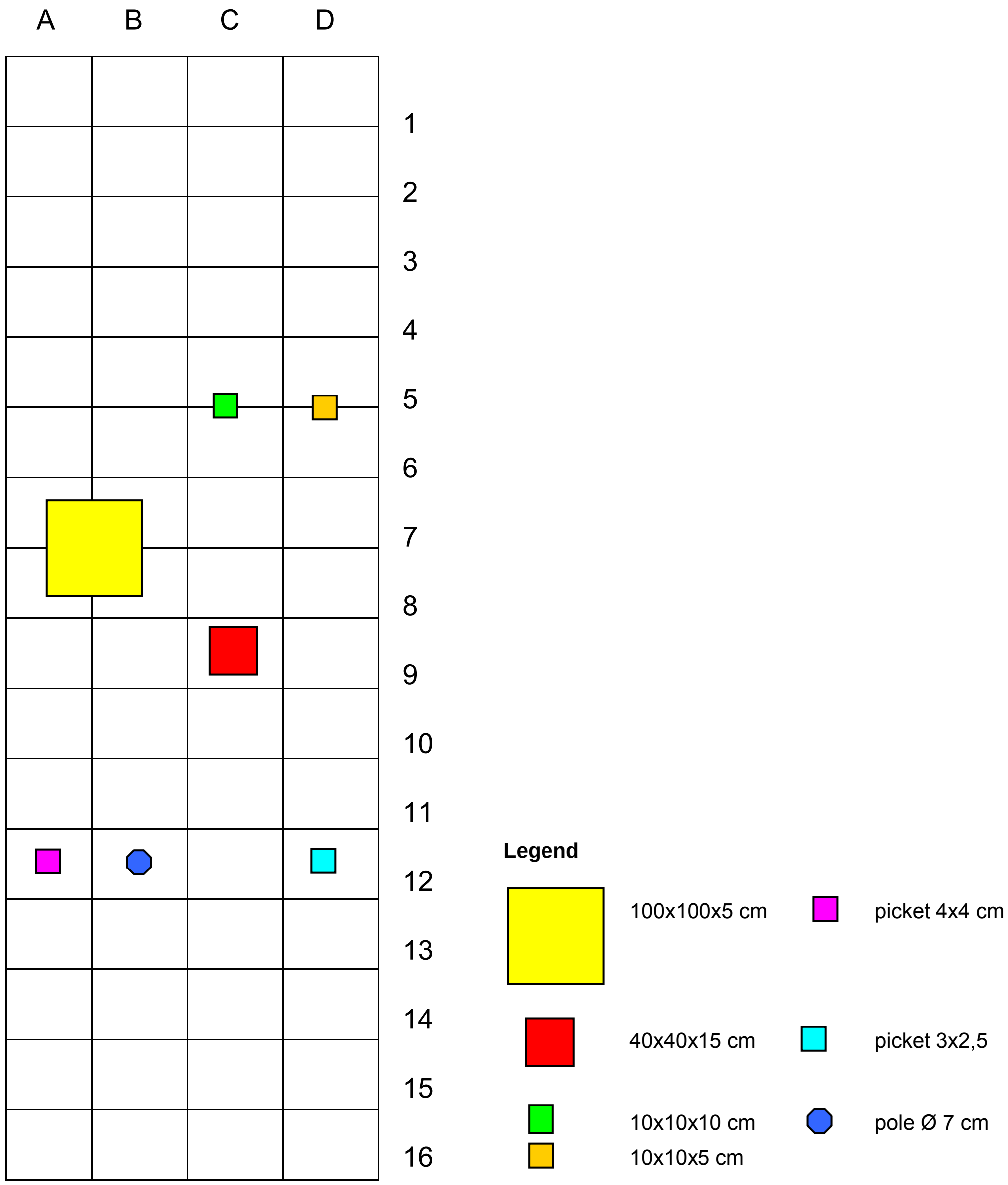


Figure 2-7: Pattern of initial erosion (depth x width x length) and obstacles introduced to the unreinforced and the reinforced test section

Apart from the bare clay tests, within SBW additional measurements have been carried out on the unreinforced as well as reinforced grass test sections. These additional acitivities are the infiltration tests (Chapter 6), the additional grass inspection (Section 5.2.2), the additional soil investigation (Section 5.3.3), as well as the velocity and flow depth measurements (Chapter 7).

3 BRIEF DESCRIPTION OF THE SMART GRASS REVETMENT (SGR)

3.1 Function and characteristics of the SGR

In this Chapter, a review is given of the selection and installation of the Smart Grass Reinforcement (SGR). More information can be found in the detailed report on the selection of the SGR (Royal Haskoning & Infram, 2005) and on the installation of the SGR (Royal Haskoning & Infram, 2006).

The SGR is a specifically selected geosynthetic that is installed under the upper part of the grass cover on a sea dyke, as to allow perfect intertwinement with the grass roots within one growing season. This allows full recovery of the grass cover before arrival of the next storm season. The system has two major functions:

1. <u>improvement of the surface erosion resistance of the grass cover</u>, by:
   a. providing grip to the grass roots;
   b. reducing eroding forces at bared spots (were the grass cover is absent);
   c. shielding of the under laying sub-soil.
2. <u>constituting an armouring frame that can cope with tensile stresses in the upper part of the dyke</u>, as to prevent or mitigate of clay shoals (i.e. shallow-slip failure).

A proper geosynthetic for the tests was selected after extensive analysis of required functions, indicated hereafter as a Geogrid. The Fortrac3D-120 system of Huesker Synthetic (main office in Germany) was found to fit the requirements best. An impression of this Geogrid is given in Figure 3-1 below.

Top view Front view

Figure 3-1: Selected Fortrac 3D-120 system

Another system was also considered by the consortium: a relatively stiff ‘Geocell’ system. Such a system may be pushed into the grass cover, thus disturbing the grass cover the least. However, such a system was not readily available and probably should have to be engineered specifically for this type of application, including a system to push the Geocell into the grass cover. This was considered as not appropriate for the tests at the sea dyke in terms of preparation time and development costs.

3.2 Principle of installation of SGR

The installation of the Geogrid is a critical issue. Pre-engineering of feasible methods by the consortium lead to a principle in which the layer is cut, lifted up, the Geogrid placed underneath and immediately replaced, as shown schematically in Figure 3-2 below.

However, the method as indicated schematically could not be made available in time for the field tests. Hence, the Geogrid had to be installed provisionally, according to the 'Big Roll' method: first the grass cover was sliced with a thickness of about 5 cm, rolled up and replaced again after placement of the SGR. Next the growing season would allow the grass to recover and intertwine with the Geogrid, see Figures 3-3 and 3-4.

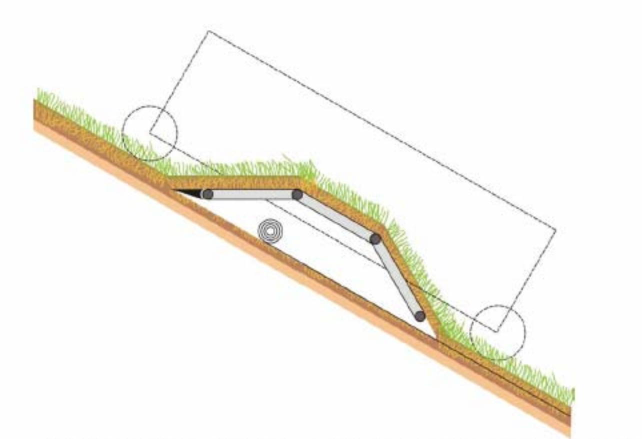


Figure 3-2: Potential system for installation of the SGR

3.3 Installation of the SGR at the test site

Prior to installation of the SGR a survey was carried out for proper placement techniques and mitigation of risks for placement. These have been reported as an appendix to the installation report (Royal Haskoning and Infram, 2006).

The placement of the SGR has been realized under the guidance and supervision of the consortium, with subcontracting services of FlevoGreenSupport (installation contractor), Queens Grass (grass farming), Huesker Synthetic (geosynthetic supplier) and the local contractor J. Kiel for additional assistance. The Waterboard provided hosting and general services and incorporated assistance of the local contractor J. Kiel. Four students of the Technical College (Hogeschool) Leeuwarden were assigned to the consortium as well for preparation of the test set-up, as well as preparation of the installation of the SGR.

The installation of the Smart Grass Reinforcement (SGR) at the sea dyke in May 2006 was to be located at a 4 m wide testing section, east of the unreinforced section, see Figure 2.2. This Geogrid also cover the upper part of the outer slope (anchoring) and extended over the crest and the inner slope up to a level of about 1.5 m above the service road (Royal Haskoning and Infram, 2006). The placement with Big Rolls provided provisional installation, that was considered as successful (but too expensive for large scale application). This methodology limited the Geogrid to be placed at a small distance below the grass cover surface of about 5 cm. An additional reinforced testing section was made as well, as a back-up section. However, this section was abandoned due to the poor grass cover growth. This was caused by the type of grass used here, which was taken from the foreshore due to a lack of proper grass rolls cut from the dyke. It should be remarked that the back-up test section was not required as well during the actual testing, as the basis test section was tested successfully. Adjacent to the test section with SGR a small monitoring section with SGR was installed as well. This section allowed intermediate inspection of the intertwinement process of the grass roots with the Geogrid.

Prior to the tests, cutting in an upward direction was considered as most appropriate as regards safety and control. After trial tests at the sea dyke, the default cutting was done from the crest down the slope, as shown in Figure 3-3. This was mainly due to the difficulty to obtain good Big Rolls in upward direction. In downward direction this proved to be easier. However, rolling up was not fully successful. This was especially caused by the extremely dry condition of the grass cover in spite of sprinkling water a day in advance.
This also caused ‘a loss’ of about half of the rolled up grass layer due to disintegration into too small pieces. Therefore, at the basic section nearly all the grass that was cut at the basic section and the back-up section was consumed. It was decided then to cover the back-up section with grass from the foreland (which by its composition turned out to be not feasible for intertwinement with the Geogrid).

The placement of the Geogrid and the result after replacement of the grass rolls are shown in Figure 3-3 and 3-4.

Figure 3-3: Finally selected methodology: cutting in downward direction

Figure 3-4: After placing the Geogrid (left), the test section was covered again with the grass from big rolls (right)

As regards the rather difficult installation, the grass experts involved in the project recommended intensive after-treatment for the reinforced grass section: replenishment of soil and additional seeding at larger seams, fertilization based on chemical analysis, sprinkling during dry weather and postponement of mowing. For comparison reasons, this after-treatment was done identically at the monitoring section and the unreinforced section.

In spite of the intensive after-treatment, the grass suffered severely during the month of July 2006 due to the extremely hot and dry weather. However, thanks to the very wet and cool month of August and moderate temperatures in autumn, the grass recovered remarkably, resulting in an acceptable grass cover rate in October 2006 and in February 2007, see Figure 3-5 and Section 5.2. It should be remarked here that the reinforced section still showed a relatively large number of small bare spots. The unreinforced grass also showed some bare spots, but much less than could be observed at the reinforced section. In conclusion, the grass seemed to be intertwined reasonably at the SGR test section but the grass coverage was rather poor prior to the tests.

Figure 3-5: Impression of the SGR test section prior to testing (winter 2006-2007)

4 DESCRIPTION OF THE WAVE OVERTOPPING SIMULATOR

4.1 Introduction

An essential device for the erosion tests at the sea dyke is the wave overtopping simulator, that has specifically been designed, constructed and calibrated for these tests. The idea of this simulator has been proposed already some years ago and copyrights have been declared by Infram (after the ideas of Dr J.W. van der Meer).

The construction of the 4 m wide simulator was preceded by a prototype of 1 m wide. This prototype was designed, constructed, calibrated and adjusted in before the summer of 2006. Then the 4 m wide simulator was constructed and tested at the dyke during some trial tests in December 2006, well before the actual tests.

In this Chapter, a review is given of the wave overtopping simulator. More information can be found in the detailed report on the wave overtopping simulator (Infram & Royal Haskoning, 2007).

4.2 Design of the simulator

Principle

The wave overtopping simulator simulates the wave overtopping tongues at the crest and inner slope of the dyke, as shown schematically in Figure 4-1.

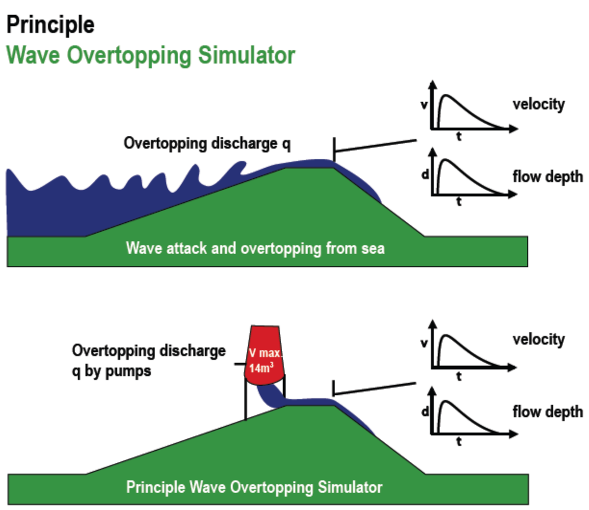


Figure 4-1: Principle of the reproduction of the overtopping wave tongues by the wave overtopping simulator

Conditions to be reproduced

The simulator is aimed to reproduce all the wave overtopping tongues that can occur during a storm, i.e. the full stochastic distribution of the overtopping tongues during that storm. In order to obtain a well-controlled process, a fixed filling rate was chosen, corresponding to the time-averaged overtopping rate. The limiting capacity of the water circulation at the test site for filling up of the simulator implies that the sequence of waves may differ from reality: e.g. highest waves may come in 'wave trains', which

cannot be reproduced well with the fixed water supply capacity. This is not considered as an important issue for the stability behaviour of crest and inner slope.
The physical parameters of the wave distribution are well-known at the outer slope of the dykes. Maximum flow velocities can be predicted rather well from present research. The flow depth at crest and inner slope is still uncertain, as regards the differences in predictions from different researchers. Hence, final verification of the wave overtopping simulator at the dyke is still pending and present verification is based on reproduction of overtopping velocities for each overtopping volume. Measurements of flow depths and flow velocities that have been performed at the sea dyke have been analysed in-depth in Chapter 7.

For the tests at the sea dyke, the boundary conditions have been assessed from average conditions along the Dutch coast, i.e.: H_s (significant wave height) = 2.0 m, T_p (peak period) = 5.7 s and T_m (mean period) = 4.7 s. In addition, the seaward slope of the dyke has been assumed to be 1:4. The maximum waves in these conditions with an overtopping rate of 30 l/s/m had a volume of 3.5 m^3 per m. Hence, the 4 m wide simulator was designed such as to store 14 m^3.
A 6-hour storm duration has been taken as a representative (conservative) value for producing the design wave overtopping distribution.

It should be noted that at other locations, conditions may differ strongly from the above. Deviating conditions should be taken into account in interpreting the results from the erosion tests in this report.

Design variables
For a practical design of the simulator many variables had to be determined, such as: the height of the simulator, the height of the outflow, the cross-sectional shape, the size and operation requirements of the valve and the outflow section. These variables were determined after a thorough analysis of the (sometimes contradictory) requirements for the outflow performance.
During the outflow, the pressure head decreases. Hence, the cross-section had to be wedge-shaped, as shown in Figure 4-2.

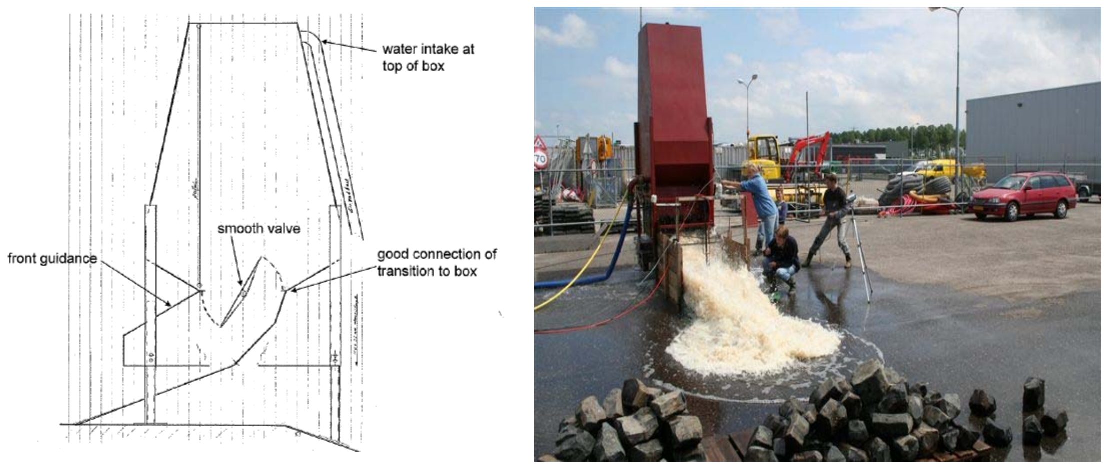


Figure 4-2: Design sketch of simulator and preliminary testing of prototype simulator

Calibration
Extensive calibration tests have been carried out, going along with ongoing improvements of the simulator with systematic variation of the design variables. Special

attention was given to the outflow structure and the optimum height of the simulator. After some trial tests, flow velocity was measured successfully with an electromagnetic device (EMS). Water depth measurements however did deviate largely from visual observation, so the instrument was found to be unusable.
After many tests, the most suitable design was selected. For this design it could be concluded that the performance of the prototype simulator was successful in terms of reproduction of maximum velocities and outflow duration.

4.3 Construction and placement of the simulator

After calibration, the construction of the final 4 m wide simulator started and final improvements could be introduced directly to the simulator. An impression of the construction and appearance of the final simulator is shown in Figures 4-3 and 4-4.

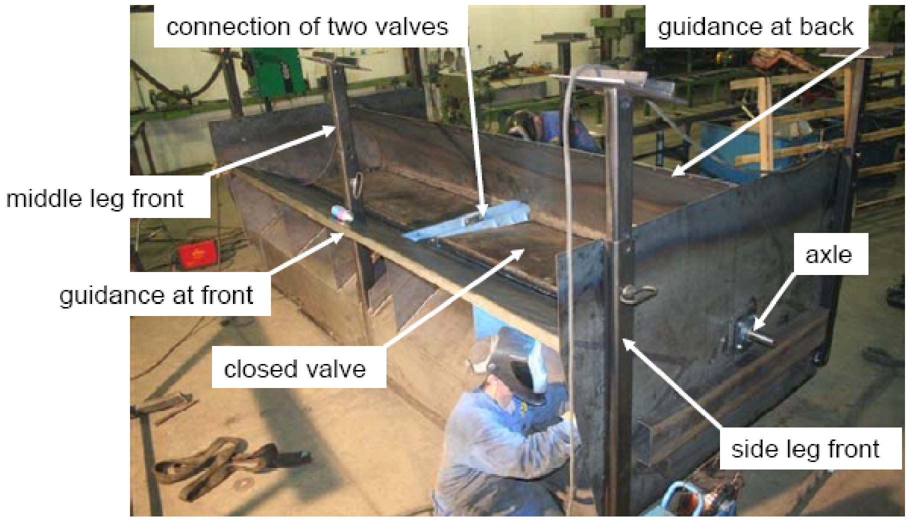


Figure 4-3: Construction of the simulator (upside down position)

Figure 4-4: Side view of the simulator (left) and rear view (after placement for trial testing at the dyke)

The trial tests in December 2006 were successful and demonstrated the capabilities of the wave overtopping simulator, including the water circulation system. An impression of this performance is given in Figure 4-5. However, some improvements were necessary for final testing: e.g. the footings of the simulator penetrated too much into the subsoil, as is shown in Figure 4-5 as well.

Figure 4-5: Trial testing of the wave overtopping simulator (left) and penetration of footing in subsoil (right)

For the real tests in February and March 2007, further improvements were implemented to the wave overtopping simulator as well as to the water circulation system .

Placement of the simulator was done with a large crane, as shown in Figure 4-6.

Figure 4-6: Placement of the simulator at the test site by a large crane

During placement much attention was paid to the footings, that had been reinforced since the trial tests, see Figure 4-7.

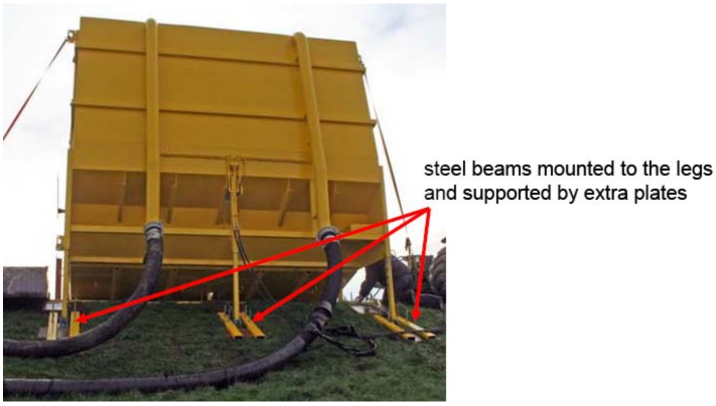


Figure 4-7: Improved footings of the simulator

4.4 Operation of the simulator

The overtopping discharge was accurately set by the pump capacity of the water circulation system, e.g. for the 10 l/s/m overtopping rate, the pump provided a capacity of 40 l/s/m for the 4 m wide simulator. Hence, the total amount of overtopping water corresponded exactly to the total amount required and the operation of the simulator was restricted to timely opening and closing of the valve. In the intermediate periods, the simulator was filled at various levels, corresponding to specific wave overtopping volumes.

