Integrated Flood Risk Analysis
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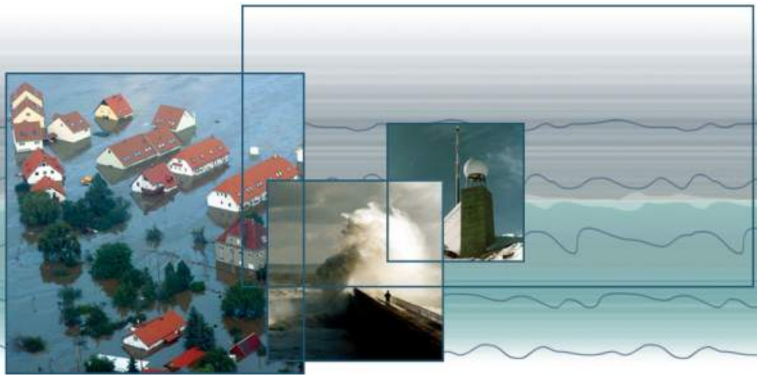
FLOODsite



# Laboratory Experiments on the Erosion of Clay

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# SUMMARY

Clay has been widely used for sea dike construction mainly for the cover layer. Clay used for dike construction can not avoid the presence of cracks as results of physical environmental changes and biological activities. Besides being exposed to the breaking wave impacts, the presence of the cracks makes the clay cover more vulnerable to erosion. These cracks become more dangerous when they are filled with water (for example during high water). Previous experimental results show that the pressures inside the water filled cracks are much higher than the air-filled cracks. These are important in order to provide dike construction guidance for the clay cover of the outer slope. Most investigations on the erosion behaviour of clay cover with and without water-filled cracks are not verified yet with laboratory experiments. Laboratory experiments are needed in order to have better understanding on the erosion resistant of clay cover with and without water-filled cracks.

The impact pressure machine in Leichtweiß-Institut für Wasserbau (LWI), Technische Universität (TU) Braunschweig has been used to investigate the erosion behaviour on clay cover with and without water-filled cracks. Two clay samples representing clay with good and moderate erosion resistance were tested with various experimental set-ups considering water content, compaction, impact magnitude, number of impacts, and clay homogeneity. For clay with water-filled cracks, crack dimension was treated additionally. The influence of water layer was also considered additionally for the test on clay without water-filled cracks.

Results showed that the existing model on erosion of clay with water-filled cracks (Führböter's theory) which only considers cohesion as the soil strength is only applicable for certain conditions. The model should consider other resistant forces such as soil weight and pore pressures. From the tests on clay without water-filled cracks, it concludes with the limitation of recommended water contents which shows the most erosion resistant behaviour. This new limitation covers both operational and functional aspects which has narrower range than previously reported. Other approaches involving theoretical and operational aspects were also discussed to describe the erosion mechanism and to develop a methodology for further research and development.

Other experiments to investigate the possibility of dike breaching initiated by breaking wave impacts have been carried out using small wave flume. A dike model built from sand as core and clay as its cover was subjected by the wave impacts generated by the flume. This qualitative experiment was intended to investigate the mechanism of clay cover failure and breaching possibility initiated by the breaking wave impacts. Seven stages of dike failure mechanisms initiated by breaking wave impacts are identified that lead to severe damage of the sea dike.

Keywords:
dike, breaking wave impact, impact machine, erosion, clay, water content, compaction

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# 1. INTRODUCTION

## 1.1 Background

Countries bordering the North Sea like the Netherlands, Germany, Belgium, Denmark, and UK share a long history in fighting against flooding threats from the sea. The need to protect these flooded-vulnerable areas which cover 40.000 km2 and home of 16 millions people has been rising since the tendency of increasing natural catastrophe threats and the important role of the threatened areas among those countries. The South Holland and the North Holland provinces which are also the most populous province in the Netherlands, the engine of country's economy, and home of important cities are in risk of flooding. The north coast of Lower Saxony State, the west coast of Schleswig-Holstein State, and the biggest seaport in Germany, Hamburg are potentially flooded during storm seasons. The south east coast of UK, the Flanders coastline, and the west coast of Denmark are also potentially affected by flooding.

A project called FLOODsite has been delivered by the European Commission to improve the understanding of the causes and their complex interactions involving physical, environmental, ecological and socio-economic aspects of floods. Damage mitigation by applying necessary measures is one the project themes that needs integrated approaches in all aspects of application. Several measures have been implemented to mitigate the damage caused by severe storms and to protect the potentially flooded areas. Coastal defence, either natural or artificial, is one of the measures to deal with flood threat. Natural coastal defences in the form of natural beaches or dunes provide sufficient protection against flood. But, since the increasingly human interferences in the coastal area that largely influence the balance of these natural coastal defences, the safety is no longer guaranteed. An artificial coastal protection is in great need to assure the flooded-free areas in a developed environment. The Netherlands and Germany are two examples where besides the natural protection systems are in place, the artificial coastal protections are also widely implemented.

There are several types of artificial coastal protections ranging from the simple mound of stones or sand bags to the most complicated ones like storm surge barriers. Among all those, dikes have been widely used as flood protection to avoid inundation, particularly in the low lying areas like the Netherlands and the North coast of Germany. To meet its function, a dike should meet certain design conditions. The design conditions are derived from both hydraulic and geotechnical characteristics and their interactions. Water levels and waves are two main hydraulic loads that are very important in dike design while geotechnical stability is contributed to the strength of the dike body. The failure in identifying these loads and the geotechnical strengths can lead to failures or even disastrous situations (breaching).

Main materials of a dike consist of sand and clay. Based on their behaviour and natural characteristics, sand is used mainly for the core of the dike and clay for the cover (revetments) of the dike. Other materials such as artificial revetments (concretes, asphalts, stones, etc), stones (for the toe protection), filter materials (geotextiles, aggregates, etc), and even grass (to prevent surface erosion) are also largely used for dike constructions.

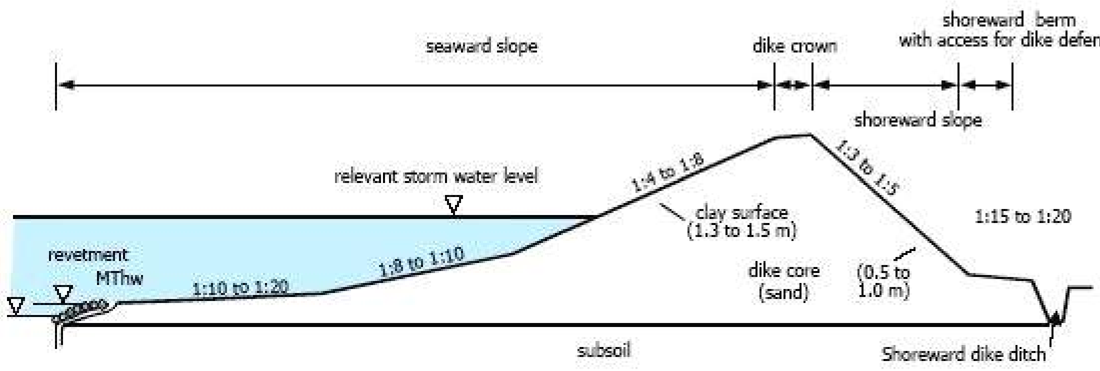



*Figure 1-1 A typical sea-dike cross section (Kortenhaus, 2002)*

A typical dike cross-section and main materials constructed along the coastline of the North Sea are shown in Figure 1-1. As it can be seen from the figure, clay cover is very important for dike stability. Clay has been widely used as a revetment material particularly reinforced with grass cover. It is excellent, sturdy, inflexible, and coherent even under the influence of water (TAW, 1996). Due to its position at the outermost side of the dike, subjected by the attacks of external force such as high water level and breaking wave impacts, clay revetment is vulnerable to erosion that can lead to breaching. Overtopping and overflow can erode clay revetment at the inner slope of the dike. At the outer slope, dike breaching can be initiated either by rising water level and/or breaking wave impacts.

Clay behavior is highly influenced by the change of its water content. The change of water content on clay revetment of the dike can be from continuous processes of drying and wetting due to tides, wave run-up/down, or rain. These changes can lead to the formation of cracks on clay layers.

The cracks on clay cover are often filled by water. Breaking wave impacts on a dike slope with cracks generates impact pressures inside water-filled cracks that can cause removal of the clay layer (Führböter, 1966). For a dike slope without cracks, surface erosion of clay revetment also potentially occurs due to continuous wave breaking impacts. Those dike erosion mechanisms had been studied and partly explained with some limited conditions because both have not fully verified yet in laboratory experiments. Laboratory tests are needed to get better understanding about physical processes and improve the existing dike erosion model that can lead to breaching.

## 1.2 Objectives

The general objectives of the research are to study one of the sea-dike failure mechanisms initiated by breaking wave impacts hitting the clay cover of outer dike slope, to understand its mechanism, to identify aspects-related failure mechanisms, and to improve the model that can explain the erosion initiated by breaking wave impacts. The break down of these general objectives is:

a) To study and analyse the erosion of compacted clay with significant water-filled cracks due to breaking wave impacts.

b) To study and analyse surface erosion on compacted clay revetment without cracks due to continuously breaking wave impacts.

c) To explain the failure mechanisms of sand dike covered by clay initiated by breaking wave impacts.

## 1.3 Methodology

To achieve the research objectives, a series of laboratory experiments have been carried out at Leichtweiß-Institut für Wasserbau (LWI), Technische Universität (TU) Braunschweig, Germany. The overall works are divided into 3 laboratory experiments. Each Laboratory experiment has its own method. They are:

- Laboratory experiments for Erosion on Water-filled Cracks,

A falling water impact machine is used to generate the impact pressures with fully control from the computer. Two types of clay are provided representing as clay with good and moderately erosion resistant clay. These clays are taken from a real dike in the North coast of Lower Saxony State, Germany. An artificial crack filled with water is then induced to the clay samples. The failure mechanism is measured by measuring the angle of failure along the shear failure line after the sample hit by a single impact. The results from these experiments will be used to explain the erosion mechanism and validate the theory of Führböter (1966).

- Laboratory experiments for Surface Erosion on Compacted Clay

The same falling water impact machine as in the experiment for Erosion on Water-filled Cracks has been used for Surface Erosion on Compacted Clay. Impact pressure of 24.75kPa (equivalent of impact pressure generated by 1.2m wave height) is used in the experiment. Two types of clay are provided representing as clay with good and moderately erosion resistant clay. These clays are taken from real dike in the North coast of Lower Saxony State. Numbers of impacts are released hitting the compacted sample until the significant surface erosions are observed. The erosion rates are documented by doing measurements every certain number of impacts depend on the observed erosion. The role of water layers and degree of compaction are additionally included in the experiments.

- Laboratory experiments for Overall Dike Breaching Mechanism Initiated by Breaking Wave Impacts

Dike models have been constructed using artificial smooth sand as dike core and moderate clay as its cover. The outer slope of the dike is 1 to 4 and the inner slope is 1 to 3. Two experiments were carried out with the same experiment setup. Investigation on Overall Dike Breaching Mechanism Initiated by Breaking Wave Impacts is done by observing the processes start from failure of the clay cover until breaching (if achieved).

Details of the methodology for three laboratory experiments will be discussed in Chapter 3.

## 1.4 Report Outline

This report starts with introduction (Chapter 1) describing insight overview of the research including the objectives and a brief methodology. Theoretical background is discussed in Chapter 2 describing clay properties, clay for dikes, the wave impacts theories, and existing theories about interaction between wave impacts and the subsoil of a dike. Chapter 3 explains more details about the methodology used for laboratory experiments. Results from the experiments are presented in Chapter 4 and analyzed more details in Chapter 5. Finally, conclusion and recommendations from the whole works are presented in Chapter 6. Some appendixes as additional information for the whole works can be found together within this report.

# 2. THEORETICAL BACKGROUND

This chapter deals with descriptions of previous works (literature review) related to the use of clay for sea dike constructions, breaking wave impacts on dike slope, and the reactions of subsoil against the wave impact pressures. Summary and conclusion of those works are described at the end of this chapter.

## *2.1 Clay Properties and Classifications*

Clay has been known as an excellent material for dike construction, mainly used for cover layers. It can be easily worked, good erosion resistant, and relatively less permeable. A Dike needs those characteristics to meet its function as flood protection. In the Netherlands and Germany, clay is also abundant, make it more advantageous.

As a cohesive material, clay consists of fine soils and is defined as a natural soil with certain mass percentages of sand, silt, and lutum. The Dutch Standard NEN 5104 limits clay composition based on percentages of 50% sand, 75 % silt, and 8 % lutum (Figure 2.1). The area bordered by those values divide the NEN 5104 Triangle into Clay, Sand, and Loam. This classification is based on the grain diameter (d) of those fine materials. They are:

- Sand : 63µm ≤ d < 2mm
- Silt : 2µm ≤ d < 63µm
- Lutum : d < 2µm

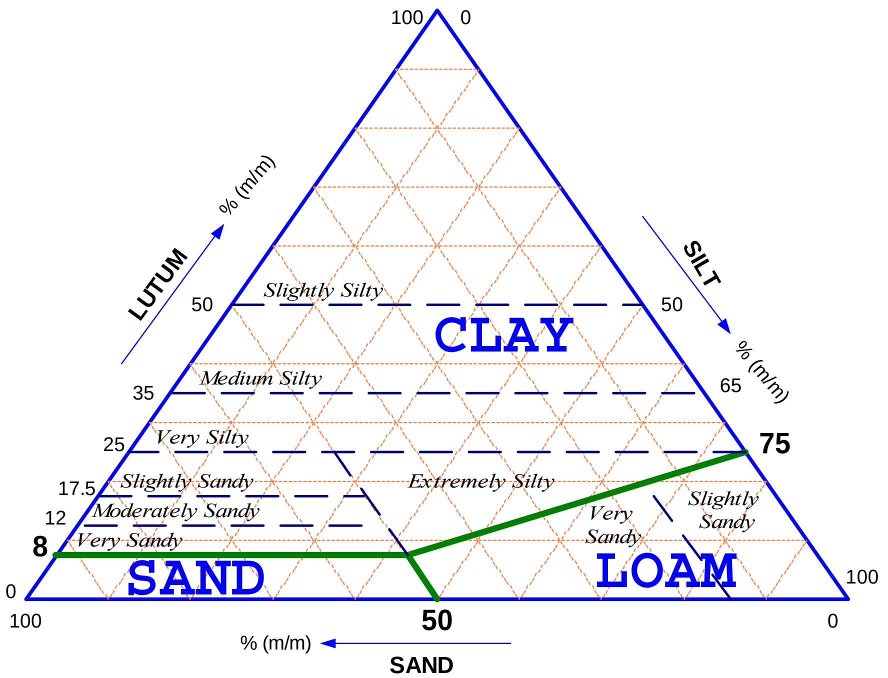



*Figure 2-1 Classification of soils based on the Dutch Standard NEN 5104 (Lubking, 2006)*

The properties of clay describe both chemical and physical aspects which contribute to the clay behaviour in dike construction. In order to understand its properties, clay should be distinguished into clay as mineral soil fraction and clay as natural soil. Clay as mineral soil fraction deals within microscopic and sub-microscopic levels while clay as natural soil describes more about its civil engineering functions.

### 2.1.1 Clay as Mineral Soil Fraction

The properties of clay as mineral soil fraction are related to cohesion and water retention capacity. Cohesion is binding processes of water molecules and fine particles. Other chemical reactions involving organic compounds and clay minerals are also contributed to the cohesion strength. Due to its dependence on water, cohesion of the clay can vary following the changes in water content. Low water content increases particles bonds and stronger cohesion. Water retention capacity is also influenced by water content changes and the physicochemical properties of water and particles. In general, cohesion and water retention capacity of clay are greatly influence by the following factors:

- Dissolved fine particles in water which contains clay minerals and other minerals such as quartz, irons, and aluminium influence the water-retaining ability of the soil and the bonds among the particles.
- Organic materials as the results of biological activities such as bacteria, fungi, organic molecules and the remains of plant or animal organism are contributed to the changes of cohesion and water retention capacity. Furthermore, they also influence other properties such as bulk density, shape retention, and deformation.
- Specific surface area of the solid particles and the surface tension determine the particles affinity for water. Specific surface area is the whole area of the outside surface of the particles. Surface tension is the result from water molecules attraction force against a solid surface. Water is retained in fine pores and among particles due to the presence of this force.
- Other factors such as temperature, humidity, presence of chemical compounds, and coarser fractions are also contributed to the performance of clay properties.

### 2.1.2 Clay as Natural Soil

As natural soil, clay has properties related to civil engineering functions such as erosion resistance, permeability, shape retention, and workability.

#### Erosion resistance

The erosion-resistant behaviour of clay is determined by both internal and external aspects. The internal aspects are related to its natural composition of sand, silt, and lutum (clay) and its physical properties. The higher sand percentages it has the weaker it will be against erosion. Other internal aspects such as water content and chemical compounds (impurities) can greatly influence the erosion-resistant properties. The external aspects come from the surrounding environments that directly cause erosion. Breaking wave impacts, current velocity, run-up/down velocity, overflow, overtopping, rain fall, etc are typical external aspects that potentially disintegrate the clay strengths.

According to Technische Adviescommissie voor de Waterkeringen (TAW, 1996) classification, clay can be classified based on the sand content and the Atterberg limits:

*Table 2-1 Classification of erosion resistant clay (TAW, 1996)*

| **Category** | **Flow Limit, $w_l$ [-]** | **Plasticity Index, $I_p$ [-]** | **Sand content, $S_p$ [%]** | **Erosion resistance** |
|---|---|---|---|---|
| 1 | >0.45 | >0.73($w_l$-20) | <40 | Erosion Resistant |
| 2 | <0.45 | >0.18 | <40 | Moderately erosion resistant |
| 3 | - | <0.18 | >40 | Little erosion resistant |

Atterberg limits describes about water contents as indications between liquid state and plastic sate. Two of them are liquid limit and plastic limit. Flow limit (wl) is the upper limit of water content where the clay is no longer in liquid state. When the water content of the clay decreases until it gets dryer and reaches plastic state, the water content at this situation is called plastic limit (wp). The difference between flow limit and plastic limit is Plasticity Index (Ip). The values of flow and plastic limit (also other Atterberg limits such as shrinkage limit and sticky limit) are determined by laboratory tests.

In Germany, Weißmann (2003) classifies clay by involving more factors and introduces classification number (N) as indication for erosion-resistant behaviour.

$$N = \sqrt[n]{B_1, B_2, B_3, ...B_n} \tag{2.1}$$

Where
N : Classification number [-]
$B_n$ : Classification factors [-]

Classification factors are defined as available clay properties and re-calculated using the following formulas

- Infiltration rate, $k_f$ ($B_1$)

$$B_1 = 0.7 - \frac{\left(\log k_f + 4\right)}{20} \tag{2.2}$$

- Decomposition time, $t_{30\%}$ ($B_2$)

$$B_2 = 0.2\log(t_{30\%}) \tag{2.3}$$

- Shrinkage limit, $V_s$ ($B_3$)

$$B_3 = 1.0 - 1.25(V_s - 0.05) \tag{2.4}$$

- Plasticity Index, $I_p$($B_4$)

$$B_4 = 0.3 + 2I_p \tag{2.5}$$

Table 2-2 shows clay classifications based on Classification numbers (N) according to Wei□mann (2003):

*Table 2-2 Classification of erosion resistant clay based on classification number (N) (Weiβmann, 2003)*

| Applicability Class | Classification number (N) | Erosion resistance |
|---|---|---|
| 1 | 1.00 ≤ N ≤ 0.85 | Very good |
| 2 | 0.85 ≤ N < 0.75 | Good |
| 3 | 0.75 ≤ N < 0.65 | Moderate |
| 4 | 0.65 ≤ N < 0.50 | Weak |
| 5 | < 0.50 | Bad |

### Permeability

Permeability is a parameter to evaluate the effectiveness of retaining water. Clay permeability is varying for different kinds of clay. For example, clay with high percentage of sand will have high permeability. Water will flow easily through the pores with good connectivity among them. If these pores are shut, for example by compaction, the permeability will be less. The immediately-after-constructed dike will have clay layer with low permeability. After some times, it will increase due to the presence of cracks, animal tunnels, etc that make the water flow easily. In general, compared to other soil types used for dikes, clay has very low permeability.

*Table 2-3 Dike material permeability related to the required compaction (Pilarczyik, 1998)*

| Material | Composition | Compaction | Soil density [kg/m³] | Permeability [m/s] |
|---|---|---|---|---|
| Sand | Silt – Clay < 5% | > 90÷98% MPD*) | 1900÷2100 | $10^{-3}$÷$10^{-2}$ |
| Clay | Sand < 25%<br>Lutum < 20÷50%<br>Humus < 3% | > 95% PD**) | 1600-1900 | $10^{-9}$÷$10^{-6}$ |
| Clay with grass | Sand ≈ 50% | - | 1400-1800 | $10^{-5}$÷$10^{-4}$ |

**) MPD-Modified Proctor Density*
***) PD-Standard Proctor Density*

### Shape Retention

Shape retention is one of the advantages while working with clay. Water content, optimum compaction and the amount of worked clay are very important in order to have optimum shape retention. Working in thick packages, for example to fill in the core of the dike, requires good shape retention of the clay in order to strengthen the construction in early stage. Attention should be paid to the water content that can increase the clay volume if it is too wet and then shrink when it gets dry.

### Workability

The easiness in working with clay depends on the water content (Atterberg limits and consistency index). Clay for dike construction must satisfy certain conditions in order to have optimum erosion resistant. For example, in order to achieve recommended compaction of at least 95% of proctor density, clay should be in optimum water content which can be represented by consistency index ($I_c$). $I_c$ is defined as:

$$I_c = \frac{w_l - w_n}{w_l - w_p} = \frac{w_l - w_n}{I_p} \tag{2.6}$$

Where:
Ip : Plasticity index
wl : Flow limit
wp : Plastic limit
wn : available water content of the soil sample

The requirement for all cover layers is $I_c \leq 0.75$ (TAW, 1996). The range of required water content based on the value of $I_c$ are:

Maximum water content $w_c$(max):

$$w_{c(\max)} = w_l - (0.75) I_p \tag{2.7}$$

Minimum water content is defined as the optimum water content from standard proctor test (wpr). Deviation of 5-10% less from the $w_{c(max)}$ is recommended as the minimum water content ($w_{c(min)}$ ) where wpr is usually still on the range.

$$w_{c(\min)} = w_{pr} \; or \; w_{c(\min)} = w_{c(\max)} - (5...10\%) \tag{2.8}$$

## 2.2 Soil Structure and Cracks Formation

Dike body is naturally exposed to the hydraulic loadings (wave impacts, currents, and water level rise), the changes of weather (temperature, humidity, rain, snow), and biological activities. Those factors together influence the clay layers properties and hence the overall dike stability. Hydraulic loadings can erode the clay layers. The weather changes and biological activities inside dike body weaken the clay layers stability due to the presence of cracks and large pores. The present of cracks and pores significantly reduce the properties of clay such as strength (cohesion) and permeability. Consequently, the waves or currents may easily erode the weakened clay layers.

The change of water content due to wetting and drying processes causes the clay to shrink and to expand. When clay shrinks, the volume decreases and when it expands the volume increases. These volume changes create cracks and pores. Two types of cracks are defined by TAW, they are;

- Pull-cracks appear when clay shrinks, found in the form of large vertical cracks.
- Shear cracks appear when the soil swells, observed in the form of smaller cracks in shear areas in all directions.

Both crack formations and also pores made by burrowing animals produce soils that consist of aggregates in various shapes and dimensions. This soil formation is called 'Soil Structure' (TAW, 1996). Soil structures can be found at 1 to 2 meter deep of clay layers, particularly when the clay layers remain unsaturated. Soil structure provides better aeration for grass but and many cases it also greatly changes the clay properties and has negative influence on erosion resistant.

*Figure 2-2 Cross section of a clay layer showing soil structures in the forms of cracks and pores as results of water content changes and biological activities (TAW, 1996)*

During dike construction, one should be aware the importance of optimum water content that can be applied in order to avoid large cracks (pull-cracks) due to volume shrinkage. Volume shrinkage can be avoided by applying well compacted clay and well compacted clay needs optimum water content. This optimum water content can be obtained from laboratory tests such as Proctor Density tests. Generally, the optimum water content is difficult to be applied in practice. Therefore, the so-called recommended water contents which are obtained from the same tests are favorable. The recommended water contents and the optimum water content are not the same for each type of clays. By applying these water contents, the required degree of compaction of at least 95% Proctor Density test can be achieved (see Table 2-3). The recommended water contents are also very helpful during construction process with relatively less energy needed and not sticky to the compacting machines. TAW mentions the range of these recommended water contents as already explained in *section* Workability.

## 2.3 Clay as top layer of a dike

A dike geometrically is divided into several sections. Each section has its own functions with specific materials. Three main sections that contribute to the structural strength of the dike are top layers, dike cores, and bed layers. Clay has been used for those sections, particularly the top layers and the cores in various types and design specifications.

Top layers of a dike play very important role as the first shield against possible erosion that can lead to breaching from (mainly) hydraulic loadings. The clay used for top layers requires a good erosion resistant surface against all possible loadings. Clay category I (based on TAW classification) is the most suitable clay for the top layer. Clay category II still can be used for top layer with certain design conditions. Thickness of the clay layer is made such that in the form of wedge shape from top to bottom to maintain the stability of the most heavily loaded area from infiltration and erosion. A clay thickness of at least 1 meter is also usually maintained in all part of the top layers such as outer bank, inner bank, and crown layer.

The thickness of clay layer should cope with the presence of soil structures (e.g. cracks). Pohl (2006) recommends the dimension of top layer by considering not only hydraulic loadings but also the structured clay, strength and softening processes, and the influence of infiltration due to damming and overtopping. Based on those factors, the clay thickness ($d_L$) of the top layer should exceed the depth of the cracks ($d_R$) plus an additional depth ($\Delta$d):

$$d_L = d_R + \Delta d \tag{2.9}$$

Pohl (2006) recommends the depth of the cracks ($d_R$) should be larger than 0.75m. Due to the difference in loadings and its main functions, inner and outer slope should be treated differently in estimating the depth of the cracks ($d_R$). The additional depth ($\Delta$d) should not be less than 0.5m for the outer slope and 0.25m for the inner slope.

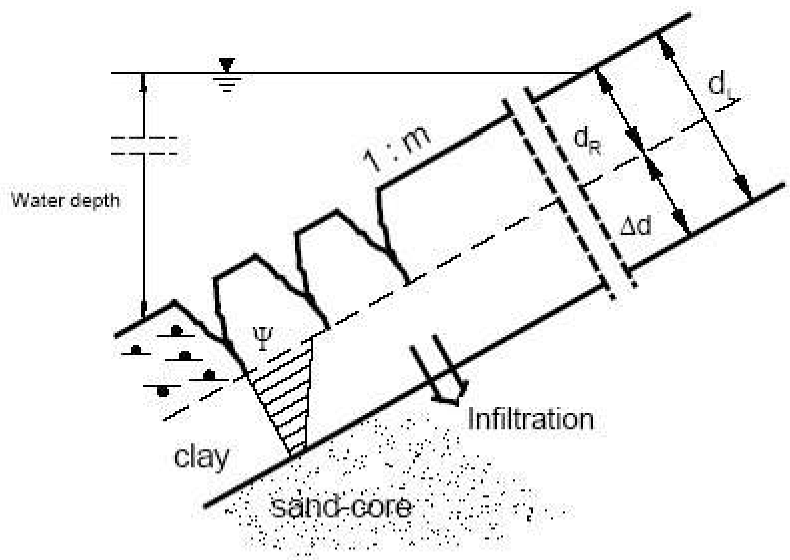



*Figure 2-3 Design of clay as top layer (Pohl, 2006)*

To increase erosion resistance, the clay layer is often reinforced by grass revetments. The interaction between the grass and the clay provides stronger and more durable defence against wave attacks and run-off from overtopping. To have good erosion resistance, during construction, the clay layer should be divided into two parts. The under layer part is the clay layer which is designed to meet civil engineering purposes; well compacted, impermeable, and stiff. The upper layer part should give better aeration and be less compacted in order to allow vegetation to grow, especially in early stages.

The important part in the reinforced-grass clay is the sod area. The sod area is the area where the root system is well developed (Figure 2-5). Dense and good rooting systems in the sod are main factor influencing the erosion resistance of grass-reinforced clay cover (*Scheldebak* Test, 1994). From the *Deltagoot* test in 1992, the clay with strong sod system shows no damage at the inner bank by water run-off from the overtopping up to 25 l/s (TAW, 1999). With the wave impacts of 0.75 m in more than 12 hours, the sod of the outer bank only suffers minor damage. The sod starts to collapse after being hit by the wave of 1.4 m high for more than 16 hours.

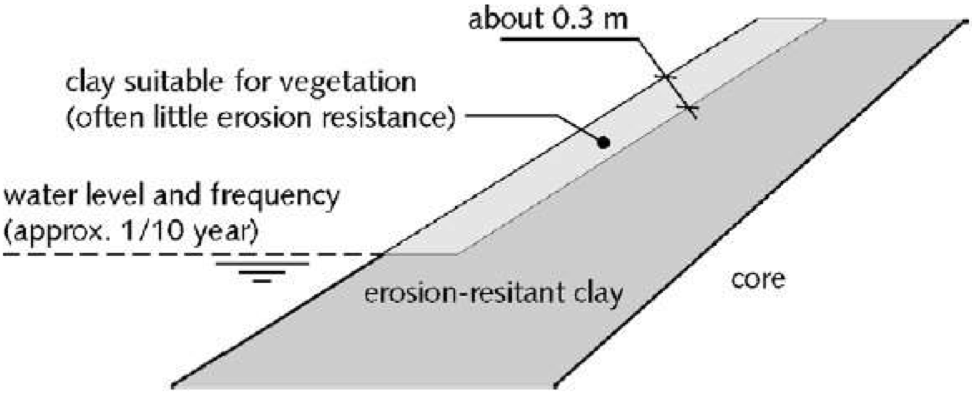



*Figure 2-4 The upper layer part for growing the grass and the under layer for erosion resistance (TAW, 1996)*

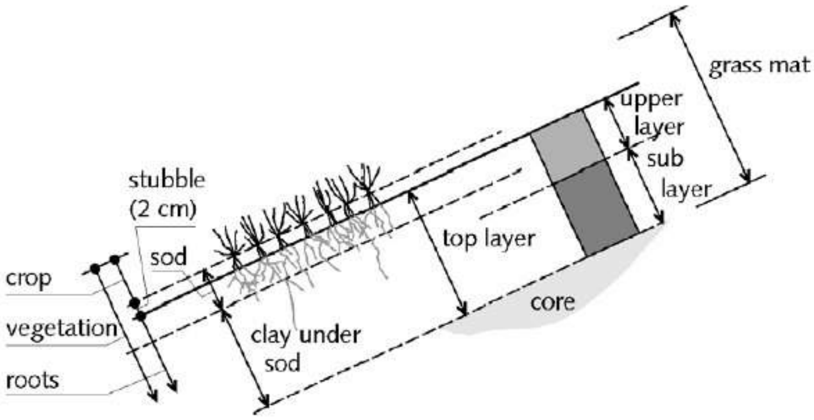



*Figure 2-5 Structure of top layer with grass revetment (TAW, 1999)*

Hard coverings are also used on top of the clay layer. In the area where the hydraulic loadings are severe, block of concretes or asphalt layers are used to replace grass. In this condition, the clay layer should be well compacted and well filtered. A filter layer should be in place to avoid interface instability between the hard coverings and the clay layer. Filters can be in the form of gradation aggregates or geotextiles.