The valve was opened hydraulically by means of tractor power. Initially the operation was done manually from the tractor. Prior to the actual tests however, joystick-operation was installed from the measuring cabin, as shown in Figure 4-8. This enabled accurate and well-controlled emptying of the wave simulator with the help of a list of opening times of the valve, in real time during the 6-hours period of the storm.

Figure 4-8: Joystick-operation of the wave simulator

During the first test series, the water circulation capacity had to be increased as to accurately reproduce the largest waves during 30 l/s/m with a volume of 3.5 m^3 per m. As a consequence, the smallest overtopping rate of 0.1 l/s/m could not measure accurately enough. Hence, the 0.1 l/s/m series were performed with intervals that were speeded up ten-fold while using 1 l/s/m capacity.

The formal opening event of the test site enabled pre-testing at full capacity, as this was done at the location where the bare clay was to be tested at a later stage and some erosion to the grass cover was allowable. This testing was successful. An impression is given in the series of snapshots in Figure 4-9.

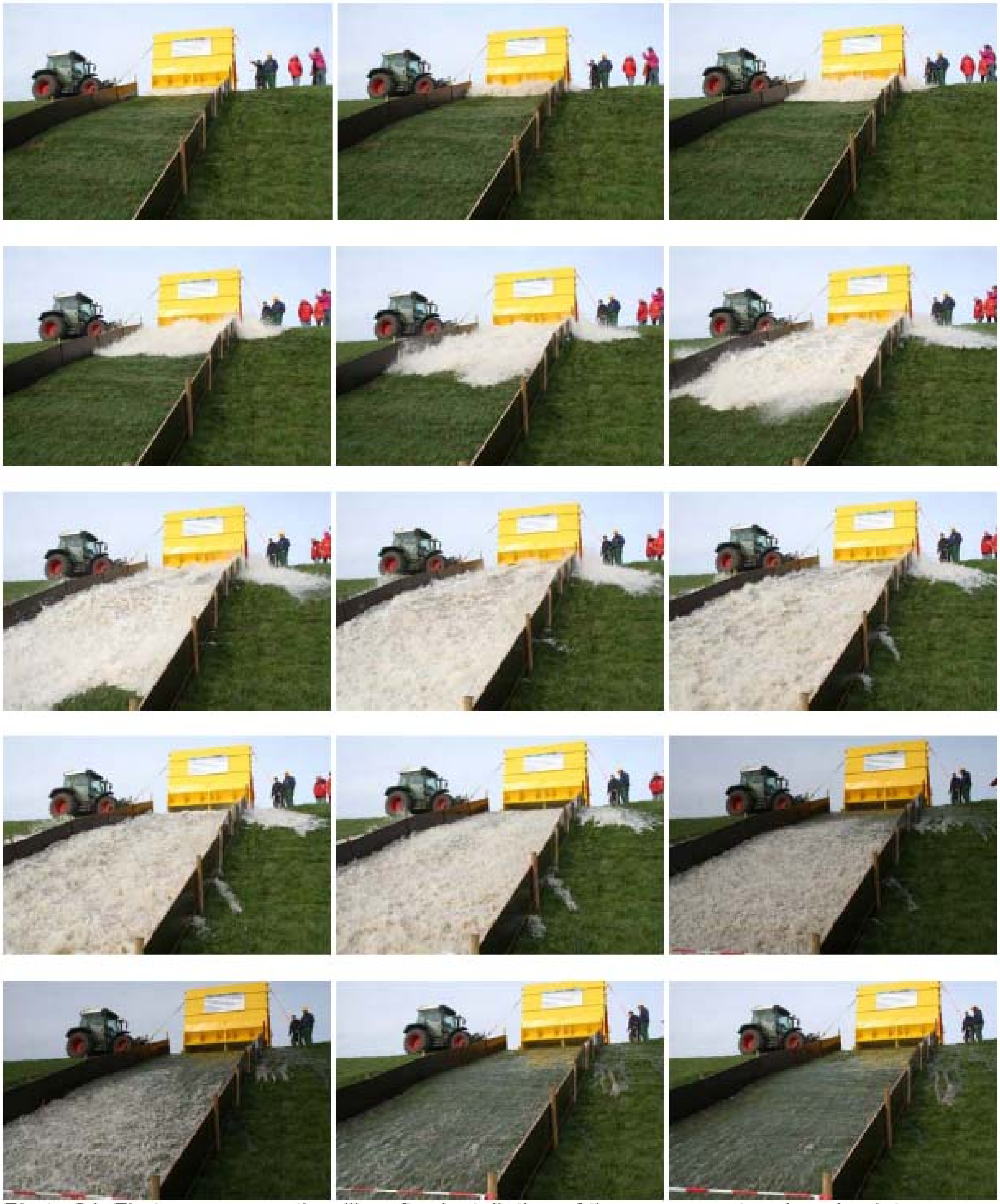


Figure 4-9: Impression of wave overtopping during final trials outside of test sections (before placement of scaffolding and sand bags)

5 CLAY AND GRASS COVER INSPECTION AND STRENGTH DETERMINATION

5.1 Introduction

Different kind of tests have been carried out to determine the factual conditions of grass and soil before and after installation of the SGR (May 2006) as well as before and after completion of the wave overtopping tests.

In chronological order the following studies have been carried out:
1. Visual inspection at the site (June 2004), outside of scope of ComCoast and SBW.
2. Grass analysis (April 2006) to determine the condition and resistance against erosion of the grass before installation of the SGR.
3. Soil analysis (May 2006) to determine the physical composition and the strength conditions of the samples.
4. Grass analysis in the winter period prior to testing (February 2007) to determine the condition and resistance against erosion of the grass at both the SGR and the unreinforced test section after the follow-up treatment (spray of water and fertilization). For comparison grass analysis was done outside the sections with follow-up treatment as well.
5. Soil analysis (July 2007) to determine the shear stress of the grass.
6. Grass analysis (June 2007) to determine the condition and resistance against erosion of the grass after completing the tests (Frissel, 2007).

The above items are briefly reported hereafter. For more detailed information is referred to separate reports. The studies numbered 1, 2 and 3 are discussed in (SBW, 2007), the study with number 4 in (Geodelft, 2007). The soil investigation of June 2004 has been reported by Fugro (2004). The Fugro results of 2006 can be found in (Heikes and Zwang, 2006).

Figure 5-1 shows the location of the different measurements: grass investigation (A and B) in red and soil investigation (1 to 6) in blue.

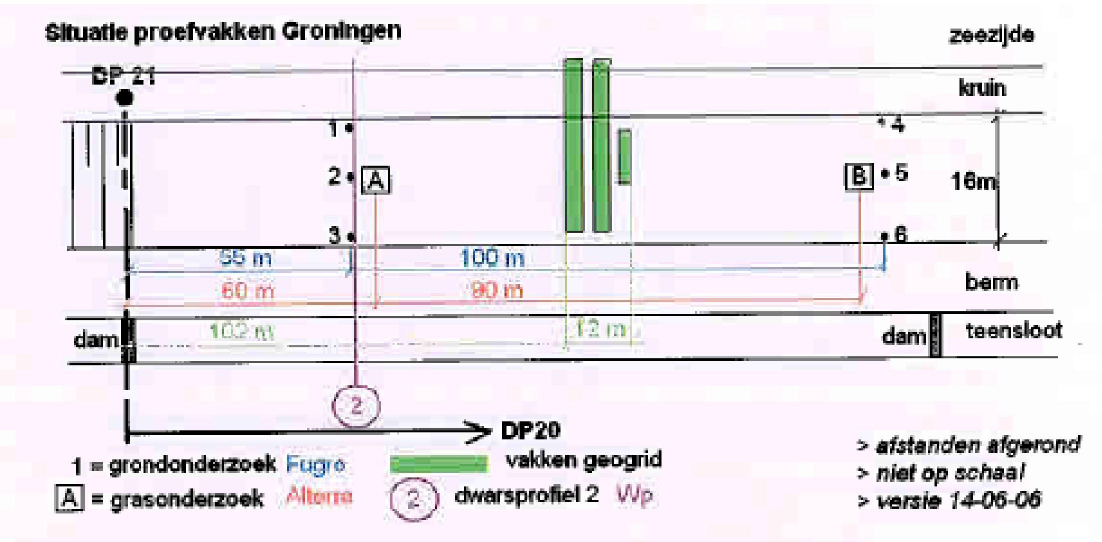


Figure 5-1: location of measurements (SBW, 2007)

5.2 Grass investigation by Alterra

The objective of the grass inspection is to determine the condition and resistance against erosion of the grass in the different stages (with and without after-treatment both prior and after the tests) at the SGR-test section as well as at the normal test section. Grass inspection was carried out in April 2006 prior to placement of the SGR, in February 2007 prior to the actual overtopping tests (determination of root system) and in June 2007 after the overtopping tests (determination of the vegetation). The inspection of 2006 has been reported in Alterra (2006) and the inspection of 2007 in Frissel (2007).

5.2.1 Grass investigation April 2006

In April 2006 Alterra was commissioned by the CUR to carry out a grass inspection on the Groningen sea dyke. The objective of the grass inspection was to determine the conditions and resistance against erosion of the grass at that moment. Therefore the root system and the covering rate of the grass were assessed. A vegetation survey was done as well. The results of the grass inspection were used to determine the score in quality for resistance against erosion according to the VTV: 'Safety assessment of the primary flood defences in the Netherlands' (VTV, 2004).

Figure 5-2: Overview of test section location before placement of the SGR (spring 2006)

The test sections are located at the dykes inner slope, which is facing the south. The slope of the dyke is 1:3. No activity of mice and mole has been observed. According to the water board the dyke slope has been pealed (in dutch: 'klepelen') every two or three weeks since the year 2000. Regarding the high mowing frequency and the abundance of nutrients because the hay is not removed, a shallow rooting grass vegetation can be expected.

The total coverage of the grass was found to be high in 2006. The covering rate of grasses and herbs proves to be within 98 -100%. The coverage rate of grasses varies within 95 – 100%, the covering of the herbs varies within 5 – 12%. The covering of moss is extremely low (<3%). The two grasses that are found most are Lolium perenne (in Dutch: Engels raaigras) and Fetuca rubra (in Dutch: Rood zwenkgras). For more details reference is made to (SBW, 2007).

The appraisal of the quality of the grass turf layer for situation in April 2006 is based on the VTV (VTV, 2004) and can be summarized as follows:

1. Defining the quality of the turf based of the type of grass maintenance. The score was found to be 'moderate' in April 2006.
2. Defining the quality of the turf based on the composition of the vegetation. This score was 'moderate' as well.
3. Defining the quality of the turf based on the rooting. The score was 'moderate' as well.

Because both 1 and 2 scores were moderate, the final score depends on the classification of the rooting system. Hence the final score for both sections A an B of Figure 5-1 is 'moderate' in spite of the good coverage rate of the vegetation.

It should be remarked that this score (according to the standard procedures) is somewhat in contrast to the personal impression in the field by Alterra that the erosion resistance might be quite good.

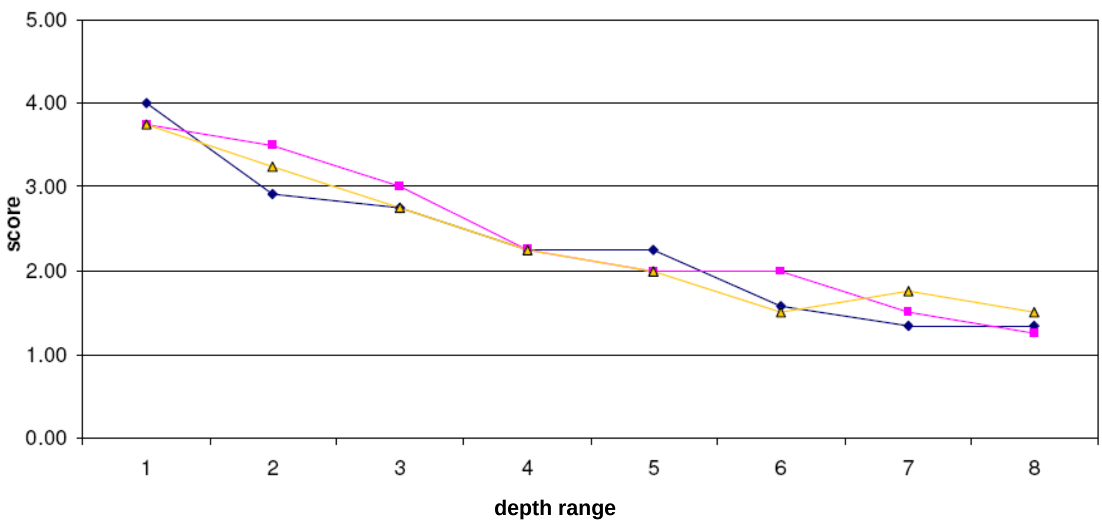
5.2.2 Grass investigations February and June 2007

In February and June 2007 Alterra performed a follow-up grass investigation at the Groningen sea dyke. Primary goal was to track possible yearly and seasonal changes and the influence of the after-treatment of the test sections (fertilization and sprinkling), as regards the restoration of the grass cover at the SGR section. It should be remarked here that the after-treatment was applied to the unreinforced section as well, as to provide a good comparison basis for the influence of the SGR. In addition, the grass condition was assessed also at an unreinforced section further east of the testing area, which did not receive the after-treatment. An impression on the appearance of the grass cover has been presented before in Figure 3-5.

A remarkable finding in February 2007 was that the after-treatment seemed to have caused a less homogeneous grass cover, even for the unreinforced section, although the average coverage rate was nearly the same for all sections (80 %). This observation was strongest however for the reinforced section: the grass cover showed more open spaces and a more 'patched' pattern. The original untreated grass cover had a more regular coverage. The vegetation in June 2007 at the monitoring section (with SGR and after-treatment) as well as at the unreinforced section east of the testing area (no SGR and no after-treatment) showed an increased coverage rate of 90 to 95 %.

The grass root development of the three investigated locations (reinforced section, unreinforced section with after-treatment and the unreinforced section without after-treatment) showed a highly comparable root development, as is illustrated in Figure 5-3. t must be remarked here that the mild winter temperatures may considerably have added to the root development of the grass at the reinforced section. This may have compensated to some extent the effects of the severe heat and drought that occurred in July 2006 shortly after installation of the SGR.

The inspection of the vegetation in June 2007 showed that the number of grass species was rather poor: 9 and 8 species at the monitoring section (reinforced and with after-treatment) and the eastern location (unreinforced and without after-treatment) respectively. According to the VTV the vegetation type was denoted as W1 (in dutch: Beemdgras-Raaigrasweide), which is considered as relative erosion sensitive. The vegetation at all sections was dominated by Lolium perenne (Dutch: Engels raaigras). The composition at the unreinforced and reinforced sections that received the after-treatment was largely the same. In the untreated section a relatively large coverage of Festuca rubra (Dutch: Rood zwenkgras) could be observed. The above composition is characteristic for the differences in treatment.



section with Geogrid and after-treatment
section with Geogrid without after-treatment
section with no Geogrid without after-treatment

score:		depth range:
0 = no roots		1 = 0,0-2,5 cm
1 = very few roots	:1-5	2 = 2,5-5,0 cm
2 = few roots	: 6-10	3 = 5,0-7,5 cm
3 = moderate	:11-20	4 = 7,5-10,0 cm
4 = many roots	:21-40	5 = 10,0-12,5 cm
5 = very many roots	:>40	6 = 12,5-15,0 cm
		7 = 15,0-17,5 cm
		8 = 17,5-20,0 cm

Figure 5-3: Root density February 2007 at three sections Groningen sea dyke

The quality of the grass turf layer, based on the VTV (2004) for the 2007 situation can be summarized as follows:

1. The score for the type of grass maintenance is denoted as ‘moderate’, based on pealing every two or three weeks without removal of the hay.
2. The score for the composition of the vegetation is set as ‘poor’
3. The score for the rooting is indicated as ‘poor’.

The rooting score is decisive for the total assessment score, as the maintenance score is ‘moderate’ and the vegetation score is ‘poor’. Hence, the VTV methodology leads to the overall score ‘poor’ for the grass condition in 2007 prior to testing. This is worse than in 2006, when the overall score was ‘moderate’. A possible explanation may be found in the yearly fluctuation of grass quality (possibly influenced by the extremely hot summer of 2006), as well as the activities at and near the test site.

Testing was mainly done in March 2007. As a consequence, some additional root growth may have occurred as compared to the winter situation of the grass in February. The mild winter may have caused the grass to be in 'spring condition' rather than in 'winter condition'. Future tests may clarify this issue in retrospective. Moreover, the differences in VTV scores in 2006 and 2007 indicate large yearly fluctuations as well, which may even go beyond the seasonal fluctuations.

5.3 Soil investigations by Fugro and GeoDelft

5.3.1 Soil investigation June 2004 (Fugro)

Waterboard Hunze en Aa's performed the most recent five yearly safety assessment for 26 km of dyke, including the test location, in 2004. Support was given by Fugro (Fugro, 2004) through a visual inspection of the grass cover. This inspection was performed on 7 June 2004. Main purpose was the revetment on the seaward side, but also the grass cover on seaward side, crest and inner side were inspected.

The middle of the test section with the SGR was located at km DP 20.888 m (112 m east of DP 21). Results of the inspection for dyke section DP 19.5 to DP 24.4 were:

> "The grass on seaward and inner side is maintained by regular mowing (i.e. once per 2 to 3 weeks). The grass cover at DP 23.5 is dry and not in good condition. At DP 22 and DP 23.5 hand borings have been performed".

Results show that at DP 19.5 the upper 0.9 m consists of (extensively) sandy clay. Below this layer clay exists over 0.3 m. Borings had a depth of 1.2 m.

5.3.2 Soil investigation May 2006 (Fugro)

Fugro has performed a soil investigation specific for the test site end of May 2006, which has been described in (Heikes and Zwang, 2006). Below, a summary of results is presented.

Measurements were performed on both sides of the test section, at DP 20.945 and at DP 20.845. With the middle of the test section with SGR at DP 20.888, this means that the test sections for soil investigations were about 40 – 50 m on each side of the test sections for the overtopping tests. At each location 3 borings were made, one at the inner crest line, one at the toe of the inner slope and one half way the inner slope). Borings were numbered 1-3 and 4-6 from above to below and with numbers 1-3 at DP 20.945 and 4-6 at DP 20.845. Furthermore, the permeability was measured halfway the slopes (at numbers 2 and 5).

Borings
Boring 1 at the inner crest line was performed up to a depth at 3.5 m below the surface. Only clay was found, no sand. Boring 4, also at the inner crest line, was performed up to a depth of 1.5 m below surface and also showed only clay. Both borings half way the slope, numbered 2 and 5, were performed till 1.5 m below surface and showed only clay. Borings 3 and 6 were performed at the toe of the inner slope to a depth of 1.6 m. Till 0.4 m below surface clay was found, then almost a half metre thick layer of sand was found and from there till 1.5 m depth again clay. Under this clay layer sand was found.

Results of the clay

The organic content of the clay at both locations was about 4% (m/m) and the chalk content about 7% (m/m). The salt content was lower than 0.5 g/l of bottom moisture. Other measures were different at the both locations, see Table 5-1. The clay at locations 1-3 was better than the clay at locations 1-6. At latter locations the sand content is fairly large.

Parameter	**Unity**	**Clay 1-3**	**Clay 4-6**
Sand content > 63 μm	% (m/m)	18	38
Lutum content < 2 μm	% (m/m)	40	28
Water content	% (m/m)	35	25
Liquid limit	% (m/m)	64	49
Plastic limit	% (m/m)	26	17

Table 5-1: Averaged results for borings at locations 1-6

Strength results of clay samples

At locations 2 and 5, half way the inner slope undisturbed samples have been taken to undergo a CU-triaxial test. Results of friction angle and cohesion were determined at a displacement of 1%. Results are shown in Table 5-2.

Parameter	**Unity**	**Clay 2**	**Clay 5**
Distorted samples			
Sand content > 63 μm	% (m/m)	4	13
Lutum content < 2 μm	% (m/m)	52	37
Water content	% (m/m)	41	35
Liquid limit	% (m/m)	70	65
Plastic limit	% (m/m)	21	22
Undistorted samples			
Wet density	kN/m^3	19.3	18.8
Dry density	kN/m^3	14.6	14.7
Cohesion	kN/m^2	24	27
Friction angle	°	13	9

Table 5-2: Averaged strength results for clay samples 2 and 5

Results of sand

The sand at the toe showed similar composition for the two borings at the two locations (3 and 6). There were slight differences between the upper layer of sand and the lower layer, see Table 5-3.

Parameter	**Unity**	**Upper layer**	**Lower layer**
Sand content > 63 μm	% (m/m)	82	91
Lutum content < 2 μm	% (m/m)	7	4
Water content	% (m/m)	11	20
Organic content	% (m/m)	2.1	1.6
Chalk content	% (m/m)	2.7	1.3
Salt bottom moisture	g/liter	1.3	0.3

Table 5-3: Averaged results of sand for borings at locations 3 and 6

Infiltration tests
At location of borings 2 and 5, half way the inner slope, the permeability has been measured by means of an infiltration test. An aluminum box was used, without bottom, with dimensions of 1 by 1 m. The box was pushed into the soil to a certain depth and then filled with water. The decrease of water level in time was measured to give an idea of the permeability of the upper layer of clay. The permeability at location 2 appeared to be 11 m/day and at location 5 this was 22 m/day. The larger sand content at location 5 is probably the reason for the larger permeability.