## 2.4 Breaking Waves

As waves travel from deep to shallow water, changes on their characteristics are observed. The processes known as shoaling, refraction, and diffraction led to the changes of wave height, wave direction, etc. As waves feel the shallower water depth, a wave of certain characteristics will be unstable and break releasing its highest energy. Later on, energy dissipation occurs at the surf zone (the region where most of the waves are breaking stretching from the dry beach to seaward limit of breaking) in the form of turbulence and friction against the bottom. The location where the waves break at some depth is known as the breaker line.

Wave breaking types are identified in many forms. Galvin (1968) distinguished 4 types of waves breaking:

1. Surging
   Surging waves occur at very steep beaches with strong reflection. These waves move up and down the slope.

2. Plunging
   Plunging waves are characterized very steep wave fronts which then fall downwards into part of the wave trough. Different from surging waves, plunging waves are having air-trapped inside the water mass that can result big blows when breaking.
3. Spilling
   Such waves occur at mild beaches characterized by breaking starts at the crests.
4. Collapsing
   Collapsing is identified as combination of plunging and surging.

Those types of breaking waves depend on the angle of beach slope ($\alpha$) and the wave steepness. Battjes (1974) defines what is called surf similarity ($\xi$) to indicate breaking types.

$$\xi = \frac{\tan\alpha}{\sqrt{\frac{H}{L_o}}} \tag{2.10}$$

Where:
$\xi$ : Surf similarity [-]
$\tan\alpha$ : Beach slope [-]
H : Wave height [m]
Lo : Deep water wave length [m]

Plunging waves are more interesting to coastal engineers since this type of breaking waves potentially produces high impact pressures to coastal structures. Most coastal structures, for example sea dikes, have front slopes which are ideal for creating plunging waves. Removal of revetments or big scour holes are often found as the results of breaking wave impacts from plunging waves (Figure 2-7). Plunging waves are responsible for most of the damage at the sea side of sea dikes during storms.

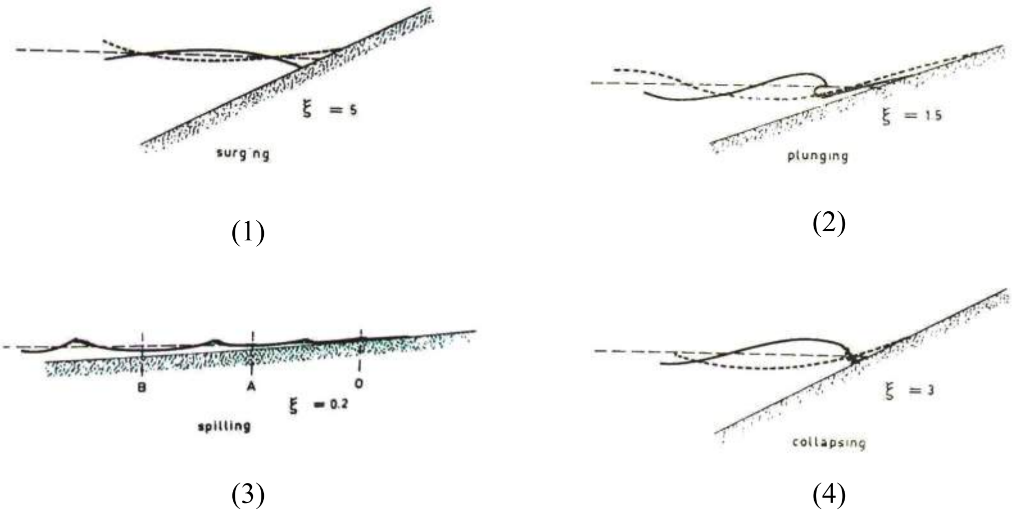



*Figure 2-6 Types of breaking waves according to Galvin,1968 (TAW, 1990)*

*Figure 2-7 Damage at the sea side of a dike due to breaking wave impacts (Stephan, 1981)*

Breaking wave impacts can generate shock pressures into the revetment and the subsoil of the dike. This shock pressure works at a relatively very short time in the range of 10 – 60 milliseconds with maximum pressure can reach up to 350 kPa (Figure 2-8). The magnitude and the time history of these shock pressures mainly depend on the local sea states characteristics and the dike slopes. Dike slopes determine what types of breaking waves will occur. Breaker types have substantial influence on the magnitude and frequency of shock pressure. Steeper slopes are more likely to have plunging and collapsing waves which have higher generated shock pressures than other breaking types (Führböter (1976), Grüne (1988), Führböter & Sparboom (1988)). Aeration processes and thickness of backrush-water also influence the amplitude and the occurrence of shock pressures (Oumeraci (1984), Führböter (1976, 1986), Grüne (1988), and Führböter & Sparboom (1988)).

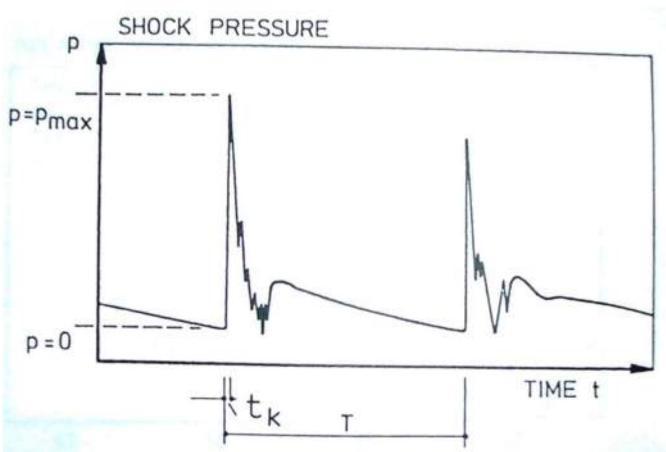



*Figure 2-8 Shock pressure of breaking wave impacts (Führböter & Sparboom (1988))*

The locations of impact pressures are defined as the area where the maximum pressures are observed along the dike slope. These locations depend on the wave characteristics and the slope of the dike. In general, the location of maximum impact pressures is below the mean water level (MWL). In experiment with regular waves, the location of maximum impact pressure on the dike slope was observed at fixed positions of below the MWL (TAW, 1990).

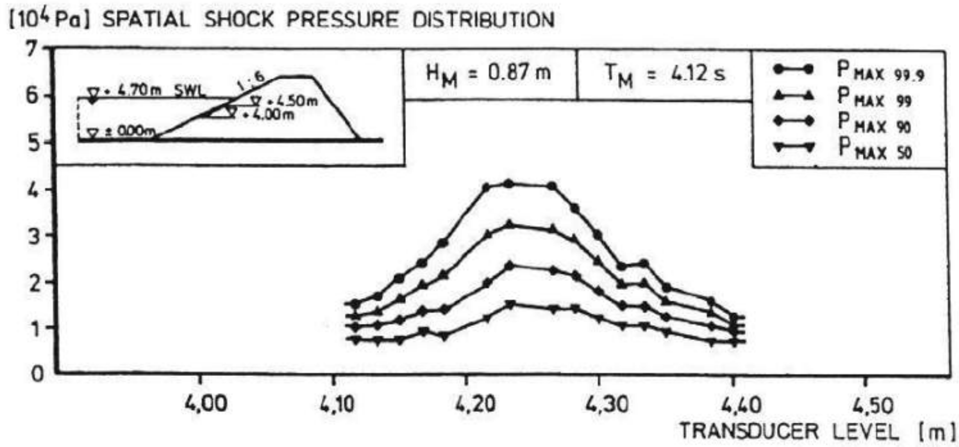



*Figure 2-9 Locations of maximum impact pressures are located below the MWL (Führböter & Sparboom (1988))*

Besides the impact pressures, the dike slopes are also subjected to the pressures generated by wave run-up and run-down. When the waves break at the slope, it is followed by run-up and run-down that significantly contributes to the surface erosion of the dike slope. The magnitude of run-up and run-down depends on local wave characteristics, slope angles, surface roughness, and permeability of the slope.

## 2.5 Subsoil reactions against the impact pressures

Dike slopes at sea side are usually protected by various revetments. These revetments are constructed for protection purposes against erosions caused by breaking wave impacts. Besides direct hits from the breaking wave impacts, the dike slope also experiences dangerous pressure changes during wave run-up and run-down due to exerting drag force from the subsoil. This exerted drag force can uplift the revetment of the dike. There are 5 mechanisms (Figure 2-10) that can lead to erosion of sea side slope of the dike due to breaking wave impacts:

1. Direct hit of shock pressures (A)
2. Splash water around the impact area that can erode the slope surface with very high velocity (B)
3. Uplift forces during wave run-down (D).
4. Wave run-up and run-down velocities that continuously cause erosion over the surface (C)
5. Propagated shock pressures inside the water-filled cracks

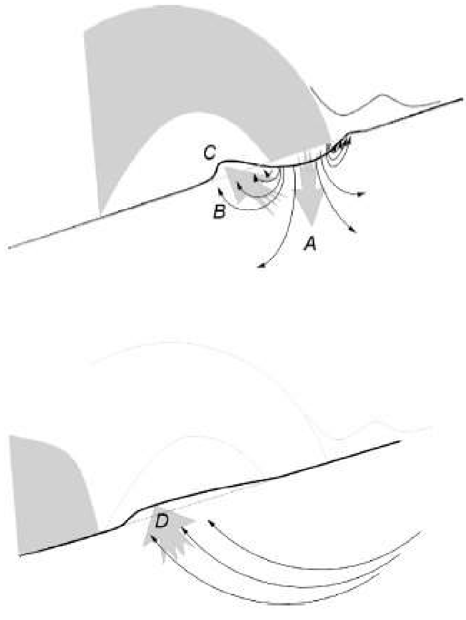



*Figure 2-10 Mechanisms of damaging factors due to wave impacts on soil surface without cracks (TAW, 1997)*

Interactions between breaking wave impacts and dike revetment including the subsoil have been investigated both in field and laboratory. Groups of wave impacts effects investigations can be divided into:

1. Field Measurement.
2. Laboratory experiment with scale model
3. Full-scale laboratory experiment
4. Laboratory experiment with water-jets

Some results of those investigations are described in the following sections with main concern on the reactions of clay covers; the erosion on clay with significant water-filled cracks and the erosion on compacted clay without cracks.

### 2.5.1 Erosion on clay covers with water-filled cracks

#### Basic concept

A model explaining the failure mechanism on the clay surface with significant water-filled cracks was developed by Führböter (1966). When a water-filled crack is hit by the impact, the impact pressures will be instantly distributed on both side of the surface wall of the crack with an equivalent speed of sound (pressure propagation) of 1485 m/s. The model calculates the forces ($F_{crack}$) acting along both wall surface of the crack as a result of the instantly transferred impact pressure ($p_{max}$). The shear stress force (S) acts as counterforce against the $F_{crack}$ holding it from failure. The $F_{crack}$ can remove the soil body along the shear failure angles ($\alpha$) as it overrides critical situation ($F_{crack}$ >S).

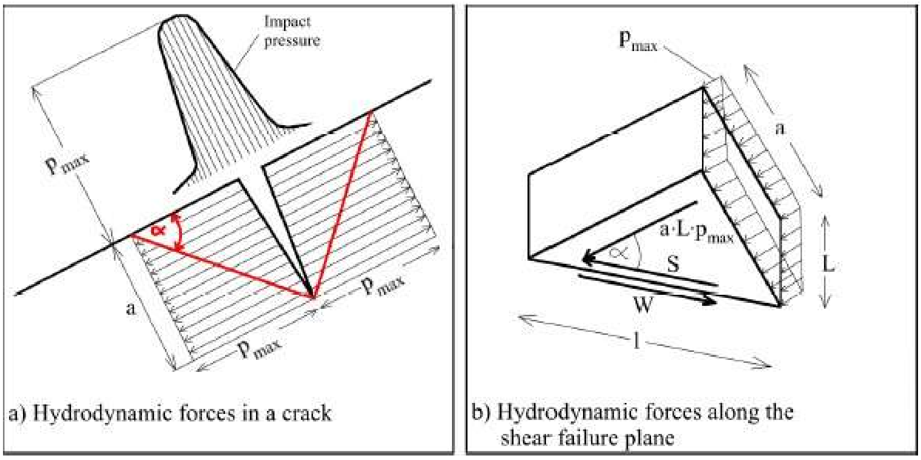



*Figure 2-11 Impact pressure distribution and forces acting inside the water-filled crack according to Führböter (1966) (Stanczak, 2006)*

The model considers the depth (a) and the length (L) of the cracks. The width of the crack is ignored. Since the weight of the soil body is assumed to be very small, the only resistant force considered in this model is the shear strength (S) represented by soil cohesion (c).

The force acting along both wall surface of the crack ($F_{crack}$) is calculated as follow:

$$F_{crack} = aLp_{\max} \tag{2.11}$$

Where:
$F_{crack}$ : Force acting on the wall of crack [N]
a : Depth of the crack [m]
L : Length of the crack [m]
$P_{max}$ : Maximum impact pressure acting on the surface [Pa]

The shear stress force (S) is acting along the leaning plane with an angle:

$$S = aLp_{\max} \cos\alpha \tag{2.12}$$

This resistant force is provided by the soil cohesion (c)

$$W = lLc \tag{2.13}$$

Where:
S : Shear force
W : Resistance force
c : Shear strength/cohesion
α : The angle of leaning plane to the surface
l : Length of shear failure

The length of the shear failure (l) is (see Figure 2-11b):

$$l = \frac{a}{\sin\alpha} \tag{2.14}$$

Therefore, it gives:

$$W = \frac{aLc}{\sin\alpha} \tag{2.15}$$

By solving the limit state equation S=W, the angle of shear failure (α) is

$$\sin\alpha = \sqrt{\frac{1}{2} \pm \sqrt{\frac{1}{4} - \left(\frac{c}{p_{\max}}\right)^2}} \tag{2.16}$$

From equation (2.16), the critical impact pressure that can lead to the failure in the form of the block soil is:

$$p_{\max} = 2c \tag{2.17}$$

### Graphical interpretation

A graphical analysis to estimate possible failure mechanism due to impact pressure on water-filled cracks was investigated by Richwien, 2003 (Figure 2-12). The line that closes the polygon of resisting forces is the maximal force that can be absorbed by soil without failure ($S_{poss}$). By considering other factors that are neglected in Führböter theory such as weight of the soil (G), reaction of the soil (Q), and pore water pressure (U), the failure is defined as the situation when the shock pressure force (S) is larger than the maximum force the soil can be absorbed ($S_{poss}$). The $S_{poss}$ may decrease, for example, because of poor cohesion (c), or the resisting forces are not working at the same time.

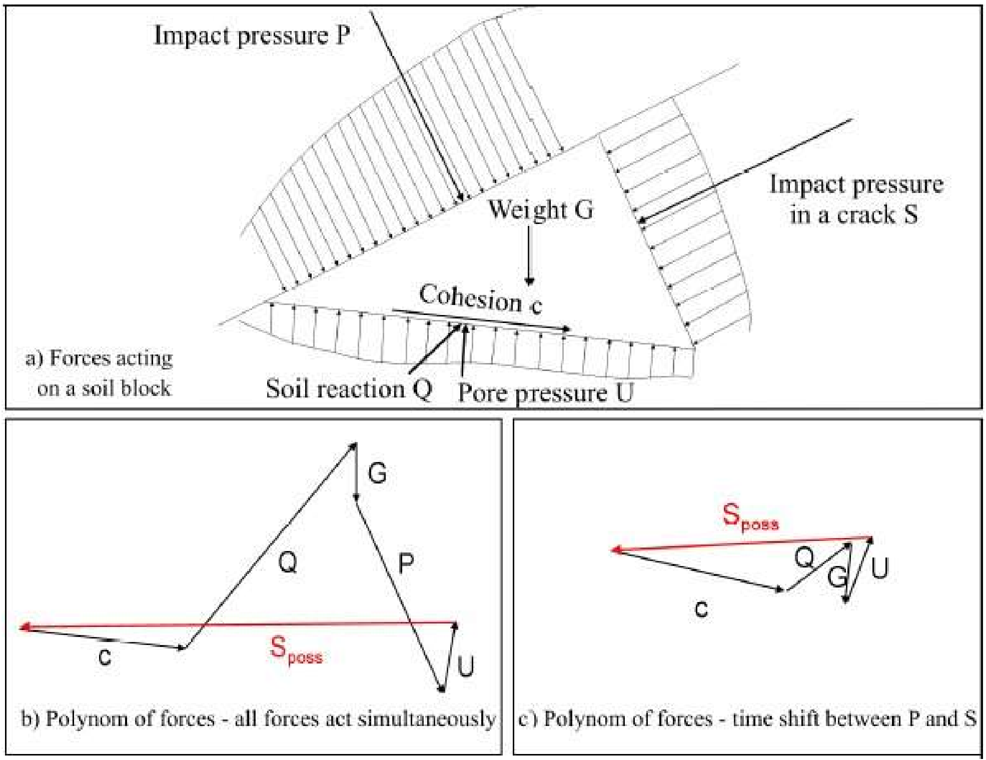



*Figure 2-12 Richwien graphical approach in failure mechanism of water-filled cracks hit by impact pressure, G= Weight of the soil, Q=Reaction of the soil, U= Pore water pressure, P=Hydrostatic pressure, c=Cohesion, S=Impact pressure (Stanczak, 2006)*

### Pressures propagation and distribution

The characteristic of pressure pulses propagating through water-filled cracks was investigated by Müller, G at all (2003). At the beginning, a series of experiment was conducted using a wave tank with various cracks dimension. It was found out that the impact pressure entering the water-filled cracks travels at speeds of 70-150 m/s. This pressure attenuates rapidly but increases in duration. Furthermore, inside the crack with a closed end, several factors that reduce the pressures such as reflection, subsequent doubling of the pressure pulse, and water-air mixture were observed.

In order to have more controllable environment, a new apparatus was developed to investigate pressure pulse propagation in more detail. This allows one to have the generation of pressure pulses with controlled magnitude and duration traveling on various cross sections of water-filled cracks. Figure 2-13 shows the working principle of the apparatus which consists of a dropping piston that generate controlled pressure, a pressure chamber to hold the piston connected with the crack model that can be closed/opened at the end. With a drop height of 50mm, the piston hits the water surface, creating a pressure pulse inside the water-filled chamber. The pressure inside the chamber is the same as the pressure at the crack entrance. Pressure propagation along the water-filled crack is measured by a series of pressure gauges.

Results from this experiment show that the speed of pressure pulse propagation inside the water-filled crack increases from 50-60 m/s for 0.5 mm cracks to 250-300 m/s for 10-18.25 mm wide cracks. It means the wider the cracks, the faster the pressure pulse propagates. Water-air mixture can slow down the propagation speed while the model stiffness and geometry changes of the cracks do not. The pressure pulses also experience superposition, reflection, and damping (attenuation process). In general, the impact pressures are at maximum at the crack entrance and then gradually weaken in increasing distance inside the water-filled crack (Figure 2-14).

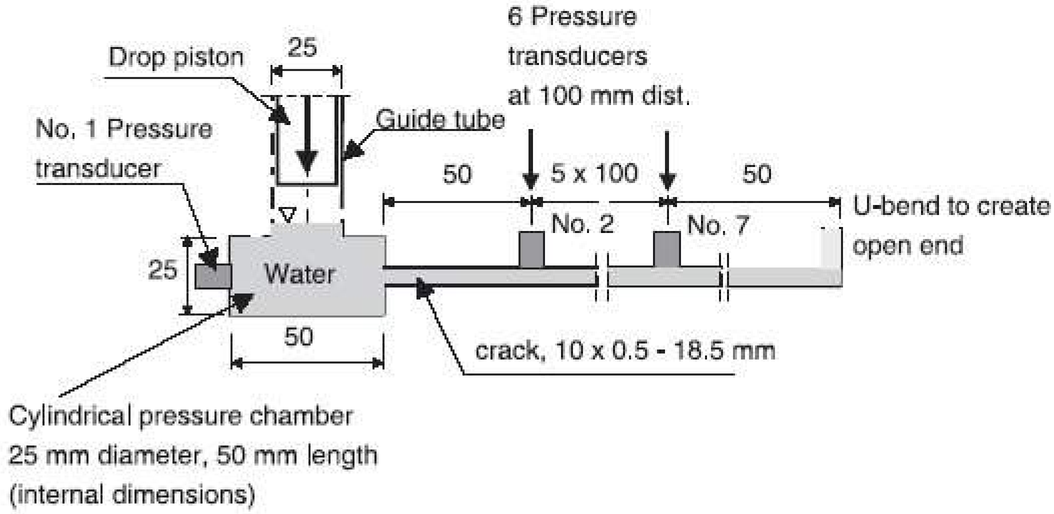



*Figure 2-13 Principles of drop test apparatus (Müller et al, 2003)*

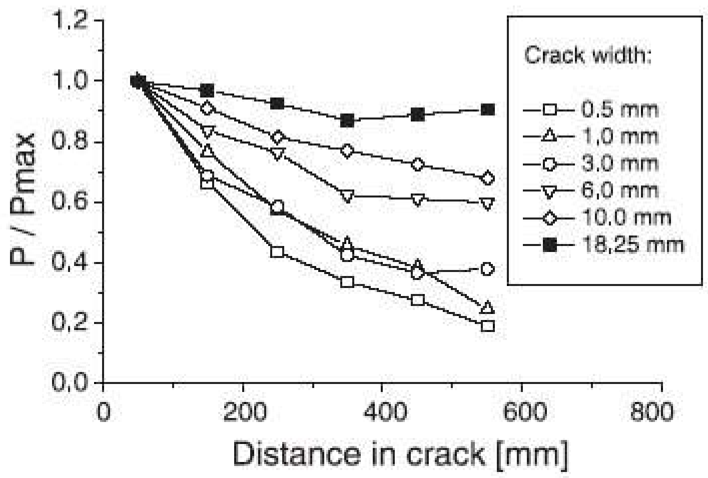



*Figure 2-14 The pressure magnitude decreases with increasing distance from the crack entrance (Müller et al, 2003)*

Recent works on the impact pressure propagation inside water-filled cracks were carried out by Pachnio (2005) at Leichtweiß-Institut für Wasserbau (LWI), Technische Universität (TU) Braunschweig, Germany. An impact pressure machine was constructed using a falling water mass with various amounts of water and fall heights (Figure 2-15). A metal plate with a gap was put below the water mass tube. A series of pressure gauges were mounted vertically under the gap and horizontally next to the gap. Two scenarios was considered during the experiment; air-filled and water-filled crack. The results of these were compared and analyzed to explain wave impact propagation inside water-filled crack and its mechanism. Other factors such as inclination of the gap, amount of water masses, variation in gap width, and variation in fall height are also considered.

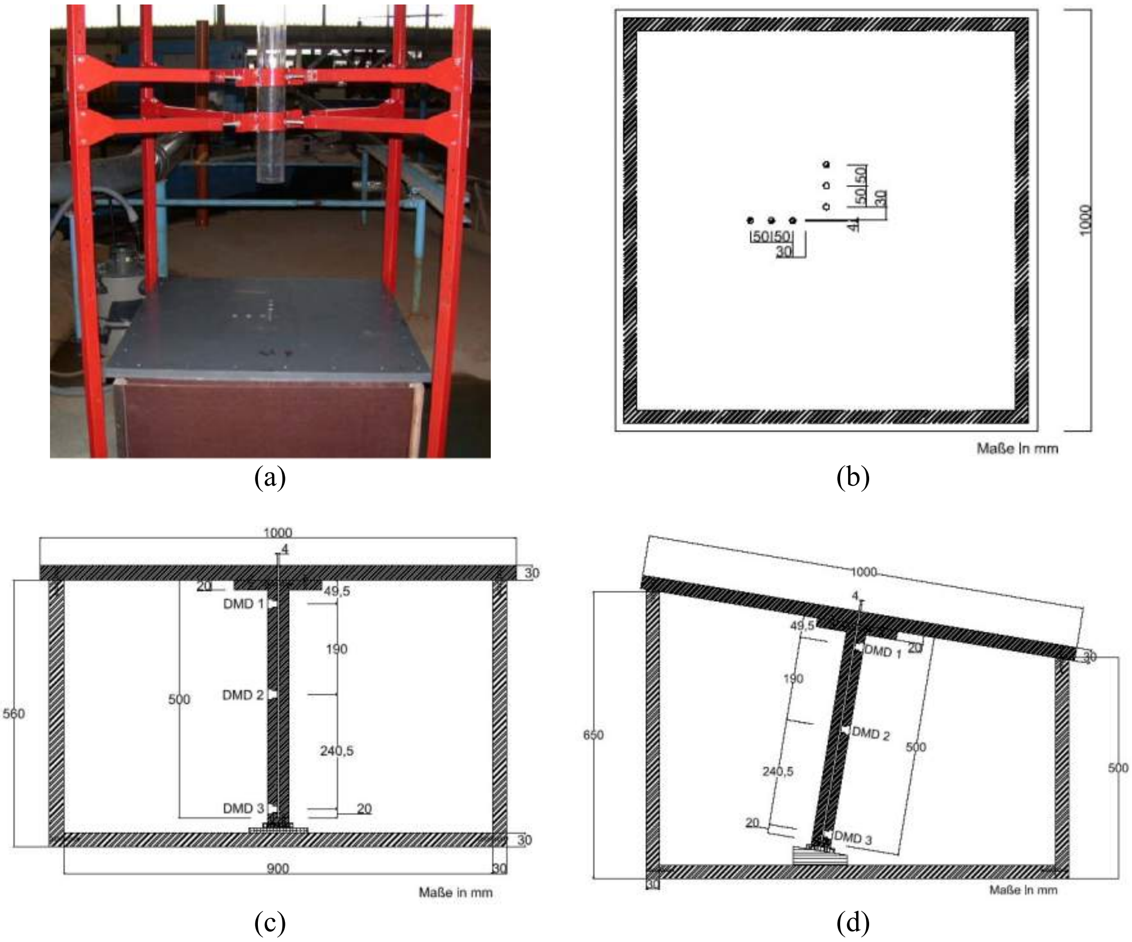



*Figure 2-15 (a) Overview of the impact pressure machine (b) Top view of steel plate with its 6 mounted pressure gauges next to the gap (c) Side view of the gap and its pressure gauges (DMD) in a plane surface (d) Side view of the gap and its pressure gauges (DMD) in an inclined surface (Pachnio, 2005)*

Pachnio (2005) measured the maximum impact pressure ($p_{max}$) as a function of fall height ($h_f$) and water masses ($h_w$).

$$p_{\max} \frac{2h_w\rho\sqrt{2gh_f}}{\Delta t} \tag{2.18}$$

where:

$p_{max}$ : Maximum impact pressure [Pa]
$h_w$ : Water level in the tube [m], the tube has 10 cm diameter.
$h_f$ : Fall height [m]
$\Delta t$ : Impact duration [s]
$\rho$ : Water density [1000 kg/m$^3$]
g : Gravity acceleration [10 m/s$^2$]

The impact duration flat depends on the water level in the tube. The fall height can be lower/higher by removing the tube up and down until maximum 125 cm above the gap.

Results from the series of experiment show that the impact pressure inside the water-filled gaps is much higher than in the air-filled gaps. The average pressure inside the air-filled gaps reaches 31 % below the reference pressure (the pressure at the gap entrance). Meanwhile, the average pressure inside the water-filled gaps is 41 % higher than the reference pressure. In vertical direction, the pressure increases and reaches its maximum at farthest distance from the entrance gap (Figure 2-16).

The increase of the impact pressure propagation is obvious for water-filled gaps. In horizontal direction, the pressure propagation decreases by distance (Figure 2-17). The maximum measured pressure (100% of the reference pressure) in horizontal direction is at the perpendicular pressure gauges just next to the gap. A narrower gap seems to have less impact pressure (Figure 2-16) and there are no significant impact pressure differences between plane and inclined gaps.

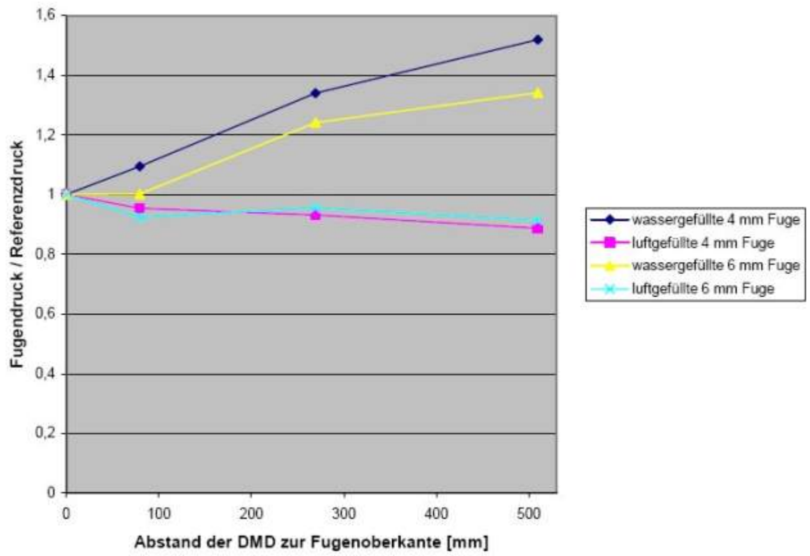



*Figure 2-16 Comparison of vertically impact pressures propagation inside the air-filled and water-filled gap for different gap widths (4mm and 6mm) (Pachnio, 2005)*

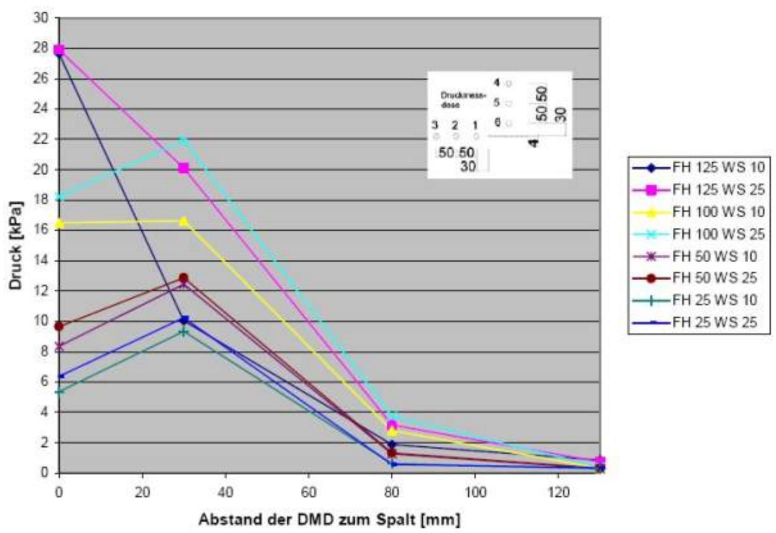



*Figure 2-17 The decrease impact pressures in horizontal direction next to the gap with various fall heights (FH) and water masses (WS) (Pachnio, 2005)*

### Recent experiments on clay with water-filled cracks

Experiments on water-filled crack subjected to breaking impact pressures were carried out by Rohloff and Stanczak (2006) as the implementation of Pachnio findings. There are 3 types of clay from *Cäciliengroden*, *Elisabethgroden km-9.0* and, *Elisabethgroden km-3.5* (Lower Saxony State, Germany) which are categorized as good, moderate, and bad clay respectively. A clay sample was put in a transparent box which has dimension of 90cm long, 10cm wide, and 60cm high. The clay sample was then compacted in six layers. For each type of clay, an artificial water-filled crack was introduced which has dimension of 1cm wide, 10cm long, and 15cm deep. The water-filled crack was subjected to an impact with different fall heights (50, 75, 100, 125, and 165cm) and repeated for 5 times. The results are as follows:

For bad clay, the samples have water content ranging from 23.3% - 36.7% with angle of failures ranging from 75.26$^o$ – 84.35$^o$. The angle of failure differences between measurement and calculation using Führböter theory is in the range of 6.53% - 19.06%.

For moderate clay, the samples have water content ranging from 27.3% - 71.1% with angle of failures ranging from 41.72$^o$ – 84.94$^o$. The angle of failure differences between measurement and calculation using Führböter theory is in the range of 4.04% - 33.12%.

For good clay, the samples have water content ranging from 27.9% - 52.6% with angle of failures ranging from 38.28$^o$ – 76.46$^o$. The angle of failure differences between measurement and calculation using Führböter theory are totally different because the theory says that the shear strengths of the clay are larger than the impact pressures. The failure occurred in the experiment due to the presence of hard lumps connected by soft fraction of soils.

The liquid limit (LL) of moderate clay is 41%. According to the German DIN-specification consistency (Table 2-4), 10 out of 25 tests carried out by Rohloff and Stanczak (2006) are in *liquid state* (Larger than the liquid limit), 5 out of 15 are in *slushy* condition ($_{Ic}$<0.25). The rest 9 samples are in *very soft* condition (0.25<$_{Ic}$<0.50). There is only 1 sample with *soft* condition ($_{Ic}$>0.50).

There is no information about compaction. The eroded soils for all experiments are always in the form of particles and small aggregates.

*Table 2-4 Relation between consistency and strength relation according to DIN-18122 (Lubking, 1998)*

| consistency index $I_c$ | consistency according DIN 18122 | liquidity index $I_L$ |
|---|---|---|
| < 0 | liquid | > 1.0 |
| 0 | liquid limit $w_L$ | 1.0 |
| 0 - 0.25 | slushy | 0.75 - 1.0 |
| 0.25 - 0.50 | very soft | 0.50 - 0.75 |
| 0.50 - 0.75 | soft | 0.25 - 0.50 |
| 0.75 - 1.0 | stiff | 0 - 0.25 |
| 1.0 | plastic limit $w_p$ | 0 |
| 1.0 - 1.25 | semi-firm | (- 0.25) - 0 |
| > 1.25 | firm | < (- 0.25) |

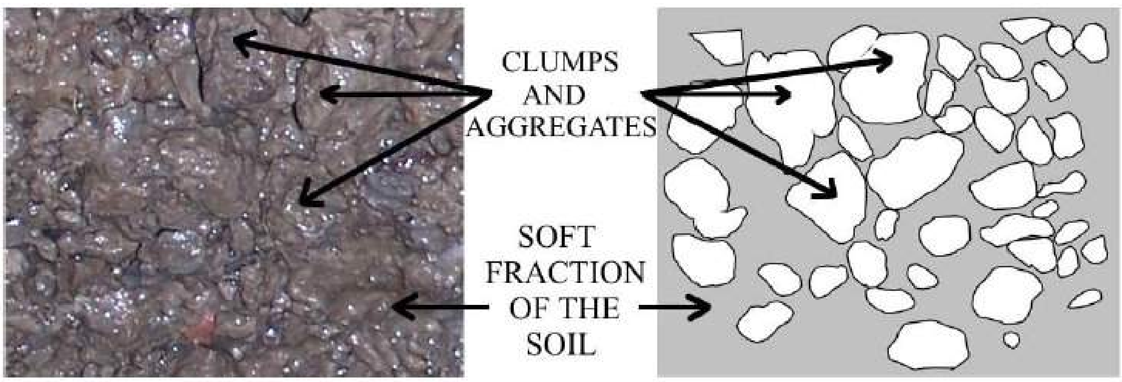



*Figure 2-18 Hard lumps structure found in strong clay (Stanczak, 2006)*

### 2.5.2 Erosion on surface erosion on clay covers without cracks

Investigations on the reactions of dike revetment and its subsoil had been reported by several authors. Richwien and Wehner (1988) investigated the stresses distribution of non-cohesive subsoil under the influence of breaking wave pressures. Führböter (1976) carried out an experiment to check revetment stability of block concrete stones hit by the breaking waves. Führböter & Sparboom (1988) investigated spatial shock distribution and shock pressure transfer in the subsoil in the Large Wave Channel. Führböter & Sparboom (1988) reported that more compacted cover layers can reduce the generated pressures up to 30 %. In the Netherlands, Deltagoot test (1992) and Scheldebak test (1994) were carried out in full-scale experiment to investigate the strength of various grass revetments under the loading of breaking wave impacts. Laboratory experiments with water-jets were carried out simulating the breaker tongue of plunging waves. Führböter (1966, 1969) used this technique to investigate the maximum generated impact pressure acting on various dike slopes. The influence of water layers are also studied additionally. From those investigations, the clay cover stability and its subsoil against the breaking wave impacts are not well explained. Clay, one of main dike construction materials mainly used for cover layers, often suffers from severe erosion due to breaking waves that further can initiate breaching. Therefore, knowledge on clay erosion behavior against the impacts is very important for safety design of sea dikes.

Investigation on clay erosion due to impact pressures has been widely reported outside the application of breaking wave impacts. The approach to estimate erosion of cohesive material subjected to impact loads starts from the stress based detachment equation.

$$\varepsilon = k_d (\tau_e - \tau_c)^a \tag{2.19}$$

Where:

$\varepsilon$ : Amount of eroded soil per single impact
$k_d$ : Detachment (erodibility) coefficient
$\tau_e$ : hydraulic boundary stress
$\tau_c$ : Soil critical stress to initiate erosion
a : Exponent of other aspects (e.g. water layer)

Hanson and Cook (2004) have been extensively investigating the erosion behaviors of clay types using jet erosion test (JET) with submerged samples (Figure 2-19). They used this apparatus to measure the erodibility and the critical stress of the soil both in field and laboratory. Influence of compaction and variation in water content are also investigated as comparison. The hydraulic boundary stress ($\tau_e$) was defined as the maximum shear stress acting upon the bed in the impingement region represented by the maximum velocity. Since the sample is submerged, the hydraulic boundary stress consists of stress due to direct impact in the impingement region (potential core) and the stress due to diffusion. The critical stress ($\tau_c$) was determined from the measurements where the scour depth of erosion reaches stable condition.

$$\tau_e = C_f \rho U_o^2 \quad for\, H \le H_p \tag{2.20}$$

$$\tau_e = C_f \rho \left( C_d U_o \frac{d_o}{H} \right)^2 \quad for\, H > H_p \tag{2.21}$$

Where:

$C_f$ : Friction coefficient (reported value is 0.00416)
$C_d$ : Diffusion coefficient (reported value is 6.3)
$U_o$ : Jet velocity
$D_o$ : Nozzle diameter at the origin
$H_p$ : Height of potential core
$H$ : Distance from the original bed to the nozzle entrance

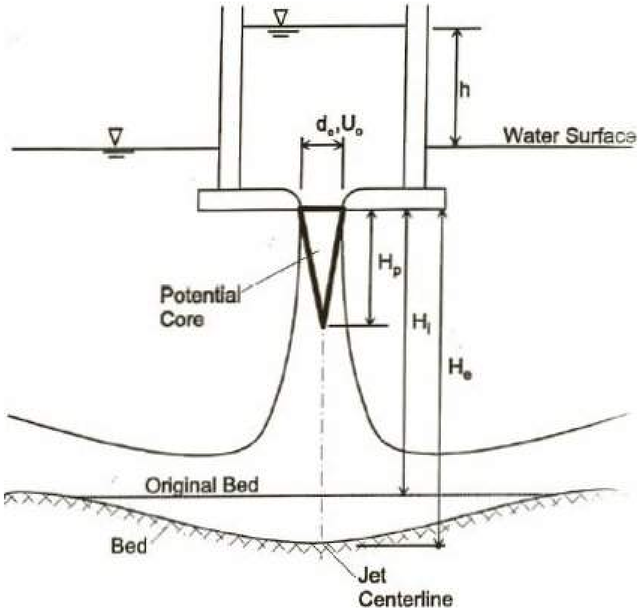



*Figure 2-19 JET Apparatus (Hanson, 1997)*

Other works related to soil erosion due to impact were mostly carried out for erosion investigation due to rain fall (splash erosion). Soil erosion due to rain drop impacts was investigated by Woolhiser (1990). Hydraulic boundary stress was expressed as the kinetic energy of the impact. He proposed an empirical formula to calculate the amount of eroded soil due to splash erosion with known kinetic energy:

$$R_d = k_d E_k e^{-wh} \tag{2.22}$$

Where:
$R_d$ : Volume of eroded soil after a single impact [cm$^3$]
$k_d$ : Empirical detachability coefficient [cm$^3$/J]
$E_k$ : Kinetic energy of an impact [J]
w : Empirical coefficient representing the effectiveness of a water layer to damp impact pressure
h : water layer thickness [cm]

Compared to the approach used by Hanson (1997), relevant parameters are included in Eq. (2.22) such as kinetic energy of the impact, water layer thickness (damping) and soil erodibility coefficient.

Recent works on surface erosion on compacted clay were investigated by Rohloff and Stanczak (2006) by using the falling water impact machine. Three types of clay representing good, moderate, and bad clay were tested with various impacts (or the fall height of the impact machine). The influence of water layers were also investigated by setting-up different water layer thicknesses (1cm, 2cm, 2.5cm, and 4cm). The presence of water layers significantly decreases the erodibility of soil. By applying the model concept of Wollhiser (1990) with calibrated coefficients, the experiment results showed good agreement with the model. The calibrated coefficients are derived from the generated maximum impact pressures not the kinetic energy as originally proposed by Wollhiser (1990). Kinetic energy ($E_k$) was replaced by maximum impact pressure ($p_{max}$). According to Stanczak (2006), Eq. (2.22) becomes:

$$R_{d,p} = k_{d,p} p_{\max} e^{-wh} \tag{2.23}$$

Where:
$R_{d,p}$ : Volume of eroded soil after a single impact [cm$^3$]
$k_{d,p}$ : Empirical detachability coefficient [cm$^3$/kPa]

$p_{max}$ : Maximum impact pressure [kPa]
w : Empirical coefficient representing the effectiveness of a water layer to damp impact pressure
h : water layer thickness [cm]

## 2.6 Summaries and conclusions from literature review

Up until now, investigations on the impact pressures generated by the breaking wave impacts have been widely known. On the other hand, the knowledge on the reactions of the subsoil, particularly clay, of the outer slope due to breaking wave impacts are still not well understood. For the erosion due to breaking wave impacts on water-filled cracks, the old theory of Führböter (1966) has some drawbacks:

1. Resistant force is only from cohesion; other forces such as the weight of the block soil and the pore pressure are ignored.
2. The width of the cracks is neglected.
3. The pressure distribution is assumed to be the same from top up to the bottom of the crack
4. No verification yet with experimental results.

Recent investigations in laboratory experiments done by Rohloff and Stanczak (2006) have left some questions:

1. Results from the tests of good clay which need to be repeated due to non-homogenous clay samples
2. Degree of compaction was unknown
3. Conditions of samples for moderate clay which were prepared with $_{Ic}$<0.50 are in questions. Clay with $_{Ic}$<0.50 is in very soft condition which is likely to be impossible to have cracks due to suction pressures.

The surface erosion of compacted clay due to wave impacts also has not been well understood. Difficulties in dealing with cohesive materials combined with the stresses generated by the impacts and limited past research on this topics resulted in big gaps between the knowledge on the impact pressure generated by breaking wave impacts and the reaction behavior of the subsoil of a dike. Some developed models in explaining the erosion of soils due to impacts are mainly for land erosion due to rainfall known as splash erosion. The concept of stress based detachment equation and splash erosion concept will be used in this report for deep investigation of surface erosion tests on compacted clay due to breaking waves. The Woolhiser (1990) formula to determine the erodibility of soil due to impact will be validated with the results from the experiments. The approach of Hanson et al (1997, 1999, 2006) in investigation of soil erodibility is also used for comparison and in experimental set-up.

As first step, these existing models could be used to investigate erosion due to impacts from the breaking waves. Therefore, experimental investigations in laboratory are greatly needed to examine and validate those existing models/theories. Furthermore, the knowledge on this topic will improve the design requirements for the outer slope of sea dike which is still not well developed.

# 3. Experimental Methods

## 3.1 Falling water impact machine

The falling water impact machine is used for the investigation of clay erosion due to breaking wave impacts. The impact machine consists of 3 main parts as follows:

- The water tube (water container). It functions as a holder of the water mass. The water inside the tube can be filled and released by control from the computer. The pressure inside the tube is maintained by a compressor in order to avoid leaking. The water level inside the tube is maintained at 25cm high.

- The steel frames. This steel frame is used for lowering or raising the tube in order to get desired fall height. These frames consist of 4 braces and 4 legs that can be fixed manually together by bolts. In this experiment, the fall height is always 162cm high.

- The wooden box. The clay sample is put inside the wooden box by placing transparent walls with 60 high, 90 cm long and 10 cm wide.

## 3.2 Clay Samples Preparation

The clay samples used for the experiment are from the dikes at Cäciliengroden and Elisabethgroden km-9.0, lower Saxony State, Germany. The clay properties of these clays are shown on Table 3-1**.** Grain size distributions are shown in Error! Reference source not found.**.**

*Table 3-1 Properties of clay samples*

| **Parameters** | **Cäciliengroden** | **Elisabethgroden km-9.0** |
|---|---|---|
| Clay percentage [%] | 35 | 20 |
| Silt percentage [%] | 53 | 45 |
| Sand percentage, $S_p$ [%] | 12 | 35 |
| Proctor Density, $\rho_{pr}$[g/cm$^3$] | 1.458 | 1.643 |
| Optimum water content, wpr [-] | 0.259 | 0.185 |
| Infiltration Rate, $k_f$ [m/s] | 1.37. 10-9 | 1.22. 10-8 |
| Decomposition time $t_{30\%}$ [s] | >259200 | 97263 |
| Shrinkage $V_s$ [%] | 48.61 | 30.12 |
| Plasticity index, $I_p$ [-] | 0.45 | 0.2067 |
| Undrained cohesion [kPa] | 22.6-70.7 | 18.6-40.0 |
| Natural water content, $w_l$ [-] | 0.40...0.50 | 0.22...0.26 |
| Flow limit $w_l$ [-] | 0.77 | 0.41 |
| Plastic limit, $w_p$ [-] | 0.32 | 0.2044 |
| Consistency index, $_{Ic}$ [-] | 0.60...0.82 | 0.73...0.92 |

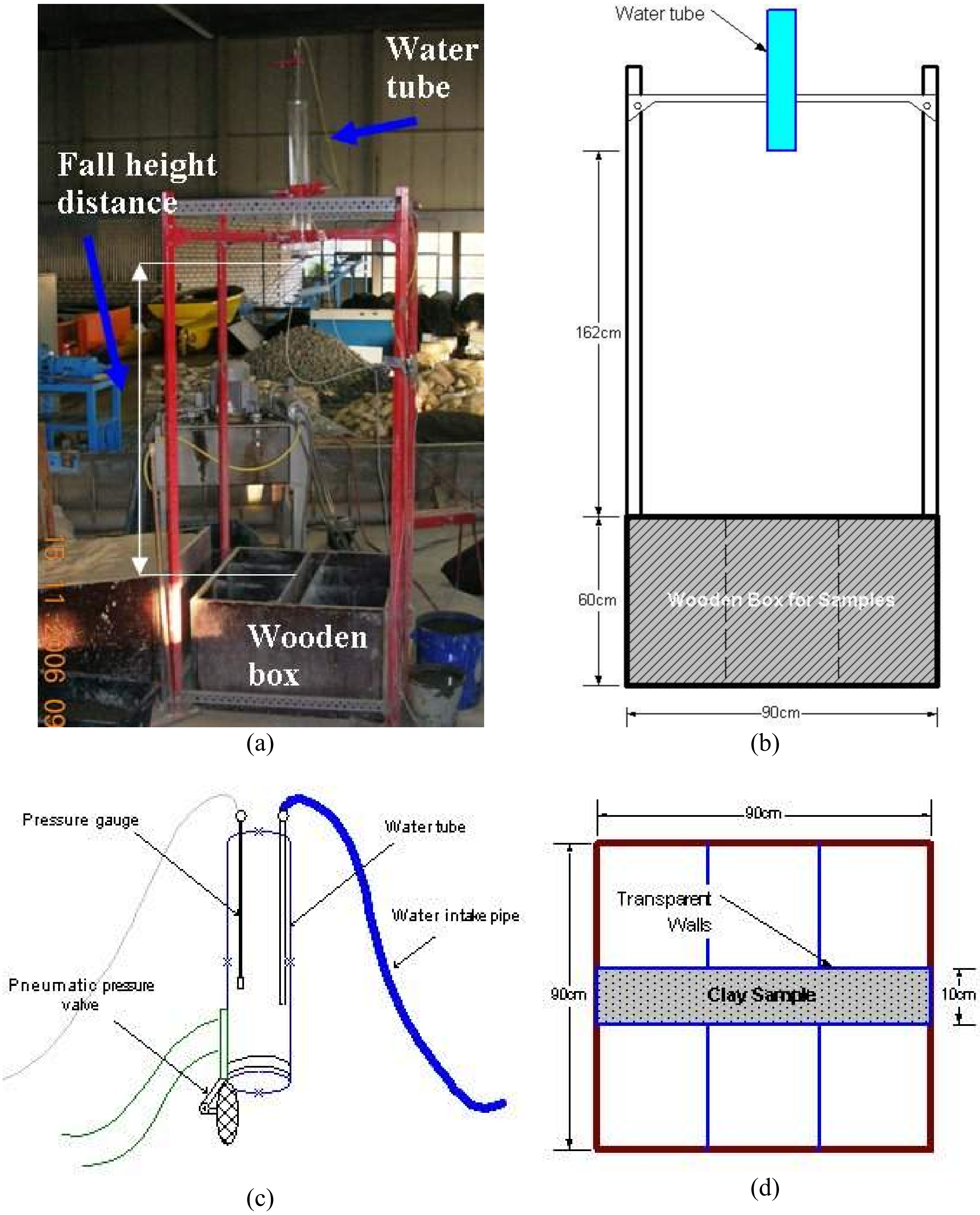



*Figure 3-1(a) The impact pressure machine overview (b) Schematisation of the impact pressure machine (c) the water tube principles (d) Schematisation of the wooden box with clay sample inside transparent wall*

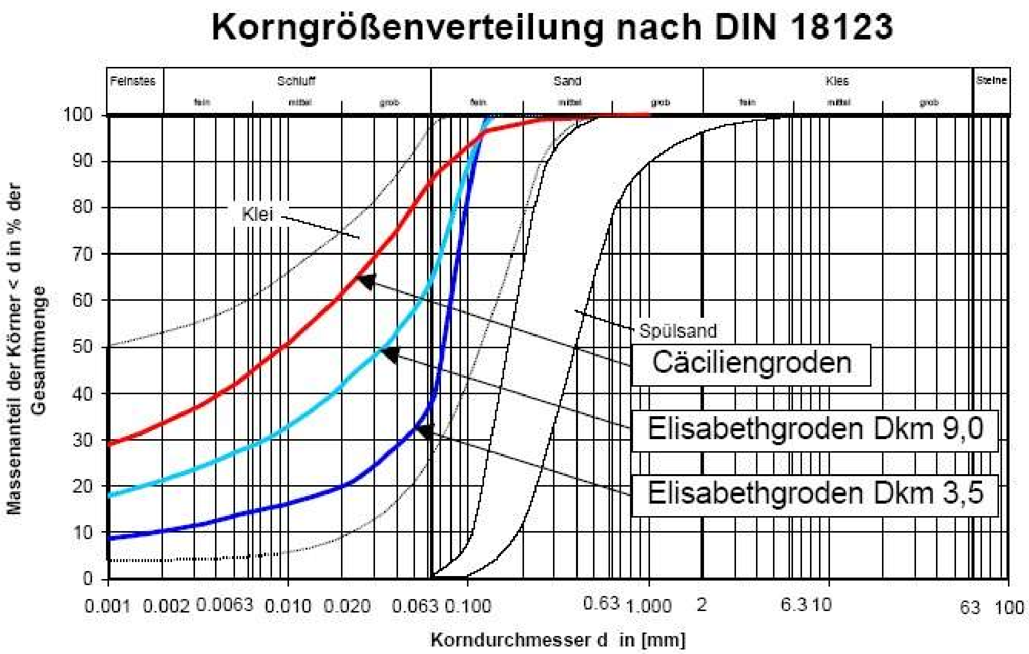



*Figure 3-2 Grain size distribution of Clay from Cäciliengroden and Elisabethgroden km-9.0*

According to *Technische Adviescommissie voor de Waterkeringen* (TAW, 1996) classification (see Table 2-1), those clay samples can be classified based on the sand content and the Atterberg limits:

*Table 3-2 Clay samples classification according to TAW, 1996*

| **Clay from** | **$w_l$ [-]** | **$I_p$ [-]** | **Sand content, $S_p$ [%]** | **Classification** |
|---|---|---|---|---|
| Cäciliengroden | 0.77 | 0.45 (>41,6) | 12 | Category 1 |
| Elisabethgroden km-9.0 | 0.41 | 0.208 | 35 | Category 2 |

According to Weiβmann (2003) (see section 0), the clay samples are classified as very good and good quality as shown in Table 3-1**.**

*Table 3-1 Clay samples classification according to Weiβmann (2003)*

| **Parameters** | **Cäciliengroden** | **Elisabethgroden km-9.0** |
|---|---|---|
| Infiltration rate ($B_1$) | 0.94 | 0.90 |
| Decomposition time ($B_2$) | 1.08 | 1.00 |
| Shrinkage ($B_3$) | 0.45 | 0.69 |
| Plasticity Index ($B_4$) | 1.20 | 0.71 |
| Classification Number (N) | 0.86 | 0.81 |
| Erosion resistant quality | Class 1 | Class 2 |

In this report, the erosion resistant classification terms from TAW (1996) will be used. Good clay for category 1 and moderate clay for category 2. Therefore, the term of good clay is for clay from Cäciliengroden and moderate clay is for clay from Elisabethgroden km-9.0.

### *3.2.1 Aspects related to sample preparation*

During the experiment, there are several aspects that should be considered that can influence the erosion resistant behavior of the clay

1. Water content

Water content of the clay can largely influence the strength of the soil particles that leads to erosion resistant capability. Kortenhaus (2003) describes the relationship between the water content and the undrained shear strength ($c_u$) of the clay for 3 types of clay including clay in Cäciliengroden and Elisabethgroden km-9.0.

- Clay of Cäciliengroden $c_u = 7230e^{-12w_c}$
- Clay of Elisabethgroden km-9.0 $c_u = 2800e^{-20w_c}$
- Clay of Elisabethgroden km-3.5 $c_u = 2550e^{-33w_c}$

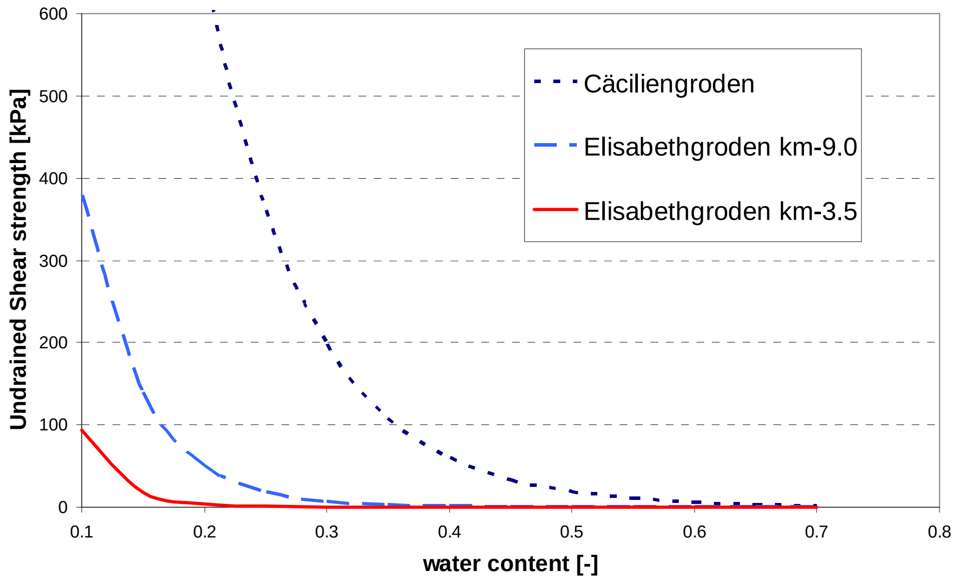



*Figure 3-3 Relationship between water content and undrained shear strength of the clay (Kortenhaus, 2003)*

2. Homogeneity

In order to have a controllable environment and more stable results, the soil sample should be homogeneous. The clay should be free from hard lumps that can cause undistributed shear strength (and water content) over the body of the clay samples. The only measure to get a homogeneous condition is by crushing the hard lumps and continuously blending up the clay sample prior to the experiment.

3. Impurities

The clay sample should not contain all kinds of substances or obstacles such as stones, woods, organic materials, etc.

4. Compaction

TAW (1996) recommends to use consistency index ($_{Ic}$) values as a reference to achieve an optimum compaction for erosion resistant clay. $_{Ic}$ is defined as:

$$I_c = \frac{w_l - w_n}{w_l - w_p} = \frac{w_l - w_n}{I_p} \tag{3-1}$$

Where:

$I_p$ : Plasticity index

$w_l$ : Flow limit
$w_p$ : Plastic limit
$w_n$ : available water content of the soil sample

The requirement for all cover layer should be $I_c \leq 0{,}75$. In order to have easiness in doing compaction, TAW gives a range of water content $w_c$ based on the value of $I_c$.

Maximum water content $w_c$(min):

$$w_{c(\max)} = w_l - (0.75)I_p \qquad (5.3.1)$$

$$w_{c(\max)} = w_l - (0.75)I_p \qquad (3.2)$$

Minimum water content is optimum water content from proctor test (wpr) or maximum $w_c$(max) minus 5 up to 10 %.

$$w_{c(\min)} = w_{pr} \text{ or } w_{c(\min)} = w_{c(\max)} - (5...10\%) \qquad (5.3.3)$$

Based on those criteria, the clay samples are prepared in 3 groups as shown in Table 3-2.

*Table 3-2 Preparation of samples based on water content classification of TAW (1996)*

| Clay Sample | Dry $w_n$ | Recommended $w_n$ | Wet $w_n$ |
|---|---|---|---|
| Category 1 | < 25.9 | 25.9≥wc≥43.2 | >43.2 |
| Category 2 | <18.5 | 18.5≥wc≥25.6 | >25.6 |

### 3.2.2 Sample preparation procedure

In order to have desired water content for the sample, the procedure below was carried out:

1. Measure the water content of the available water content by considering all forms of the physical appearances (dust, particles, small aggregates, hard lumps)
2. Calculate the needed water content by means of water mass comparison
3. Prepare the required clay by putting them into basket. Moisture the available soil by pouring water based on the needed water content per required volume (Figure 3-4).
4. After few hours, blend up the moistured clay several times. Let the clay to hydrate for at least 24 hours.
5. Sometimes water remains at the bottom of the basket. Thus, the clay at the bottom is much wetter than the clay above it. If this happens, separate the clay sample half way, and put the wetter clay on top of porous material (geotextile) to dry out some water.
6. Crush manually the remaining lumps (if there are any)

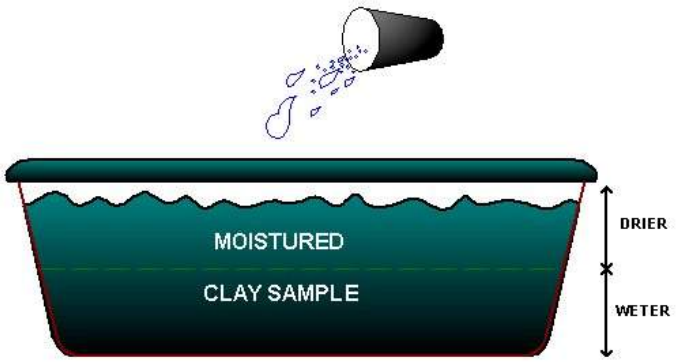



*Figure 3-4 Moistured clay inside the basket consist of upper part which has drier clay and lower part which has wetter clay*

## 3.3 Erosion Experiments on Compacted Clay with Water-filled Cracks

### 3.3.1 Experimental scenario

The test on water-filled crack on clay cover was carried out by following the scenario as shown at Table 3-3

*Table 3-3 Experimental scenario for water-filled crack tests*

| Experiments | Aspects | | | | | | Repetition |
|---|---|---|---|---|---|---|---|
| | Compaction | Water content | Soil types 3) | Fall Height (Impacts) | Number of Impacts | Soil homogeneity 5) | |
| Scenario 1 | Recommended 1) | Recommended & high 2) | I & II | Maximum 4) | 1 | yes | 5 |
| Scenario 2 6) | Recommended 1) | Recommended & high 2) | I&II | 125cm | 1 | yes | 5 |

Captions:
1) The same degree of compaction (above 95% of maximum proctor density)
2) Between PL and LL
3) Clay category I (god clay) and category II (moderate clay)
4) Maximum fall height is 162 cm equivalent to the impact generated by 1,2m wave height
5) Free of hard lumps
6) Scenario 2 will be abandoned if no failures in scenario 1

Samples with water content below the recommended ones are not considered because from the operational point of view it is hardly possible to make artificial cracks on the surface of dry clay and the water inside the crack will be drained quickly. Samples with water content near the flow limit are also not considered for the same reason. It is easy to make an artificial crack with water content close to flow limit but it is also easy to collapse by itself. Maximum fall height of 162 cm was chosen because from the estimation of soil shear strength (Figure 3.3). The failure is only possible by applying the maximum generated impact pressure.
Hard lumps can cause unreliable results because of undistributed water content and strength inside the samples. Soil homogeneity should be guaranteed.