5.3.3 Soil investigation March 2007 (GeoDelft)

A geotechnical investigation was carried out by GeoDelft within the framework of SBW 22 March 2007 (GeoDelft, 2007). Shallow drillings have been executed at 8 equidistant locations (1.75 m spacing) along the inner slope, east of the test sections. This location was outside of the influencing zone of the tests. The samples, taken from these drillings have been subject to direct shear tests, as well as to plasticity assessment by the 'Atterberg limits'. The shear tests were done at samples from a depth of about 0.30 m below the surface for drillings 1, 3, 5, 7 (in downward direction) and from a depth of about 0,10 m for drillings 2,4,6 and 8, see Figure 5-4.

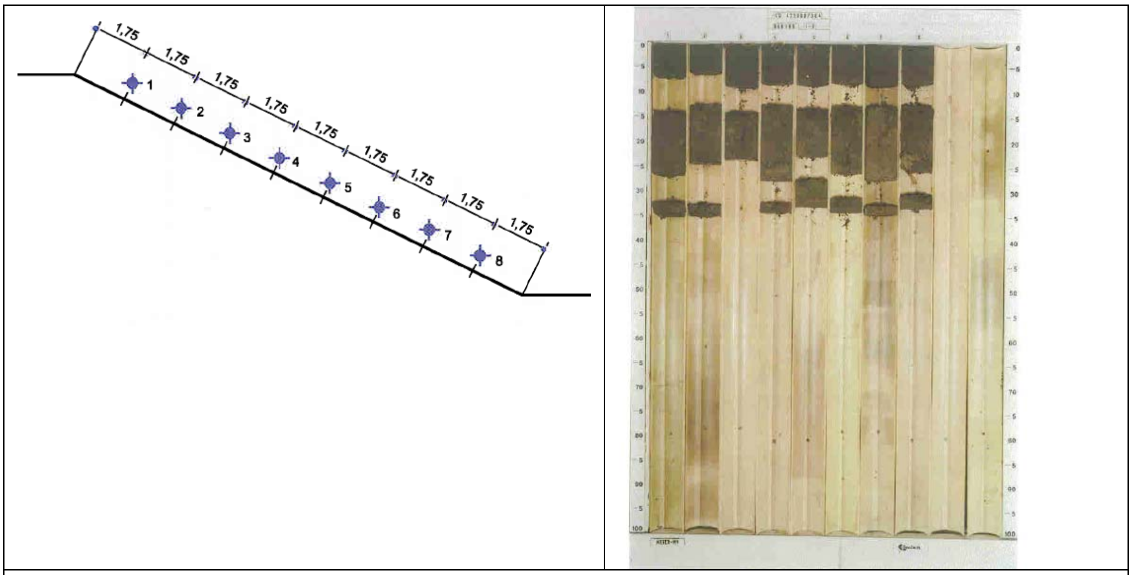


Figure 5-4: Location of drillings and indication of clay samples

All samples showed a composition of predominantly clay, moderately silty, with a low humus content, occasional small sand clogs and remains of roots.

The results will be analyzed further within the context of the SBW-program. As a preliminary finding, it can be remarked here that for the limited number of samples no clear correlation was found between the measured maximum shear force and the number of roots in the samples.

5.4 Impression of root intertwinement with Geogrid prior to tests

Prior to the start of the wave overtopping tests, parts of the monitoring sections were dug up to check if the roots of the grass had sufficiently grown through the Geogrid. This was done two times: the first time in October 2006 (after completing the after-treatment) and the second time in February 2007 (prior to testing).

Figure 5-5: Details of dug up SGR samples of monitoring section 6 October 2006

In October 2006 the results of the dug up samples showed uncertain results. It was doubtful whether the roots would sufficiently intertwine with the Geogrid (Figure 5-5). In February 2007 the results of the dug up Smart Gras Reinforcement samples were more favourable. The (longer) roots of the grass had grown well through the Geogrid (see figure 5-6). Apparently the grass had managed to overcome the severe conditions in summer (long drought and high temperatures), favoured by the mild winter temperatures.

Figure 5-6: Details of dug up SGR samples of monitoring section 13 February 2007

6 INFILTRATION TESTS AND RESULTS

6.1 Working methodology

Infiltration of water into the crest and dyke slope may influence the failure mechanisms of shallow slip failure and internal erosion (mircro instability). Infiltration tests give an indication on the sensitivity for slip failure, especially on the danger for sliding of part of the clay and grass cover.

An infiltration test with constant discharge was performed at 23 February 2007 at the unreinforced grass section, Figure 6-2a. Infiltration took place by pumping a constant flow in a perforated PVC-pipe, which was placed on top of the slope (Figure 6-2b). The flow rate has been measured by means of a flow meter and turned out to be constant in time at an average of 0,91 l/m/s. To determine the velocity by which the water infiltrated at 3 depths (0.4, 0.8 and 1.2 m) 2 sets of so called water tension meters were placed by GeoDelft (Figure 6-1). The locations of the two sets were at 1/3rd of the top and at 2/3rd of the top along the slope in the middle of the 4 meter wide dyke section (Figure 6-4). Each set consisted of three meters, extending 0.4, 0.8 and 1.2 m below the surface. Prior to the installation of the water tension meters, the grass cover layer was removed with a special cutting device and carefully replaced after installation of the meters. As to promote the conductivity with the surrounding soil, the ceramic tip of the meters was placed in sand with a diameter of 73 microns. The remainder of the drilling holes were sealed with clay granules that had a very low permeability after sufficient swelling (24 hours). Connecting cables were fixed to angled poles and transferred to the measuring cabin.

Figure 6-1: Preparations and placing of the water tension meters at the unreinforced test section

Figure 6-2a: Location of the water tension meters (poles)

Figure 6-2b: Infiltration at the dyke crest

After completion of the permanent infiltration tests, the water tension meters remained at their position for registration of the water tension during the wave overtopping tests.

6.2 Grass cover during the permanent infiltration test

The grass cover has been inspected prior, during and after the permanent infiltration test. It turned out that, apart from the existing bare spots becoming more visible by the adaptation of the grass swords to the flow, the grass cover had not been eroded by shallow sliding such as 'turf sliding' (this is sliding of a major part of the grass cover), see Figures 6-3a, 6-3b and 6-3c. After the test no damage such as cracks or surface erosion was visible at the outflow position at the inner crown line as well.
During the test some loose ground particles were washed away from the slope surface. After the test the more heavy sand particles could be found in a small amount at the toe of the dyke.

Figure 6-3a: 9:54 hr (after 1 hr)

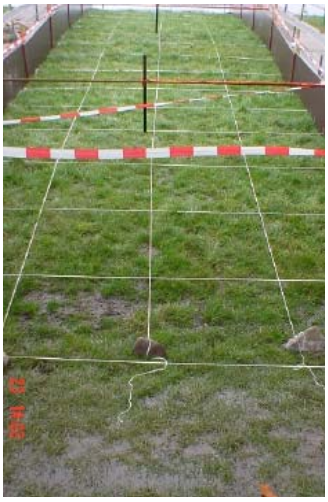


Figure 6-3b: 14:02 hr (after 5 hrs)

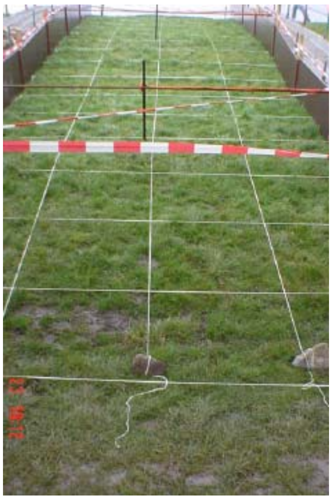


Figure 6-3c: 16:12 hr (after 7 hrs)

6.3 Infiltration results and preliminary conclusions

Data acquisition of the water tension meters was done by a CR10 logger of GeoDelft and were transmitted to the GeoDelft office by GSM-connection. The permanent infiltration test was performed at 23 February 2007. During the wave overtopping tests the measurements were recorded continuously, starting at 1 March 2007 at and finishing at 19 March 2007.

The data logger could measure with two different intervals; every two minutes or every 10 seconds. During the wave overtopping tests the data were recorded every 10 seconds. For the sake of reduction of the amount of data the interval was switched to two minutes when no tests were ongoing. Direct monitoring at the testing site of the data was not possible: looking back, this was felt as an omission and real-time monitoring at the site should be pursued. The locations of the water tension meters is shown in Figure 6-4 and a typical result of the measurements in Figure 6-5.

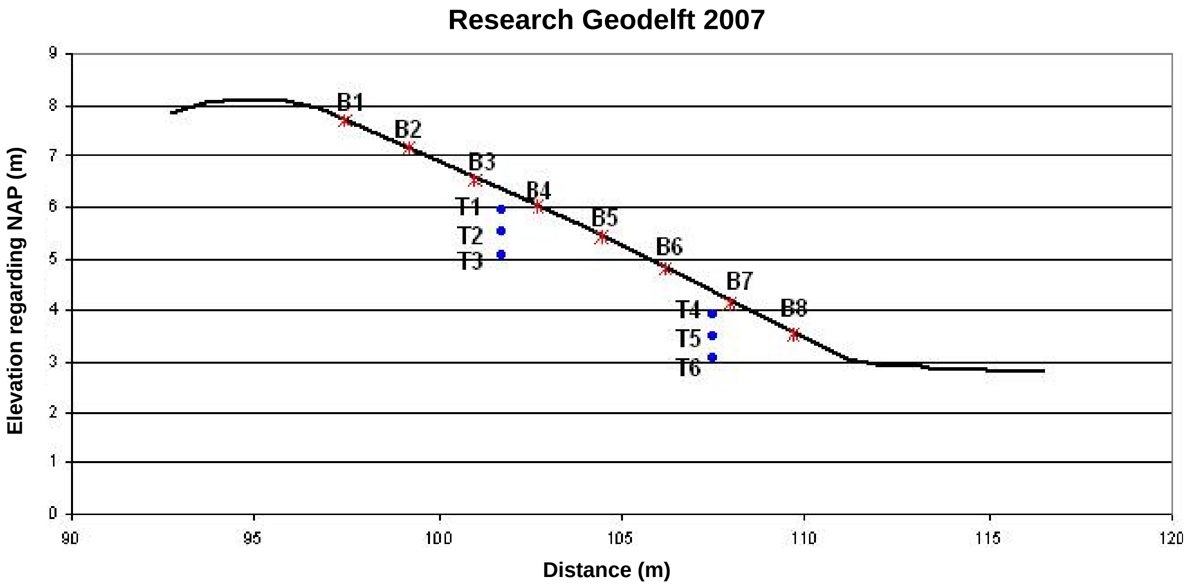


Figure 6-4: location of the water tension meters (T1 to T6) in the dyke (un-reinforced test section)

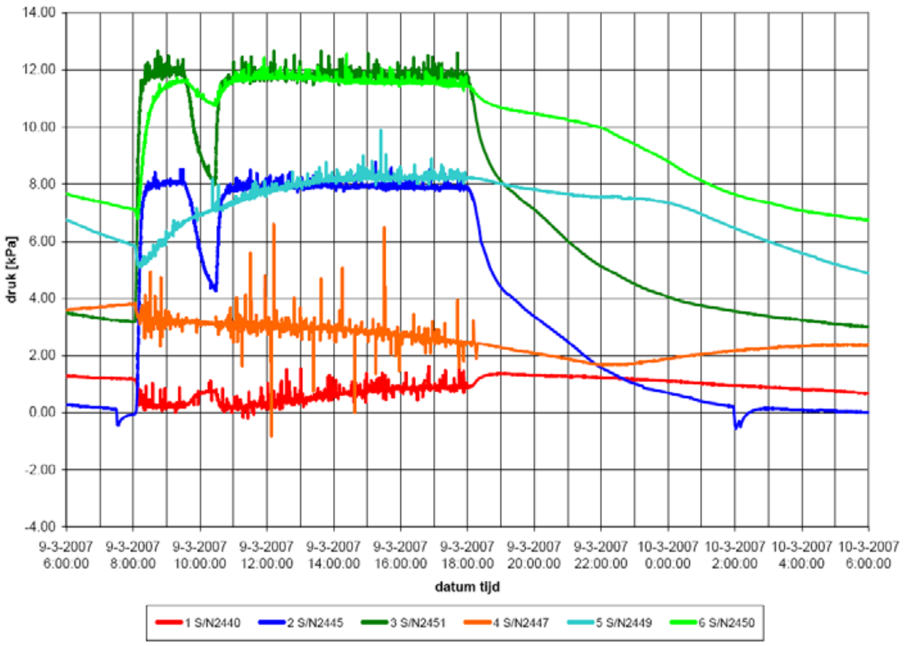


Figure 6-5: Illustrative result of the water pressure measurements

In a consistency analyses by Ronald van Etten (Etten, 2007), he concluded that the two water tension meters (T1 and T4) that were placed at the smallest depth (40 cm below the surface) should be considered as not reliable and thus should be ignored for further analysed.
Some other preliminary findings of this analysis are:

- The response of the meters was very quick after the start of the infiltration test, as can be seen in Figure 6-5. The depletion of the pressure upon the end of the infiltration is much slower. This leads to a different permeability: $2*10^{-3}$ and 10^{-4} respectively. Assuming a permeability of 1 to $2*10^{-4}$, this is quite permeable as regards to the predominantly heavy clay in the top layer. This may be explained a.o. from the many wormholes that are present up to a great depth.
- However, as regards the very quick response of the pressure at the start of the tests, leakage or enclosure of air at the tension meters could not be excluded as well. This item should be verified in future applications.
- During the wave overtopping tests, full saturation of the top layer occurred for overtopping rates of 1 l/s/m and more.
- The water tension meters at 1/3 of the crest reacted faster than the meters at 2/3 of the crest. However, pressure difference remained absent at different depths.
- During tests at the adjoining reinforced test section, fluctuations could be observed, which is attributed to the spreading of ground water flow (under about $45\ ^{0}$).
- Rainfall may only influence the pressures when very abundant.

Illustrative is that the effect of the largest overtopping waves can be observed in the pressure data, as shown in Figure 6-5.

The measurements are described in (GeoDelft, 2007) and the preliminary analysis of the measurements in (Etten, 2007). Within the SBW program, the results of the measurements may be used at a later stage for validation of the Plaxflow model that simulates groundwater response in porous media.

7 WAVE OVERTOPPING MEASUREMENTS AND ANALYSIS

7.1 Set-up of measurements

Measurements of flow velocities and depths of the wave overtopping tongues have been carried out at two locations: one location near the crest and one location about halfway the inner slope. The location near the crest changed over time: for the unreinforced grass section the instruments were placed near the crest line and for the reinforced grass section at 2.2 m distance from the crest at the inner slope.
The flow velocity measurements were done with an electromagnetic type (EMS) meter and the flow depth measurements with a thin wire conductivity meter, as shown in Figure 7-1 below. These instruments were hired from Delft Hydraulics which adapted these instruments especially for this application and also supported the data-acquisition.

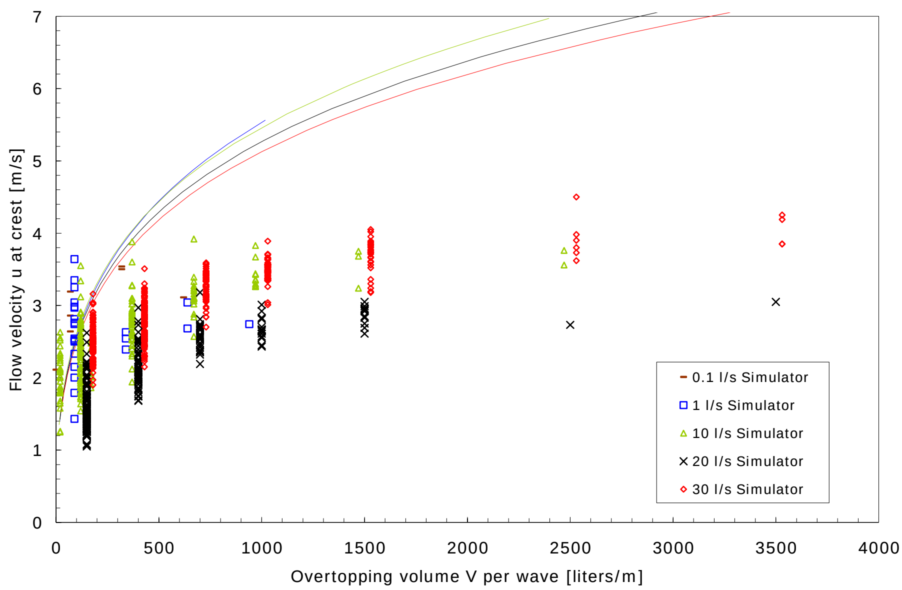
Figure 7-1: 1) velocity meter (EMS), 2) wave thickness meter (GHM) and 3) both meters fixed to a measuring frame

The flow measurements have been carried close to the grass cover: about 2cm above the grass cover for the unreinforced grass and about 5 cm above the grass cover for the reinforced grass.

The measurements have been reported in the separate report on the wave overtopping simulator (Infram & Royal Haskoning, 2007). Related to this research, Gijs Bosman performed a MSc. thesis on these measurements and analysed literature data (Bosman, 2007). Based on his findings, he arrived at new design criteria for wave overtopping behaviour. His major findings are summarized in (Infram & Royal Haskoning, 2007) as well.

7.2 Measurements

The raw flow velocity and flow depth measuring data were processed by Delft Hydraulics.
The velocity measurements showed a large scatter and measured velocities were considerably smaller at high overtopping rates than might have be expected from previous research. At the inner slope these velocities tend to decrease even stronger, as can be seen in Figure 7-2.



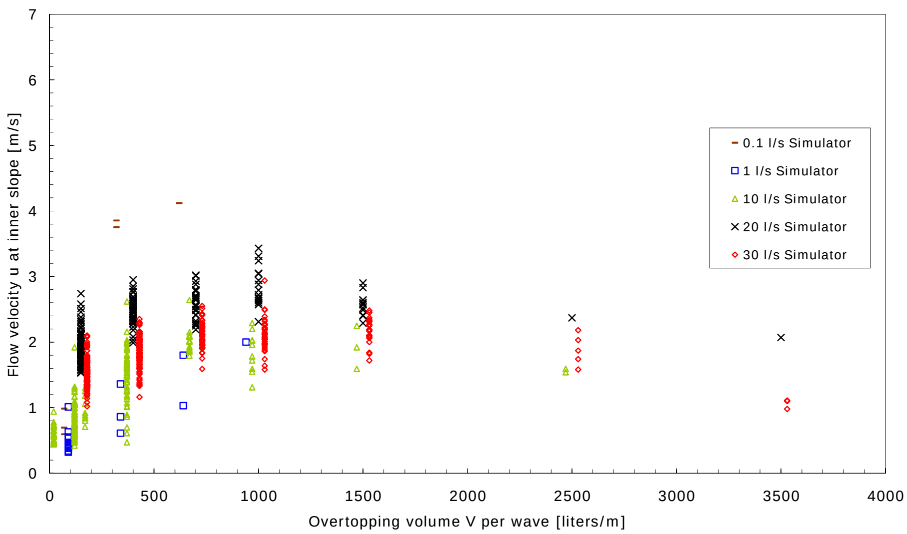
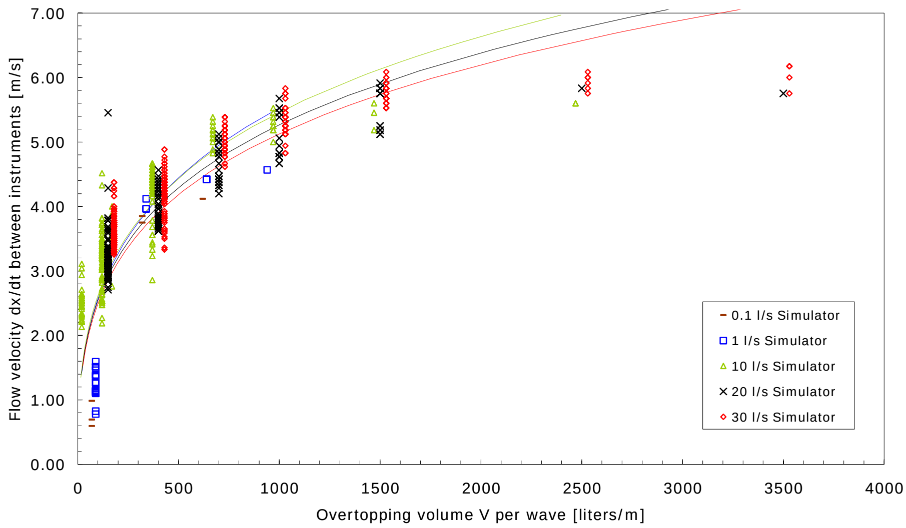


Figure 7-2: Measured flow velocities near crest (upper Figure) and halfway the inner slope (lower Figure) for the natural grass slope

Flow velocities at the SGR section do follow the same trend, albeit that the velocities are somewhat higher, probably due to the higher elevation of the EMS probe above the grass surface.
When plotted against each other, the velocities at crest and inner slope do hardly relate.