### 3.3.2 Experimental procedure

After the clay with desired water content is ready, the following procedure can be started (details of this procedure including pictures can be seen in Appendix C):

1. Position the water tube into the required fall height. The maximum impact pressure that can be generated by the machine is 24.75 kPa for the fall height (fh) of 1.62 m, water column (fw) of 0.25 m, and impact duration of 0.115 s (Pachnio, 2005).

2. The transparent box is filled in by the clay and compacted in six layers (every 10 cm thickness) using a hammer which has dimension of 9.9cm x 17.4cm and weight of about 2.258 N.

3. Measure the soil density using sand replacement method about 30 cm from the artificial crack (details in Appendix E).

4. Position the crack location so that the water mass will fall exactly at the crack.

5. An artificial crack can be made by inserting 2 plates into the clay sample. The soil between the plates then is removed. The crack dimension is 15 cm deep, 1 cm wide and 10 cm long (Figure 3-5).

*Figure 3-5 The making of artificial crack*

The soil between 2 inserted thin plates is digged out

6. The artificial crack is filled up by water.

7. Start the test by automatically filling and releasing the water mass from the tube hitting the water-filled crack.

8. After the impact. If there is a failure, a picture is taken and the shear angle failure is measured. If there are no any failures, the impacts will be carried on until significant failure is observed (Figure 3-6).

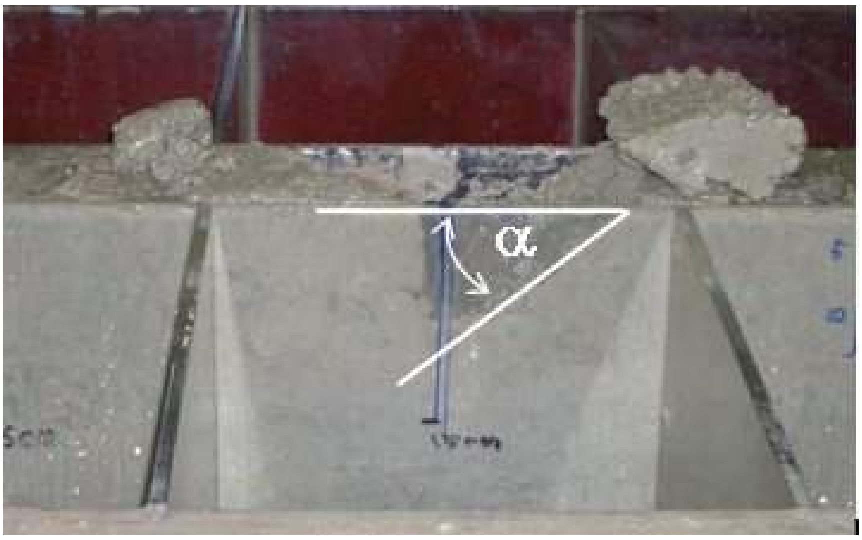



*Figure 3-6 Shear angle failure measurement*

9. Measure the actual water content from the side of the crack for shear strength estimation.

## 3.4 Erosion Experiments on Compacted Clay without Cracks (Surface Erosion Tests)

### 3.4.1 Experimental Scenario

Tests on surface erosion of compacted clay cover were carried out by following the scenario as shown at Table 3-4.

*Table 3-4 Experiment scenario for test on surface erosion of compacted clay*

| Experiments | Aspects | | | | | | | Repetition |
|---|---|---|---|---|---|---|---|---|
| | Compaction 1) | Water content | Clay types 3) | Fall Height 4) | Number of Impacts 5) | Water layer [cm] | Soil homogeneity 7) | |
| Scenario 1 | Recommended | Recommended 2) | I | Maximum | 100 - 500 | 0 | yes | 5 |
| Scenario 2 | Recommended | Dry & High 6) | I | Maximum | 50 | 0 | yes | 10 |
| Scenario 3 | Recommended | Recommended 2) | II | Maximum | 50 - 200 | 0 | yes | 5 |
| Scenario 4 | Recommended | Dry & High 6) | II | Maximum | 50 | 0 | yes | 10 |

Captions:
1) The same degree of compaction (above 95% of maximum proctor density)
2) Based on TAW recommendation between PL and LL
3) Clay category I (god clay) and category II (moderate clay)
4) maximum fall height is 162 cm equivalent with impacts generated by 1,2m wave height
5) Variation in number of impacts depends on the reached scour depth of more or less 10cm deep
6) High water content means above the recommendation. Dry water content is below the recommendation of TAW
7) Free of hard lumps

Preliminary scenario was carried out by considering the influences of water layers and variation in fall height. From previous tests carried out by Rohloff and Stanczak (2006), it was clear that water layers absorb the impact energy causing less erosion on the clay layers. Results from preliminary scenarios also showed the same tendency where the water layer significantly reduces erosion rate of the compacted clay. Variation in fall height was also abandoned due to the fact that the prepared clay samples are strong enough against the maximum impacts generated by the machine. Preliminary scenarios were also carried out by lowering own the fall height. Results showed that the lower the fall height the longer it takes to erode. From practical point of view, applying the maximum impact (24.75 kPa for 162cm of fall height) is more feasible for design purposes.

Compaction was planned to be at least 95 % of proctor density. Clay samples were prepared in 3 groups; the group of recommended, high, and dry water content based on TAW classification (see Section 3.2.1). The group of recommended water content is the samples with water contents between optimum water content ($w_{pr}$ or $w_c$(min)) up to maximum recommended water content ($w_c$(max)). The group of high water content is the samples with water contents above the maximum recommended water content ($w_c$(max)) and below the liquid limit. The group of dry water content is considered as the water contents below the optimum water content ($w_{pr}$ or $w_c$(min)).

During the process of releasing impacts, measurements of eroded clay were carried out, for example, every 50 impacts for impact number of 500.

Hard lumps can cause unreliable results because of undistributed water content and strength inside the samples. Soil homogeneity should be guaranteed.

### 3.4.2 Experimental procedure

After the clay with desired water content is ready, the following procedure can be started (details of this procedure including pictures can be seen in Appendix C):

1. Position the water tube into the required fall height. The maximum impact pressure that can be generated by the machine is 24.75 kPa for the fall height ($f_h$) of 1.62 m, water column ($f_w$) of 0.25 m, and impact duration of 0.115 s (Pachnio, 2005).

2. The transparent box is filled in by the clay and compacted in six layers (every 10 cm thickness) using a hammer which has dimension of 9.9cm x 17.4cm and weight of about 2.258 N.

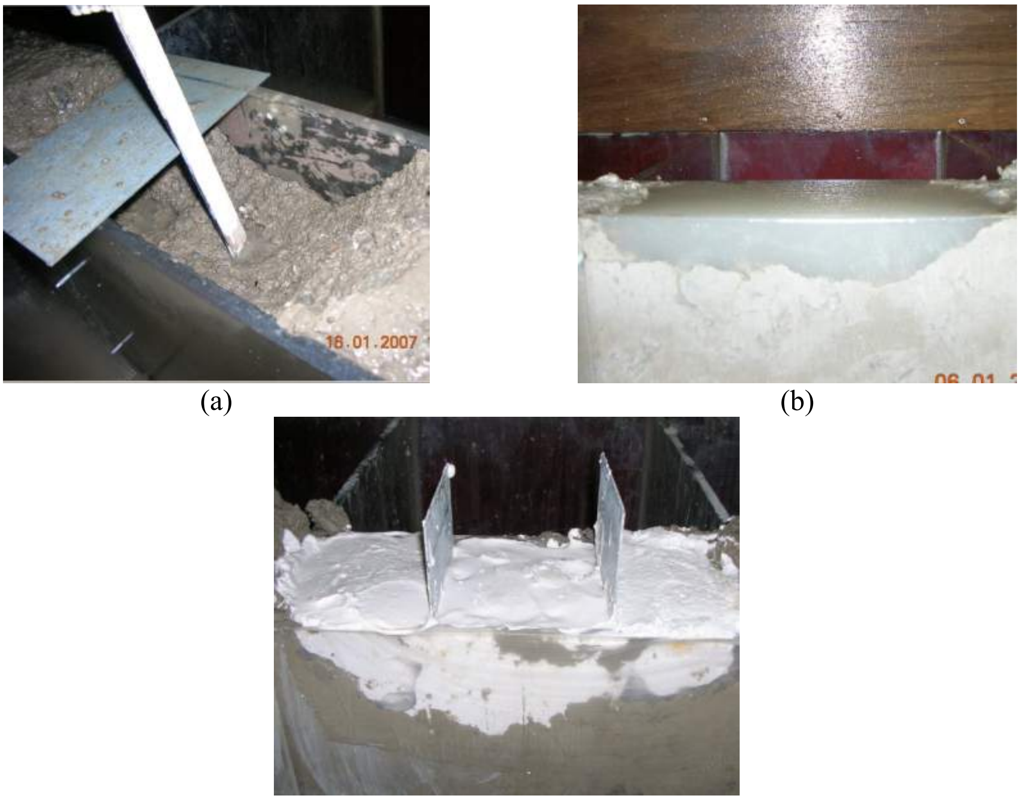
3. Measure the soil density using sand replacement method about 30 cm from the artificial crack (details in Appendix E).

4. Start the test by automatically filling and releasing the water mass with desired number of impacts. The number of impacts can be set automatically via computer control.

5. During the process of releasing impacts, measurements of eroded clay were carried out to see the progress of erosion, for example, every 50 impacts for impact number of 500 until finish. The measurement of eroded clay are divided into 3 (see Appendix D for details):
   - Eroded soil measurement by gypsum cast
   - Eroded soil measurement by filling water into the scour hole.
   - Eroded soil measurement by using a ruler to measure only the maximum depth of scour hole

6. Picture is taken for every progress of eroded soil. Important moments also should be documented such as development of cracks or removal of big block soils.

7. If there is doubt due to certain circumstances, measurement of water content after completing the impacts is worth to do.



(a) (b)

(c)

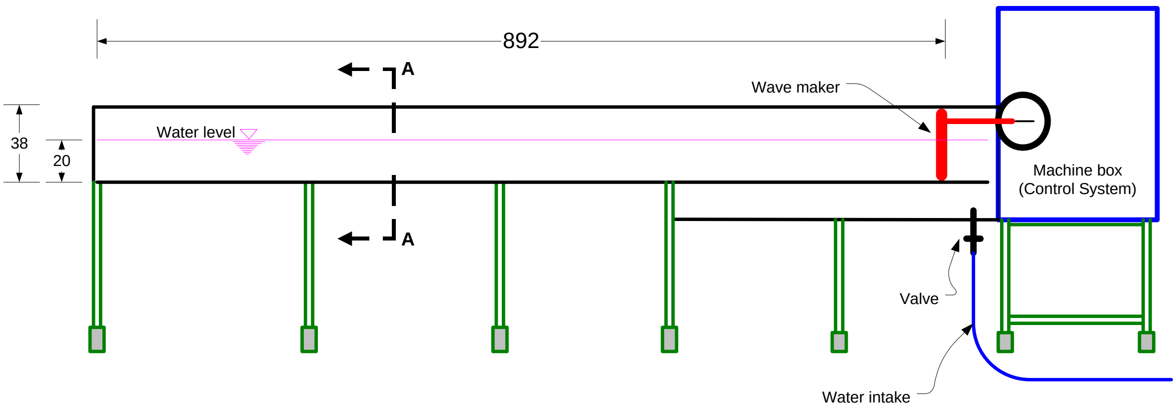
*Figure 3-7 Method of eroded soil measurements (a) measurement the scour depth by a ruler (b) Scour volume measurement by filling water (c) Scour volume measurement by gypsum cast*

## 3.5 Experiment on Failure Mechanism of Clay Cover Due to Breaking Wave Impacts

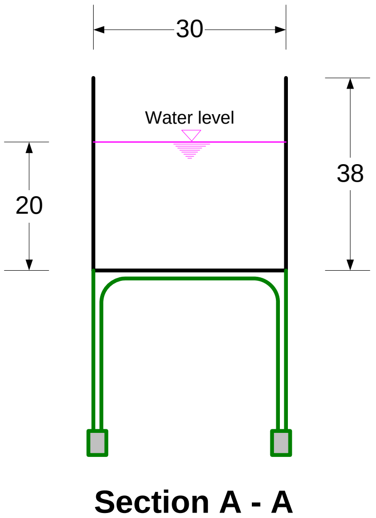
### 3.5.1 Flume description

The flume for this experiment only generates regular waves. It has transparent walls with several sections in length. The length of the flume can be modified by adding/removing the sections. The wave maker is pedal type which can move back and forth by means of electro-mechanical motor. The generated wave height can be adjusted by changing the stroke distance. The maximum stroke distance is 10cm. The frequency of the stroke can be adjusted manually from the switch-on/off handle. The intake water is from the bottom of the flume near the wave maker. The outflow is located at the bottom of the end side of the section. The length of the flume used in the experiment has length of 892m from the wave maker till the end of the section. The width is 30cm and the height is 38cm. At the machine box, the frequency of the stroke can be seen digitally.

(a)




(b)




(c)

*Figure 3-8 (a) The flume overview (b) General dimensions of the flume in centimetre (c) Cross section of the flume*

### 3.5.2 Material Preparation

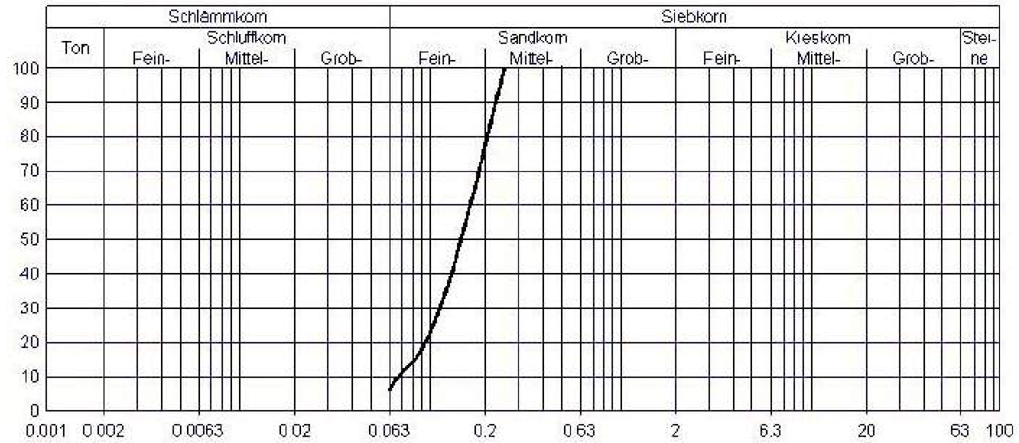
The dike model consists of 4 materials:

- Clay

Clay used for the cover layer is moderate clay (clay from Elisabethgroden km-9.0, see Section 3.2). The water content of the clay was set to have wn=30%. This water content was chosen because it is soft enough for shaping the dike and it is strong enough to hold the clay body stable (Not flowing, not too easy to deform)

- Sand

The sand used for the core of the dike is artificial sand. It is very soft with d50=0.103mm. The grain size distribution of this type of sand is shown in Figure 3-9




| Bodenart | Bd-Gruppe | U | C | T | U | S | G | | Kf(Mallet & Pacquant) |
|---|---|---|---|---|---|---|---|---|---|
| fS, ms, u' | SU | 2,4 | 1,1 | - | 6 | 94 | - | [Gew%] | 1,68 E-05 [m/s] |

*Figure 3-9 Grain size distribution of the artificial sand for dike core*

- Geotextile

Geotextile is used for the protection of the bottom of the inner slope to avoid failure due to piping. This geotextile is laid down between the outer surface of the sand core and the clay cover.

- Stones

Stones are used for protection of the bottom of the outer slope to avoid damage at the sea-side toe due to reflected waves. These stones are to be placed around the dike toe.

### 3.5.3 Dike Model Design

The dike was design based on the maximum wave height that can be generated by the flume. Measurement of wave characteristics was done by using a dummy dike. The dummy dike is made from plywood which has 1:4 slope at the sea side (Figure 3.10). Several trials were carried out to have impression of the wave characteristics, the water level and the stroke were set to 20cm and 8cm respectively to generate the maximum wave height (H) of 10cm. The location of wave measurement is at 30cm in front of the dike toe. Other characteristics such as run-up and period are shown in Table 3.7.

The crest of the dike should be high enough to prevent huge amount of overtopping. From Table 3.7, it is decided to design the crest level up to 38cm which is the height of the flume. With this crest level, small amount overtopping will occur. This overtopping will not cause damage to the inner side of the dike

The thickness of the clay cover is scaled down based on the maximum generated wave height of 10cm. In practice, the clay layer thickness is about 0.8 – 1.5m for design wave height H=1m. Therefore, for wave height H=10cm, the clay cover thickness for the model is set to 8cm. Taking account the crest width of the crest in practice of about 3m, the crest width of the model is then 30 cm.

The dike model inside the flume is put as far as possible from the wave maker. The model also has to be clear enough for observation. Therefore, the dike model is put 556cm from the wave maker to the toe.

The cross section of the dike model is shown in Figure 3.11.

*Figure 3-10 Dummy dike made from plywood for wave characteristics investigation before real experiments.*

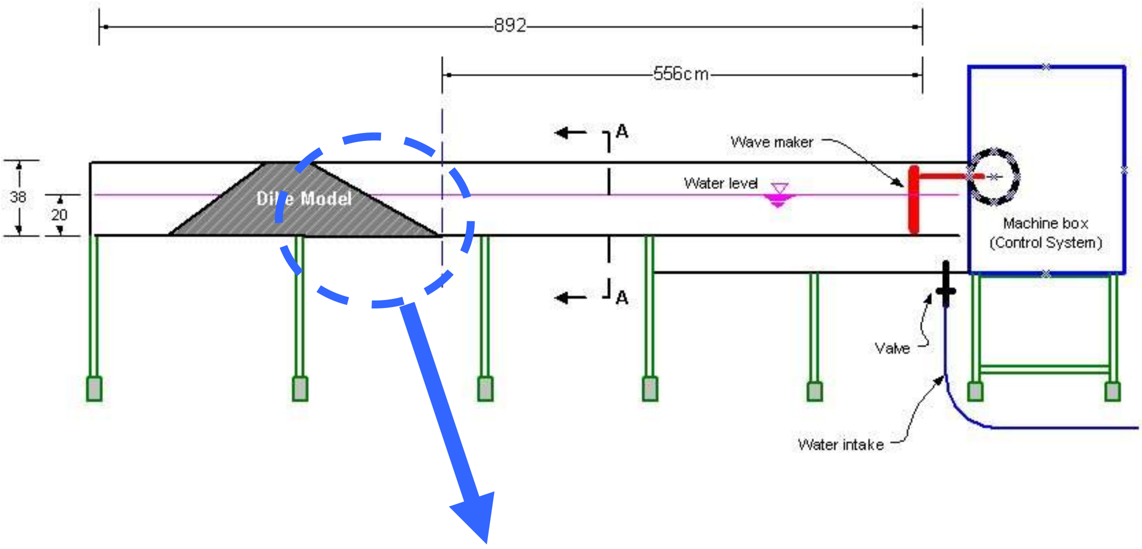
*Table 3-5 Wave characteristics measurement*

| No | f [Hz] | H[m] | Time/50waves [s] | T[s] | L[m] | ξ | Run-up [m] | Crest elev. [m] |
|---|---|---|---|---|---|---|---|---|
| 1 | 25 | 0.08 | 80.829 | 1.617 | 2.15 | 1.296 | 0.104 | 0.384 |
| 2 | 26 | 0.08 | 77.386 | 1.548 | 2.05 | 1.266 | 0.101 | 0.381 |
| 3 | 27 | 0.09 | 74.584 | 1.492 | 1.96 | 1.167 | 0.105 | 0.395 |
| 4 | 28 | 0.09 | 71.824 | 1.436 | 1.88 | 1.143 | 0.103 | 0.393 |
| 5 | 29 | 0.08 | 69.392 | 1.388 | 1.81 | 1.189 | 0.095 | 0.375 |
| 6 | 30 | 0.09 | 67.024 | 1.340 | 1.74 | 1.099 | 0.099 | 0.389 |
| 7 | 31 | 0.1 | 65.793 | 1.316 | 1.7 | 1.031 | 0.103 | 0.403 |
| 8 | 32 | 0.1 | 63.064 | 1.261 | 1.62 | 1.006 | 0.101 | 0.401 |

**Notes:**

ξ : Surf Similarity

Crest Elev : Water depth (d)+Wave height (H)+Run-up(R )



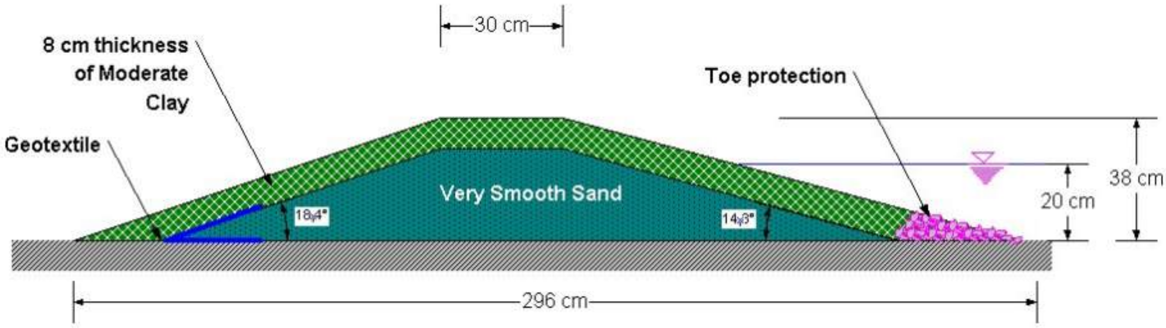



*Figure 3-11 Cross section of the dike model*

### 3.5.4 Experiment Scenario and Observation

The idea of this qualitative experiment is to have understanding on breaching mechanism initiated by breaking waves. Overtopping and other factors that can cause damage/failure to the dike are avoided. Only breaking waves are allowed to destroy the dike model. Two dike models are prepared with the same cross section as shown in Figure 3.11. For observation, two cameras are positioned at the back side to see back view of the dike model and at the side of the model to see cross-section view. When the flume is working, observation is carried out by means of filming the dike for about 15 minutes for every hour. For important moments, filming can be done longer. Timing aspects and specific events such as start of failures and damage progress are recorded.

# 4. EXPERIMENTAL RESULTS

Experimental Results from the three experiments (Tests on compacted clay with water-filled cracks, surface erosion tests on compacted clay without cracks, and flume test on clay cover failure) are described in this chapter. Explanations and some comments on those results are also briefly discussed.

## 4.1 Test Results on Compacted Clay with Water-filled Cracks

In total, 12 tests were carried out for the water-filled crack experiments consist of 7 tests for clay type I (tests 1-7) and 5 tests for clay type II (tests 53-57). The required density of >95% PD could not be achieved due to operational difficulties. Transparent walls that hold the sample are made from glass that is vulnerable if receiving too much force. In the other hand, compaction in the range of 80-95% of PD as shown for clay with recommended water and <80% of PD for clay with high water content is realistic. In general, there are no failures after first impact for all tests (except for a sample with very high water content where damage occurs in the form of almost round shape around the hit area of falling water mass). Some failures occur after the samples hit by the impact pressure several times. Some of the failures show clear angle of failure but some are difficult to identify due to irregular damage. Blocks of removed soil are also observed mostly for the samples with high water content in both types of clay. Complete results of the tests are shown in Table 4-1. Minus signs in column ($p_{max}$-2c) mean theoretically failure will not occur (Führböter, 1966).

### 4.1.1 Results for clay from Cäciliengroden (Good Clay)

#### Test with recommended water content

There are 3 experiments for clay samples with recommended water content; test 1, 3, and 4 with 37.5%, 38.5% and 38.2% and the estimated cohesion ($c_u$) using **Figure 3.3** are 80.32 kPa , 71.24 kPa, and 73.85kPa respectively. All tests show no failure after the 1st impact.. After releasing impact pressures several times, damage occurs without angle of failures (unrecognized).

Clay sample on test 1 was compacted with 88% of proctor density. The compaction >95% could not be reached in this test due to difficulties in operational. After the 1st impact resulted with no failure, the process of releasing impact pressure to the sample was then carried on. Failure was not observed even after the 10th impact. Small damage was observed after the 12th impact. This small damage occurs at the mouth of the crack from top down to few centimeters along the weak line. This weak line appears as the result of imperfect process of crack formation (**Figure 4.1a** & **b**).

Clay sample on test 3 was less compacted than test 1. Sample of test 3 was compacted with 20 blows of hammering (compare to 30 blows for test1). The idea behind this is to look at the influence of degree of compaction to the erosion resistant behavior of the clay. Result showed no failure after the 1st impact, even after the 5th impact. Damage at the surface of the crack was observed after the 6th impact. This damage seems not the result of the impact pressure propagation inside the water-filled crack but it is more likely as the result of direct hit from the repeatedly impact pressures (**Figure 42**).

*Table 4-1 Summary of test results on compacted clay with water-filled crack*

| Test ID | Clay type | Fall height [cm] | Impact (Pmax) [kPa] | The n[th] impacts | $w_n$ [%] | Compaction [gr/ml] | Estimated cohesion ($c_u$) [kPa] | (Pmax-2c) [kPa] | Failure angel [deg] | Notes |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | I | 162 | 24.75 | 1 | 37.5 | 1.278 | 80.32 | -135.89 | - | Well compacted, no damage |
| | I | 162 | 24.75 | 7 | 37.5 | 87.7 % | 80.32 | -135.89 | - | No damage, weak point observed |
| | I | 162 | 24.75 | 8 | 37.5 | | 80.32 | -135.89 | - | Soil around the weak poit is lifted |
| | I | 162 | 24.75 | 9 | 37.5 | | 80.32 | -135.89 | - | Damage is started aroung the weak point |
| | I | 162 | 24.75 | 10 | 37.5 | | 80.32 | -135.89 | - | All soil bodies around the weak point are removed |
| 2 | I | 162 | 24.75 | 1 | 44.9 | 1.162 | 33.05 | -41.35 | - | Lifted soil |
| | I | 162 | 24.75 | 4 | 44.9 | 79.7 % | 33.05 | -41.35 | - | Lifted soil is obvious, crack is widened |
| | I | 162 | 24.75 | 5 | 44.9 | | 33.05 | -41.35 | - | Block soil removed |
| 3 | I | 163 | 24.82 | 1 | 38.5 | - | 71.24 | -117.65 | - | Less compacted, no damage |
| | I | 162 | 24.75 | 5 | 38.5 | | 71.24 | -117.72 | - | Minor damage |
| | I | 162 | 24.75 | 6 | 38.5 | | 71.24 | -117.72 | - | Side crack damage observed |
| 4 | I | 162 | 24.75 | 1 | 38.2 | - | 73.85 | -122.95 | - | Almost no compaction, side crack damage observed |
| | I | 162 | 24.75 | 2 | 38.2 | | 73.85 | -122.95 | - | Almost no compaction, severe damage observed |
| | | | | | | | | | | Water flows through large pores |
| 5 | I | 162 | 24.75 | 1 | 45.2 | 1.19 | 31.88 | -39.01 | - | No damade, crack is widening |
| | I | 162 | 24.75 | 3 | 45.2 | 81.6 % | 31.88 | -39.01 | - | Side wall started to lift, crack is widening |
| | I | 162 | 24.75 | 11 | 45.2 | | 31.88 | -39.01 | 27.50 | Lifted soil at its maximum, crack is widening |
| | | | | | | | | | | Crack observed on the side wall |
| 6 | I | 162 | 24.75 | 1 | 48.5 | 1.08 | 21.46 | -18.16 | - | No damade, crack is widening |
| | I | 162 | 24.75 | 3 | 48.5 | 74.1 % | 21.46 | -18.16 | - | Side wall started to lift, crack is widening |
| | I | 162 | 24.75 | 11 | 48.5 | | 21.46 | -18.16 | - | Lifted soil at its maximum, crack is widening |
| | I | 162 | 24.75 | 12 | 48.5 | | 21.46 | -18.16 | 14.00 | Block soil removed |
| 7 | I | 162 | 24.75 | 1 | 53.2 | 1.04 | 12.21 | 0.33 | 21.80 | Failure occures |
| | | | | | | 71.3 % | | | | Clay is extremely wet |
| 53 | II | 162 | 24.75 | 1 | 23.2 | 1.45 | 27.04 | -29.33 | - | Well compacted, no damage |
| | II | 162 | 24.75 | 5 | 23.2 | 88.3 % | 27.04 | -29.33 | - | Minor damage |
| | II | 162 | 24.75 | 9 | 23.2 | | 27.04 | -29.33 | 11.90 | Slightly wall damage, 6 cm removed soil |
| 54 | II | 162 | 24.75 | 1 | 27.6 | 1.38 | 11.22 | 2.32 | - | Well compacted, no damage |
| | II | 162 | 24.75 | 2 | 27.6 | 84.0 % | 11.22 | 2.32 | - | Side wall started to lift, crack is widening |
| | II | 162 | 24.75 | 5 | 27.6 | | 11.22 | 2.32 | - | Minor damage around crack |
| | II | 162 | 24.75 | 9 | 27.6 | | 11.22 | 2.32 | 33.60 | Lifted soil at its maximum, crack is widening |
| 55 | II | 162 | 24.75 | 1 | 21.7 | 1.38 | 36.50 | -48.26 | - | Well compacted, no damage |
| | II | 162 | 24.75 | 5 | 21.7 | 84.0 % | 36.50 | -48.26 | - | Minor side damage from side effect |
| | II | 162 | 24.75 | 9 | 21.7 | | 36.50 | -48.26 | - | Minor becomes significant damage |
| | | | | | | | | | | Crack at 3cm depth observed, no failure |
| 56 | II | 162 | 24.75 | 1 | 28.1 | 1.4 | 10.15 | 4.45 | - | Well compacted, no damage |
| | II | 162 | 24.75 | 2 | 28.1 | 85.2 % | 10.15 | 4.45 | - | Lifted soil, gap widen |
| | II | 162 | 24.75 | 4 | 28.1 | | 10.15 | 4.45 | 14.28 | Failure occures, cracks at the wall side of lifted soil |
| 57 | II | 162 | 24.75 | 1 | 27.4 | 1.41 | 11.67 | 1.40 | - | Well compacted, no damage |
| | II | 162 | 24.75 | 2 | 27.4 | 85.8 % | 11.67 | 1.40 | - | Lifted soil, gap widen |
| | II | 162 | 24.75 | 6 | 27.4 | | 11.67 | 1.40 | 25.5 | Failure occures, block soil removed |


*Figure 4-1 Water filled crack with its weak line as the result of imperfect cracking process.*

*Figure 4-2 Small damage along the weak line after 12th impact.*

Role of compaction is clear when test 4 was carried out. With very little compaction, the sample has a lot of pores which are later filled by water. The water-filled crack suffered damage after the 2nd impact because its stability drastically decreases due to less compacted and large pore pressures.