The front flow velocities, however do fit quite well to the expected values , as is shown in Figure 7-3..



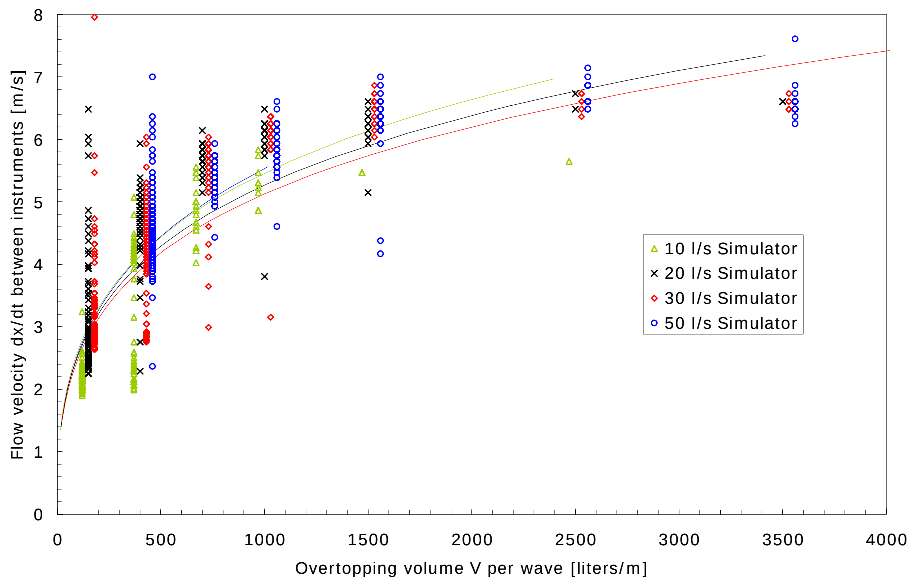


Figure 7-3: Measured velocities of the overtopping front for the unreinforced section (upper Figure) and the reinforced section (lower Figure)

Measured flow depths were also much less than the depths that could be expected e.g. according to Van Gent (2002). Maximum depths halfway the inner slope tend to go up to 0.1 to 0.2 m at the highest overtopping volumes.

Finally, the measured overtopping volumes, following from integration of velocity and water depth, can be compared with the actual volumes that have been released from the simulator. For the larger overtopping volumes the volume by integration is about half the actual volume, for the unreinforced section well as for the reinforced section. For the unreinforced section this is shown in Figure 7-4.

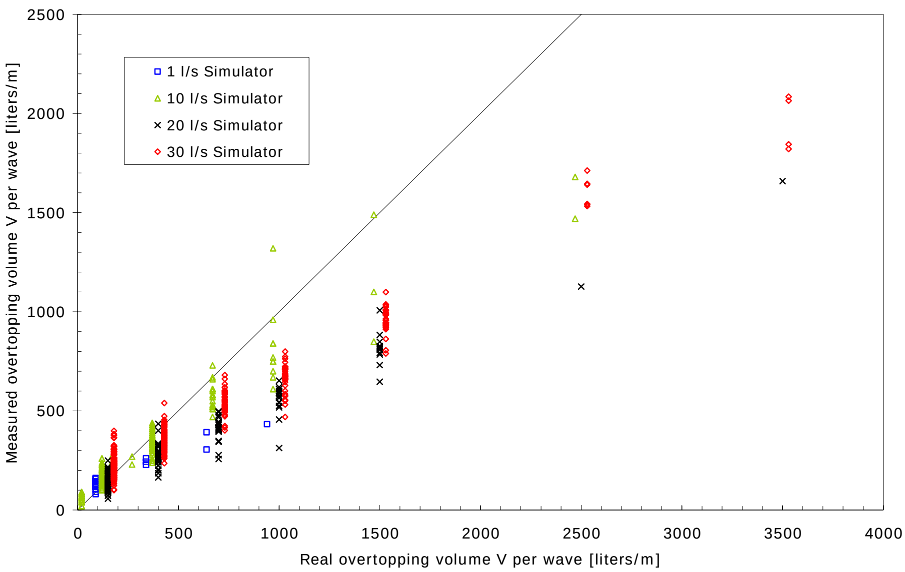


Figure 7-4: Calculated overtopping volumes from measurements versus actual volumes (unreinforced section)

The overtopping times do show a rather confusing picture halfway at the inner slope.

In addition, there seems to be hardly any correlation between the velocities near the crest and at the inner slope, which is highly unlikely.

From these results it must be concluded that the instruments that recorded velocity and flow depth did not measure properly under the simulated conditions. It appears that the highly turbulent and aerated overtopping flow is difficult to measure. Hence, the EMS and thin wire gauge need further improvement in future, or alternative instruments should be deployed. The measured front velocities between the two instrument locations were in good agreement with anticipated velocities. Visual analysis of the video recordings sustained this agreement further. This justifies the conclusion that as regards the velocities of the wave overtopping tongues the simulator performed well and in agreement with the expectations.

Within the scope of his MSc. thesis Gijs Bosman further analysed a discrepancy between earlier data from Van Gent (2002) and Schüttrumpf et al. (2002), under the guidance of Dr J.W. van der Meer. For a detailed description of his work is referred to his MSc thesis (Bosman, 2007). His major findings are also included in the final report on the simulator (Infram & Royal Haskoning, 2007).
As a major outcome Bosman showed that the discrepancy can be solved by some corrections in the measurements of Schüttrumpf and by taking into account the upstream slope angle of the dyke. These slopes were different in both research studies: 1:4 for Van Gent and 1:6 for Schüttrumpf. The analysis of Gijs Bosman resulted in new prediction formulae for wave overtopping behaviour, which included the effect of the upstream slope angle. These formulae can be used to set-up the wave overtopping simulator more accurately for further testing in the future.

8 METHODOLOGY OF EROSION MEASUREMENTS

8.1 Protocol

The detailed methodology for erosion assessment is laid down in the protocol. This Chapter summarizes the methodology for the erosion measurements.

8.2 Outline of the tests program

In Table 8-1 an outline of the test program is presented.

Unreinforced test section	
T1	0,1 l/s/m
T2	1,0 l/s/m
T3	10 l/s/m
T4	20 l/s/m
T5	30 l/s/m
T7	‘50’ l/s/m
T8	‘50’ l/s/m with initial damage
Reinforced test section (SGR)	
V1	1,0 l/s/m
V2	10 l/s/m
V3	20 l/s/m
V4	30 l/s/m
V5	‘50’ l/s/m
V6	‘50’ l/s/m with initial damage
Bare clay test section	
K1	1,0 l/s/m
K2	5,0 l/s/m
K3	10 l/s/m

Table 8-1: Outline of the test program

8.3 Video

A professional digital video camera has been used to record the overtopping tongue flowing down the slope and to record the overall erosion pattern at each overtopping tongue event. The camera was mounted at a frame at the observation point in the measuring cabin, about 4 m above the toe of the dyke. The camera recordings were continuously stored on auxiliary hard disks (permanent and backup). This however necessarily occurred with a time lag (increasing up to some days), as another video camera had to be used for real-time for processing of the data.
A digital time-clock was placed within the view of the camera at the rim of the test section. Here, additional information on data and testing number was shown as well, for adequate time reference. The minimum time interval of this clock was 1 second.

8.4 Photogaphs

8.4.1 Overall photographs

After two hours of testing (except for the 0.1 l/s/m tests) the tests were stopped and so was the digital clock. An overall photograph of the erosion was made from the measuring cabin with a digital photo camera. This was repeated after four hours of simulation and after 6 hours upon completion of the simulation. These photographs have been included in this report at the end of Chapter 9 (unreinforced grass), Chapter 10 (reinforced grass) and Chapter 11 (bare clay).

8.4.2 Detailed photographs of erosion during the tests

In addition to the overall photographs, in the two-hourly intervals also detailed photographs were taken with a digital photo camera during which spots with a characteristic erosion were registered with a measuring stick next to the eroded spot. Erosion proved to be fairly minor during most of the tests, so these photographs remained quite limited in number.

8.4.3 Additional photographs

Besides the aforementioned photographs, additional photographs have been taken prior to testing as well as after the tests at every square meter of dyke. This had been done to enable comparison of the initial conditions of the grass cover prior to the new test with the situation upon completion of the new test (after 6 hours storm duration).
In addition, after every two hours of testing (except for the 0,1 l/s/m tests) side view photographs of the section had been made from both sides of the scaffolding staircases. This was done for every metre dyke along the length of the inner slope.

Figure 8-1: Example of a side-view picture taken and of a detailed picture at each square metre dyke after completion of the test.

8.5 Visual inspection

8.5.1 Grass coverage rate

An attempt was made to assess the grass coverage rate visually by one and the same person after every two hours of testing with the naked eye. This was done by estimating the grass cover percentage (e.g. 90%). It was expected that the results of the grass cover inspection would show a clear decline in coverage rate. However, after analysis of the visual results no clear pattern of gradually decrease could be obtained. This may partly be attributed to the minor erosion that occurred. Hence these measurements were abandoned for further analysis.

8.5.2 Depth measurements for the tests with initial damage

As no major erosion was observed at the unreinforced as well as at the reinforced test sections, artificial initial damage was introduced for the final tests with an overtopping rate of '50' l/s/m. Measurement of the erosion progression was mainly done by taking photographs. However, the unreinforced section showed the strong gully formation along the slope downstream of the damage location 1*1 m and 0.4 *0.4 m after local removal of the grass layer. The size of the gullies at the reinforced section was roughly measured with a measuring stick upon completion of the test. In addition, a top view sketch was made of the shape of the gullies.
For the reinforced test section, the erosion depth below the Geogrid dit not increase at endangered spots, making depth measurements redundant.

8.5.3 Depth measurements at the bare clay tests

After two hours of testing (except for the 1 l/s/m tests), the tests were stopped and the erosion pattern was measured. This was done by vertical measurement of the depth at fixed points using a 1m * 1m grid at the most eroded locations. At a later stage of the tests, when headcut erosion occurred, a denser grid of 0.5 m * 0.5 m was used. This was repeated after the next interval of two hours and upon completion of the simulation.

The erosion depth of the holes and gully's was measured from fixed heights at cross-sections parallel to the crest line. This was done by following method:

- At every metre-line (parallel to the inner crest line) poles were driven into the ground at about 30 cm from the side walls bordering the test section (were no subsidence was expected).
- The elevation of the top of these poles regarding NAP (= reference level: Normal Amsterdam Water level) was determined by means of a professional GPS (Geographic Positioning System) by surveyors of the Waterboard.
- Next a beam of wood (crossing the test section) was placed for each cross-section on top of the poles at each side of the test section.
- The erosion depth was measured vertically extending from the underside of the beam (which level was equal to the top of the poles). For this purpose a piece of plumb was tied up to the measurement cord to allow exact vertical measurements.
- During processing of these data, the scour depths were retrieved and converted to a level with respect to NAP.
- Finally, all measurements were transformed from vertical values to values perpendicular to the slope to determine the actual erosion depths.

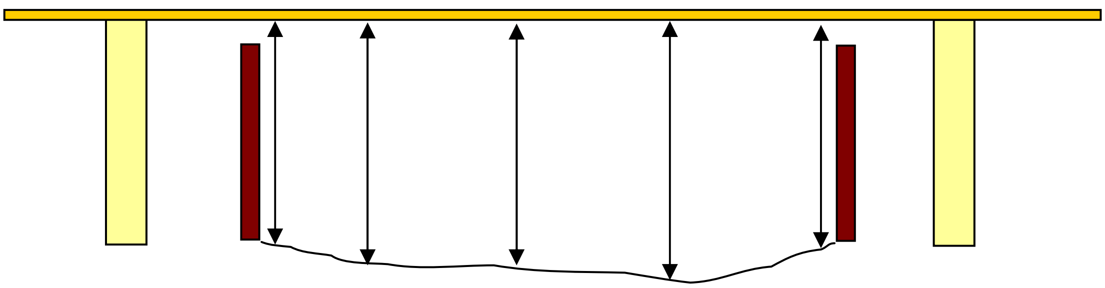


Figure 8-2: Schematic sketch for measuring the erosion depth

9 EROSION TESTS AT PRESENT GRASS COVER

9.1 Introduction

The testing program started with testing of the unreinforced section. This test was basically focused on observation of erosion at the present grass cover caused by the overtopping waves. The results of the test series are shortly discussed hereafter. The close-up photographs are presented such that the flow is directed approximately from the top of the page to the bottom of the page.

Overall photographs taken from the measuring cabin have been added at the end of this Chapter, together with an impression of the overtopping of the largest waves during the tests. These photographs have been taken at the start, at 1/3, at 2/3 and at the end of each test.

It should be noted that all overall photographs for the grass tests show a visual disturbance in the middle of the picture. This is attributed to a physical disturbance in the inner slope of the dyke, where the slope was somewhat steeper than the adjoining slope sections. Although this seems to be evident on the photographs, in reality this deviation was quite limited.

9.2 Test results 0.1 l/s/m (T1)

For a wave overtopping of 0.1 l/s/m the number of overtopping waves is very limited: hence the intermediate periods between the waves were accelerated 10 times. Measurement of the erosion was done after completion of the test (36 minutes).

After finalization of the first test no erosion could be observed. The only observation that could be made was that the bare spots became more clearly visible because of the flattening by the sward of the grass by the waves. These bare spots were already present before testing, but were hidden by the grass sward facing straight up.
Another observation during this first test series was, that the first waves (50 l and 150 l volume) did not even reach the toe of the dyke, as they fully were absorbed into the subsoil at the slope.

9.3 Test results 1 l/s/m (T2)

The 1 l/s/m test was carried out in real-time. Every 2 hours the erosion was observed and documented. To the naked eye no more substantial erosion became visible. The only difference compared to the 0.1 l/s/m test is that the point of attachment between the grass leaves and the root system became more exposed (see Figure 9-1).

Figure 9-1: Detailed photographs of some bare spots after the 1 l/s/m test at the unreinforced test section

9.4 Test results 10 l/s/m (T3)

The overall picture at the end of the subsequent 10 l/s/m test is quite similar to the 1 l/s/m test. Still no clear erosion spots can be observed. The only noticeable difference is near the root system, that at some spots is becoming more exposed.

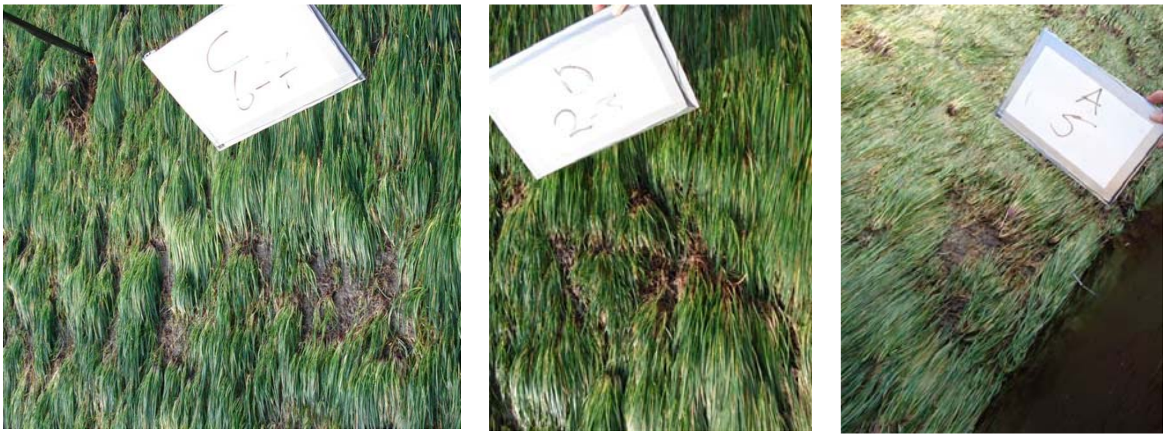


Figure 9-2: Detailed photographs of some bare spots after the 10 l/s/m test at the unreinforced test section

9.5 Test results 20 l/s/m (T4)

After the 20 l/s/m test still no major erosion appeared. The roots of the grass clumps are becoming more exposed, but are still strongly anchored to the underlying clay layer. Some details are shown in Figure 9-3.

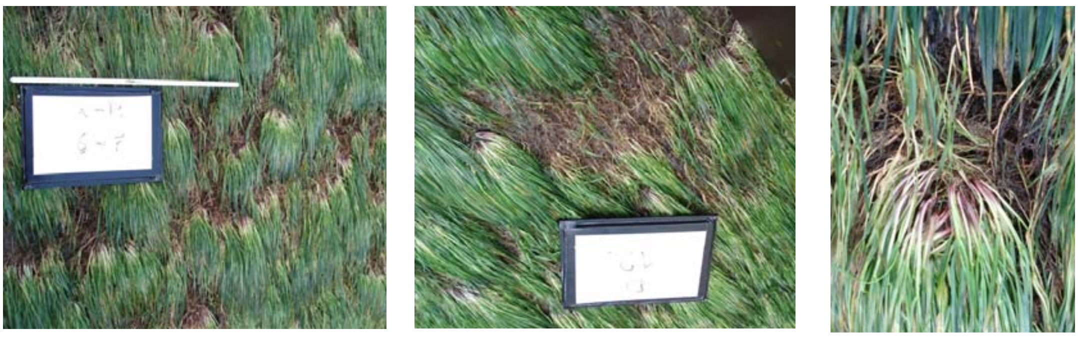


Figure 9-3: Detailed photographs of some bare spots after the 20 l/s/m test at the unreinforced test section

9.6 Test results 30 l/s/m (T5)

After the 30 l/s/m test (the maximum overtopping rate of the initially proposed test program) still no major erosion was observed. Although the roots became more visible at the entire test section, see Figure 9-4, still no grass sods were washed away.

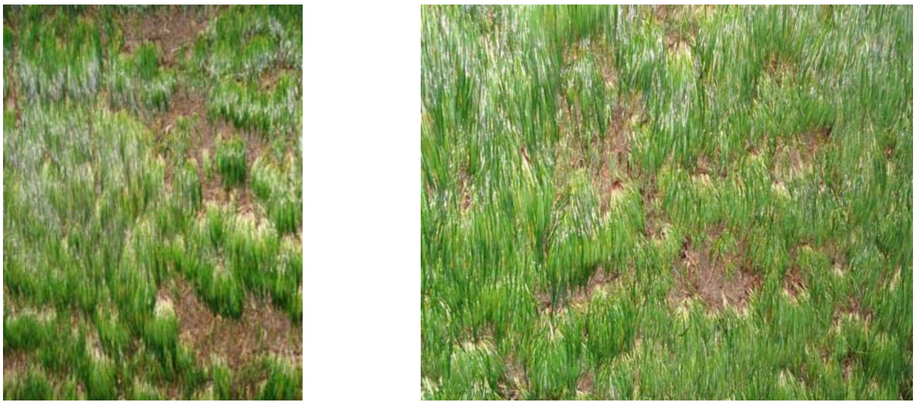


Figure 9-4: Detailed photographs of some bare spots after the 30 l/s/m test at the unreinforced test section

9.7 Test results '50' l/s/m (T7)

The initial test program was extended by a '50' l/s/m test as no major erosion was observed during the 30 l/s/m test. It turned out that grass vegetation could also cope with this load, as can be seen from Figure 9-5. Although the roots became increasingly exposed and partly lost contact with the subsoil, no major erosion could be noticed.

Figure 9-5: Detailed photographs of some bare spots after the '50' l/s/m test at the unreinforced test section

9.8 Test results '50' l/s/m with initial damage (T8)

After introducing artificial damage, according to the method as shown in Section 2.5.2, the '50'l/s/m test was repeated. The initially bare spots with poor grass cover did do not show additional erosion. Around sticks and poles erosion did not increase either.
The artificially damaged location of 1.0 * 1.0 * 0.05 m however increased some 5 cm in depth, whereas the artificially damaged location of 0.4 * 0.4 m * 0.15 m showed no increase in depth at all.

Apart from the increase in depth in the largest artificially damaged location, the most threatening erosion was the gully formation downstream of these both spots, see Figures 9-8 and 9-9. Growth of the two gullies was concentrated along the length of the slope, rather than growth in width and depth, and was preceded by local removal of the grass sods. Typical depth and width dimensions of the gullies at the end of the test were: 0.2 to 0.3 m wide and 0.1 to 0.2 m deep. A longitudinal section of this gully erosion at the end of the test is shown in Figure 9-6 and the length growth can be seen in Figure 9-7.
Figures 9-8 and 9-9 clearly show the typical flow concentration towards the gullies during depletion phases of the overtopping flow.