*Figure 4-3 Damage around the surface of the water-filled crack*

### Tests with high water content

Samples with high water content are more vulnerable to erosion. From experience, dike breaching is often initiated when the clay cover is in its weak condition, loosing its strength due to high water content. The clay samples here are defined to have high water content if they have water content larger than the recommended maximum water content ($w_{c(max)}$) according to TAW specification. Therefore, for clay category I, the water content should be larger than 43.2% (see **Section 3.2, Chapter 3**).

From field measurement, the natural water content of good clay is 40% to 50%. In this experiment, samples from test 2, 5, 6, and 7 have 44.9%, 45.2%, 48.85%, and 53.2 % respectively. Test 2, 5, and 6 show the same behaviours as follows:

- No failure after the 1st impact. The estimated cohesions are still larger than the generated impact pressure.

- After each impact, the crack width is getting wider. Clay with high water content is easy to deform when subjected to direct forces.

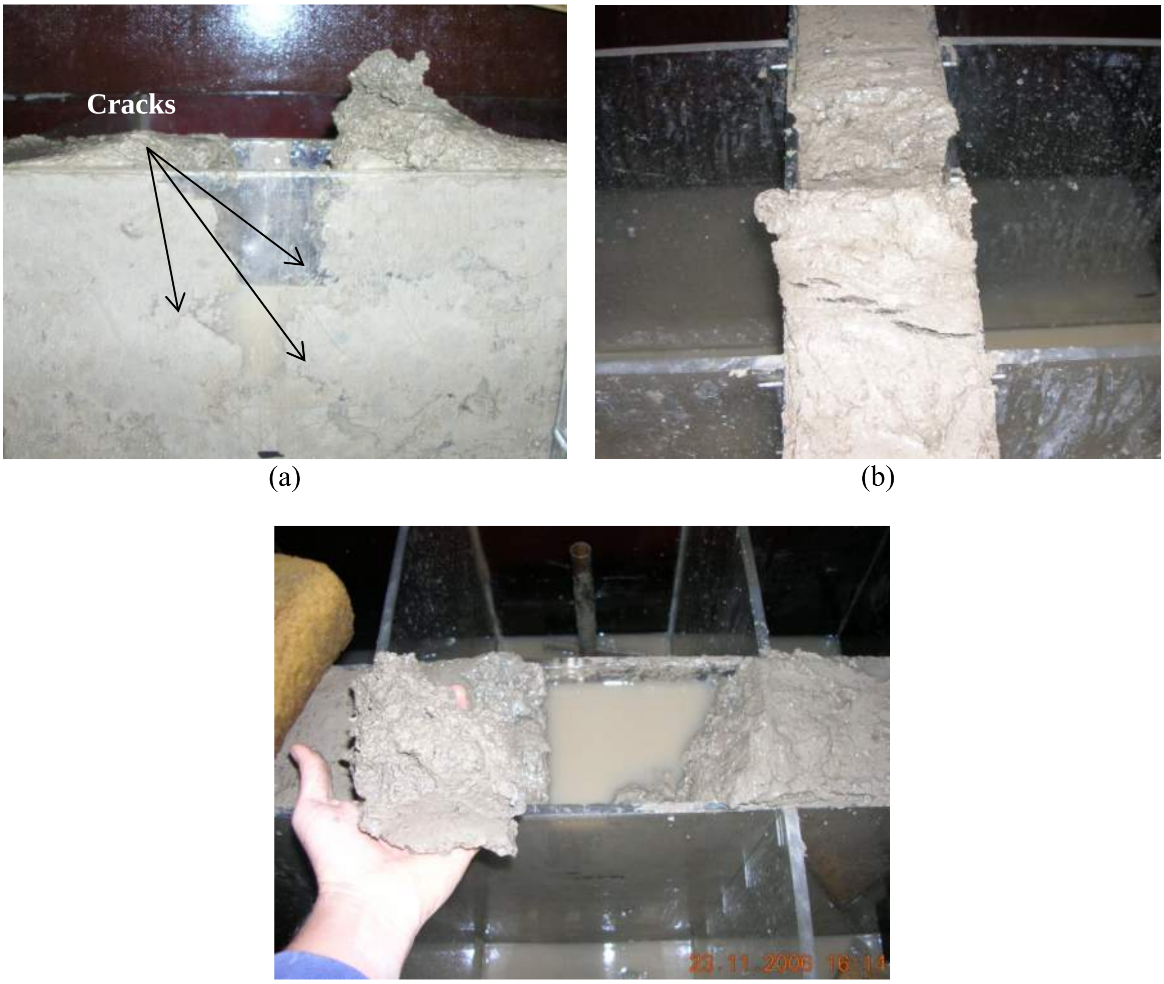
- Lifted blocks of soil were observed after several impacts. The pressure inside the water-filled crack pushes out the side wall of the crack and lifts it. A single pressure in these experiments seems not strong enough to lift up the clay as described by Führböter (1966).

- After being hit several times by the impacts, cracks were observed inside the water-filled crack walls (**Figure 4.3a**). These cracks are getting wider and finally the whole block of soil was removed. The angels of failure of this block soil removal are measured as shown at **Table 4.1**.

- At the surface, next to the water-filled crack, horizontal cracks are also observed as the reaction against the repetitions of impact pressures (**Figure 4.3b**). Furthermore, these cracks become the weak point where the block of soil is finally removed (**Figure 4.3c**).

- Small aggregates and particles from the damage at the surface are found at the bottom of the water-filled cracks. These seem to prevent further damage reaching the bottom of the water-filled crack after hit several times by the impacts.

For samples with very high water content, 53% (test 7), severe damage occurred after the 1st impact. This damage was mainly caused by the fact that the clay is extremely wet. It can be seen from the crater-like shape of the eroded area (**Figure 4.4**).




(a) (b)

(c)

*Figure 4-4 Experiments with high water content (a) The crack is getting wider after each impact. Block soil is lifted and lateral cracks appear at the sidewalls of the crack (b) Cracks at the surface (c) Block of soil is finally removed after hit several times by the impacts.*

*Figure 4-5 Crater-like shape of the eroded surface for the test with very high water content.*

### 4.1.2 Results for clay from Elisabethgroden km-9.0 (Moderate Clay)

#### Test with recommended water content

Generally, the moderate clay samples also show the same behaviours as the good clay. There are two samples with recommended water content; test 53 and 55 with water content of 23.2% and 21.7% respectively. Both show no significant damage after being hit by the impacts for several times. Minor damage at the surface around the crack was observed in both tests. In test 53, the angle of failure was easily recognized but for the angle of failure in test 55 was difficult to identify due to irregular damage. Small particles were also found in the bottom of the water-filled cracks as results of eroded soil in the surface.


*Figure 4-6 Damage at the surface after the sample hit by the impacts several times*

#### Test with high water content

There are 3 samples with high water content; test 54, 56, and 57 with water content of 27.6%, 28.15, and 27.4% respectively. Most of the results from the tests are the same results as in the test for good clay

- No failure after the 1st impact. The estimated cohesions are still smaller than the generated impact pressure but still no failure.
- After each impact, the crack width is getting wider. The same as in the test for good clay
- Lifted block of soil were observed after several impacts. Later on, block of soil is removed (**Figure 4.6**). The same as in the test for good clay.
- After hit several times by the impacts, cracks were observed inside the water-filled crack walls. The same as in the test for good clay.
- At the surface, next to the water-filled crack, horizontal cracks are also observed as the reaction against the repeatedly impact pressures. The same as in the test for good clay.
- Small aggregates and particles from the damage at the surface are found at the bottom of the water-filled cracks. The same as in the test for good clay.


*Figure 4-7 Blocks of soil are removed after releasing several impacts*

### 4.2 Test Results on Compacted Clay without Cracks (Surface Erosion Tests)

Total experiments are 44 tests consist of 29 tests for good clay and 15 tests for moderate clay. Good clay has more tests because at the beginning some preliminary tests should be done. All preliminary tests were carried out using samples from good clay. Preliminary tests are important to keep the on-going research on the right direction. During preliminary tests some important aspects are revealed:

1. Determination of impact numbers.
   Number of impacts should be determined before hand because it is related to time acquisitions for the whole experiments. It was found out that the tests should be stopped when the eroded soil reach depth up to more or less 10cm. The reason behind this is the fall height is getting higher. When it reaches 10cm deep, the fall height is not the same anymore and the impact is getting larger, consequently the clay layer suffer more and more erosion. For good clay with recommended water content the number of impacts needed to erode 10cm deep takes up to 500 impacts. For moderate clay, the needed impacts are less, about 200 impacts.

2. Determination of impact magnitudes
   Impact magnitudes depend on the fall height of impact machine and the observed reaction of the compacted clay. From the observation, it was found out that the maximum generated impact (fall height of 162cm or 24.75 kPa) was suitable to achieve the objectives of this experiment.

3. Water layers
   It is clear that water layers cause less erosion to the clay surface. Surface erosion on compacted clay with water layers is less but more distributed due to increasing water splash velocities in all direction after each impacts. Because of its clear role on erosion, test with water layer was then abandoned

4. Compaction
   Though it is mentioned in many reports that compaction is key parameter in erosion of soil. Some tests were carried out by applying different degree of compaction. Working on clay with various water contents needs specific skills to understand the relationship between water content and its degree of compaction.

5. Role of remaining water
   When scour hole started to form due to repetition of impacts, some water can not escape from the scour hole. This water showed its behavior as water layers absorbing some impact energy. Therefore, actions should be taken to dry out this water every time after each impact.

6. Eroded soil measurement
   Eroded soil measurements are very important in this test. By looking at the progress of erosion, the determination of when measurement should be done is revealed. For example, the eroded soil measurement for good clay with recommended water content can start at least after 100 impacts. Less than 100 impacts, hardly any erosion occurs. After that, measurement every 50 impacts look reasonable to follow the erosion progress.

7. Limitation of eroded scour measurements
   The area of measurement should be limited, otherwise it can go anywhere and uncontrollable. From the observation, it was found that the area of 10cmx30cm is feasible to be used as spatial boundary condition for eroded soil measurement.

Detail results of all tests both for good and moderate clay are shown in **Appendix A**.

### *4.2.1 Results for clay from Cäciliengroden (Good Clay)*

From the total 29 tests, only data from 15 tests are considered to be representative for further analysis (tests from 23 - 37). The remaining tests, some of them are part of preliminary tests, are analyzed when they are needed for additional information. Those 15 tests have complete parameters and consistent procedures as well.

During the tests, it was found out that controlling clay samples with desired water content are difficult. Some of the samples with desired water content could not be prepared. The maximum recommended water content for good clay is ($w_{c(max)}$ ) 43.2% and the optimum water content ($w_{pr}$) is 26% (see **Section 4.2, Chapter 4**). There are 6 samples with water content above $w_{c(max)}$ and 11 samples with recommended water content. By looking at the second expression (Equation 4.3), determination for the recommended water content according to TAW can be 5% up to 10% lower than the $w_{c(max)}$. It means the lower limit of recommended water content has another value, not $w_{pr}$, but $w_{c(min)}$=32.8%. By applying this approach, there are 2 samples with water content below the $w_{c(min)}$ and the samples with recommended water content are 7 samples.

The tests with recommended water content required large number of impacts to have significant erosion. Small erosion was observed after 100 impacts so that the measurement of eroded soil can be done. To see the progress of erosion, measurement of eroded soil was carried out for every 50 impacts. The measurement was stopped after 500 impacts or until the scour hole reached more or less 10cm deep.

The tests with high water content required a smaller number of impacts to have significant erosion. Erosion was observed usually after 10 impacts. Later on, the progress of erosion was measured every 10 impacts. The measurement was stopped after 50 impacts or until the scour hole reached more or less 10cm deep.

The tests with dry water content required more than 50 impacts. The erosion progress was measured for every 50 impacts or until the scour hole reached more or less 10cm deep.

For all tests, the volume of eroded soil was measured by filling in water into the scour hole (see **Appendix D** for details). **Table 4.2** shows the experiment results of surface erosion on compacted clay for good clay.

The progress of erosion seems to be consistent for the 3 groups of different water content. The samples with recommended water content are more erosion resistant than the others. It can be seen from the number of impacts needed for making significant erosion. The samples with recommended water content required impacts up to 500 to make erosion less than 2000cm3. For the same amount of erosion, the samples with high water content only need 50 impacts. The samples with dry water content show unique behaviors. The first 50 or 100 impacts, the samples suffered little erosion. But, the following impacts changed it drastically. The erosion was getting severe as more impacts hit the samples. The changes in water content are probably the cause of this kind of behavior. **Figure 4.7** shows the relationship between number of impacts and amount of eroded soil

*Table 4-2 Summary results of surface erosion tests on compacted clay for good clay*

| Test ID | | Clay type | Fall height [cm] | Impacts [kPa] | Number of Impacts | $w_n$ [%] | Water layer [cm] | Eroded soil [$cm^3/300cm^2$] | Compaction [$gr/cm^3$] | Notes |
|---|---|---|---|---|---|---|---|---|---|---|
| 8 | | I | 165 | 24.75 | 200 | 38.6 | 0 | 260 | | Pump is used |
| 9 | | I | 165 | 24.75 | 200 | 36.6 | 1 | 200 | | |
| 10 | | I | 165 | 24.75 | 200 | 39.6 | 2.5 | 190 | | |
| 11 | | I | 125 | 21.53 | 200 | 35.6 | 0 | 20 | | |
| 12 | | I | 125 | 21.53 | 200 | 37.1 | 0 | 90 | | |
| 13 | | I | 125 | 21.53 | 200 | 36.8 | 1 | 120 | | |
| 23 | a | I | 165 | 24.75 | 200 | 34.7 | 0 | 75 | 1.3041 | Optimum wc. Using the same soil. Score hole measurement was done using water in order to maintain undisturbed condition of the sample |
| | b | I | 165 | 24.75 | 250 | 34.7 | 0 | 180 | 1.3041 | |
| | c | I | 165 | 24.75 | 300 | 34.7 | 0 | 305 | 1.3041 | |
| | d | I | 165 | 24.75 | 350 | 34.7 | 0 | 480 | 1.3041 | |
| | e | I | 165 | 24.75 | 400 | 34.7 | 0 | 650 | 1.3041 | |
| 24 | a | I | 165 | 24.75 | 200 | 37.3 | 0 | 205 | 1.2861 | |
| | b | I | 165 | 24.75 | 250 | 37.3 | 0 | 310 | 1.2861 | |
| | c | I | 165 | 24.75 | 300 | 37.3 | 0 | 450 | 1.2861 | |
| | d | I | 165 | 24.75 | 350 | 37.3 | 0 | 500 | 1.2861 | |
| | e | I | 165 | 24.75 | 400 | 37.3 | 0 | 690 | 1.2861 | |
| 25 | a | I | 165 | 24.75 | 50 | 42.06 | 0 | 150 | 1.2804 | |
| | b | I | 165 | 24.75 | 75 | 42.06 | 0 | 310 | 1.2804 | |
| 26 | a | I | 165 | 24.75 | 25 | 46.02 | 0 | 170 | 1.1879 | |
| | b | I | 165 | 24.75 | 50 | 46.02 | 0 | 520 | 1.1879 | |
| 27 | a | I | 165 | 24.75 | 100 | 41.35 | 0 | 25 | 1.2055 | |
| | b | I | 165 | 24.75 | 150 | 41.35 | 0 | 45 | 1.2055 | |
| | c | I | 165 | 24.75 | 200 | 41.35 | 0 | 65 | 1.2055 | |
| | d | I | 165 | 24.75 | 250 | 41.35 | 0 | 135 | 1.2055 | |
| | e | I | 165 | 24.75 | 300 | 41.35 | 0 | 275 | 1.2055 | |
| | f | I | 165 | 24.75 | 350 | 41.35 | 0 | 415 | 1.2055 | |
| | g | I | 165 | 24.75 | 400 | 41.35 | 0 | 570 | 1.2055 | |
| | h | I | 165 | 24.75 | 450 | 41.35 | 0 | 740 | 1.2055 | |
| | i | I | 165 | 24.75 | 500 | 41.35 | 0 | 790 | 1.2055 | |
| 28 | a | I | 165 | 24.75 | 100 | 42.7 | 0 | 25 | 1.1641 | |
| | b | I | 165 | 24.75 | 150 | 42.7 | 0 | 45 | 1.1641 | |
| | c | I | 165 | 24.75 | 200 | 42.7 | 0 | 70 | 1.1641 | |
| | d | I | 165 | 24.75 | 250 | 42.7 | 0 | 85 | 1.1641 | |

| Test ID | Clay type | Fall height [cm] | Impacts [kPa] | Number of Impacts | $w_n$ [%] | Water layer [cm] | Eroded soil [$cm^3/300cm^2$] | Compaction [$gr/cm^3$] | Notes |
|---|---|---|---|---|---|---|---|---|---|
| e | I | 165 | 24.75 | 300 | 42.7 | 0 | 155 | 1.1641 | |
| f | I | 165 | 24.75 | 350 | 42.7 | 0 | 195 | 1.1641 | |
| g | I | 165 | 24.75 | 400 | 42.7 | 0 | 285 | 1.1641 | |
| h | I | 165 | 24.75 | 450 | 42.7 | 0 | 365 | 1.1641 | |
| i | I | 165 | 24.75 | 500 | 42.7 | 0 | 860 | 1.1641 | |
| 29 a | I | 165 | 24.75 | 100 | 43 | 0 | 175 | 1.1603 | |
| b | I | 165 | 24.75 | 150 | 43 | 0 | 285 | 1.1603 | |
| c | I | 165 | 24.75 | 200 | 43 | 0 | 655 | 1.1603 | |
| d | I | 165 | 24.75 | 250 | 43 | 0 | 1000 | 1.1603 | |
| e | I | 165 | 24.75 | 300 | 43 | 0 | 1260 | 1.1603 | |
| f | I | 165 | 24.75 | 350 | 43 | 0 | 1520 | 1.1603 | |
| g | I | 165 | 24.75 | 400 | 43 | 0 | 1790 | 1.1603 | |
| 30 a | I | 165 | 24.75 | 10 | 48.6 | 0 | 215 | 1.0756 | |
| b | I | 165 | 24.75 | 20 | 48.6 | 0 | 415 | 1.0756 | |
| c | I | 165 | 24.75 | 30 | 48.6 | 0 | 720 | 1.0756 | |
| d | I | 165 | 24.75 | 40 | 48.6 | 0 | 980 | 1.0756 | |
| e | I | 165 | 24.75 | 50 | 48.6 | 0 | 1340 | 1.0756 | |
| 31 a | I | 165 | 24.75 | 10 | 47.64 | 0 | 390 | 1.0364 | |
| b | I | 165 | 24.75 | 20 | 47.64 | 0 | 1020 | 1.0364 | |
| c | I | 165 | 24.75 | 30 | 47.64 | 0 | 1655 | 1.0364 | |
| 32 a | I | 165 | 24.75 | 10 | 48.32 | 0 | 190 | 1.1241 | |
| b | I | 165 | 24.75 | 20 | 48.32 | 0 | 480 | 1.1241 | |
| c | I | 165 | 24.75 | 30 | 48.32 | 0 | 780 | 1.1241 | |
| e | I | 165 | 24.75 | 50 | 48.32 | 0 | 1650 | 1.1241 | |
| 33 a | I | 165 | 24.75 | 10 | 50.2 | 0 | 310 | 1.0587 | |
| b | I | 165 | 24.75 | 20 | 50.2 | 0 | 660 | 1.0587 | |
| c | I | 165 | 24.75 | 30 | 50.2 | 0 | 1110 | 1.0587 | |
| d | I | 165 | 24.75 | 40 | 50.2 | 0 | 1480 | 1.0587 | |
| e | I | 165 | 24.75 | 50 | 50.2 | 0 | 2100 | 1.0587 | |
| 34 a | I | 165 | 24.75 | 10 | 50.73 | 0 | 740 | 1.1288 | |
| b | I | 165 | 24.75 | 20 | 50.73 | 0 | 1490 | 1.1288 | |
| c | I | 165 | 24.75 | 30 | 50.73 | 0 | 2050 | 1.1288 | |
| 35 a | I | 165 | 24.75 | 100 | 35.17 | 0 | 0 | 1.2637 | |
| b | I | 165 | 24.75 | 150 | 35.17 | 0 | 50 | 1.2637 | |
| c | I | 165 | 24.75 | 200 | 35.17 | 0 | 90 | 1.2637 | |
| d | I | 165 | 24.75 | 250 | 35.17 | 0 | 145 | 1.2637 | |
| e | I | 165 | 24.75 | 300 | 35.17 | 0 | 235 | 1.2637 | Computer error occur |
| f | I | 165 | 24.75 | 350 | 35.17 | 0 | 300 | 1.2637 | |
| g | I | 165 | 24.75 | 400 | 35.17 | 0 | 360 | 1.2637 | |
| h | I | 165 | 24.75 | 450 | 35.17 | 0 | 420 | 1.2637 | |
| i | I | 165 | 24.75 | 500 | 35.17 | 0 | 485 | 1.2637 | |

| Test ID | | Clay type | Fall height [cm] | Impacts [kPa] | Number of Impacts | $w_n$ [%] | Water layer [cm] | Eroded soil [cm$^3$/300cm$^2$] | Compaction [gr/cm$^3$] | Notes |
|---|---|---|---|---|---|---|---|---|---|---|
| 36 | a | I | 165 | 24.75 | 50 | 27.71 | 0 | 500 | 1.2084 | erosion started from side wall, assymetric |
| | b | I | 165 | 24.75 | 90 | 27.71 | 0 | 2810 | 1.2084 | |
| 37 | a | I | 165 | 24.75 | 50 | 27 | 0 | 90 | 1.2607 | |
| | b | I | 165 | 24.75 | 100 | 27 | 0 | 180 | 1.2607 | |
| | c | I | 165 | 24.75 | 150 | 27 | 0 | 360 | 1.2607 | erosion started from side wall, asymmetric |
| | d | I | 165 | 24.75 | 200 | 27 | 0 | 1300 | 1.2607 | |
| | e | I | 165 | 24.75 | 238 | 27 | 0 | 4100 | 1.2607 | |

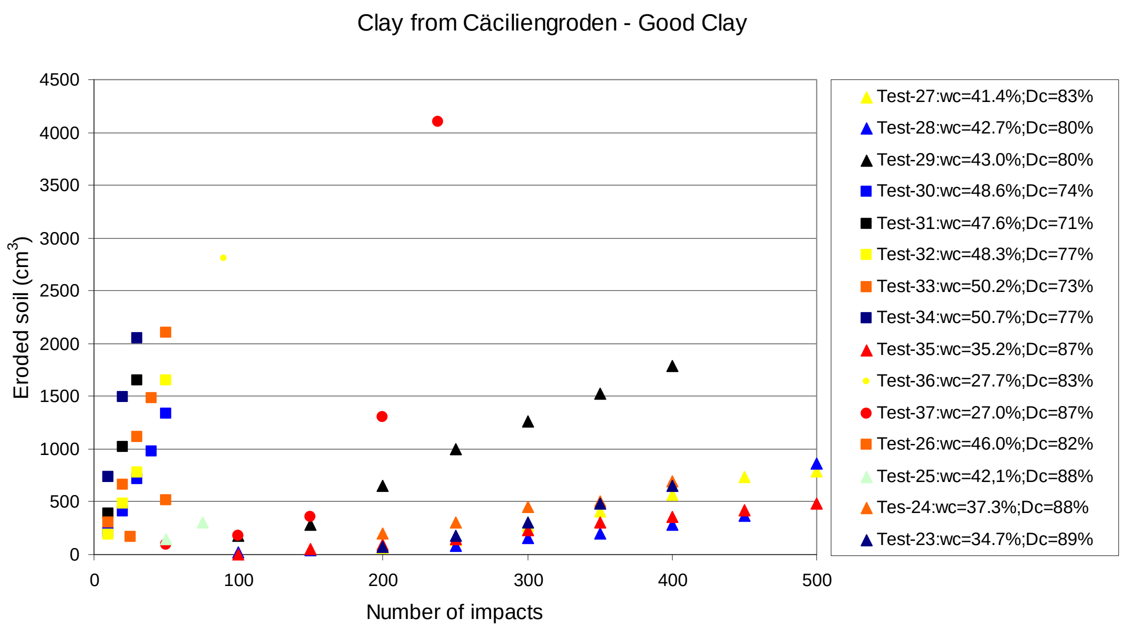



*Figure 4-8 Amount of eroded soil vs number of impacts for good clay (wc: water content, Dc: degree of compaction)*

### 4.2.2 Results for clay from Elisabethgroden km-9.0 (Moderate Clay)

For moderate clay, all data from 15 tests are taken for further analysis. Similar to the good clay, controlling samples with desired water content are difficult to get. There are 5 samples with high water content (water content above $w_{c(max)}$=25.6%), 9 samples with recommended water content (water content between $w_{c(min)}$ and $w_{c(max)}$; 18.5% - 25.6%), and 1 sample with high water content (water content below $w_{pr}$=$w_{c(min)}$=18.5%).

Samples with high water content show a consistent behaviour. The compacted clays were eroded quickly by applying impacts less than 50 impacts. The samples with water content close to the border ($w_{c(max)}$) showed a little bit erosion resistant until finally eroded fast by 150 impacts.

Samples with recommended water content show some different behaviour. Samples with water content below the $w_{c(max)}$ =25.6% and above 23% are the most erosion resistant with amount of erosion soil below 2000 cm$^3$ for more than 200 impacts. There are 4 samples (out of 9 samples) with water content still in the range of recommended one but showing different behaviour with the other 5 samples. These samples have water content below 23% and above $w_{pr}$=$w_{c(min)}$=18.5%. They suffered heavy erosion by less than 150 impacts (some could not survive by 50 impacts). The only sample with water content below the optimum water content ($w_{pr}$=$w_{c(min)}$=18.5%) could not withstand more than 50 impacts. Complete results are tabulated in **Table 4.3**.

Beside measurement by water, the eroded soils are also measured by ruler to identify the maximum scour depth of the scour hole. The tendency of the results from measurement using ruler are in agreement with the measurement using water. **Figure 4.8** and **4.9** show the relationship between number of impacts and amount of eroded soil and depth of eroded soil respectively.

*Table 4-3 Summary results of surface erosion test on compacted clay for moderate clay*

| Test ID | | Clay Type | Fall height [cm] | Impacts [kPa] | Number of Impacts | $w_c$ [%] | Water layer [cm] | Eroded soil [$cm^3/300cm^2$] | scour depth [cm] | Compaction [$gr/cm^3$] | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 38 | a | II | 162 | 24.75 | 50 | 21.14 | 0 | 250 | 2.4 | 1.3806 | side erosion |
| | b | II | 162 | 24.75 | 75 | 21.14 | 0 | 420 | 3.7 | 1.3806 | asymmetric |
| | c | II | 162 | 24.75 | 100 | 21.14 | 0 | 900 | 6.3 | 1.3806 | maybe compaction |
| | d | II | 162 | 24.75 | 125 | 21.14 | 0 | 1570 | 9 | 1.3806 | problem |
| | e | II | 162 | 24.75 | 150 | 21.14 | 0 | 2920 | 10.5 | 1.3806 | |
| 39 | a | II | 162 | 24.75 | 25 | 26.02 | 0 | 270 | 3.3 | 1.3923 | 5cm eroded soil - |
| | b | II | 162 | 24.75 | 50 | 26.02 | 0 | 640 | 5 | 1.3923 | observed |
| | c | II | 162 | 24.75 | 75 | 26.02 | 0 | 1020 | 7.4 | 1.3923 | no cracks - |
| | d | II | 162 | 24.75 | 100 | 26.02 | 0 | 1660 | 9.5 | 1.3923 | observed |
| 40 | a | II | 162 | 24.75 | 25 | 24.2 | 0 | 40 | 1 | 1.4316 | Lateral crack |
| | b | II | 162 | 24.75 | 50 | 24.2 | 0 | 220 | 2.5 | 1.4316 | |
| | c | II | 162 | 24.75 | 75 | 24.2 | 0 | 370 | 3.5 | 1.4316 | |
| | d | II | 162 | 24.75 | 100 | 24.2 | 0 | 600 | 4.6 | 1.4316 | |
| | e | II | 162 | 24.75 | 125 | 24.2 | 0 | 770 | 5.6 | 1.4316 | |
| | f | II | 162 | 24.75 | 150 | 24.2 | 0 | 1140 | 6.3 | 1.4316 | |
| | g | II | 162 | 24.75 | 175 | 24.2 | 0 | 1370 | 8.3 | 1.4316 | |
| | h | II | 162 | 24.75 | 200 | 24.2 | 0 | 1680 | 10.4 | 1.4316 | |
| 41 | a | II | 162 | 24.75 | 25 | 22.01 | 0 | 100 | 2 | 1.4248 | side erosion |
| | b | II | 162 | 24.75 | 50 | 22.01 | 0 | 170 | 2.8 | 1.4248 | comp error |
| | c | II | 162 | 24.75 | 75 | 22.01 | 0 | 470 | 3.5 | 1.4248 | valve error |
| | d | II | 162 | 24.75 | 100 | 22.01 | 0 | 870 | 5 | 1.4248 | leftside big erosion |
| | e | II | 162 | 24.75 | 125 | 22.01 | 0 | 1780 | 8 | 1.4248 | |
| | f | II | 162 | 24.75 | 150 | 22.01 | 0 | 3800 | 13 | 1.4248 | |
| 42 | a | II | 162 | 24.75 | 25 | 20.56 | 0 | 240 | 2.5 | 1.3006 | lefttside erosion |
| | b | II | 162 | 24.75 | 50 | 20.56 | 0 | 1440 | 9.5 | 1.3006 | |
| | c | II | 162 | 24.75 | 60 | 20.56 | 0 | 2530 | 13.5 | 1.3006 | |
| 43 | a | II | 162 | 24.75 | 10 | 27.59 | 0 | 130 | 1.5 | 1.4284 | lifted |
| | b | II | 162 | 24.75 | 20 | 27.59 | 0 | 360 | 3 | 1.4284 | cracks |
| | c | II | 162 | 24.75 | 30 | 27.59 | 0 | 850 | 5.5 | 1.4284 | tongue |
| | d | II | 162 | 24.75 | 40 | 27.59 | 0 | 1490 | 7.7 | 1.4284 | 10cm block soil |
| | e | II | 162 | 24.75 | 50 | 27.59 | 0 | 2370 | 10.5 | 1.4284 | |
| 44 | a | II | 162 | 24.75 | 10 | 26.31 | 0 | 40 | 1.5 | 1.4745 | lateral cracks |
| | b | II | 162 | 24.75 | 20 | 26.31 | 0 | 90 | 1.8 | 1.4745 | 6cm eroded soil |
| | c | II | 162 | 24.75 | 30 | 26.31 | 0 | 230 | 3.5 | 1.4745 | |
| | d | II | 162 | 24.75 | 40 | 26.31 | 0 | 445 | 4.1 | 1.4745 | |
| | e | II | 162 | 24.75 | 50 | 26.31 | 0 | 680 | 5.7 | 1.4745 | |
| | f | II | 162 | 24.75 | 75 | 26.31 | 0 | 1290 | 8.8 | 1.4745 | |
| | g | II | 162 | 24.75 | 100 | 26.31 | 0 | 2020 | 11.5 | 1.4745 | |
| 45 | a | II | 162 | 24.75 | 20 | 19.5 | 0 | 410 | 4 | 1.219 | left erosion |
| | b | II | 162 | 24.75 | 30 | 19.5 | 0 | 960 | 7.5 | 1.219 | |

| Test ID | | Clay Type | Fall height [cm] | Impacts [kPa] | Number of Impacts | $w_c$ [%] | Water layer [cm] | Eroded soil [$cm^3/300cm^2$] | scour depth [cm] | Compaction [$gr/cm^3$] | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | c | II | 162 | 24.75 | 40 | 19.5 | 0 | 1970 | 11.3 | 1.219 | |
| 46 | a | II | 162 | 24.75 | 25 | 24.07 | 0 | 40 | 0.3 | 1.512 | |
| | b | II | 162 | 24.75 | 50 | 24.07 | 0 | 70 | 1.2 | 1.512 | |
| | c | II | 162 | 24.75 | 75 | 24.07 | 0 | 195 | 2 | 1.512 | |
| | d | II | 162 | 24.75 | 100 | 24.07 | 0 | 325 | 2.7 | 1.512 | |
| | e | II | 162 | 24.75 | 125 | 24.07 | 0 | 480 | 3 | 1.512 | |
| | f | II | 162 | 24.75 | 150 | 24.07 | 0 | 560 | 4 | 1.512 | |
| | g | II | 162 | 24.75 | 200 | 24.07 | 0 | 970 | 5.7 | 1.512 | |
| 47 | a | II | 162 | 24.75 | 25 | 24.14 | 0 | 25 | 0.5 | 1.466 | |
| | b | II | 162 | 24.75 | 50 | 24.14 | 0 | 55 | 1.1 | 1.466 | |
| | c | II | 162 | 24.75 | 100 | 24.14 | 0 | 220 | 1.9 | 1.466 | |
| | d | II | 162 | 24.75 | 150 | 24.14 | 0 | 385 | 2.7 | 1.466 | |
| | e | II | 162 | 24.75 | 200 | 24.14 | 0 | 610 | 4 | 1.466 | |
| 48 | a | II | 162 | 24.75 | 50 | 24.85 | 0 | 50 | 0.5 | 1.417 | Hit by the impacts after 4 days |
| | b | II | 162 | 24.75 | 100 | 24.85 | 0 | 95 | 1.8 | 1.417 | |
| | c | II | 162 | 24.75 | 150 | 24.85 | 0 | 185 | 2.5 | 1.417 | |
| | d | II | 162 | 24.75 | 200 | 24.85 | 0 | 310 | 3.8 | 1.417 | |
| 49 | a | II | 162 | 24.75 | 10 | 16.75 | 0 | 180 | 2.4 | 1.354 | side erosion |
| | b | II | 162 | 24.75 | 20 | 16.75 | 0 | 420 | 3.3 | 1.354 | side erosion |
| | c | II | 162 | 24.75 | 30 | 16.75 | 0 | 835 | 4.5 | 1.354 | side erosion |
| | d | II | 162 | 24.75 | 40 | 16.75 | 0 | 1420 | 6.1 | 1.354 | |
| | e | II | 162 | 24.75 | 50 | 16.75 | 0 | 2300 | 9.8 | 1.354 | leftside erosion |
| 50 | a | II | 162 | 24.75 | 10 | 29.27 | 0 | 270 | 2.4 | 1.370 | lifted |
| | b | II | 162 | 24.75 | 20 | 29.27 | 0 | 560 | 4.1 | 1.370 | 10cm block soil |
| | c | II | 162 | 24.75 | 30 | 29.27 | 0 | 1850 | 7.7 | 1.370 | removed |
| | d | II | 162 | 24.75 | 40 | 29.27 | 0 | 2540 | 9.3 | 1.370 | |
| 51 | a | II | 162 | 24.75 | 10 | 28.48 | 0 | 405 | 3.3 | 1.404 | cracks observed |
| | b | II | 162 | 24.75 | 20 | 28.48 | 0 | 1015 | 6.5 | 1.404 | lifted |
| | c | II | 162 | 24.75 | 30 | 28.48 | 0 | 1725 | 9.5 | 1.404 | 4-7cm block soils |
| | d | II | 162 | 24.75 | 40 | 28.48 | 0 | 2320 | 11.5 | 1.404 | removed |
| 52 | a | II | 162 | 24.75 | 50 | 23.8 | 0 | 110 | 1.2 | 1.481 | |
| | b | II | 162 | 24.75 | 100 | 23.8 | 0 | 305 | 2.7 | 1.481 | |
| | c | II | 162 | 24.75 | 150 | 23.8 | 0 | 590 | 4.1 | 1.481 | |
| | d | II | 162 | 24.75 | 200 | 23.8 | 0 | 910 | 5 | 1.481 | |

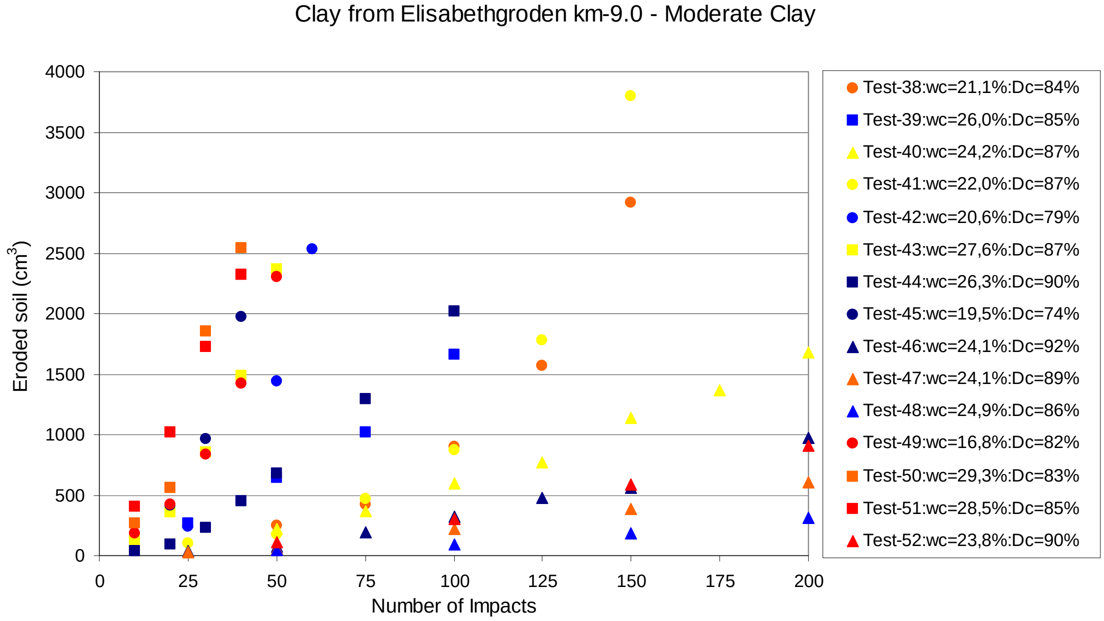



*Figure 4-9 Amount of eroded soil vs. number of impacts for moderate clay (wc: water content, Dc: degree of compaction)*

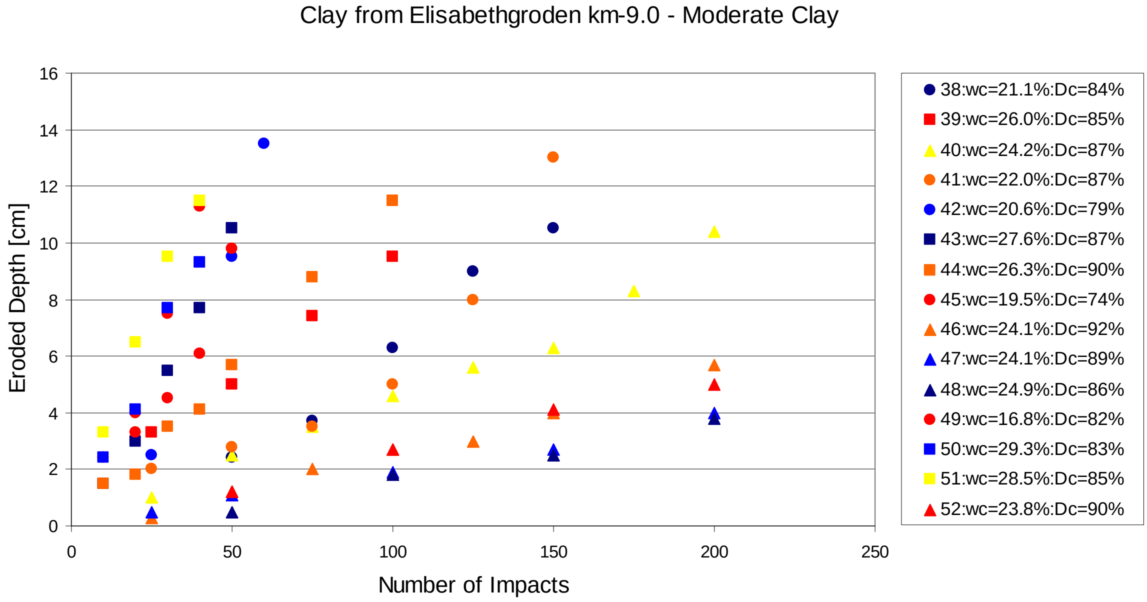



**Figure 4-10 Depth of eroded soil vs. number of impacts for moderate clay (wc: water content, Dc: degree of compaction)**

## 4.3 Test Results on Failure Mechanism of Clay Cover Due to Breaking Wave Impacts (Flume Tests)

The test lasted 78 hours for model 1 and 45 hours for model 2. Both dike models suffered severe damage at the outer slope but breaching did not occur. After suffering severe damage at certain point, the outer slope went to balance, forming very gentle slope. Piping and flow slide at the inner slope was also observed in both models.

### 4.3.1 Model 1

Important features in model 1 are as follows:

1. The wave characteristics are H=10cm, T=1.3s.
2. Small amount of overtopping was observed as predicted. This overtopping did not cause damage to the inner slope.
3. The type of breaking waves was plunging (Figure 4-11)

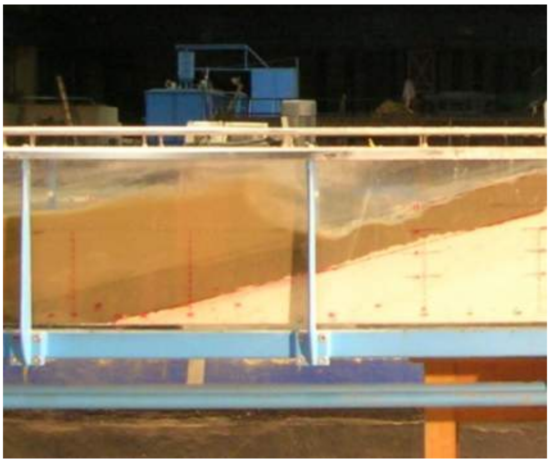


*Figure 4-11 Plunging breaking type for the model*

4. The breaking wave impacts located at 60cm from the toe or at the slope with 15cm from the bottom or 5cm below the mean water level. This point sometime moved back or forward due to reflection (Figure 4-12).

*Figure 4-12 Location of breaking waves below the mean water level creating impacts on the dike slope.*

5. The scour hole at the breaking was clearly observed after 12 hours. Besides breaking wave impacts, wave run-down also played important role on erosion rate. This scour hole was located at the middle of the slope.


(a) (b)

*Figure 4-13 (a) Run-down velocities increase the erosion rate (b) Scour hole due to breaking wave impacts.*

6. The clay cover starts to collapse after 14 hours.

7. The failure of clay cover was followed by seepage that travelled for about 3 hours to make the whole dike body became saturated (Figure 4-14).

*Figure 4-14 Seepage after clay cover failure*

8. Piping was observed at the bottom of the inner slope.

9. The failure of clay cover was also followed by sand undermining, accelerating more damages to the rest of remaining clay cover (Figure 4-15).

*Figure 4-15 Sand undermining causes more damages to the clay cover*

10. The fallen blocks of clay cover due to sand undermining acted as natural breakwater that prevents further erosion for some time. Erosion continued after all the fallen blocks of clay cover washed out (Figure 4-16).

*Figure 4-16 Fallen block of clay naturally protecting the slope from more damages for some time.*

11. Sand bar was building up at the location where the breaking waves were observed before failure occurred (Figure 4-17).

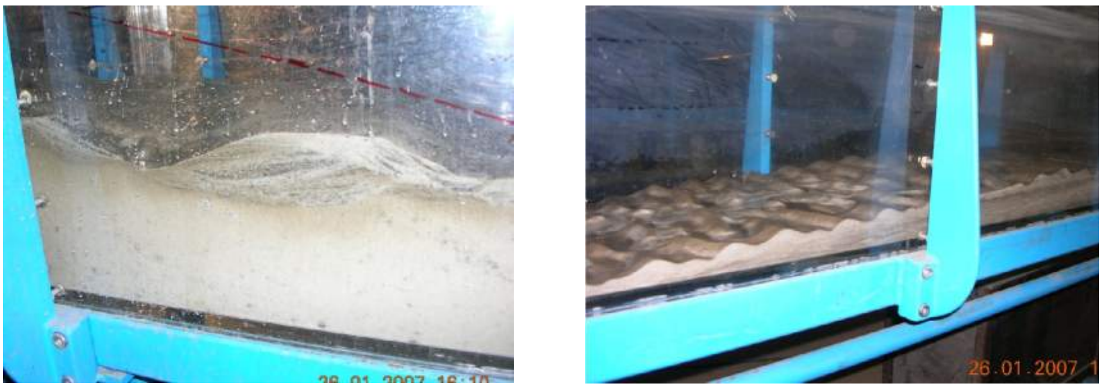


*Figure 4-17 Sand bar formation along the new slope of damaged dike*

12. Flow slide occurred at the inner slope causing large cracks at the clay cover due to gravity (Figure 4-18).


*Figure 4-18 Flow slide cause the whole body of inner slope to move. Cracks appeared due to movement of the sand core.*

13. Once the slope went to gentle, creating s-shape slope with gentle berm, the type of breaking waves was no more plunging waves but collapsing and surging waves (Figure 4-19).


*Figure 4-19 A new slope balancing the dike profile*

14. The changes in type of breaking waves and slope reduced overtopping until finally no-overtopping at all.

15. The final slope after 78 hours running model was 0.056 (Figure 4-20).

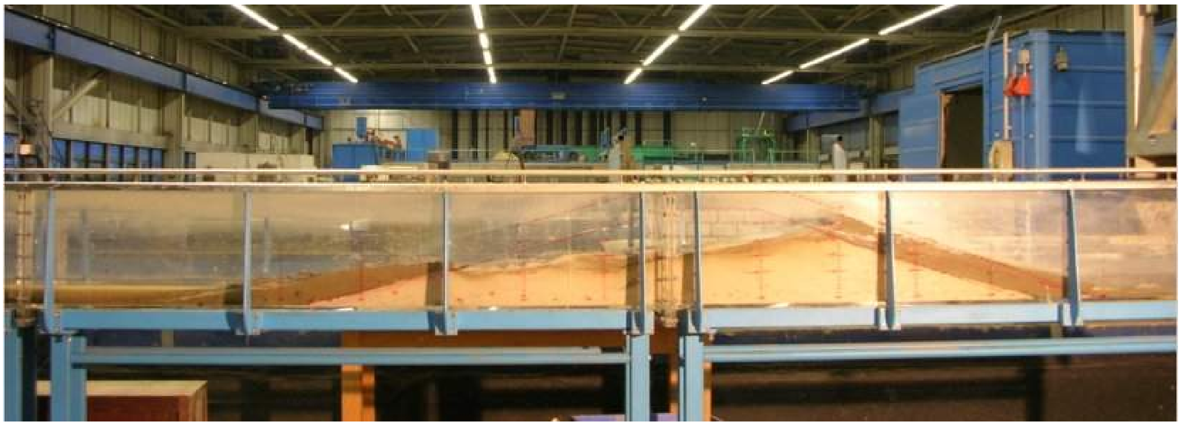


*Figure 4-20 Final Profile of the damaged dike after 78 hours. No breaching occurred due to single factor of breaking wave impacts.*

### 4.3.2 Model 2

In general, most of the features in model 2 are the same as in model 1. Some differences are observed as follows:

1. The erosion did not start at the middle of the slope, but went to the left side. After some time, after erosion reached the crest, all part of dike slope are at the same level of damage (Figure 4-21).

*Figure 4-21 Damage did not start from the middle of the slope*

2. The flow side did occur but the scale was smaller.
3. The erosion was halted up to crest level due to the fact that no significant changes in the last 24 hours. The final slope of the damaged dike is 0.07 (Figure 4-22).

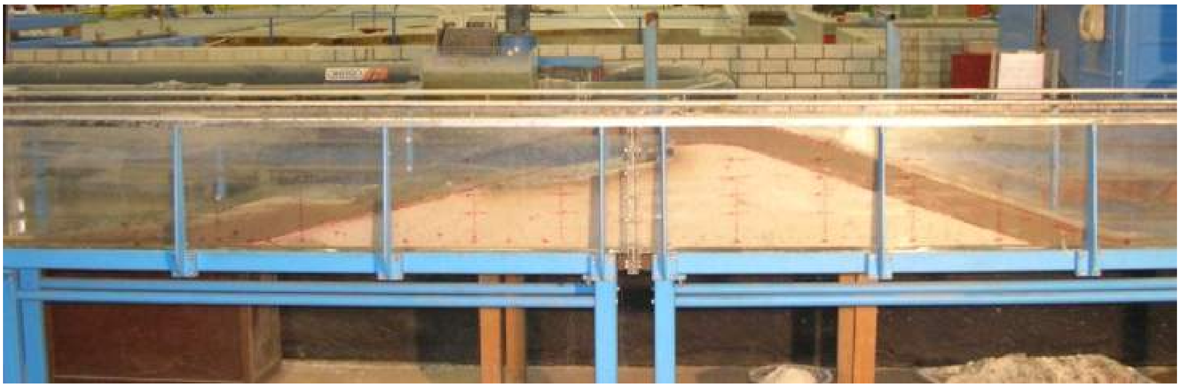


*Figure 4-22 Final Profile of model 2 after being hit by breaking wave impacts for 45 hours.*

# 5. ANALYSIS OF THE RESULTS

## 5.1 *Analysis on Compacted Clay with Water-filled Cracks*

### *5.1.1 Experiment Results vs. the Theory of Führböter (1966)*

The theory of Führböter (1966) mentioned 2 important aspects related to erosion due to impact pressures on water-filled cracks. They are:

- The failure occurs if the maximum impact ($p_{max}$) larger than 2 times of the undrained cohesion ($c_u$). The failure will be along the shear surface where the soil strength from the cohesion ($c_u$) forming an angle called angle of failure.
- The removal of eroded soil in the form of block mass of soil

Results from the current experiments confirm some part of that theory with some extensions. For good clay, There is 1 sample with Ic~0.50 (test 7) suffered failure instantly after 1 impact. The angle of failure is observed ($\alpha$=21.8$^o$) and the estimated undrained cohesion of 2$c_u$ is less than the maximum impact pressure ($p_{max}$). This result confirms the Führböter's theory. Due to its condition which is close to mud, the eroded soils are in the form of small fraction of soils. This does not confirm the Führböter's theory. Questions arise regarding the sample and crack formation in such wet water content. Preparing such samples with artificial crack appears to be difficult. Cracks formations in clay with very high water content are almost impossible.

A simple experiment was carried out to look at the influence of water content on cracks formation over time. Clay category I was prepared with initial water content of 35% (Ic=0.90) and 52% (Ic=0.56). The samples were put at a steel container which has a dimension of 36cm long, 22.5cm wide and 4.5 cm high. The samples were then exposed to the room temperature (about 15$^o$C) for about 3 weeks.


(a) (b)

*Figure 5-1Cracking processes in laboratory scale for good clay after 3 weeks (a) Sample with initial water content of 52% (Ic=0.56) (b) Sample with initial water content of 35% (c=0.90)*

After 3 weeks, the volume of both samples decreased. The sample with high water content (Ic=0.56) shows horizontal and vertical cracks formation in a quite significant size. The widest crack measured was 13mm. For the sample with recommended water content (Ic=0.90), no single crack was observed. The water content of both samples after 3 weeks are close and in the range recommended water content. They are 31.8% and 27.7%. This indicates that formation of significant cracks depend on the water content. Cracks appearance within clay with Ic<0.50 seem to be impossible.

For other sample with high water content (0.50<$I_c$<0.75), failure does occur along shear surface for some samples, but this failure occur after the sample being hit many times by the maximum impact

($p_{max}$) of 24.75 kPa. None of those samples were failed by a single impact. By looking at the estimated undrained cohesion ($c_u$) of the samples, it appears that the maximum impact pressures ($p_{max}$) are still below the value of estimated $2c_u$. Again these results confirm the Führböter's theory. The forms of eroded soils are observed in both small fractions and block of soils. These results might confirm the theory. Only larger impacts seem to be appropriate to investigate more about these phenomena.

For good clay samples with recommended water content (Ic>0.75), small scale failure occurs only at the surface of the crack entrance. The $c_u$ of the samples are far too strong than the $p_{max}$. Moderate clay samples with recommended water content (Ic>0.75) also show the same behaviours as the good clay samples. These results confirm the Führböter's theory.

Moderate clay samples with high water content ($0.50<I_c<0.75$) have low estimated undrained cohesion ($c_u$). The $p_{max}$ is larger than the $2c_u$ but failure did not occur. Failure occurs after samples were hit by impacts more than once. These do not confirm the theory. The forms of eroded soils are observed in both small fractions and block of soils. Like results on good clay, these results might confirm the theory.

From those results, the mechanism of erosion inside the water-filled crack is influenced not only by the undrained shear strength ($c_u$) and the magnitude of the impacts but also by the following factors such as soil density, water content, crack geometry, and clay type.

### 5.1.2 Comparison with results from Rohloff and Stanczak (2006)

Results from good clay are completely different from Rohloff and Stanczak (2006). Good clay samples with water content up to 50% are obviously too strong for the maximum generated impact pressure of 24.75 kPa. The eroded soils are not only in the form of small aggregates and particles but also blocks of soil.

Results from moderate clay are also different from Rohloff and Stanczak (2006). No failure observed in the 1st impact. It means the strength of the clay samples is also larger than the maximum generated impact pressure of 24.75 kPa. **Table 5.1** shows general comparison between current experiment and previous one.

These differences are possibly caused by the differences in pre-treatment of clay samples and some experimental procedures. In Rohloff and Stanczak samples, hard lumps were found in strong clay due to no pre-treatment of samples to avoid lumps. The high values of water content in moderate clay samples (Ic<0.50) could be caused by the way of taking samples and compaction effort (see **Section 2.5.1.3**). Low compacted soil tends to absorb more water due to the presence of large pores. The samples with water-filled pores can have higher water content. In this situation, estimation of soil shear strength ($c_u$) from **Figure 3.3** could be underestimated. The estimation of soil shear strength based on **Figure 3.3** requires well-compacted samples.

*Table 5-1Comparison between current experiment and experiment done by Rohloff and Stanczak (2006)*

| Parameters | Good Clay | | Moderate Clay | |
|---|---|---|---|---|
| | Current experiments | Rohloff & Stanczak (2006) | Current experiments | Rohloff & Stanczak (2006) |
| Fall height | Maximum | Varied | Maximum | Varied |
| Failure after 1st impact | No | Yes | No | Yes |
| Observed angel of failure ($\alpha$) | No | Yes | Yes | Yes |
| Compaction measurement | Yes | No | Yes | No |
| Presence of hard lumps | No | Yes | No | No |
| Water content [%] | 37.5 – 53.2 | 27.9 - 52.6 | 21.7 – 28.1 | 27.3 - 71.1 |

Cracks in clay are impossibly found in mud conditions (liquid state or slushy). In very soft condition, even without cracks, the clay will be easily destroyed by small impacts. The natural water content of moderate clay is in the range of 22% - 26% (see **Table 3.1**).

### *5.1.3 Cracking processes*

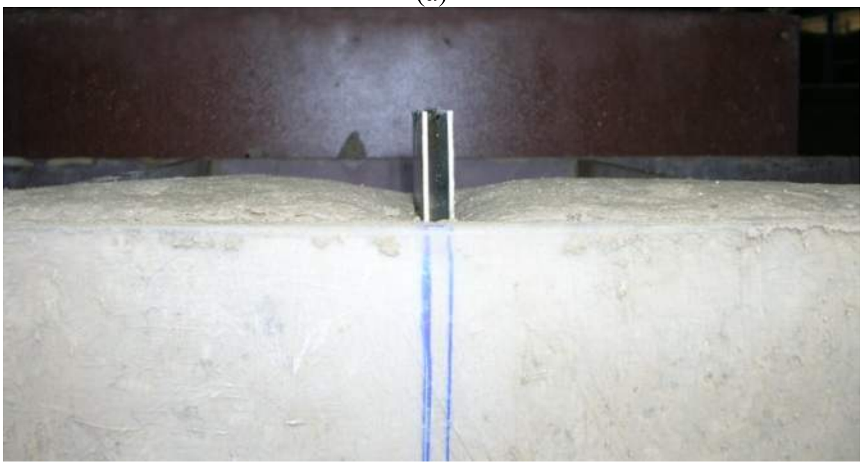
The process of making cracks highly influences the results of the test. The results from the test on both good and moderate clays (both with recommended and high water content) show the importance of cracking processes. Problems arising in cracking processes are:

- Weak lines at the surface of the crack wall are produced during insertion and removal of the plates. Skin frictions between the surface of the plates and the clay cause this problem. These weak lines, later, cause early damage as shown on most the tests with recommended water content.

- When punching the plates down to the sample, swelling of the clay body appears next to the crack (**Figure 5.2**). This condition was observed for the samples with high water content and it was more obvious for moderate clay. For moderate clay, swelling is also followed by cracks at the surface around the artificial crack. Additional efforts to prevent this are needed by putting load at the surface of the swelled area.

Although there is no exact measurement, those problems clearly decrease the estimated soil strength which is highly relying on the solid condition of well compacted clay. A well-compacted clay was then disturbed by forces produced by hammering down and withdrawing up the plates. A better method to overcome these problems is needed for future development of this research (See **Recommendations**).

(a)



(b)

*Figure 5-2 Swelling during the making the artificial crack (a) Unwanted cracks at the surface around the artificial crack due to swelling (b) Swelling of the clay body during hammering down the plates*

### 5.1.4 Crack dimension and pressure magnitude propagation

Pressure propagation inside the water-filled crack depends on its dimension, magnitude of impacts, and air-water mixture (Müller, 1997, Cox and Cooker, 2000, Müller, 2003, Wolters and Müller, 2004, Pachnio, 2005). In this experiment, it is difficult to quantify the air-water mixture phenomena. Therefore, only the magnitude of the impacts and the crack dimension will be discussed.

Müller et all (2003) reported that the pressure magnitude decreases with increasing distance from the crack entrance and it increases with increasing crack width (see **Section 2.5.1**). In contrast, the experiment carried out by Pachnio (2005) showed different results. The pressure magnitude increases with increasing distance from the crack entrance and it decreases with increasing crack width. These differences probably are caused by the difference of the apparatus used and the model set-up. Müller apparatus has water interface in side the chamber between the water-filled crack and the dropping impact. The water-filled crack is put in horizontal position. It is possible that the recorded impact pressure at the crack entrance has been considerably reduced by the presence of water inside the chamber. Therefore, the weakened pressure then has to travel through the water-filled crack in lateral direction and attenuate.

In Pachnio (2005) case, the impact hit a vertical water-filled crack directly. Therefore, the received impacts are pure and travel down through the water filled crack. The gravity also play important role here as it can be seen in the relationship of the generated maximum impact pressure (**Equation 2.18**).

The crack with 4mm and 6mm wide were tested by Pachnio (2005). The results from Pachnio showed that crack geometry influence the pressure propagation. The wider the crack, the lower the pressure magnitude travels through the water-filled cracks. The water-filled crack in the experiment with clay has dimension of 150mm deep, 100mm long and 10mm wide. Therefore, the pressure magnitude at the water-filled crack with 10 mm wide must be lower than the 4mm and 6mm wide.

The pressure magnitude at the surface known as the reference pressure can not be treated as the maximum impact pressure ($p_{max}$) inside the water-filled crack. The increases of pressure magnitude are quite significant. Unfortunately, there is no data/model to estimate the pressure magnitude inside the water-filled crack with 10mm wide.

### 5.1.5 Erosion Mechanisms

Based on observation, erosion mechanisms for water-filled crack hit by the impacts can be divided into 2 categories based on water content; erosion on recommended water content and erosion on high water content.

1. Erosion mechanism on water-filled crack with recommended water content (**Figure 5.3**)

a) No failure after 1$^{st}$ impact
b) Depends on clay type, after several impacts, the entrance wall of the crack starts to erode.
c) Some of the eroded soils in the form of small aggregates and particles are filling in the cracks, replacing the water.
d) The damage at the entrance is getting bigger forming a scour hole with the crack filled by eroded soils and water.
e) Erosion continues following the repetition of the impacts.