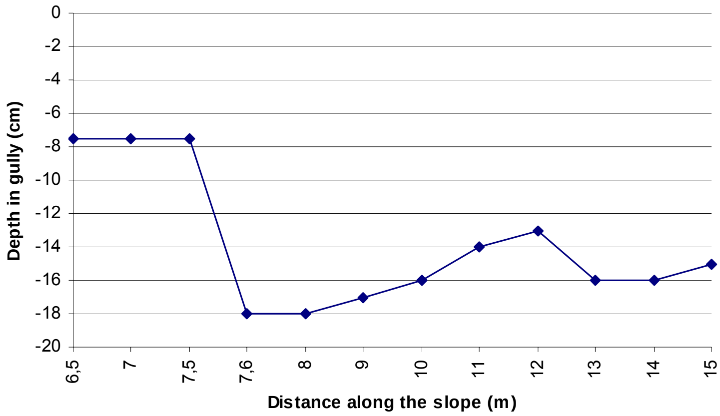


Figure 9-6: Measured erosion depths in the gully behind the artificially damaged spot (1m x 1m x 0,05m) at the unreinforced test section: the location where the gully starts is at 6.5 m

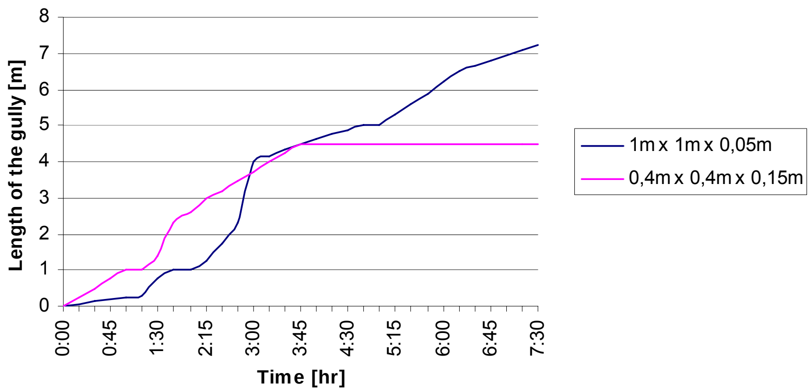


Figure 9-7: Growth of the gully length in time behind the two artificially damaged spots at the unreinforced grass test section

Figure 9-8: Start of the gully formation downstream of the damaged areas

Figure 9-9: Typical flow concentration at the gullies

T1

start | at 1/3 | at 2/3 | after

Figure 9-10: Situation during overtopping of 0,1 l/s/m. The biggest wave shown here had a volume of 0.7 m^3 per m width

T2

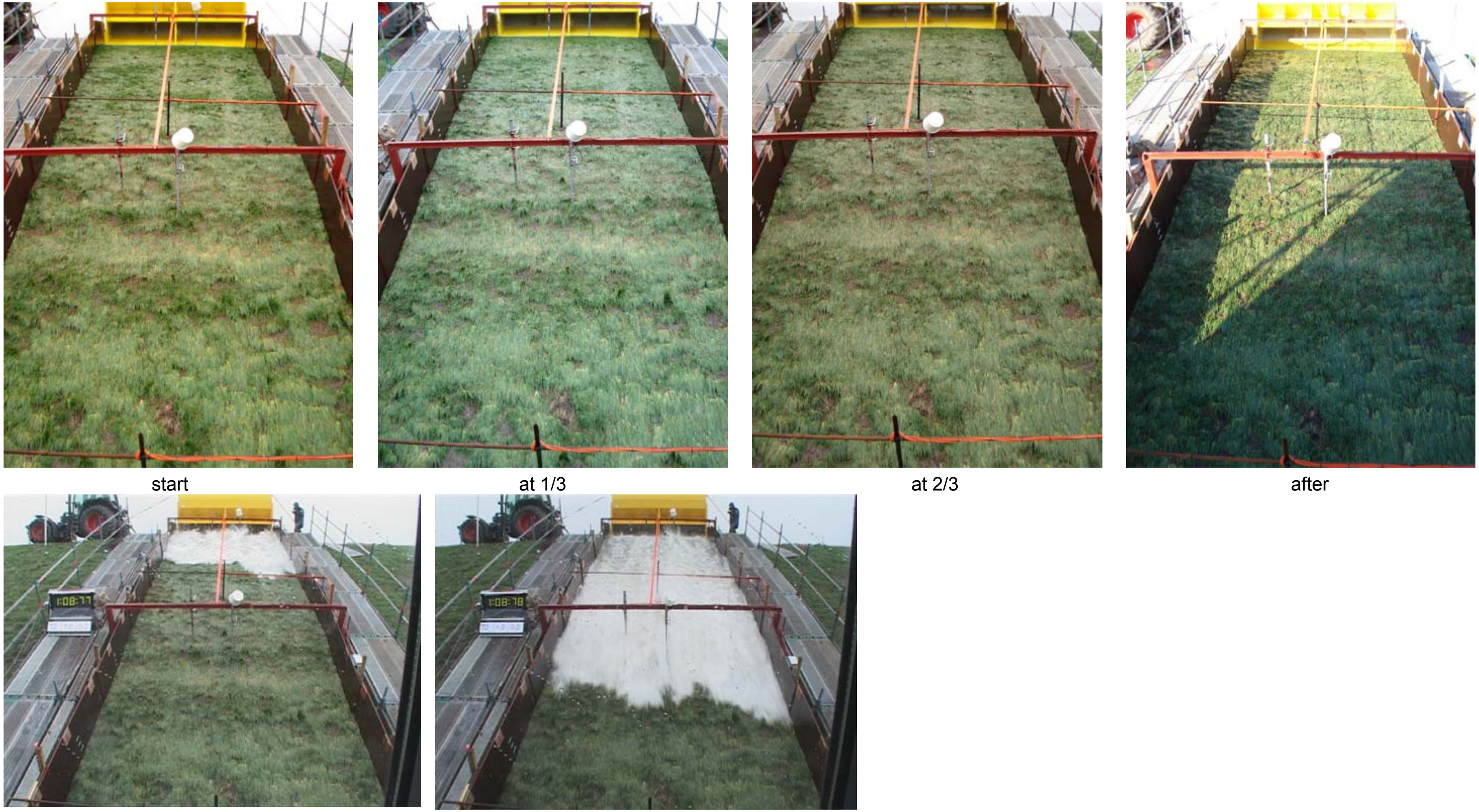


Figure 9-11: Situation during overtopping of 1 l/s/m. The biggest wave shown here had a volume of 1 m^3 per m width

T3

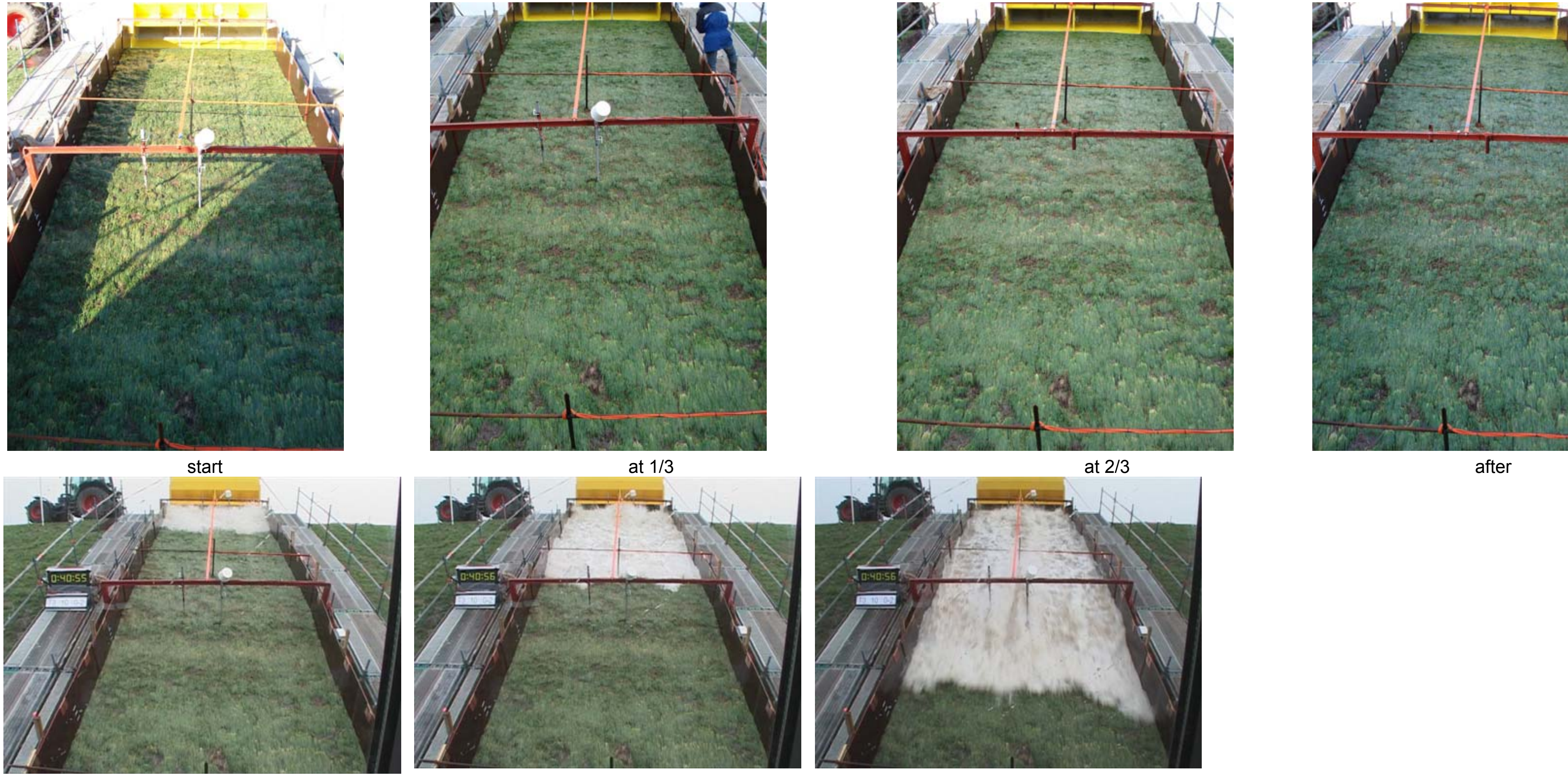


Figure 9-12: Situation during overtopping of 10 l/s/m. The biggest wave shown here had a volume of 2.5 m^3 per m width

T4

start at 1/3 at 2/3 after

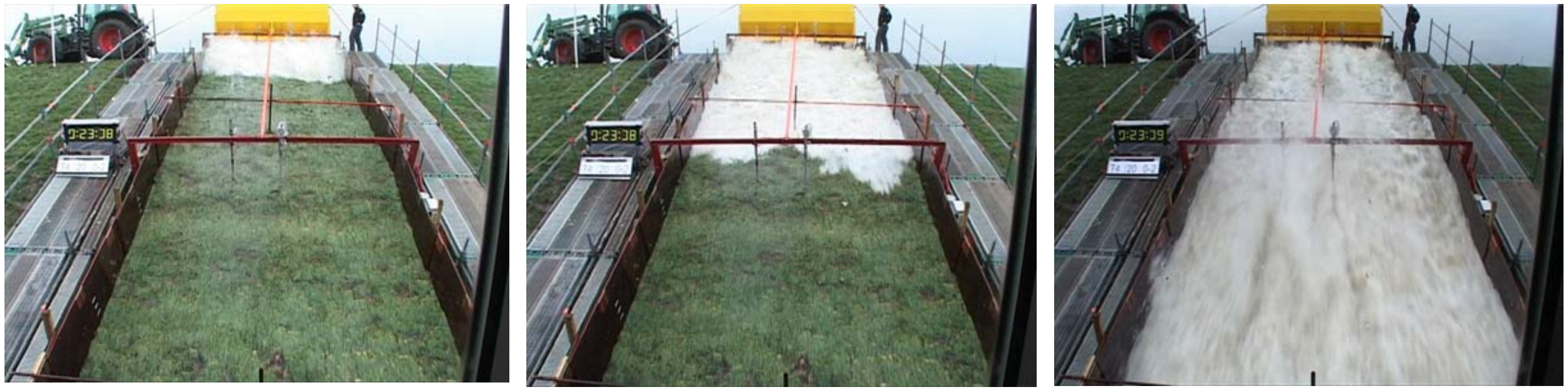
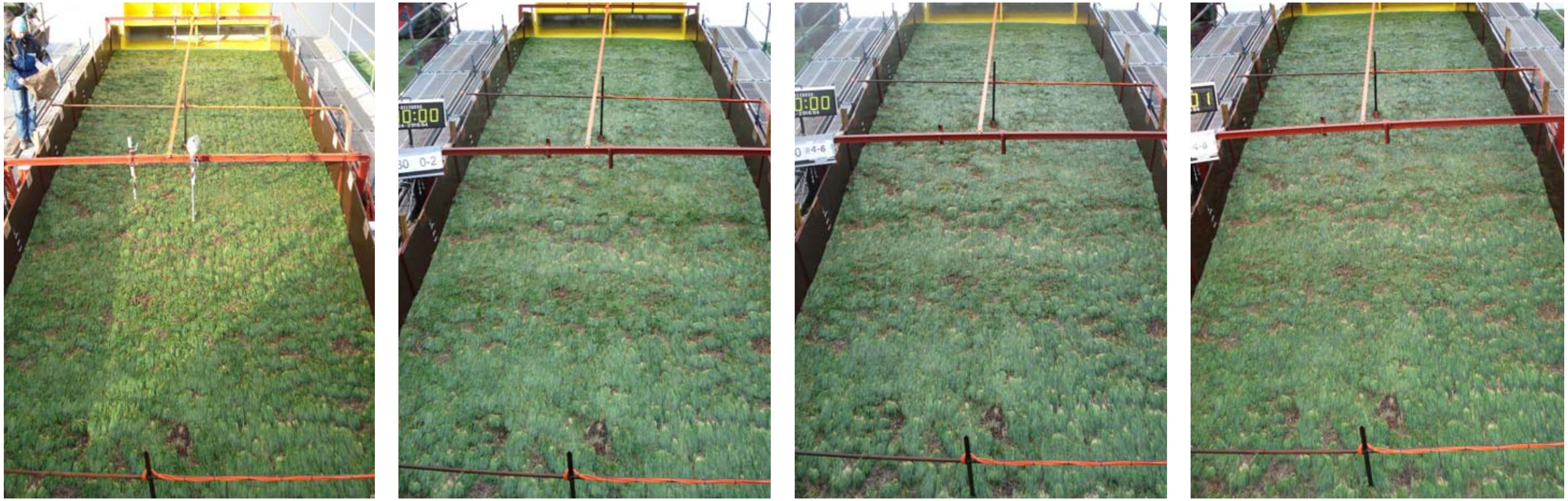


Figure 9-13: Situation during overtopping of 20 l/s/m. The biggest wave shown here had a volume of 3.5 m^3 per m width.

T5



start | at 1/3 | at 2/3 | after

Figure 9-14: Situation during overtopping of 30 l/s/m. The biggest wave had a volume of 3.5 m^3 per m width (see Figure 9-10)

T7

start | at 1/3 | at 2/3 | after

Figure 9-15: Situation during overtopping of ‘50’ l/s/m. The biggest wave had a volume of 3.5 m^3 per m width (see Figure 9-10)

T8

start at 1/3 at 2/3 after

Figure 9-16: Situation during overtopping of ‘50’ l/s/m with initial damage. The biggest wave had a volume of 3.5 m^3 per m width (see Figure 9-10)

10 EROSION TESTS AT REINFORCED GRASS COVER

10.1 Introduction

The results of the test series with the reinforced grass cover are shortly discussed hereafter. The close-up photographs are oriented such that the flow is directed approximately from the top of the page to the bottom of the page.

As mentioned before the overall grass coverage at the reinforced section was rather poor. Initially quite a number of bare spots (with a typical diameter of 0.1 m) were visible here, on average larger than the bare spots at the unreinforced test section.

Overall photographs taken from the measuring cabin have been added to the end of this chapter, together with an impression of the overtopping of the largest waves during the tests. These photographs have been taken at the start, at 1/3, at 2/3 and at the end of each test.

It should be noted that all overall photographs for the grass tests show a visual disturbance in the middle of the picture. This is attributed to a physical disturbance in the inner slope of the dyke, where the slope was somewhat steeper than the adjoining slope sections. Although this seems to be evident on the photographs, in reality this deviation was quite limited.

10.2 Test results 0.1 l/s/m

For the 0.1 l/s/m overtopping test, the number of overtopping waves is very limited so erosion was not to be expected based on the experience from the previous tests. Hence, the intermediate periods between the waves were accelerated 10 times. Measurement of the erosion was done after completion of the test (36 minutes).This test was not recorded as no erosion was expected and, indeed, did not occur. The only observation that could be made was that the bare spots became slightly more exposed.

10.3 Test results 1 l/s/m (V1)

To save one full testing day the 1 l/s/m test was shortened as well by accelerating the intermediate periods between the waves by a factor 10. At the end of the test grass sods had become somewhat more exposed by the adjustment of the grass orientation to the flow. Loose materials (e.g. felt) being washed away, the locally sparse grass cover exposed small bare spots.

Figure 10-1: Detailed photographs of the grass cover after the 1 l/s/m test at the reinforced test section

10.4 Test results 10 l/s/m (V2)

The erosion pattern at the end of the 10 l/s/m test is quite similar to the 1 l/s/m test. The root system is however becoming a bit more exposed because of the additional loose clay particles that are being washed away.

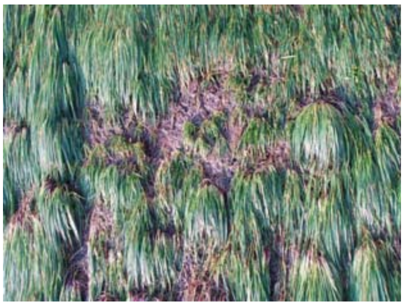


Figure 10-2: Detailed photographs of some bare spots after the 10 l/s/m test at the reinforced test section

10.5 Test results 20 l/s/m (V3)

The bare spots in between the grass sods have been eroded several centimeters. Although the Geogrid is still not exposed, it is clearly visible that at some locations the clay has been nearly washed out up to the location of the Geogrid. The root system of the grass has clearly become more uncovered. No grass sods are, however, washed down by the wave load.


Figure 10-3: Detailed photographs of some bare spots after the 20 l/s/m test at the reinforced test section

10.6 Test results 30 l/s/m (V4)

At the end of the 30 l/s/m test the Geogrid became visible at several places (5 locations). From the observations it is obvious that the root system is well-intertwined with the Geogrid, as can be seen in Figure 10-4. In this way the Geogrid prevents the grass sods from being washed away. For these typical locations the erosion depth is in the order of some centimeters.

Figure 10-4: Detailed photographs of some bare spots after the 30 l/s/m test at the reinforced test section

10.7 Test results ‘50’ l/s/m (V5)

After the ‘50’ l/s/m test the erosion has increased. The Geogrid has become visible at more spots and at the same time the spots that were already eroded in the previous test (30 l/s/m) have grown further. However, no further growth in depth could be observed because of the presence of the Geogrid protecting the underlaying clay. At a few locations (about 3) some grass clumps have been washed away, as indicated in Figure 10-5.

Figure 10-5: Detailed photographs of some bare spots after the ‘50’ l/s/m test at the reinforced test section

10.8 Test results ‘50’ l/s/m with initial erosion (V6)

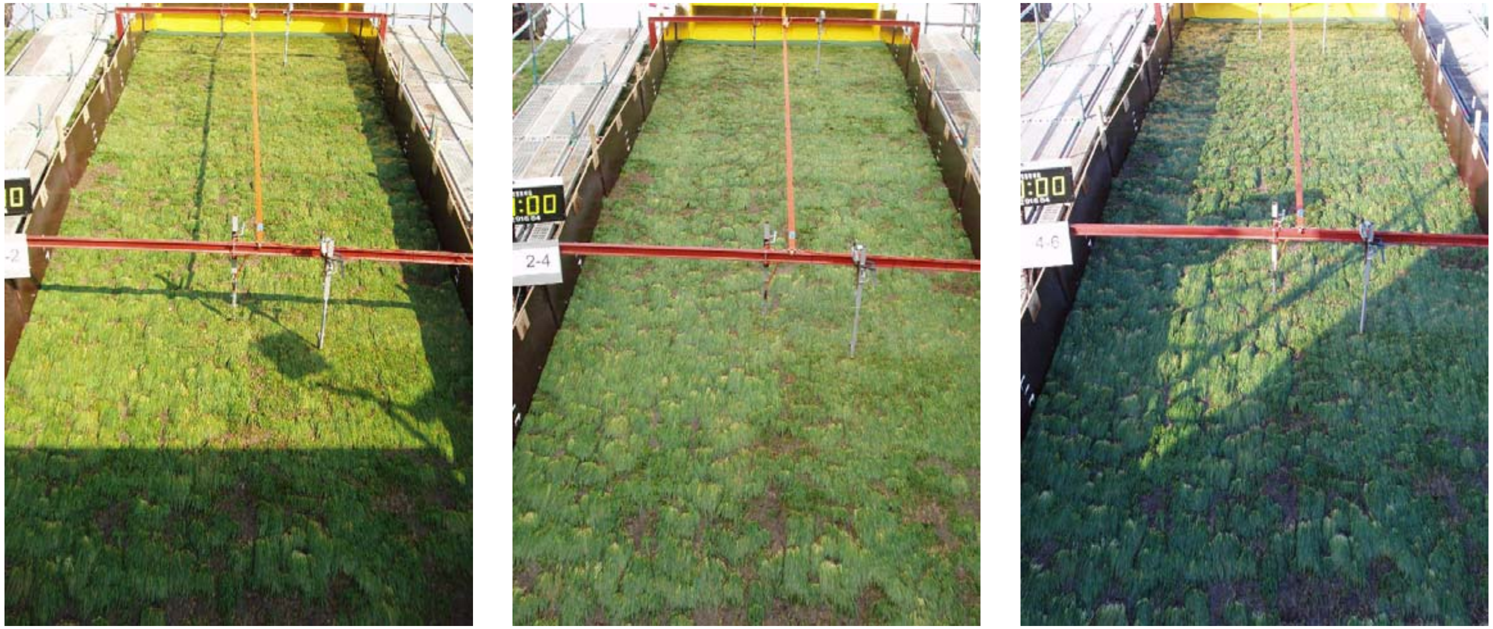
The artificially damaged locations of 0.1 * 0.1 m, as well as initially bare spots with poor grass cover, did do not show additional erosion. Around sticks and poles, erosion did not increase either.

The artificially damaged location of 1.0 * 1.0 * 0.05 m did not increase in depth, obviously due to the presence of the Geogrid. In addition, gully erosion was very moderate, as compared to the unreinforced grass. In fact, locally the grass cover was partly removed, but the clay was not eroded noticeable. In addition, there was no sign of progressive erosion as well. Figure 10-6 shows that the grass sods at the rim of the removed grass layer could hold on well to the Geogrid by the root anchoring to the Geogrid. The damaged location of 0.4 * 0.4 m * 0.15 m showed no further deepening. Gully erosion did hardly occur downstream of the 0.4 * 0.4 m spot. It could be observed that the Geogrid at the downstream side curled upwards to some extent, which added to the protection of the downstream grass cover. All in all, erosion development was much less than with the unreinforced grass.

Figure 10-6: Anchoring of grass roots to the Geogrid near the artificially removed grass cover

The progression of the erosion can be seen from Figure 10-11.

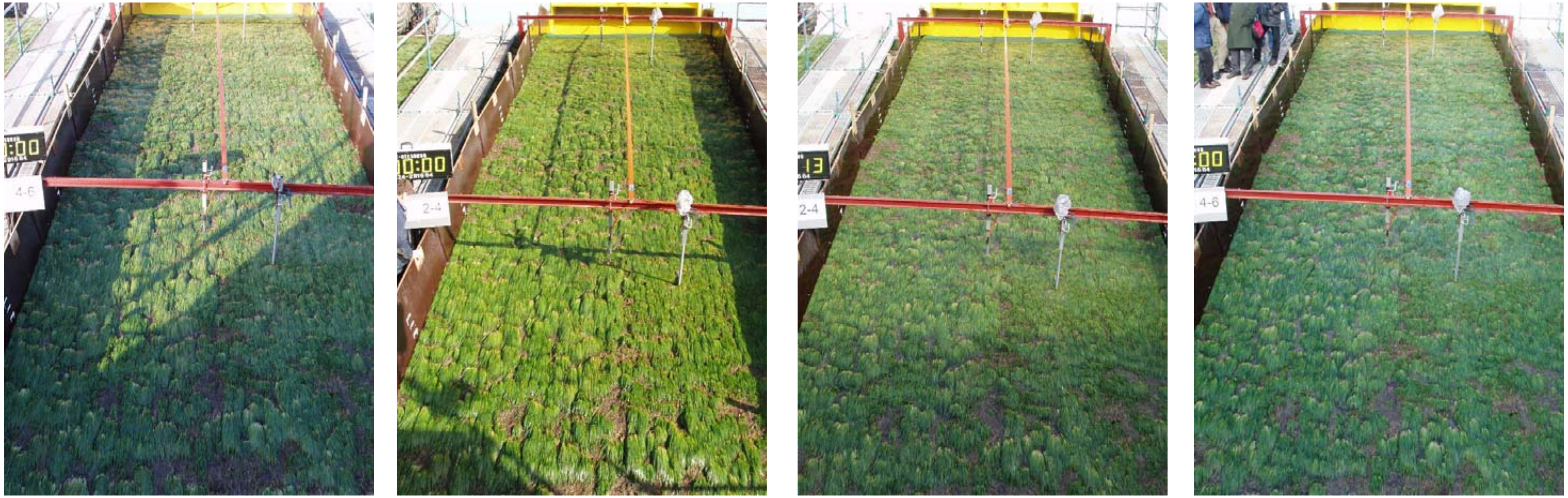
V2



start at 1/3 at 2/3 after

Figure 10-7: Situation during overtopping of 10 l/s/m. The biggest wave shown here had a volume of 2.5 m^3 per m width

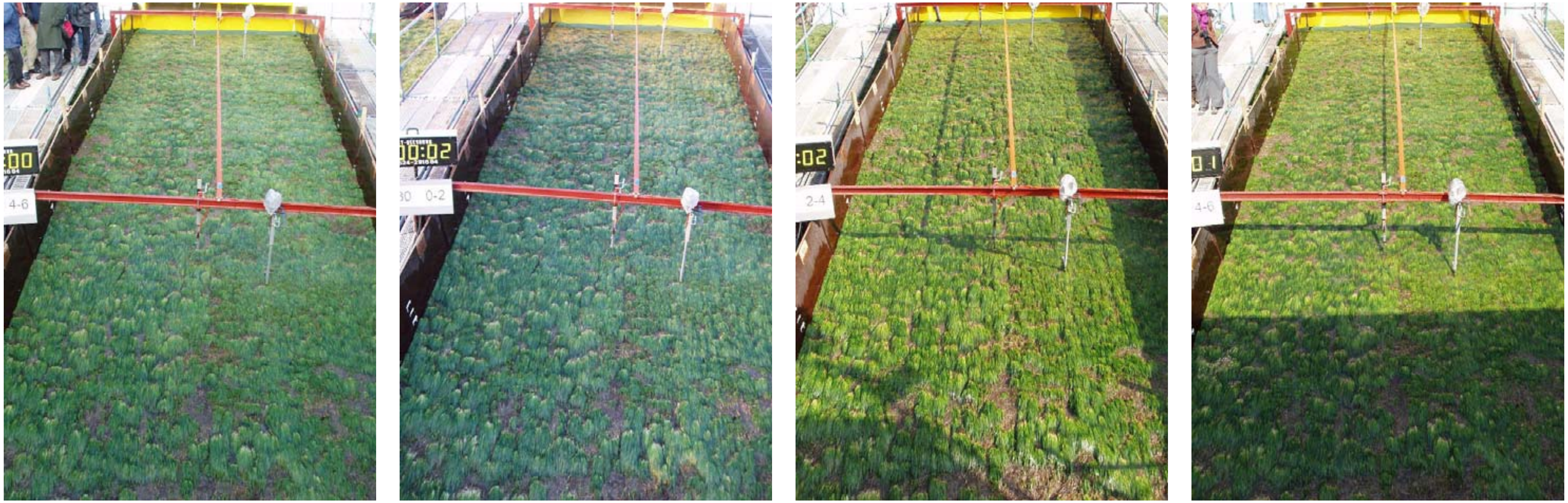
V3



start at 1/3 at 2/3 after

Figure 10-8: Situation during overtopping of 20 l/s/m. The biggest wave shown here had a volume of 3.5 m^3 per m width

V4



start | at 1/3 | at 2/3 | after

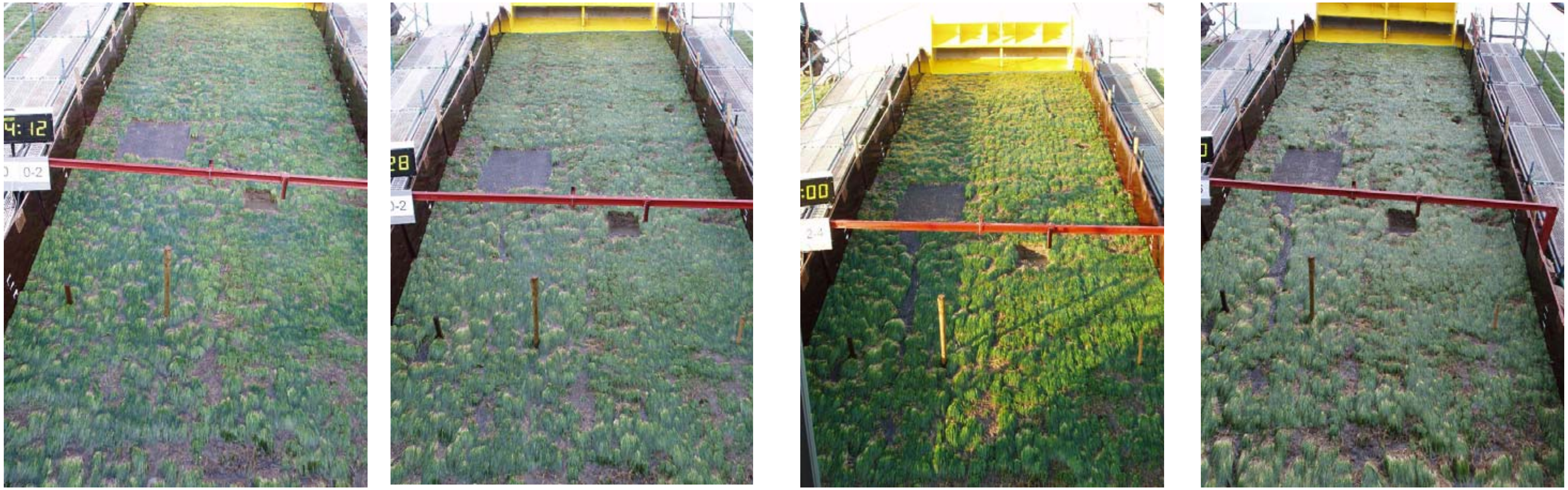
Figure 10-9: Situation during overtopping of 30 l/s/m. The biggest waves had a volume of 3.5 m^3 per m width (see Figure 10-8)

V5

start at 1/3 at 2/3 after

Figure 10-10: Situation during overtopping of '50' l/s/m. The biggest waves had a volume of 3.5 m^3 per m width (see Figure 10-8)

V6



start | at 1/3 | at 2/3 | after

Figure 10-11: Situation during overtopping of ‘50’ l/s/m with initial damage. The biggest waves had a volume of 3.5 m^3 per m width (see Figure 10-8)

11 EROSION TESTS AT BARE CLAY

11.1 Introduction

An extra test has been performed on bare clay, for which the upper 20 cm of the grass sod had been removed. Hence, this test allows for a comparison of the erosion performance of the bare clay layer with the integrated system of grass cover and clay layer. Under SBW, prediction models have been developed for the behaviour of the inner slope by wave overtopping. The results of the tests will be used to validate or modify these prediction models (for a preliminary analysis see Chapter 12).

Overall photographs taken from the measuring cabin at the start, at 1/3, at 2/3 and at the end of each test have been added to the end of this Chapter, together with an impression of the overtopping of the largest waves during the tests.

11.2 Preparations

The removal of the grass cover was done by using two cranes with a digging bucket that was used as a ‘knife’ to cut the grass cover. Full removal of the grass cover at the inner slope was done by placing one crane on the dykes crest and the other crane at the toe of the inner slope. By working both from above and down the slope it was not necessary to drive on the slope and thus disturb the under layer. The digging bucket arms of the cranes were just long enough to reach to each other. To make sure that only the upper 20 cm was removed, measurement tape was used to determine the actual depth based on which proper instructions could be given to the driver of the crane.

Figure 11-1: Beginning of removal of the grass cover by deployment of two cranes and protection of the bare clay layer against drying out before testing

After removal of the grass cover a plastic sheet was used to cover the slope and prevent the clay layer from drying out. At the toe of the inner slope so called ‘straw mats’ were used. These mats are being used by the Water Board as emergency measure, and were pinned to the clay under-layer to serve as protection against undermining of the service road. At the same time, the Water Board could gain more experience with this natural product . The installation and protective action of these straw mats are not part of this study and are not further elaborated in this study for that reason.

Accelerated testing with a wave overtopping rate of 0.1 l/s/m, as had been done at the grassed test sections, was started at the clay section as well. At the grassed test sections, this was done without measurement of erosion, as erosion was not noticeably and the test with 0.1 l/s/m was merely done for reproduction purposes (infiltration). However, at the clay section erosion was not insignificant, so this test was canceled immediately after the first minutes. Obviously, even for the smallest overtopping rates, measurements in real time would be needed. A test with 0.1 l/s/m in real time was not possible as the pump could not provide such a small discharge rate with sufficient accuracy. Therefore the overtopping rate was increased to 1 l/s/m, which test was monitored further in real-time.

The initial short-duration wave overtopping at 0.1 l/s/m already showed a 'roughening' of the surface of the clay with local small erosion pits, as shown in Figure 11-2.

Figure 11-2: Impression of starting erosion of bare clay surface after interrupted 0.1 l/s/m test

11.3 Test results 1 l/s/m (K1)

As could be expected from the interrupted 0.1 l/s/m test, after a couple of waves it became clearly visible that the bare clay was sensitive to surface erosion. In addition, it could be observed that entire clay layer was perforated by worm holes and small fissures, which may have added to the sensitivity for erosion.

Erosion was most severe at the obvious weakest spots of the clay surface somewhere halfway the slope, see Figure 11-3 and 11-7. This weak area probably originated from two factors.
1) The removal of the grass layer was realized by two cranes, a heavy crane working from the toe of the dyke and a smaller crane working from the crest. The reach of both cranes was limited and they did hardly overlap. Where the reach was maximum, they scraped the grass cover, while loosening the subsoil to some extent. This especially applied to the heavy crane at the toe of the dyke when it had its longest reach.
2) The loosened subsoil matched the area with the small 'step' (steeper slope) in the inner slope profile.
We think that weak spots will generally exist in many field situations, e.g. caused by preceded sliding of the grass layer down the slope, by damaged spots and so on. Moreover, even in

idealistic flat conditions lightly weaker spots will exist that will attract stronger erosion as can be seen from laboratory tests.

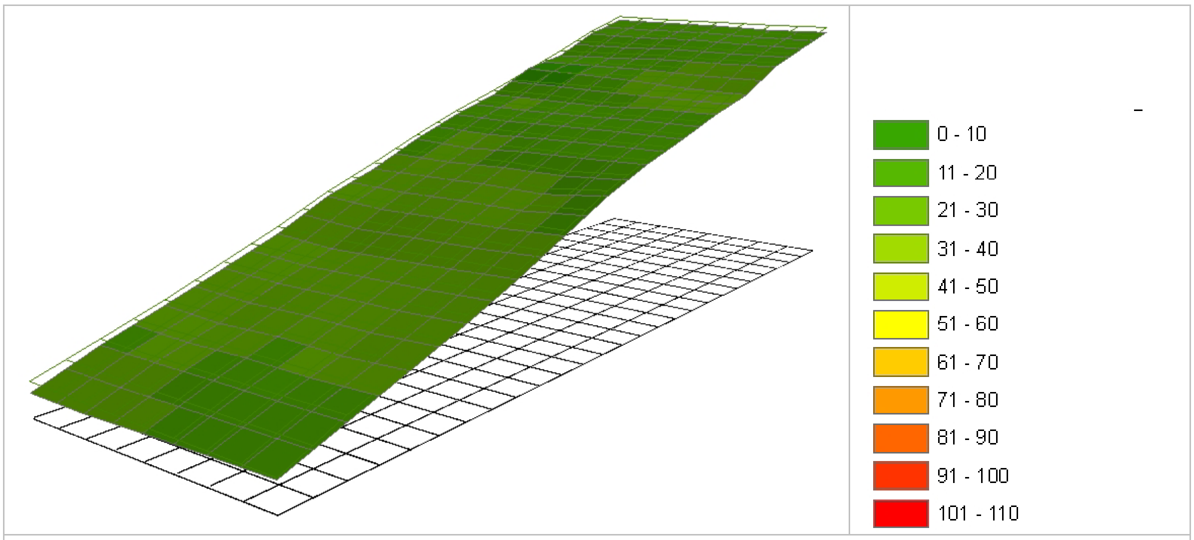


Figure 11-3: 3-D presentation of erosion pattern (depth in centimetres) after the 1 l/s/m test , compared to the initial situation

11.4 Test results 5 l/s/m (K2)

For the 5 l/s/m test, strong and ongoing erosion took place predominantly at the initial erosion location, leading to the development of two distinct scour holes with nearly vertical cliffs.

Erosion occurred almost continuously: even long after passage of the maximum flow of the wave tongues, flow converged strongly towards the cliff areas. Moreover, this flow was supplied by seepage flow, see Figure 11-6. The formation of steep cliffs with a nearly horizontal base matches very well with observations from literature on flow over bare clay slopes, starting at the weakest points at the slope and intensifying there. The cliff progresses towards the crest, causing 'head-cut' erosion. This is caused by the occurrence of vertical fissures near the crest of the cliff and gradual sliding and subsequent removal of the deposited toe material.

At the upper part of the slope, surface erosion could be noticed. A striking feature here was the presence of the remains of the deep grass root system which may have averted localized erosion (Figure 11-4). At the top of the slope erosion was more equally spread and remained limited.

Figure 11-4: Remains of the root system becoming visible after some surface erosion

The progression of the erosion can clearly be seen from the 3D-processed depth measurement data in Figure 11-6.

11.5 Test results 10 l/s/m (K3)

Initially erosion at the western cliff dominated but was overtaken after some time by the eastern cliff. At the end of the tests, after a storm with an overtopping rate of 10 l/s/m, two marked scour holes with a steep upstream cliffs had developed starting about halfway from the inner slope of the dyke. The largest hole was about 1 m deep, see Figures 11-5 and 11-10.

Retrogressive ‘head-cut’ erosion of the cliffs took place, while maintaining the nearly vertical cliff slope, going along with a nearly horizontal erosion base. The velocity of this head-cut erosion was about 0.25 m per hour (measured horizontally).

The final surface erosion at the upper part of the slope after the 10 l/s/m test was within 5 and 10 centimeters.

In Figure 11-5 the impressive extent of scouring is clearly visible, after dewatering and dismantling of the test arrangement.

Figure 11-5: Deep scour holes after wave overtopping of 10 l/s/m

The progression of the erosion can clearly be seen from the processed depth measurement data in Figures 11-6 and 11-7, as well as from the photographs in Figures 11-9 and 11-10.

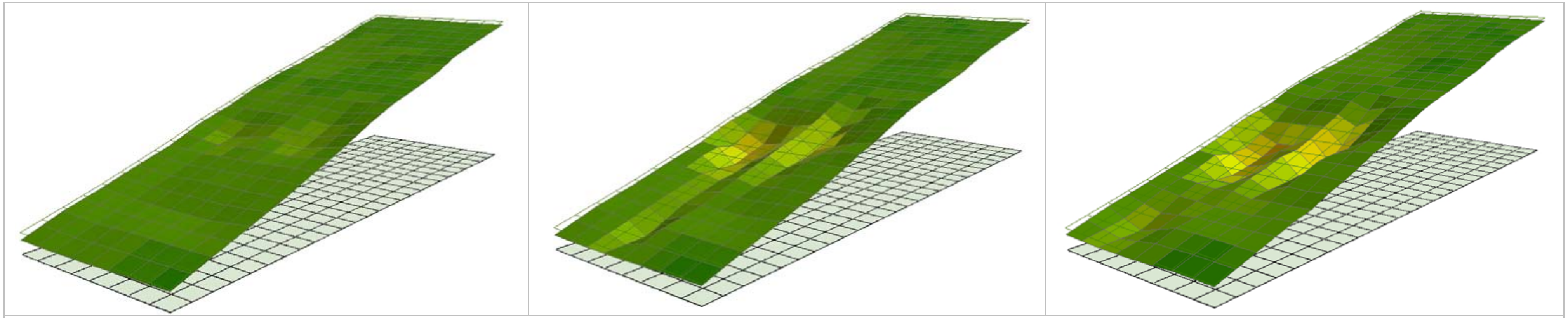


Figure 11-6: 3-D presentation of erosion pattern compared to the initial situation during the 5 l/s/m test after 2, 4 and 6 hours respectively (for a legend see Figure 11-3)

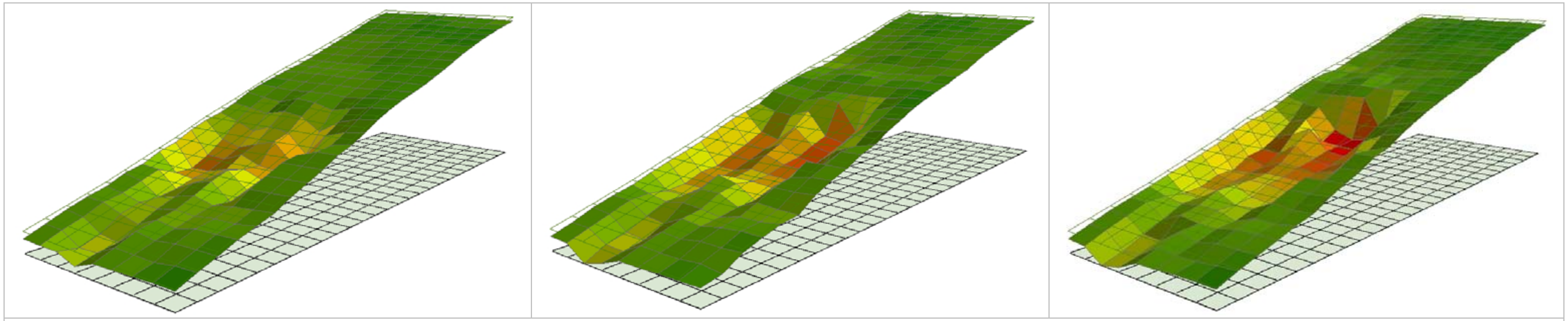


Figure 11-7: 3-D presentation of erosion pattern compared to the initial situation during the 10 l/s/m test after 2, 4 and 6 hours respectively (for a legend see Figure 11-3)

1

before | at 1/3 | at 2/3 | after

Figure 11-8: Situation during overtopping of 1 l/s/m. The biggest wave shown here had a volume of 1 m^3 per m width

K2

Figure 11-9: Situation during overtopping of 5 l/s/m. The biggest wave shown here had a volume of 2.0 m^3 per m width

K3

before at 1/3 at 2/3 after

Figure 11-10: Situation during overtopping of 10 l/s/m. The biggest wave shown here had a volume of 2.5 m^3 per m width

12 PRELIMINARY ANALYSIS OF TEST RESULTS

12.1 Introduction and remarks

Hereafter, a preliminary analysis of the tests results is given against the background of earlier experience and expectations. This analysis needs to be extended more in-depth within another framework, such as the SBW program. However, we think it is useful to give our first impressions here, as otherwise the relevance of some observations may not be noticed and be lost at a later stage of analysis.