In general, the erosion with recommended water content takes more impacts to reach final stage (e.g more than 10 impacts for the moderate clay)

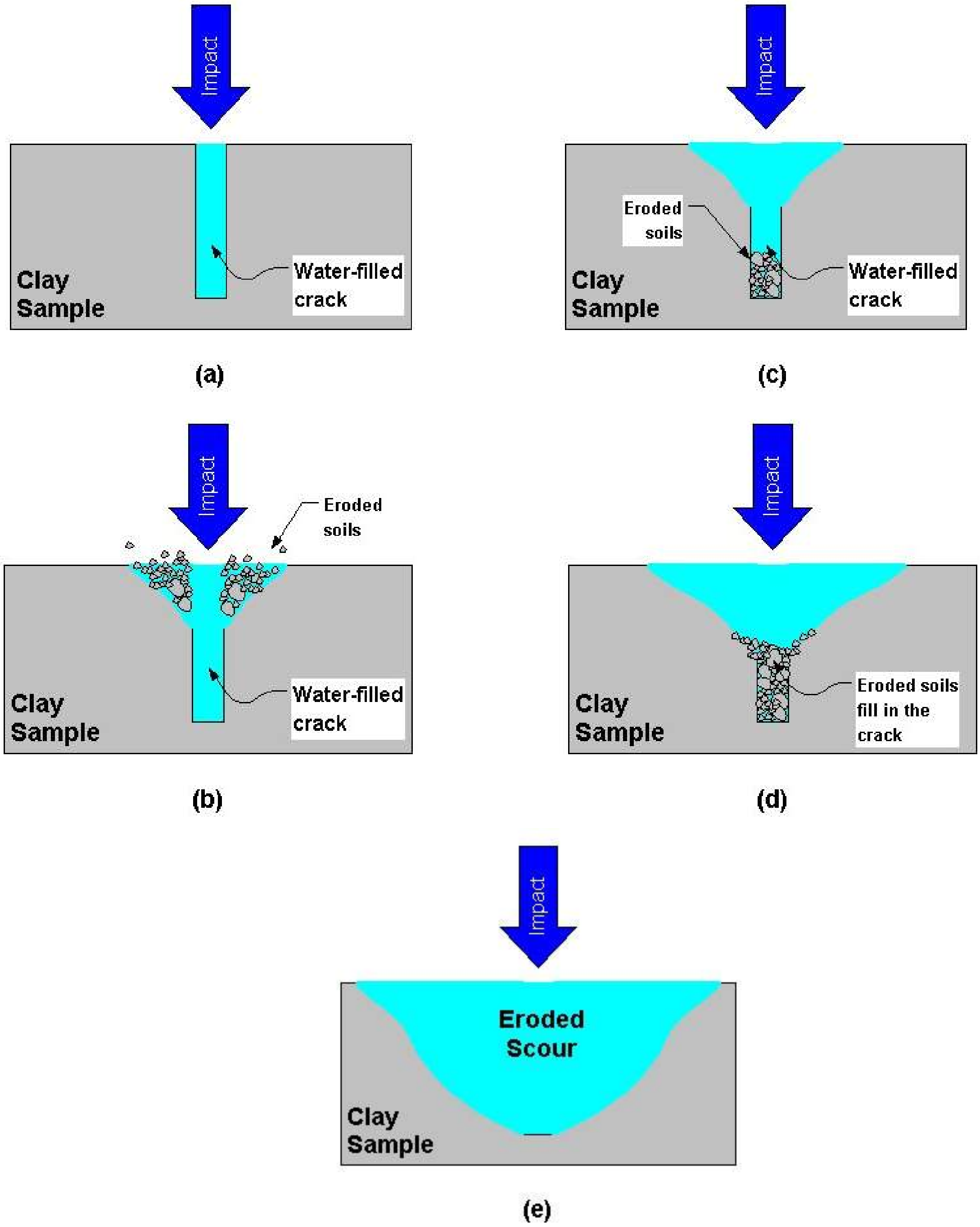



*Figure 5-3 Erosion mechanisms on water-filled crack with recommended water content ($I_c \geq 0.75$) for $p_{max} < 2c$*

2. Erosion mechanism on water-filled crack with recommended water content (**Figure 5.4**)

a) No failure after 1st impact
b) Depends on clay type, after several impacts, the entrance wall of the crack starts to erode. At the same time, the crack width is getting wider and the block walls of the clay in both side of the crack are lifted
c) Small amount of the eroded soils in the form of small aggregates and particles are filling in the cracks, replacing the water.
d) After being hit several times by the impacts, cracks were observed inside the water-filled crack walls. At the same time, at the surface, next to the water-filled crack, horizontal cracks are also observed as the reaction against the repeatedly impact pressures.

e) Furthermore, the cracks formation along the crack wall and at the surface becomes the weak points where the blocks of soil are finally removed.

Erosion is faster than for the clay with recommended water content.

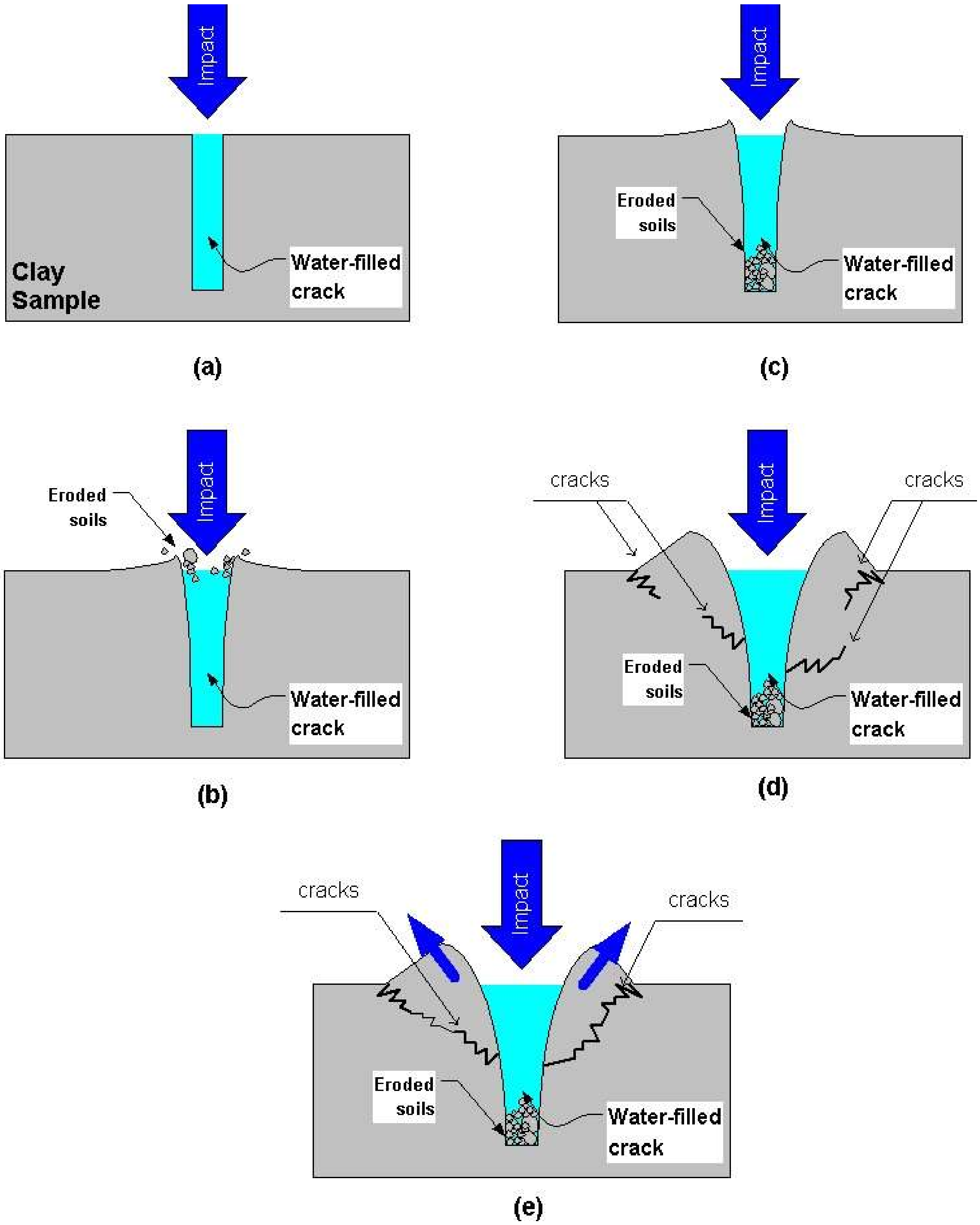



*Figure 5-4 Erosion mechanisms on water-filled crack with high water content ($I_c$<0.75) for $p_{max}$<2c*

### 5.1.6 Building erosion model on clay with water-filled cracks

From the results, some aspects reveal to be very important in developing a model on erosion-resistant behaviors of the water-filled cracks:

1. Clay type

Good clay (clay from *Cäciliengroden*) is proven to be more erosion-resistant than the moderate clay (clay from *Elisabethgroden km-9.0)*. Results show that water-filled cracks for good clay can survive more than 10 maximum impacts while for moderate clay significant damage appears in less than 10 maximum impacts.

2. Magnitude of breaking wave impacts
   The higher the breaking wave impacts, the more damaging results on the erosion of water-filled cracks. The experiments show that the maximum impacts can not destroy the water-filled cracks with one blow. It means the clay with water-filled cracks, as long as it has sufficient strength, is strong enough to withstand impact from 1.2m wave height. But, repeated actions from the impacts (number of impacts) are dangerous and cause severe damage to the cover layer.

3. Water content
   Water content determines the strength of the clay. The clay samples with recommended water content are more erosion resistant than the higher ones.

4. Compaction
   A loose-packed clay layers are vulnerable to erosion. Experiment results with good clay show drastic behaviors in erosion-resistant of the water-filled cracks. A water-filled crack in a loose-packed clay layer failed to withstand the maximum impact pressure of 24.74 kPa.

5. Crack dimension
   Although there is no experiment with variation in crack dimension, previous works and current results show the importance of crack dimension towards erosion resistance. The experiment in clay with high water content shows the change of cracks dimension after several times hit by the impacts until finally the crack walls collapse.

The results from moderate clay show the uncertainties between the $p_{max}$ and the cohesion. In 3 tests with high water content, the $p_{max}$ is always larger than the estimated shear strength but no failure occurs. The following factors could be contributing to these results:

1. The unknown pressure magnitudes inside the water-filled crack (see **Section 5.1.4**).
2. Other resistant forces within the soil such as the weight of the soils and pore pressures (Richwien, 2003) are ignored in Führböter's theory. Shear strength is only the only resistant the soil has.

With the available data, the erosion model on clay with water-filled crack can not be developed. Difficulties in developing this model arise as follows:

1. In these experiments, the maximum impact magnitude of 24.75kPa is not big enough to produce pressures inside the water-filled crack that can cause failure. Changes in experimental set-up is needed to produce higher maximum impact magnitude

2. The limitation of water content for samples of larger than Ic>0.50 causes the strength of the soils is far larger than the maximum impact magnitude generated by the machine. Since the clay properties can not be changed, the only possibility to coup this problem is by increasing the fall height of the machine to produce higher maximum impact magnitude.

3. The repetition of impacts can not be used to develop the model. The cracks are widening during impact repetitions. The change in geometry causes the change in magnitude of propagated impact pressures inside the water-filled crack. A single impact which can destroy the water-filled crack is needed

4. Due to impact repetitions, the presence of eroded soils in the form of particles and small aggregates can not be avoided. These cause drastic changes of the characteristics of water-filled crack.

Therefore, to avoid the presence of eroded soil inside the water-filled crack, a single impact which can destroy the water-filled crack is needed.

## 5.2 Analysis on Compacted Clay without Cracks (Surface Erosion Tests)

### 5.2.1 Soil density

Erosion resistant characteristics of clay depend on how well the clay is compacted (Hanson& Robinson (1996), TAW (1996), Hanson, Cook, & Simon (1999), Hanson & Hunt (2006)). In laboratory scale, soil density measurement is done either by Standard Proctor Density (PD) or Modified Proctor Density Method (MPD). The density value (represent by dry density, $\rho_d$) from the laboratory test is then used as reference to do compaction in practice. The density value itself is a function of the water content and widely known as dry density – water content relationship. In dike construction, the required degree of compaction is at least 95% of standard proctor density (PD).

For this experiment, the dry densities (PD) of good and moderate clay are 1.458gr/cm$^3$ and 1.643gr/cm$^3$ with optimum water content 25.95 and 18.5% respectively. Therefore, the minimum density values to meet the requirement (95% PD) are 1.385gr/cm$^3$ for good clay and 1.56gr/cm$^3$ for moderate clay. To get those values for the experiments, in-situ density measurement was carried out for each sample by applying Non-Standard in-Situ Sand Replacement Method (See **Appendix E** for details). By taking an assumption that the whole part of sample has the same degree of compaction, the in-situ density measurement took place at 40cm from the test location to maintain undisturbed condition of the tested area (**Figure 5.5**).

Results from the in-situ measurements showed deviations from the standard one. The revealed values of degree of compaction for good clay range from 71% - 90% PD and 74% - 92% PD for moderate clay. Density of good clay show tendency to get higher towards the plastic limit (**Figure 5.6**) and then decrease towards the optimum water content of proctor. This could be results of difficulties during compaction with water content less then the plastic limit. The same tendency is also observed in moderate clay samples (**Figure 5.7**). Getting the required density seems to be difficult when it closes to the plastic limit. The use of glass walls make operational of compaction should be done carefully. Furthermore, the Non-Standard in-Situ Sand Replacement Method has never been calibrated

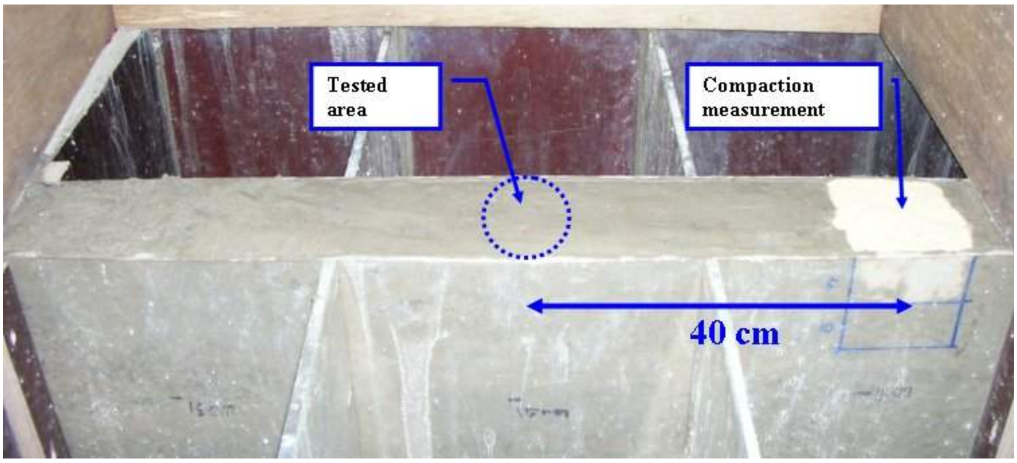



*Figure 5-5 Location of in-situ density measurement*

*Table 5-2 Summary of compaction for good clay and moderate clay samples*

| Good clay | | | | Moderate clay | | | |
|---|---|---|---|---|---|---|---|
| Test ID | $w_n$ [%] | Compaction [gr/cm$^3$] | Relative Density [%] | Test ID | $w_n$ [%] | Compaction [gr/cm$^3$] | Relative Density [%] |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 23 | 34.7 | 1.304 | 89.4 | 38 | 21.14 | 1.381 | 84.0 |
| 24 | 37.3 | 1.286 | 88.2 | 39 | 26.02 | 1.392 | 84.7 |
| 25 | 42.06 | 1.280 | 87.8 | 40 | 24.2 | 1.432 | 87.1 |
| 26 | 46.02 | 1.188 | 81.5 | 41 | 22.01 | 1.425 | 86.7 |
| 27 | 41.35 | 1.206 | 82.7 | 42 | 20.56 | 1.301 | 79.2 |
| 28 | 42.7 | 1.164 | 79.8 | 43 | 27.59 | 1.428 | 86.9 |
| 29 | 43 | 1.160 | 79.6 | 44 | 26.31 | 1.475 | 89.7 |
| 30 | 48.6 | 1.076 | 73.8 | 45 | 19.5 | 1.219 | 74.2 |
| 31 | 47.64 | 1.036 | 71.1 | 46 | 24.07 | 1.512 | 92.0 |
| 32 | 48.32 | 1.124 | 77.1 | 47 | 24.14 | 1.466 | 89.2 |
| 33 | 50.2 | 1.059 | 72.6 | 48 | 24.85 | 1.417 | 86.2 |
| 34 | 50.73 | 1.129 | 77.4 | 49 | 16.75 | 1.354 | 82.4 |
| 35 | 35.17 | 1.264 | 86.7 | 50 | 29.27 | 1.370 | 83.4 |
| 36 | 27.71 | 1.208 | 82.9 | 51 | 28.48 | 1.404 | 85.4 |
| 37 | 27 | 1.261 | 86.5 | 52 | 23.8 | 1.481 | 90.1 |
| Average | | 1.183 | 81.1 | | Average | 1.404 | 85.4 |
| Max | | 1.304 | 89.4 | | Max | 1.512 | 92.0 |
| Min | | 1.036 | 71.1 | | Min | 1.219 | 74.2 |

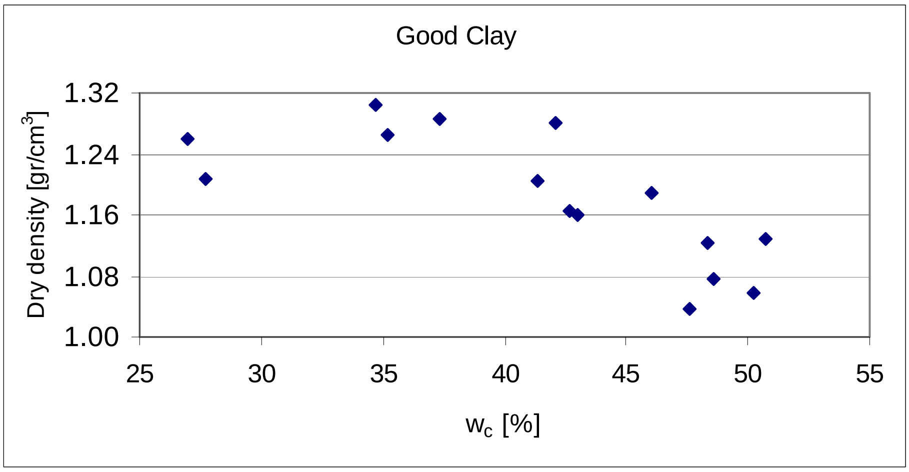



*Figure 5-6 Relationship of water content and in-situ dry density for good clay*

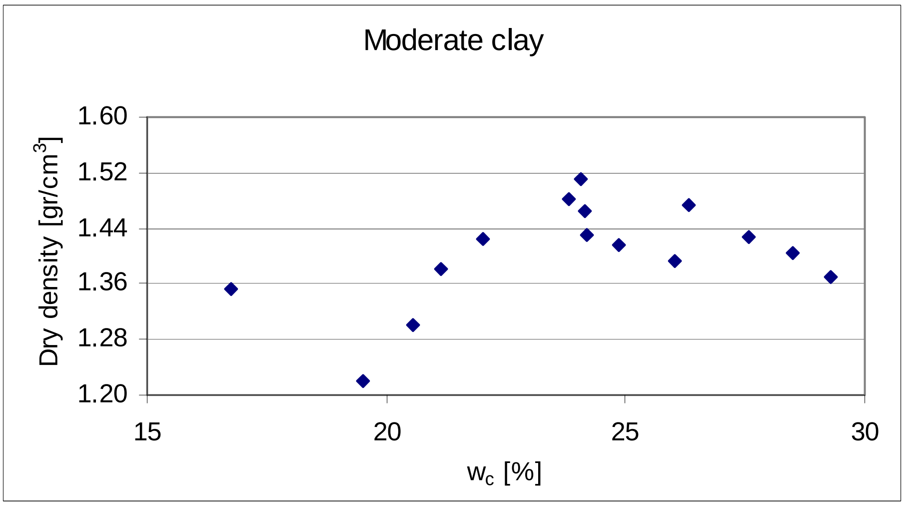



*Figure 5-7 Relationship of water content and in-situ dry density for moderate clay*

### 5.2.2 Detachment Coefficient and Critical Stress

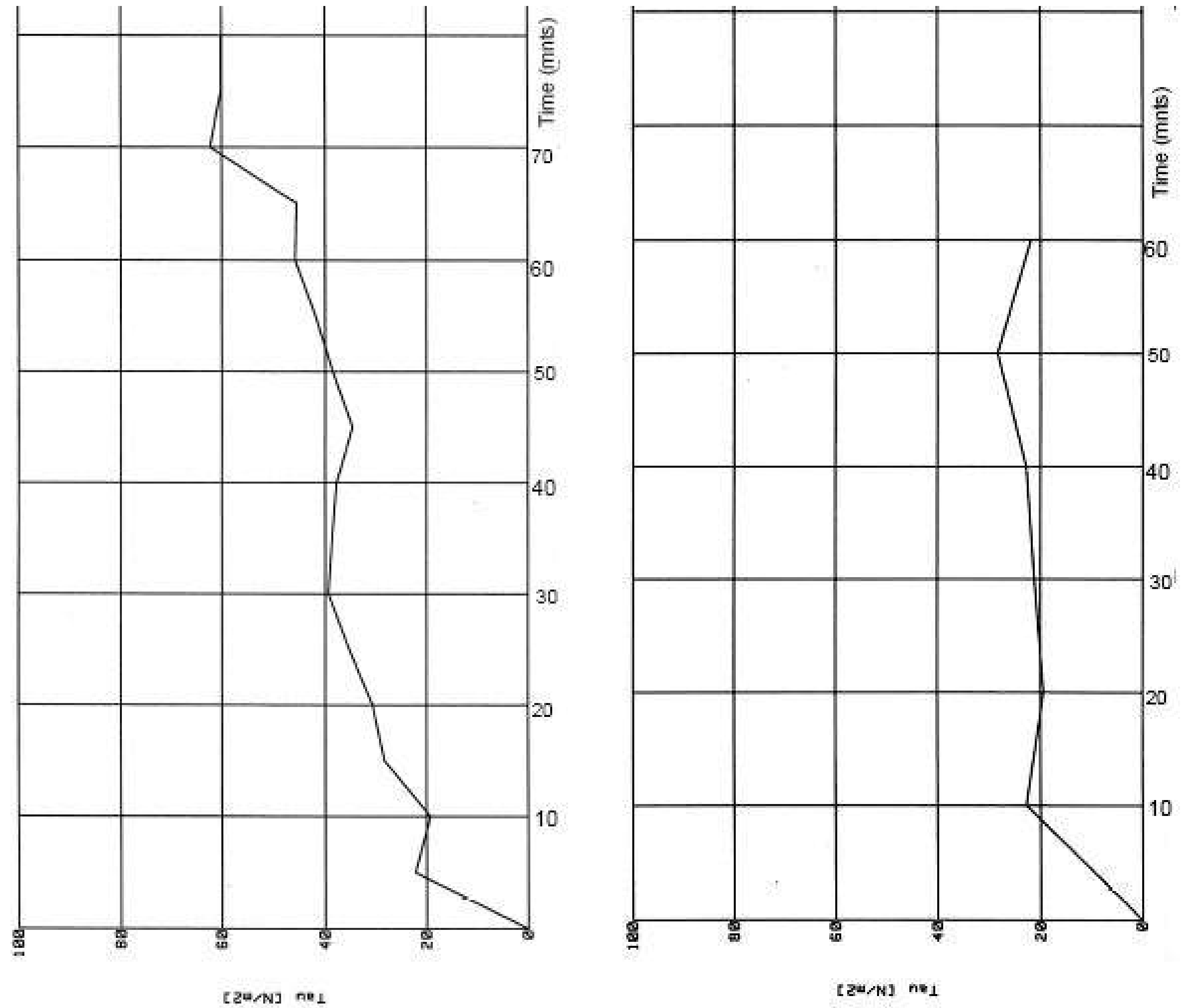
Detachment coefficient (also known as erodibility coefficient) is an important parameter to quantify erosion resistant soils. Based on the stress based detachment equation (**Equation 2.19**) and using the formula proposed by Woolhiser (**Equation 2.22**), the amount of eroded soil ($R_d$) and the hydraulic load (represented by the kinetic energy from the impact $\tau_e$=$E_k$) are measured from the experiment. The exponent term is determined to be zero since there is no water layer involved. The kinetic energy from falling water mass is defined as:

$$E_k = mgh_f \tag{5.1}$$

Where:
Ek : Kinetic energy [Joule]
m : Mass of water (here m is always 2kg)
g : Gravity (9.81 m/s)
$h_f$ : Fall height (here $h_f$ is always 1.62m)
The only unknown parameter to determined detachment coefficient ($k_d$) is the critical stress. The critical stress is the maximum stress that the clay can resist the erosion forces. The limitation to determine this critical stress for cohesive materials has been found to be difficult. The critical stress for the analysis on this report is assumed to be zero ($\tau_c$=0). The following are some considerations of assuming $\tau_c$=0:

1. From the observations, there is always erosion (in the form of removed particles/grains or shape deformation) after being hit by the 1st impact, particularly for the samples with high and dry water content.

2. The erosion test on cylindrical rotating device from Geodelft (2003) using the same remolded samples from Cäciliengroden and Elisabethgroden km-9.0 measured very small value of shear stress to start erosion (less than 0.1 kPa) compare to a single impact that measured up to 24,75kPa.




(a) (b)

*Figure 5-8 Shear stresses from erosion test using cylindrical rotating device for (a) Clay from Cäciliengroden and (b) Clay from Elisabethgroden km-9.0 (Geodelft, 2003)*

Therefore, **Equation 2.22** can be re-expressed to determine the detachment coefficient ($k_d$):

$$k_d = \frac{R_d}{nE_k} \qquad (5.2)$$

Where:

$R_d$ : Volume of eroded soil per single impact [$cm^3$]
$k_d$ : Detachment coefficient [$cm^3$/Joule]
n : Number of impact energy
Ek : Kinetic energy [Joule]

The rate of eroded soil has the same tendency as amount of eroded soil per number of impact shown in **Chapter 4, Figure 4.7**, **4.8**, and **4.9**. **Figure 5.9** and **5.10** show the rate of eroded soil for good clay and moderate clay. Both figures describe the rate of eroded soil related to number of impacts and water content. The clay samples with Ic<0.75 (high water content) and Ic>1.0 (dry water content) are more vulnerable to erosion than the clay samples in the range of PL>Ic>0.75 (recommended water content). For good clay, samples with recommended water content have erosion rate of less than 0.50$cm^3$/kPa for over 500 impacts while the samples with high and dry water content, the erosion rate can reach up to 3 $cm^3$/kPa in less than 100 impacts.

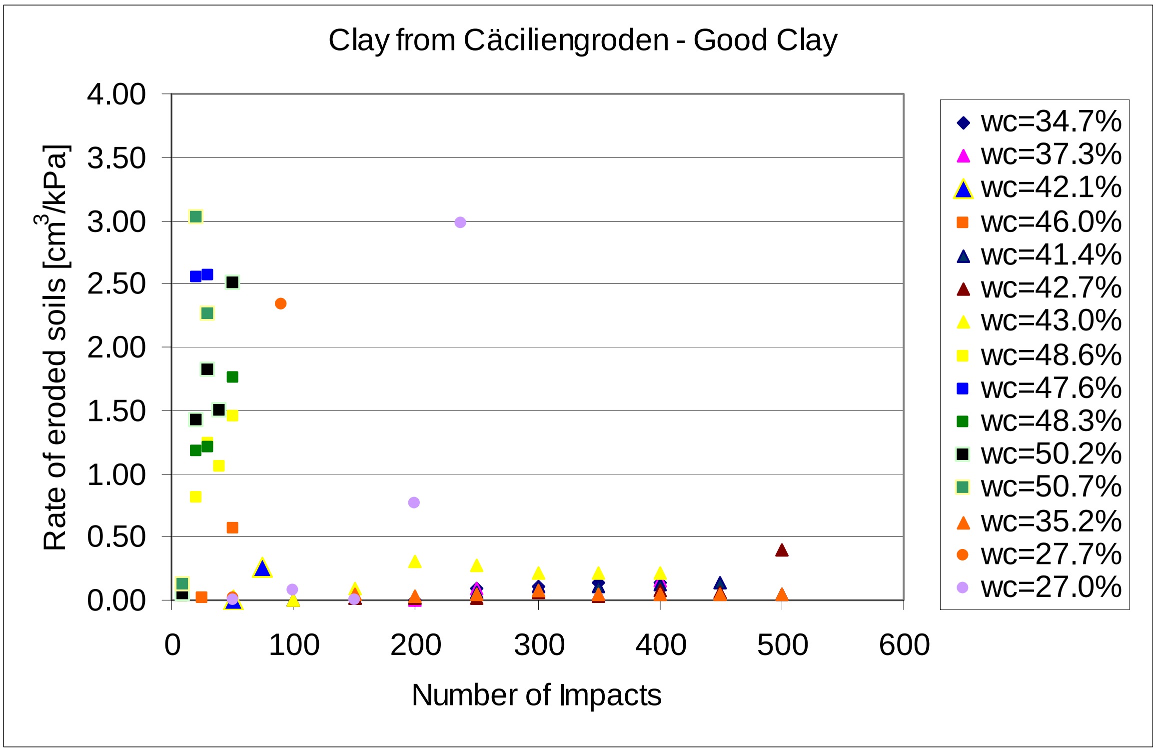



*Figure 5-9 Rate of eroded soil for good clay*

The same as good clay samples, moderate clay samples with recommended water content are more erosion resistant with erosion rate of less than 1.50cm$^3$/kPa for over 100 impacts while the samples with high and dry water content, the erosion rate can reach up to 4.5 cm$^3$/kPa in less than 50 impacts

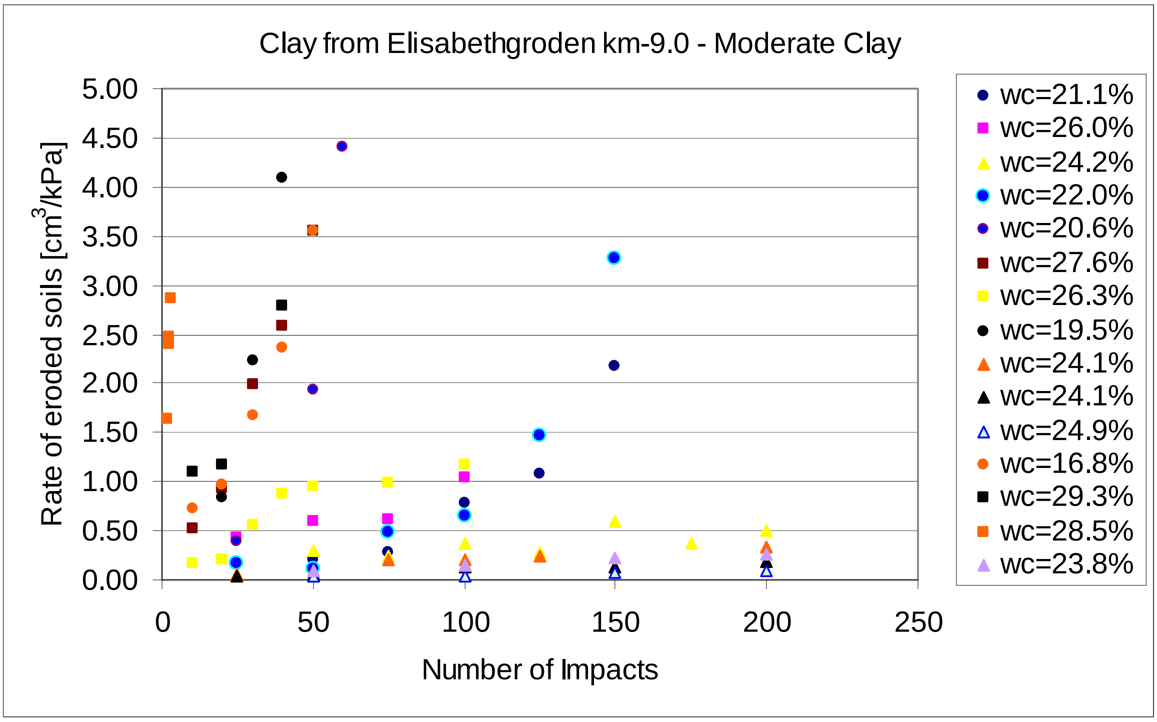



*Figure 5-10 Rate of eroded soil for moderate clay*

Detachment coefficient ($k_d$) can be used to determine in which range the optimum water content is the most erosion resistant against the impacts. The higher the value of kd, the less erosion resistant the samples are. **Figure 5.11 & 5.12** shows the detachment coefficient curves ($k_d$) for good and moderate clay. The curves are drawn to show the difference between the detachment coefficient [$cm^3$/J] as a function of impact energy and the impact itself [$cm^3$/kPa]. From both figures, the detachment coefficient due to kinetic energy of the impacts has lower values than the one using impact values.