As to promote a correct in-depth analysis at a later stage, we add the following remarks here:

- In present wave overtopping standards the accessibility of the crest during the design storm is implicitly included as to allow proper monitoring of the dyke condition. When overtopping standards would be further increased in future, adequate monitoring should be possible in a different way or the increased overtopping resiliency should be fully trustworthy.
- The wave overtopping tests as presented in this report are probably not representative for slip failure of a larger part of the grass and clay layer caused by infiltration and sliding. This is due to the massive clay core of the dyke at the test site, as well as to the restricted width (4 m) of the test sections. In addition, other failure mechanics have not been investigated in these tests, such as internal erosion resulting from infiltration, piping and heave, and macro instability. In contrast, shallow slip failure of the turf layer (turf sliding) is probably reproduced properly by the tests. When overtopping rates would be increased in future, other failure mechanisms may become more critical. We think this warning should be taken very serious as some researchers indicate even for current practices that other failure mechanisms than surface erosion may sometimes be dominant at a clayey dyke. Even more so, when the surface erosion would be tackled by a SGR, other failure mechanisms may become more critical such as sliding by infiltration and internal erosion. However, the SGR may have a mitigating effect at these failure modes as well, but this should be substantiated by adequate testing. It should be remarked that such failure phenomena may be aggravated by intensified infiltration and accompanying pressure build-up. The latter effect has been observed in the field ('Lisse-effect', as mentioned by Ronald van Etten).

Another remark to be placed here is the extremely turbulent flow and high aeration rate of the overtopping tongues, as could be observed and which is evident from the photographs in this report. Aeration measurements of comparable situations at spillways and overtopping banks indicate 30 to 50 percent of air entrained in the flow, even at lower overtopping rates than applied here. This may lead to an equivalent increase in flow depth, which should be added to theoretically predicted flow depths. Another phenomenon that is associated with high levels of aeration is the high degree of turbulence and the 'erratic' flow structure that is difficult to measure (see Chapter 7). These are aspects that need to be taken into account when measuring and interpreting flow velocities and flow depths.

It is remarked here that the analysis in this Chapter was performed by Gert Jan Akkerman of Royal Haskoning. We greatly acknowledge the valuable remarks made by Jan Willem Seijffert during a lively discussion on invitation of Royal Haskoning.

12.2 Comparison with earlier investigations and experience of unreinforced grass

The major results of the tests on the unreinforced grass section can be summarized as follows:

1. No significant erosion, even for overtopping events of ‘50’ l/s/m.
2. Artificially damaged locations of 0.1 * 0.1 m, as well as initially bare spots with poor grass cover, did do not show additional erosion for abovementioned conditions. This may be attributed by the sheltering action that the surrounding protruding grass sods offered to the bare clay. Another possibility is the presence of cohesion increasing substances in the upper part of the clay layer by e.g. excrements of worms, beetles and cohesive ‘glues’ from vegetation.
3. Erosion did not increase around sticks and poles either.
4. The artificially damaged location of 1.0 * 1.0 * 0.05 m scoured up to some 5 centimeters, whereas the artificially damaged location of 0.4 * 0.4 m * 0.15 m showed no scouring at all.
5. Apart from the increase in depth in the largest artificially damaged locations mentioned before, the most threatening erosion progression seems to be the gully formation downstream of these spots, associated with local removal of the grass cover. Such a gully formation is self-expanding, as it attracts an increasing amount of water during formation, which can be easily seen from Figure 9-9. Growth of the gullies was concentrated in the length dimension (down the slope), rather than growth in width and depth, although the length growth of the right gully stopped after some time. Typical depth en width dimensions at the end of the test were: 0.2 to 0.3 m width and 0.1 to 0.2 m deep.
6. Shallow slip failure of the grass cover (e.g. ‘turf sliding’ downstream of the crest) was not observed; in addition, horizontal fissures near the crest or inner slope have not been observed.

It should be remarked that the grass cover showed some bare spots of typically one or some decimeters in circumference, prior to the tests. During the tests, these spots did not lead to such an erosion that ongoing erosion could be observed.
A typical longitudinal section of the gully erosion is shown in Figure 9-6 and the length growth in Figure 9-7.
These findings can be evaluated against the background of two recent investigations, as outlined hereafter.

Comparison with surface erosion model for the grass cover by Van den Bos (2006), denoted ‘Bare Spots Model’ (abbreviated EPM – Erosiegevoelige Plekken Model- in Dutch)
Van den Bos supposed that the limiting strength of a grassed slope would be by bare spots that can be present due to erosion, poor grass coverage etcetera. Typically, he supposed these spots to be about 1 decimeter in flow direction. Next he predicted the development of the scour hole in the clay substrate, based on a scour formula for 3-dimensional scour, taking into account cohesion and a typical length scale λ of 0.2 m (being the approximate grass cover thickness). After calibration to data on (permanent) overflow tests on a real dyke (1970), he arrived at the following formula (for an erosion depth y_m smaller than the thickness of the grass sods:

$$\frac{y_m}{\lambda_m} = \left[\frac{(\alpha U_0 - U_c)^2}{c\Delta^{1.7}}\right] t \qquad (12.1)$$

with:

y_m	=	erosion depth	[m]
λ_m	=	characteristic length scale	[m]
α	=	erosion intensity coefficient = 3	[-]
U_0	=	depth-averaged flow velocity	[m/s]

U_c	=	depth-averaged critical flow velocity	[m/s]
c	=	1.3*10^6	[m^2/s]
Δ	=	relative density	[-]
t	=	time	[s]

The typical erosion depth for failure has been set at 0.1 m, for which it is assumed that the surrounding grass sods will lose their stability (Van den Bos acknowledged however that this may be a conservative assumption).
Equation 12.1 can be rewritten as:

$$U_0 = \frac{1}{\alpha}\left(\sqrt{\frac{y_m c \Delta^{1.7}}{\lambda_m t}} + U_c\right) \tag{12.2}$$

For a typical erosion depth of e.g. 0.1 m, this results in a graph as shown in Figure 12-1. This Figure shows a trend that is comparable to the stability curves from CIRIA (Hewlett et al., 1987).

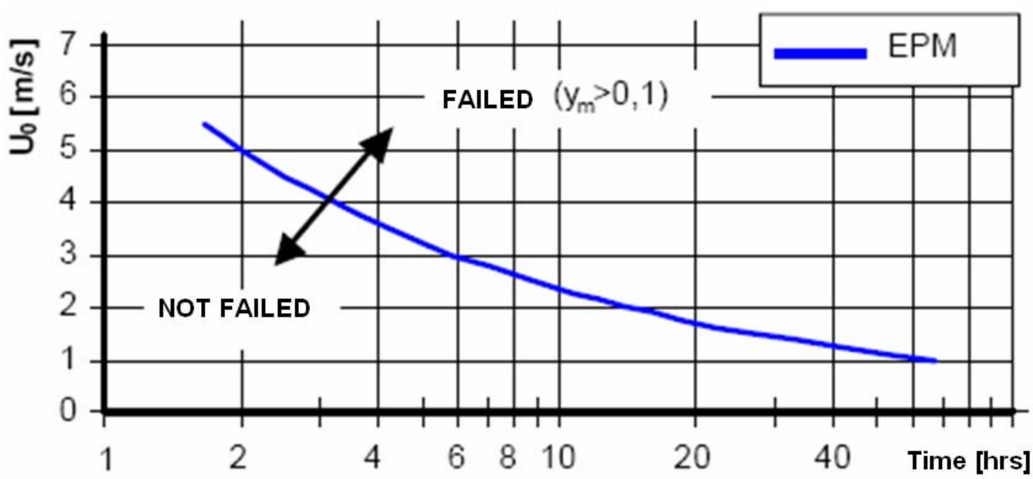


Figure 12-1: Allowable flow velocity as a function of duration (horizontal axes in hours) for an erosion depth of 0.1 m) for steady flow conditions

Van den Bos investigated also the loads by wave overtopping. Assuming equilibrium flow at the inner slope, he translated the instationary maximum wave overtopping flow velocity to a characteristic permanent velocity per overtopping event with the same erosive action according to:

$$U_{char} = \frac{1}{\sqrt{2}} U_{\max} \tag{12.3}$$

with:

U_{char}	=	characteristic permanent flow velocity	[m/s]
$U_{\max}$	=	maximum flow velocity per overtopping event	[m/s]

This leads to a prediction model for 'critical' erosion of a grass cover (erosion depth 0.1 m), in which in (12.1) $(\alpha U_0 - U_c)^2$ can be replaced by $\sum(0.7\,\alpha U_{\max} - U_c)^2$ for all overtopping events.

With the above, in the recent SBW report 05i028 of 15 March 2007 (SBW, 2007), the potential erosion has been computed for e.g. 0.1 l/s/m, 1 l/s/m, 10l/s/m and 30 l/s/m, as has been investigated in the present field tests at the Groningen sea dyke.
This has been done for the following assumptions:

0.1 l/s/m: 9 characteristic waves per 6-hour storm duration
1 l/s/m: 125 characteristic waves per 6-hour storm duration
10 l/s/m: 867 characteristic waves per 6-hour storm duration
30 l/s/m: 1324 characteristic waves per 6-hour storm duration

The following characteristic load parameters have been defined:

maximum flow velocity:	U_{max} = 1 to 5 m/s
wave tongue thickness:	0.02 to 0.08 m
duration:	1 to 5 s

Two situations have been discerned:
A: a situation with a good grass cover for which U_c = 3.4 m/s (taking root cohesion into account)
B: a situation with a grass cover with bare spots for which U_c = 0.5 m/s

This leads to the following erosion depths after a 6-hour storm:

average overtopping discharge	y_m in situation A (mm)	y_m in situation B (mm)
0.1 l/s/m	0	0
1 l/s/m	0	0-8
10 l/s/m	0-8	p.m.
30 l/s/m	0-13	p.m.

This prediction shows that with a good grass cover, the grass cover will not fail at overtopping flows of up to 30 l/s/m.
Van den Bos also concluded for two case studies for other Dutch sea dykes, that they would be able to cope with overtopping rates up to 50 l/s/m, even for ‘moderate’ grass covers.

<u>Preliminary conclusions based on the observations</u>
The predictions in the above are in line with the observed absence of significant erosion in the unreinforced grass cover for overtopping rates of well over 30 l/s/m.
Yet, the observations lead to some remarks:

1. For bare spots, the sheltering effect of the grass sods is large, which may lead to much higher critical velocities for the clay at the bare spots than 0.5 m/s mentioned above.
2. At highest overtopping rates, the actual maximum flow velocities and flow depths are larger than assumed in the above: characteristic maximum flow velocities are approximately 7 m/s, rather than about 5 m/s as assumed in the SBW report and maximum depths are up to some decimeters, rather than some 0.1 m.
3. The failure mode of gully formation progressing downstream from a bare spot seems to be more important than localized erosion at the bare spots.
4. The importance of the grass roots, such as the important role of root cohesion, is qualitatively underlined by the measurements: it could clearly be observed that stability of the grass sods was mainly obtained by their root action, rather than by the presence of the grass swards. This is where the most important feature of the Geogrid comes in: providing better anchoring of the grass by root intertwinement with the Geogrid.

Comparison with superficial sliding model of the grass cover by Young (2005)
Apart from surface erosion, superficial sliding of the grass cover at the inner slope (one of the 'shallow slip failure' modes) is attributed one of the main initial causes of failure of sea dykes in the past, e.g. as observed in the flooding disaster in 1953 in the Netherlands and in the U.K.. Such a beginning of failure is shown in Figure 12-2 below (indicated 'turf sliding).

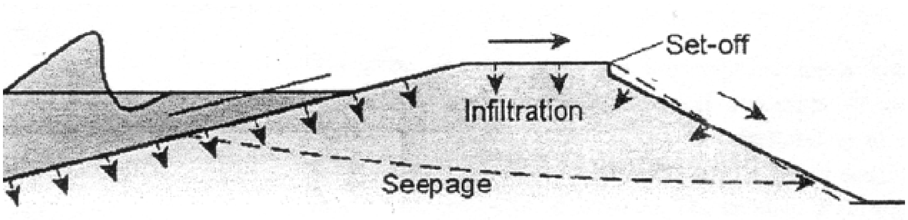


Figure12-2: Typical initiation of failure by 'turf sliding' (Oumeraci et al., 1999)

In his MSc Thesis, Young developed a model for prediction of the onset of 'turf sliding'. Important issues in this model are the grass root characteristics and strength.

In SBW (2007) the data for root strength at the present test location at the sea dyke have been introduced into the model of Young. Typically, at a greater depth, the grass roots are less dense, but the clay aggregates show less 'structuring' (weathering), which may partly compensate the reduction of strength there. At the loading side, the eroding forces by of the overtopping flow are included in the model and are based on realistic overtopping flow velocities (e.g. 7.6 m/s at the maximum wave of 3500 l).

The outcome of this stability analysis for turf sliding is shown in Figure 12-3.

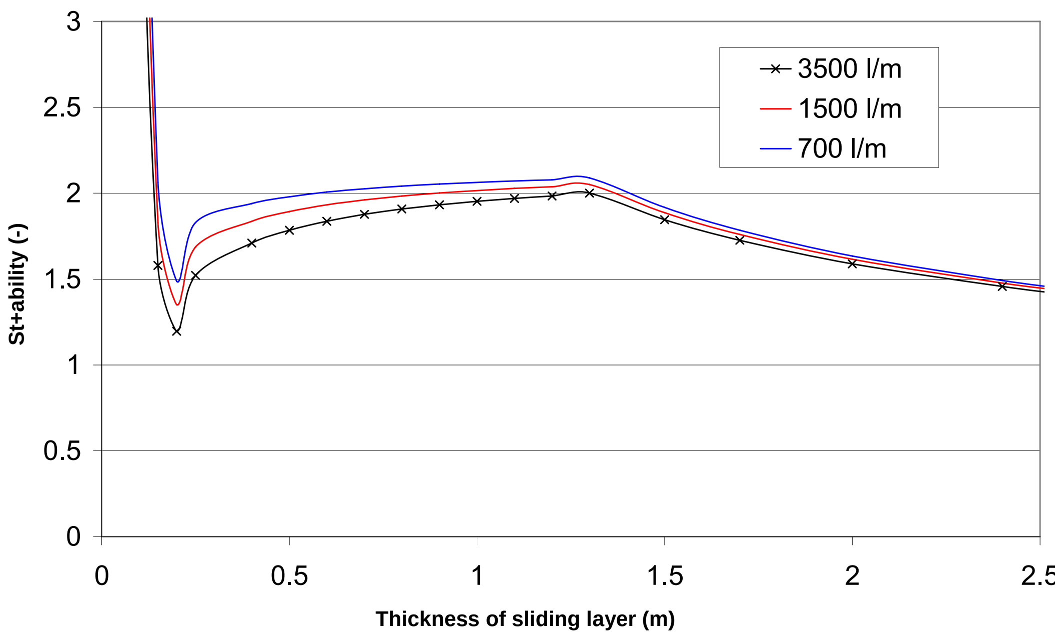


Figure 12-3: Stability coefficient (vertical axes) for different thicknesses of the turf layer that may potentially slide at different wave overtopping volumes in (SBW, 2007), based on (Young, 2005)

For the highest wave overtopping, stability is minimum at a turf layer thickness that largely corresponds to the layer thickness of the grass cover (about 0.2 m), but stability would still be ensured.

Preliminary conclusions based on the observations

The main observation relevant to superficial sliding is that this phenomenon did not occur, even during prolonged testing of severe overtopping (30 l/s/m, ‘50’ l/s/m and ‘50’ l/s/m after introducing artificial damage). No fissures, or initiation of sliding areas could be observed at all, nor significant deformation of the crest of inner slope.
It should be remarked here that the test set-up, as well as the dyke composition (massive clay) has not been representative for slip failure in general, so that sliding deeper than the grassed surface has probably not been representative in the tests.
The outcome of Young’s prediction model that sliding was not to be expected, is confirmed by the tests. The value of such a confirmation is important but should be considered with caution, as validation of the model should be based on validation of the onset of instability (which was not observed during the tests).

12.3 Comparison with earlier investigations and experience with reinforced grass

The major results of the tests on the reinforced grass section can be summarized:

1. No significant erosion, even for overtopping events exceeding 30 l/s/m.
2. Artificially damaged locations of 0.1 * 0.1 m, as well as initially bare spots with poor grass cover, did not show additional erosion for aforementioned conditions. This may be explained by the good sheltering that the surrounding grass sods offered to the bare clay.
3. Around sticks and poles erosion did not increase either.
4. The artificially damaged location of 1.0 * 1.0 * 0.05 m did not increase in depth, due to the presence of the Geogrid. In addition, gully erosion was very moderate as compared to the unreinforced grass. In fact, only the grass cover was partly removed locally, but the clay was not eroded noticeable. There was no sign of progressive erosion as well. The damaged location of 0.4 * 0.4 m * 0.15 m showed no deepening. Gully erosion did hardly occur downstream of the 0.4 * 0.4 m spot. It could be observed that the Geogrid at the downstream side curled upwards to some extent, adding to the protection of the downstream grass cover. All in all, erosion development was much less than with the unreinforced grass.
5. Shallow slip failure of the grass cover (e.g. ‘turf sliding’ downstream of the crest) was not observed. In addition, horizontal fissures near the crest or inner slope have not been observed.

Grass reinforcement systems according to CIRIA (Hewlett et al., 1987)

In this CIRIA report reference is made to two types of reinforcement systems:

1. geotextile reinforcement;
2. concrete reinforcement.

In the CIRIA report it is stated that mats and small-aperture woven fabrics offer better protection as compared to meshes during the growing process of the grass (this grass being seeded after installation) and that in the latter case a good grass cover cannot be guaranteed. The Geogrid may be placed without seeding new grass, in case that no major dyke reconstruction works are being undertaken. In that case meshes are usable as well and are in favour, as the larger opening size allows for unrestricted root development. The Geogrid-type as selected in ComCoast, is in between the mesh-type as indicated by CIRIA and the small-aperture woven type.

In the CIRIA design graph reinforcement with mesh-type geotextile (with openings larger than the Geogrid) are indicated as slightly more stable compared with the unreinforced good grass cover. Strong improvements however can be expected for open mat systems, for short durations comparable to standard concrete block systems, see Figure 12-4. A characteristic feature of the Geogrid is that it is rather stiff locally and therefore offers good anchoring possibilities for the grass roots.

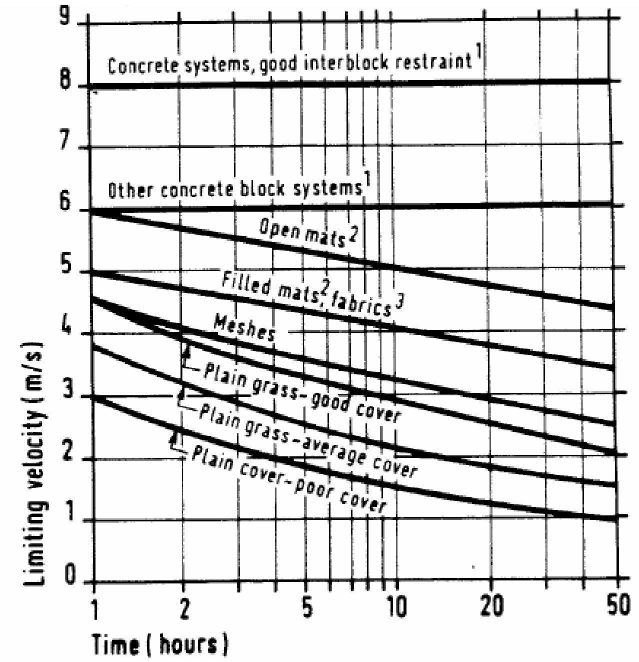


Figure 12-4: CIRIA design graph for allowable overtopping velocity as a function of time

It should be remarked here that the limiting values shown in the CIRIA graph, are likely quite conservative, when compared with the original data.

As stated before, the action of the Geogrid may be expected to lay in between Meshes and Open mat systems. This would lead to a considerable improvement of the surface erosion resistance. It should be noted that the above design graph is valid for steady overtopping flow only.

Enkamat system

Tests have been carried out in the Netherlands at an ‘Enkamat’ system. This system has a thickness of centimeters (different options) and has a sponge-like threading structure. Primary function is to provide ‘sheltering’ of the under laying soil against flow attack, rather than anchoring of the grass roots, as the system does not seem to be stiff enough. This sheltering should compensate the reduction of grass growth through the relatively dense Enkamat. Recent Enkamat systems are delivered as ‘soft’ as to prevent damage to mowing machines. Hence, we think that the additional strength to the grass and substrate layers is limited, which is considered a major disadvantage as compared to the Geogrid system.

<u>Preliminary conclusions based on the observations</u>

The specific feature of the Geogrid is that this reinforcement can be installed at the existing grassed dykes, without major reconstruction works (although some engineering for economic installation still needs to be done). When applied early in the growing season, the reinforced

grass is supposed to recover completely before the next storm season. Then, the Geogrid will add significant strength to the grass cover against surface erosion and turf sliding.
Due to the fact that the unreinforced grass cover did not show major erosion at the highest overtopping rates either, the improvements could not be validated for limit-state conditions without introducing artifical damage. After introducing artificial damage however, the erosion progression was much more moderate and the mesh structure of the Geogrid was fine enough as to protect the underlaying clay. The latter was obvious from the bared surface of 1 m squared, where no major erosion underneath the Geogrid was observed during the overtopping tests of '50' l/s/m. In addition, the function of the Geogrid in providing additional anchoring to the grass roots and lateral continuity between the grass sods was evident. Additional overtopping tests with more severe wave overtopping will still be needed to investigate the limiting conditions and to provide a good comparison with the limiting conditions of the unreinforced grass.

12.4 Comparison with earlier investigations and experience with bare clay

The major results of the tests on the bare clay section can be summarized as follows:

1. The bare clay proved to be sensitive to surface erosion. As a consequence, the tests had to be limited to 1 l/s/m, 5 l/s/m and 10 l/s/m. At the end of the tests, with an overtopping of 10 l/s/m, two marked scour holes with a steep upstream slope ('cliffs') had developed starting about halfway from the inner slope of the dyke. The largest hole was about 1 m deep.
2. From the beginning of the tests, erosion was concentrated at the weakest spots of the clay surface, leading to development of two scour holes. Initially the western scour hole dominated, but was overtaken after some time by the eastern hole. During this process the scour holes steepened at the upstream side, forming the aforementioned cliffs.
3. Erosion occurred almost continuously, as long after the maximum flow of the wave tongues had passed, depletion flows converged strongly towards the cliff areas and were supplied by seepage flow.
4. Retrogressive erosion of the cliffs (('head-cut' erosion) took place, while maintaining the nearly vertical cliff slope, going along with a nearly horizontal erosion base. The velocity of this head-cut erosion was about 0.25 m per hour (measured horizontally) at the overtopping rate of 10 l/s/m.

A longitudinal presentation of the dominant erosion is shown in Figure 12-5. In this figure the maximum values of scouring is taken in each cross-section, irrespective of the location.

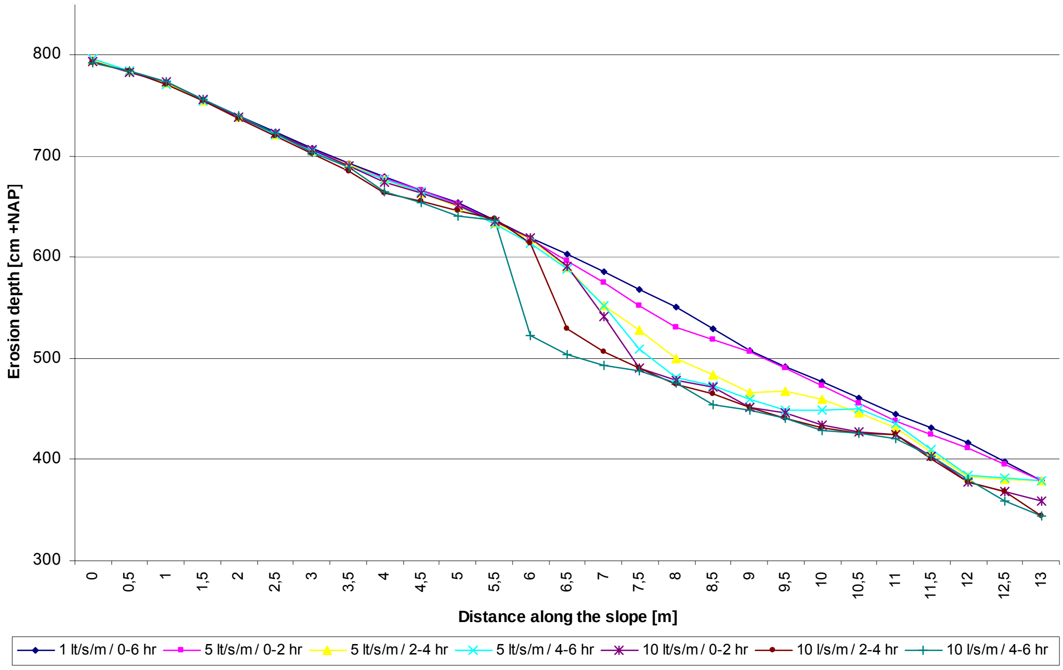


Figure 12-5: Measured erosion depths (highest values of both holes) along the slope for the test series on bare clay

LWI study on overtopping of bare clay (Oumeraci et al., 2001)
In this study in-depth research was performed on the overtopping of a clay dyke in the large scale wave flume of LWI in Braunschweig. Much attention was paid on the physics and reproduction of wave overtopping and the clay properties of the clay layer at the crest and inner slope of the dyke (thickness 0.6 m). The erosion process showed large instabilities at the clay surface, like longitudinal gullies, local scour holes and cliffs, dependent on the conditions and clay composition. These tests do also underline the ongoing formation of localized erosion, as was observed in the present field dyke tests.

SSEA-model for headcut erosion (Knoeff & Verheij, 2003)
In Knoef & Verheij (2003), the SSEA (Sites Spillway Erosion Analysis) model of Stillwater Laboratories has been analysed. A computational example of the head-cut erosion for a thick layer of clay, predicts no erosion for a constant overtopping rate of 10 l/s/m. By minor adaptation of the input parameters, an erosive velocity is predicted of some decimetres per hour, which is in line with the obervations. At dyke failure field tests in the Netherlands under steady state conditions, the Bergambacht tests, the head-cut erosion velocity was some 0.4 m per hour for a permanent overtopping flow rate of about 50 l/s/m.

<u>Preliminary conclusions based on the observations</u>

1. Development of localized erosion started at weak spots in the clay surface, leading to localized scour holes and finally to cliffs. These cliffs progress towards the crest (head-cut erosion). This corresponds to the type of erosion that can be expected from literature.
2. The erosive resistance of the bare clay was much less than the resistance of the grassed surface (even with bare spots on this surface) and the localized erosion increased strongly at ongoing overtopping. In spite of this big relative difference, the clay layer was able to resist subsequent storms with overtopping rates of 5 l/s/m and 10 l/s/m, which is beyond what is usually allowed for grassed slopes.

3. The observed head-cut erosion at the 10 l/s/m overtopping test was about 0.25 m/hour. This is more than has been predicted by Knoeff & Verheij (2003) for a comparable situation: they did not predict erosion in this situation. However, taking slightly deviating values of the indexes, some decimeters per hour are predicted, which is in line with the observed head-cut erosion rate.

13 COMMUNICATION AND PUBLICITY

13.1 Introduction

Adequate communication and publicity was considered an essential part of the present project. To enable this, communication officers of Ministry of Transport, Public Works and Water Management were involved in facilitating and organising communication by communication protocols, assistance to the 'opening event' of the dyke tests, information services and in streamlining publicity. Hereafter the communication and publicity issues are briefly summarized. This summary is based on an draft internal evaluation note by Hanneke Derksen. For due caution towards the Ministry of Transport, Public Works and Water Management, internal conclusions are not mentioned here. Hence we limit ourselves in this Chapter to the external issues.

13.2 Communication process

A communication plan was drawn up in spring 2006. This plan addressed three groups of stakeholders:

- primary internal parties directly involved in the tests;
- primary external parties, comprising neighbouring persons and parties, inhabitants of Delfzijl en of the province of Groningen;
- secondary parties such as: coastal managers at municipal, provincial and national level, water boards, universities and market parties and the Dutch public.

The relation with other communication routes were indicated and critical communication milestones defined.
Four major milestones were identified:

1. start of the SGR installation at the Groningen sea dyke: 17 May 2006;
2. testing of the prototype wave overtopping simulator in Heerenveen: 23 June 2006
3. trial testing at the Groningen sea dyke: 18 December 2006;
4. opening event of the tests: 27 February 2007.

These four moments were formally announced by press communications and 'fact sheets' that contained interesting general information were issued. Prior to the tests a special information brochure was supplied to all interested parties.

After the installation of the SGR and testing of the prototype simulator, a professional movie ('Superturf') was made. After completion of the tests, another professional movie has been made ('Superdykes'), that pictures the tests, SGR-installation, observations and outcomes.

Parallel to the communication officers of Ministry of Transport, Public Works and Water Management, the ComCoast Projectteam was very active in guiding and facilitating the communication activities at the spot. Moreover, the consortium and Waterboard was strongly involved in some PR activities, e.g.: providing signpost panels at the main routes, providing assistance in the communication process and protocol, providing an information panel at the simulator, providing an information cabin, co-operating in external communications (e.g. press activities and assigning a technical information official at the information cabin) and enabling the demonstration test at the opening event. Obviously, cooperation was given to the professional movie shooting.

13.3 Information services and publicity

Questions and answers
As to streamline the information to press and interested parties, the projectleader on behalf of Ministry of Transport, Public Works and Water Management of the ComCoast tests, Patrizia Bernardini, drafted a ‘Questions and Answers’ list, that was used by all directly related staff for answering questions to external parties. This list proved to be very valuable in the external and internal communication process.

Information panels
Two information panels were applied at the site:

- one panel mounted to the simulator, showing the major parties involved in the tests
- one panel placed along the service road at the test site that gave background information about relevant ComCoast issues, see Figure 13-1.

Figure 13-1: Information panel at the test site

Information cabin
An information cabin was placed at the parking lot at some distance from the test site. In this cabin interested parties could be hosted and information was given there via a presentation by a technical information officer (Gijs Bosman, student of the Delft Technical University). The introduction also included the do’s and don’ts for the subsequent visit at the test site. In addition, a small scale demonstration model of the wave overtopping simulator was installed, together with relevant information posters and brochures. This routine enabled the staff at the test site to remain relatively unaffected by the visiting persons. Special guests, however, could directly be invited by the staff at the test site.

Media attention
The field tests received abundant attention in the media (regional and national papers, journals, radio, television), which was promoted by the communication officers of Ministry of Transport, Public Works and Water Management. Within this Ministry the tests were also widely communicated. The consortium pro-actively enabled this media attention and contributed in writing articles and papers and giving presentations.

Opening event
The opening event was a major communication milestone, with some 100 invited guests and ample attention of press (journals, radio, television). This event was accompanied with introductory lectures and presentations at the Delfzijl theatre. At the sea dyke an official opening act was performed by opening the wave overtopping simulator and throwing balls (with the names of participating persons) at the overtopping wave tongue.
The opening event was considered as highly successful. The arrangement of this event had been quite cumbersome, e.g. as the overtopping simulation might damage the grass test sites. At the last moment, it was decided to move the wave overtopping simulator to a site that could not harm the test site (at the location were the additional bare clay tests would be carried out at a later moment).
An impression of the opening event is given in Figure 13-1.

Figure 13-1: Impression of the opening event

14 CONCLUSIONS AND RECOMMENDATIONS

14.1 Conclusions

From the tests the following conclusions can be drawn.

1. The wave overtopping erosion field tests, being unique in their kind, had been successful. Two important innovations have performed satisfactorily: the installed Smart Grass Reinforcement (SGR) and the wave overtopping simulator. The SGR had been installed provisionally in 2006 and some doubts remained about the condition of the SGR at the start of the tests. The outcome of the tests removed any doubts about the performance. The wave overtopping simulator performed very well and without any hitch throughout the tests. This can be attributed to the extensive and well-organized research and tests such as: the reliable design, adequate calibration of the 1m wide prototype, careful construction and trial testing at the dyke testing site.
2. The unreinforced grass section was able to cope well with conditions of '50' l/s/m (for the situation that initial damage was absent), in spite of the poor root conditions.
3. The good performance of the SGR was obvious from the moderate erosion development after introducing artificial damage to the grass cover, as compared to the erosion progression in the comparable situation for the unreinforced grass section. In the latter case distinct gullies were formed that progressed quickly down the inner slope. These erosion patterns could be observed after introduction of artificial damage to both sections (reinforced and unreinforced) and testing with severe wave overtopping ('50' l/m/s).
4. The favourable action of the SGR for limiting surface erosion is considered to be two-fold:
   - providing good 'anchorage' to the grass sods by intertwinement of the root system to the Geogrid: this function could clearly be observed;
   - providing shelter to the under laying clay body against the flow attack by physical protection of the clay layer and by (partial) consumption of eroding forces.
5. It should be noted here that geotechnical verification of reducing the risk of slip failure by the reinforcement action could not be verified in these tests. Although theoretical considerations indicated that 'turf-sliding' was not likely to occur for the unreinforced grass section under the conditions applied, this prediction should still be verified by limit-state field tests.
6. The bare clay slope proved to be much more susceptible to wave overtopping erosion as compared to a grass covered slope. Significant erosion occurred during 1 l/s/m conditions with progressive head-cut erosion with a deep incision (over 1 m) at the inner slope after 10 l/s/m conditions. The difference in erosion behaviour as compared to the situation with a grass cover is striking, especially as the clay at the test dyke was typically heavy and stiff clay. In addition, it must be remarked here the test dyke is a massive clay dyke with no sand core. In the latter case erosion might have progressed even stronger after the moment that the scouring would have reached the sand core. Seen from a perspective of residual strength, however, the clay was able to withstand subsequent storms with overtopping rates of 5 l/s/m and 10 l/s/m respectively, without reaching the crest of the dyke. This residual strength is larger than had been anticipated before.
7. Infiltration measurements showed a quick pressure build-up in the clay layer up to the deepest water tension meters 1.2 m below the surface. This indicated a relatively low permeability. An explanation for this relative good permeability may be found in the numerous worm holes and small fissures that could be observed during the bare clay tests.
8. The measuring data of velocities and flow depths show a large scatter and showed that the measuring instruments did not work properly. The highly turbulent and aerated flow

may have contributed to this malfunctioning. The front velocities of the wave overtopping tongues however could be identified and compared well with the velocities that may be expected from earlier investigations. This sustains the adequate performance of the wave simulator performance in producing representative wave overtopping tongues.

An overall conclusion is that even a rather poor grass cover may be very resistant to surface erosion. The presence of a grass cover is of the utmost importance for combating surface erosion at the inner slope of wave overtopped dykes. This leads to the supposition that the grass cover may be far more important than the quality of the under laying clay substrate. At the tests at the Groningen sea dyke the clay substrate was good so such a supposition needs further verification in future at sites with moderate and poor clay.
The outcome of the tests with the SGR are very encouraging. The strength of the SGR was even so good that limit-state loading could not be obtained in the test set-up at the sea dyke, even not after applying artificial damage. The situation of artificial damage may well match realistic situations at sea dykes, at which some damage may occur anyhow (sheep, fences, burrowing animals). Then, the presence of the SGR may be decisive for the stability of the dyke.

It should be remarked that the better the failure mechanism of surface erosion failure can be coped with, other failure mechanisms may become more normative, such as slip-failures and internal erosion (which could not be tested in this dyke). Although we think the SGR may also mitigate these failure mechanism as well, this should be verified in further tests.

14.2 Recommendations

Further erosion testing at unreinforced grass dykes in the Netherlands is important for exploring the influence of e.g. other overtopping conditions, grass maintenance, other grass types and substrates, and seasonal/yearly variation of grass strength for the existing dykes. Such tests are already foreseen by the SBW program (Sterkte & Belastingen Waterkeringen).

A further recommendation is to perform limit-state testing for surface erosion with and without a SGR. For this type of tests, a large wave overtopping simulator needs to be deployed. Furthermore, this will require a dyke that may be eroded to a large extent. This fits perfectly within the concept of the ‘Calibration Dyke Program’ (in Dutch: ‘IJkdijk’) in the Netherlands that is planned for the next years. In such a facility, other failure mechanisms can be explored as well. This research has already been anticipated in the above mentioned SBW program as well.
It should be noted here that the better surface erosion failure can be coped with, other failure mechanisms may become more normative such as slip-failure and internal erosion (which could not be tested at this dyke). We think the SGR may give a major contribution to mitigating such failure mechanism as well, provided that the reinforcement is properly placed. We would advise to verify this with further tests as well.

As failure by surface erosion could not be reached with the present wave overtopping simulator, it is recommended to investigate the possibility to increase to the size of the wave overtopping simulator such that more than 50 l/s/m can be produced for future tests.
In addition, it is recommended to develop instruments for measurement of flow velocity and flow depth, that can cope well with the highly turbulent and aerated flow.

In future major grass improvements can be anticipated such as in grass types and grass composition in relation to optimized maintenance. Such improvements should strongly be pursued.

For placement at dykes that unconditionally require reconstruction, ‘new-work installation’ of the SGR is straight-forward and feasible. Such reconstruction works are relevant when the crest needs to be raised anyhow or when the composition of the dyke needs to be improved or when the inner slope needs to be adapted. Here, the SGR can reduce the extent of the reconstruction works and add to the economy of these works. In addition, the SGR will contribute to the resiliency of the defences.

For placement at existing dykes, the installation of the SGR still needs some development, especially as regards a more economical and robust installation methodology. A major challenge is that the grass cover remains intact as much as possible, as to allow full recovery of the grass before the next storm season. Should such a feasible installation methodology be arrived at, economical savings may be enormous.

REFERENCES

Alterra (2006): *Grass investigations sea dyke Farmsum,*
Wageningen (in Dutch).

Bos W. van den (2006): *Erosion Resistance of grass revetments during wave overtopping,*
Ministry of Transport, Public Works and Water Management, Dienst Weg en Waterbouwkunde, Delft University of Technology, final thesis (in Dutch).

Bosman, G. (2007): *Velocity and flow depth variations during wave overtopping*,
Delft University of Technology, MSc thesis, August 2007.

EurOtop, 2007: *Wave overtopping of Sea Defences and Related Structures: Assessment Manual,*
www.overtopping-manual.com

Etten, R.van (2007): *SBW Infiltration test at 23-02-2007,*
Ministry of Transport, Public Works and Water Management, February 2007 (in Dutch).

Etten, R.van (2007): *Analysis of Water pressure measurements wave overtopping tests Delfzijl,*
Ministry of Transport, Public Works and Water Management, June 2007 (in Dutch).

Frissel, J., Huiskes, R., Melman, D. (2007): *Grass investigation sea dyke Farmsum*,
Alterra Juli 2007 (in Dutch).

Gent, M.R.A. van, 2002: *Wave overtopping events at dikes,*
ASCE, Proc. ICCE2002, Cardiff, UK.

Geodelft (2007): *SBW Wave Overtopping Tests Delfzijl. Monitoring Water Pressures,*
Geodelft, 423890-0024, version 02 Final, May 2007 (in Dutch).

Geodelft (2007): *SBW Wave Overtopping Tests Delfzijl. Geotechnical Research,*
Geodelft, 423890-0031, version 03 Final, July 2007 (in Dutch).

Heikes, J.B. and L.W.A. Zwang (2006): *Soil investigations test site sea dyke Delfzijl,*
Fugro, opdrachtnummer 1206-0044-0000 (in Dutch).

Hewlitt, H.W.M., et al. (1987): *Design of reinforced grass waterways,*
CIRIA Report 116.

Infram & Royal Haskoning (2007): *Design, construction, calibration and use of the wave overtopping simulator,*
Comcoast, September 2007.

Knoef & Verheij (2003): *Residual strength after initial failure by overflow/overtopping,*
Delft Cluster, Projectcode 02.02.01, July 2003.

Oumeraci et al. (1999): *Wave Overtopping at Seadikes; Comparison of physical model tests and numerical computations,*
OPTICREST Research Report, LWI, No.846, Braunschweig, 1999.

Oumeraci et al. (2001): *Belasting der Binnenböschung von Seedeichen durch Wellenüberlauf,* Abschlussbericht, LWI Bericht Nr 872, Braunschweig/Essen, November 2001 (in German).

Richwien W., Weissmann R. (1995): *Zur Standsicherheit von Deichenbinnenböschungen bei Wellenüberlauf,*
Ergebnisbericht, Universität GH Essen, Institut für Grundbau und Bodemmechanik (in German).

Royal Haskoning & Infram (2005): *Development of alternative overtopping resistant sea defences*
Comcoast, 10 November 2005.

Royal Haskoning & Infram (2006): *Placement of Smart Grass Reinforcement at Test Sections Groningen Sea Dyke,*
Comcoast, 15 November 2006.

Schüttrumpf, H., J Möller and H. Oumeraci, 2002: *Overtopping flow parameters on the inner slope of seadikes,*
ASCE, Proc. ICCE2002, Cardiff, UK.

SBW (2007): *Wave overtopping and strenght of inner slopes of dykes,*
Rapport Predictiespoor SBW, Infram, WL | Delft Hydraulics, Geodelft, rapport 05i028 (in Dutch).

VTV (2004): *The safety of the primary defences in the Netherlands, Safety Assessment Code for the second assesment period 2001-2006,*
Ministry of Transport, Public Works and Water Management, Januari 2004 (in Dutch)..

Young, M.J. (2005): *Wave overtopping and grass cover layer failure on the inner slope of dikes,*
Unesco-IHE, MSc Thesis WSE-CEPD-05.03, Delft.