From the **Figure 5.11**, Sample with water content about 35% is the most erosion resistant indicated by its smallest $k_d$ value (=0.03 $cm^3$/J). It is also can be seen that the recommended water contents from TAW do not occupy all range with low value of $k_d$. On the left side, the curve goes up after passing the plastic limit (PL=32%). On the right side, the curve of the upper limit of the recommended water content ($w_{c(max)}$=43%) seems to be in agreement as TAW recommendation.

For moderate clay, sample with water content 24.85% is the most erosion resistant indicated by its smallest $k_d$ value (=0.05 $cm^3$/J). Similar to the results of good clay, the recommended water contents from TAW do not occupy all range with low value of $k_d$. Moderate clay shows very narrow range of optimum water content. On the left side, the curve goes up after passing the plastic limit (PL=20%). On the right side, the curve of the upper limit of the recommended water content ($w_{c(max)}$=25%) seems to be in agreement as TAW recommendation.

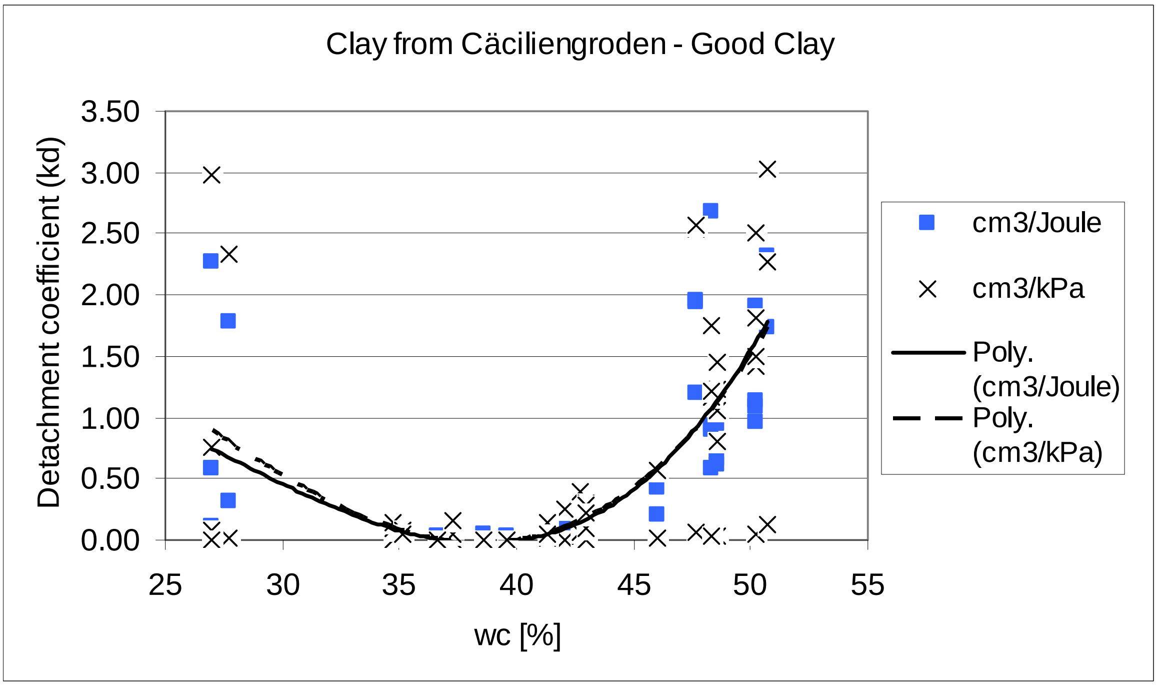



*Figure 5-11 Detachment coefficient ($k_d$) for good clay*

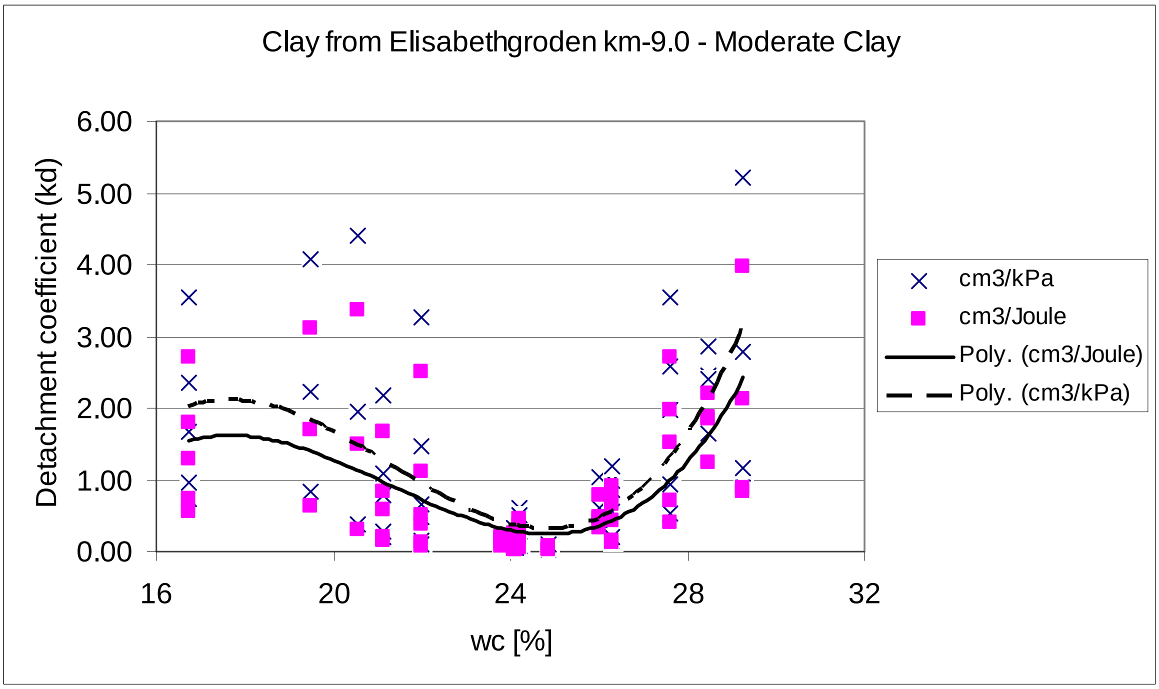



*Figure 5-12 Detachment coefficient.($k_d$) for moderate clay*

By taking Ic=0.75 as point of reference, the detachment coefficient ($k_d$) can be divided into erosion resistant regions. For good clay, the average value for kd for samples with Ic>0.75 is about 0.07 $cm^3$/J. Since there are no enough data to look at the region below the plastic limit, determination of $k_d$ can not be carried out (**Figure 5.13**).

For good clay with Ic<0.75, the average $k_d$ values fit with the exponential curve (**Figure 5.14**). Therefore, the $k_d$ values are defined as exponential function of the water content:

$$k_d = 1x10^{-7} e^{0.3286 w_c} \quad \text{(for good clay with Ic<0.75)} \quad (5.3)$$

For moderate clay with Ic>0.75, the average $k_d$ values also fit the exponential curve (**Figure 5.15**). Therefore, the $k_d$ values are defined as exponential function of the water content:

$$k_d = 11327 e^{-0.4642 w_c} \quad \text{(for moderate clay with Ic>0.75)} \quad (5.4)$$

Similar to good clay samples, moderate clay with high water content or Ic<0.75 have average kd values that fit the exponential curve (**Figure 5.16**). Therefore, the $k_d$ values are defined as exponential function of the water content:

$$k_d = 7x10^{-7} e^{0.7638 w_c} \quad \text{(for moderate clay with Ic<0.75)(5.5)}$$

Complete $k_d$ values for each region of application are shown in **Table 5.3**

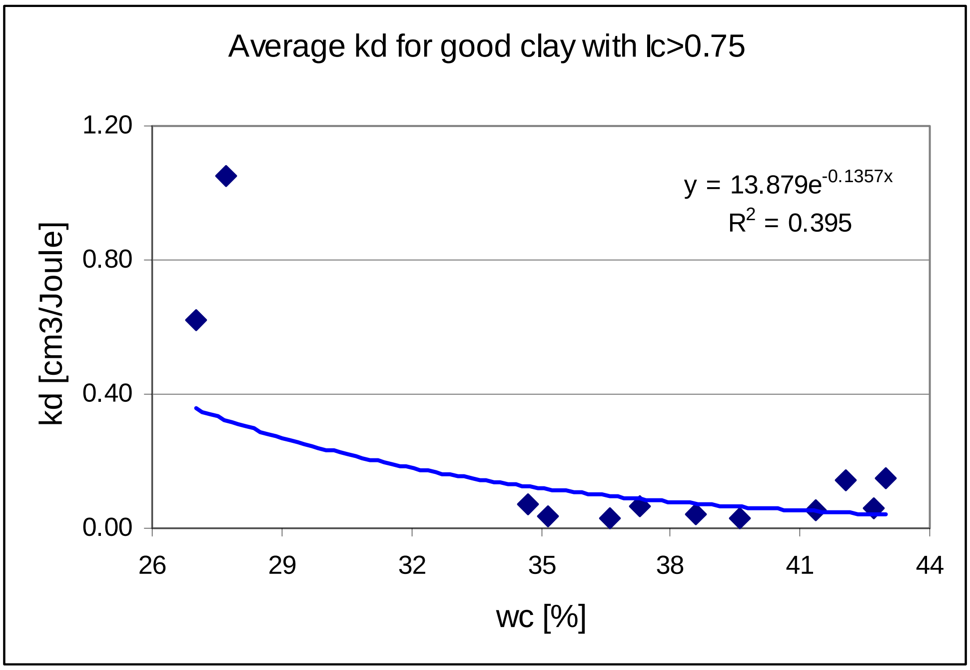



*Figure 5-13 Detachment coefficient.($k_d$) for good clay with Ic>0.75*

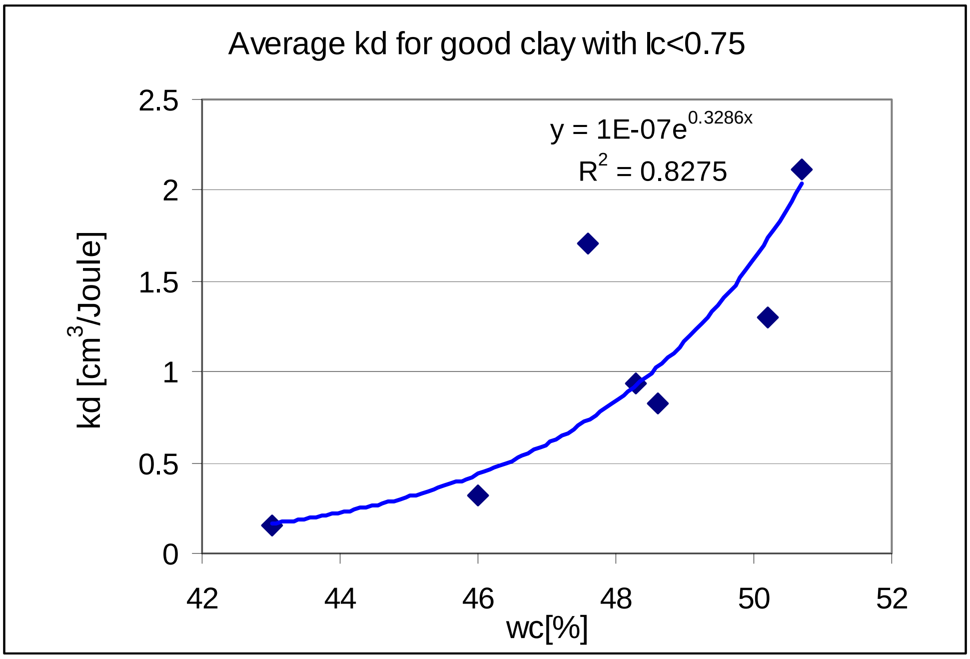



*Figure 5-14 Detachment coefficients ($k_d$) for good clay with Ic<0.75*

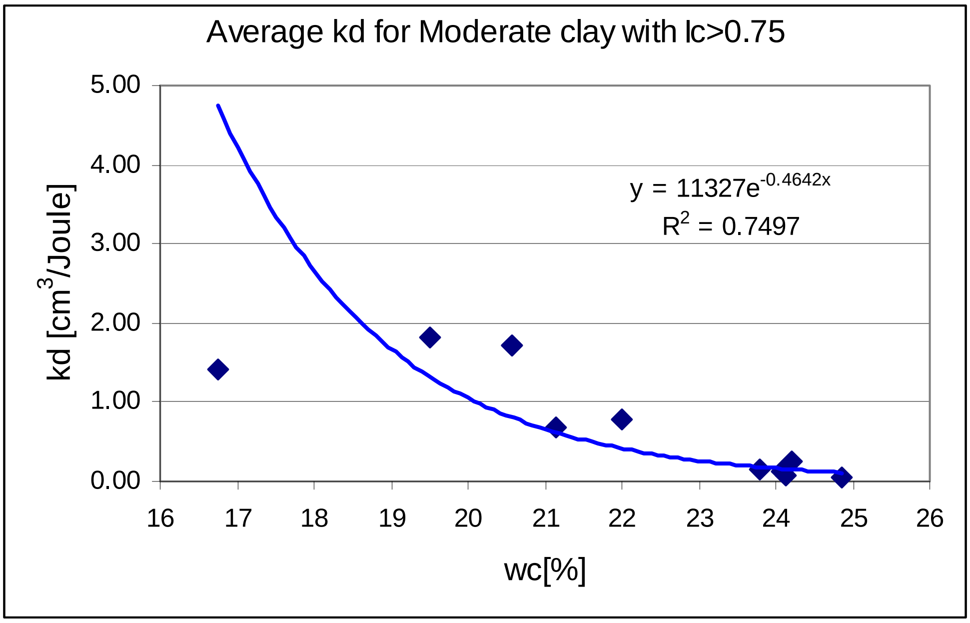



*Figure 5-15 Detachment coefficients ($k_d$) for Moderate clay with Ic>0.75*

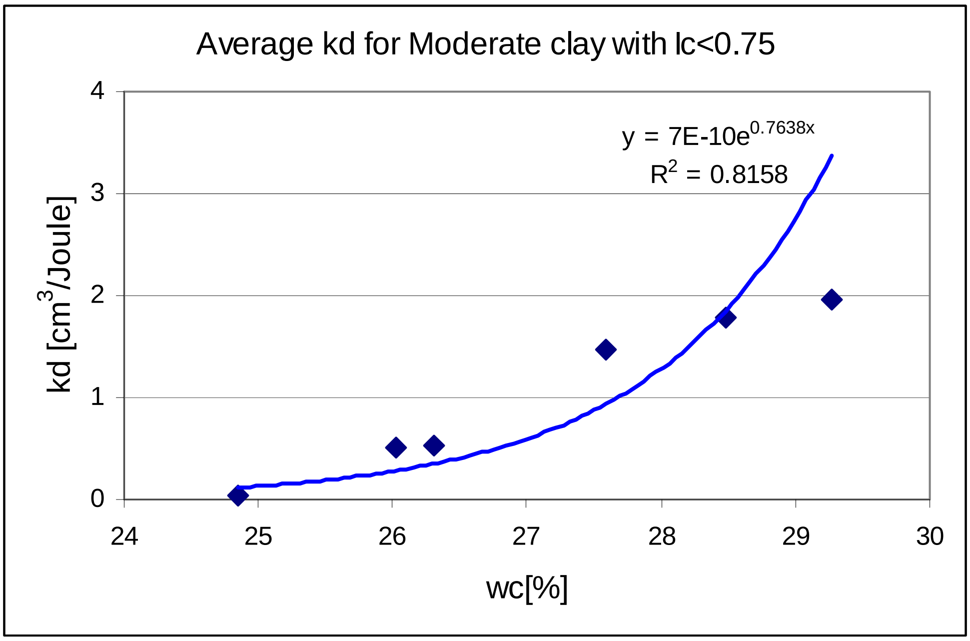



*Figure 5-16 Detachment coefficients ($k_d$) for moderate clay with Ic<0.75*

*Table 5-3 Detachment coefficients ($k_d$) for good and moderate clay*

| Region of application | Good clay | Moderate clay |
|---|---|---|
| Ic<PL | - | $k_d = 11327e^{-0.4642w_c}$ |
| 0.75<Ic<PL | 0.07 | $k_d = 11327e^{-0.4642w_c}$ |
| Ic<0.75 | $k_d = 1x10^{-7}e^{0.3286w_c}$ | $k_d = 7x10^{-7}e^{0.7638w_c}$ |

*Note: PL has $I_c$=1.0*

The samples below the minimum recommended water content PL until the optimum water content on Proctor Density are supposed to be erosion resistant as well. Difficulties in operational set up can be the main factor causing this problem. Working with clay samples which have water content close to the plastic limit (PL) is difficult and needs a lot of effort (TAW 1996). This can cause undesired condition of the sample, for example, the compaction is not as required. The data from compaction measurements (**Figure 5.6** & **5.7**) show that most samples near the plastic limit have quite low density values.

Those samples with dry water content had less compaction efforts (see **Section 5.2.1**). They suffered rapid changes in water content, loosing the strengths due to increasing pore pressures. Some measurements to look at these changes are shown in **Table 5.4**. Observations showed this problem also caused unsymmetrical shape of eroded scour. Most of samples with less compacted effort started to erode at the edge of the glass wall or side erosion (**Figure 5.17**). From these points, erosion goes faster causing severe damage to the samples.

*Table 5-4 Water content change measurements*

| Test ID | Clay Type | Water Content [%] | | $w_c$ changes [%] |
|---|---|---|---|---|
| | | Before | After | |
| 29 | I | 42.4 | 42.6 | 0.2 |
| 30 | I | 48.6 | 48.9 | 0.3 |
| 31 | I | 47.6 | 48.7 | 1.1 |
| 37 | I | 26.8 | 36.4 | 9.6 |
| 38 | II | 21.1 | 30.4 | 9.3 |
| 39 | II | 26.0 | 26.1 | 0.1 |
| 40 | II | 24.2 | 24.3 | 0.1 |
| 42 | II | 20.6 | 30.8 | 10.2 |
| 43 | II | 27.6 | 28.2 | 0.6 |
| 45 | II | 19.5 | 26.4 | 6.9 |
| 46 | II | 24.1 | 24.5 | 0.4 |
| 47 | II | 24.1 | 24.4 | 0.3 |
| 48 | II | 24.9 | 24.9 | 0.0 |
| 49 | II | 16.8 | 29.4 | 12.7 |

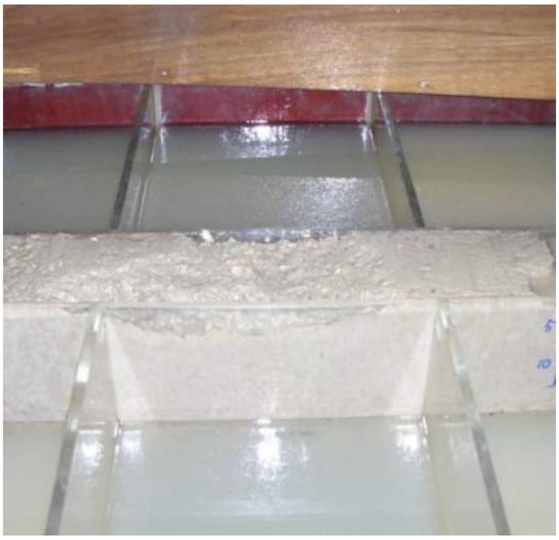


*Figure 5-17 Less compacted samples cause side erosion and lead to early severe damage*

### 5.2.3 Erosion Mechanism

The impact pressures hit the compacted clay for several times causing scour hole (erosion) up to certain depth. The mechanism of the formation of this scour hole was observed and can be described in 3 stages:

***1. The release of water mass***

The water mass was released from the tube (see **Figure 3.1c**) by an automatic computer-control devices consist of:

- a water intake pipe with its valve which is responsible for filling in the tube by water
- a pressure gauge that maintain the water level at $h_w$=25 cm
- a pneumatic valve that control the releasing of the water mass after it reaches $h_w$=25 cm

The mechanism of the releasing water mass controlled by the pneumatic valve has side effect to the shape of the water mass. The pneumatic valve is opened downward to release the water mass causing imperfectness of the water mass shape. The water mass has 'a tongue' as a result of this opening valve process.

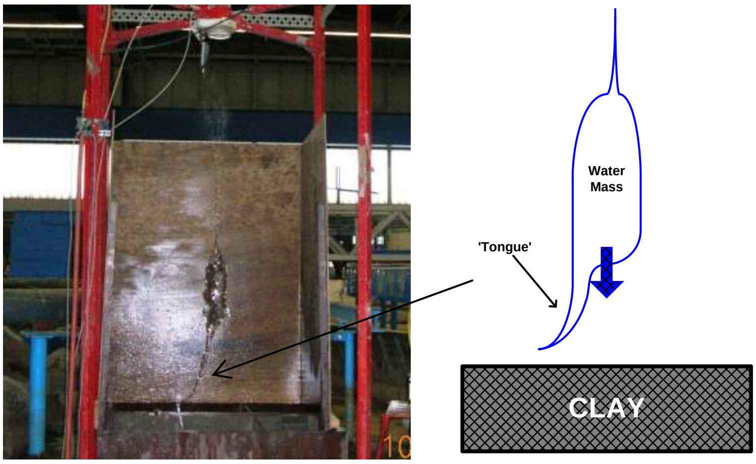



*Figure 5-18 The tongue as the result from the valve opening mechanism. The tongue touches the clay at first before the whole body of water mass hit the clay surface*

**2. The impact**

When the water mass hits the clay, the pressures are distributed around the impact point and the clay sample will react differently depends on several factors such as water content, homogeneity, degree of compaction, etc. Generally, from the observation, after water mass hit the clay surface, soil disintegration occurs around the impact area forming cracks in all direction. These cracks are hardly observed for the clay with high water content (Ic<0.75). These cracks are visible when the surface has been eroded up to certain depths. These cracks could be the results of almost-detached soil after being hit by the impacts.

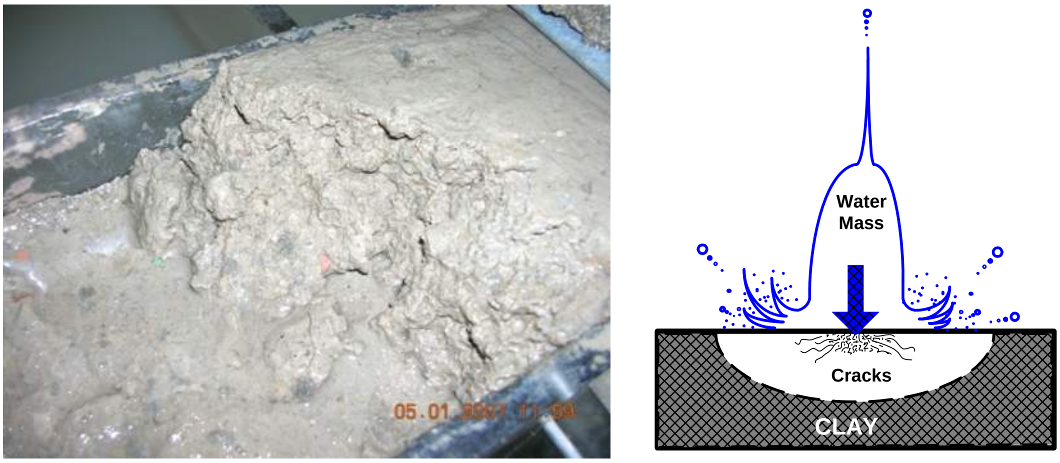



*Figure 5-19 Soil disintegration forming cracks after hitting by repeated impacts*

**3. The scour formation and the eroded clay**

A crater-like hole is developed at the surface after the impacts. For a perfect shape of water mass, the scour hole will be symmetrical (Ghadiri, 2004). Due to imperfectness of the water mass shape, which has a tongue, general shape of the scour hole is not symmetrical. It has a slope at one edge (where the tongue hit first) and a sharp edge at the other side due to pure impact from the rest of the impact water mass.

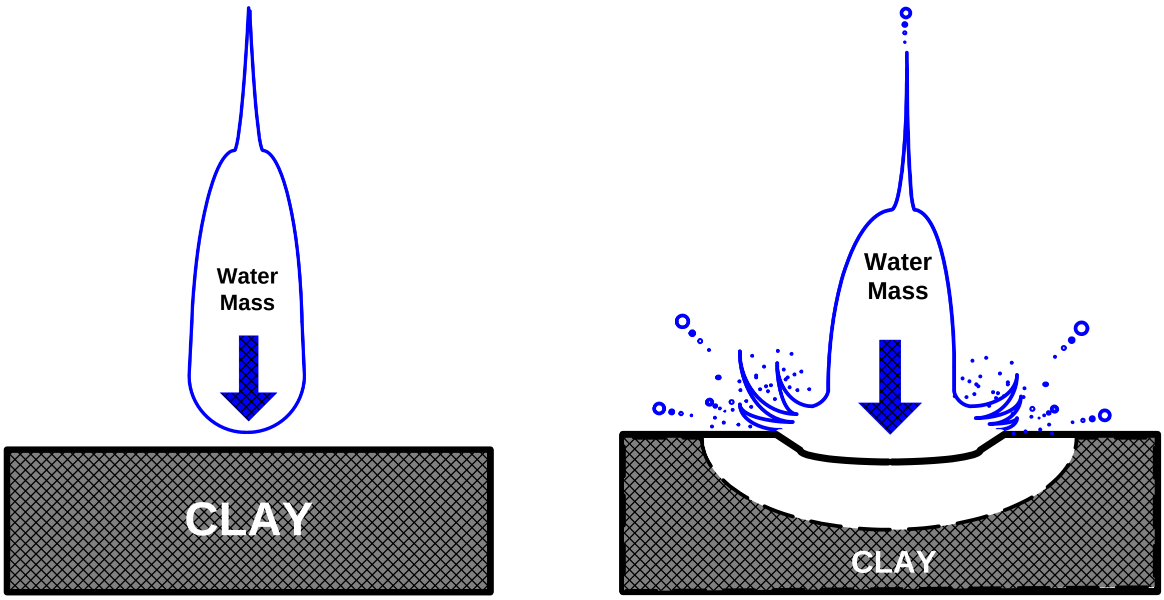



*Figure 5-20 A perfect falling-water mass and its symmetrically crater-like scour hole*

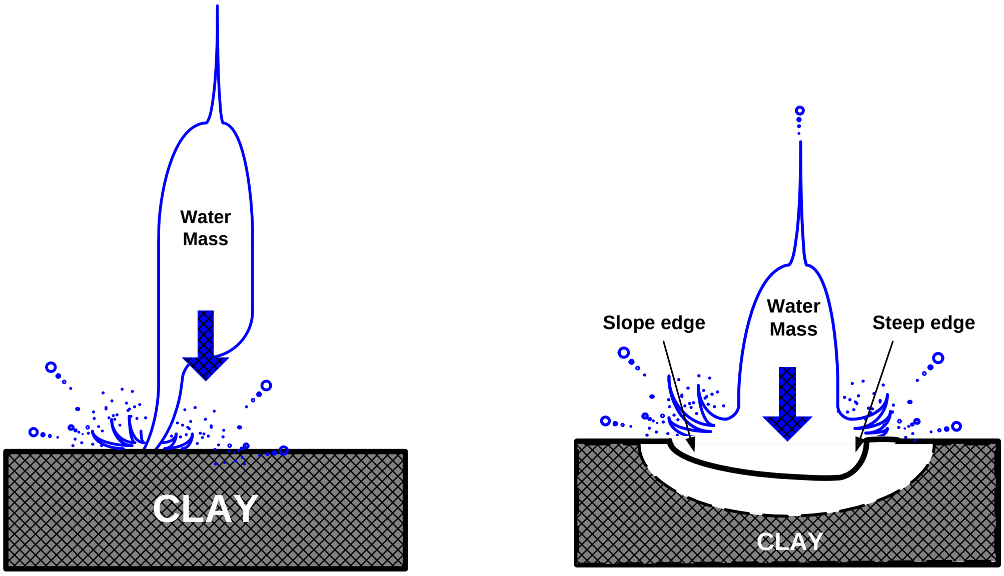



*Figure 5-21 An imperfect falling-water mass and its asymmetrically crater-like scour hole*


*Figure 5-22 Example of unsymmetrical scour hole*

The eroded clays are in the form of big block soils and small particles or aggregates. The largest big block soil can reach a dimension up to 10 cm long. These eroded soils were produced due to repeating impacts. Some aspects are contributed to the formation of small particles and big blocks of eroded soil:

– The direct impacts cause uplift reaction pressures around the impact area. This uplift reaction pressures form small vertical cracks around the impact area in all directions. The vertical cracks have much smaller size than the cracks in lateral directions. Larger cracks were developed laterally due to the same process but some impacts energy is able to be released laterally.

– The splashed water as the result of water mass expansion. This splash water creates very high velocities that produce high pressure on the side wall scour hole and accelerate the process of erosion.

– The sidewall effect. The use of glass and the limited space of soil sample width (10 cm) could be contributed to the formation of big block soil erosion. The lifted soil can be seen after releasing several impacts followed by removal of block of soils.

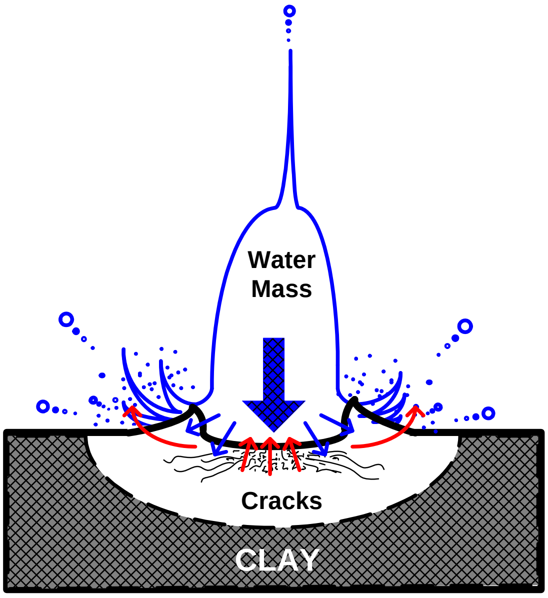
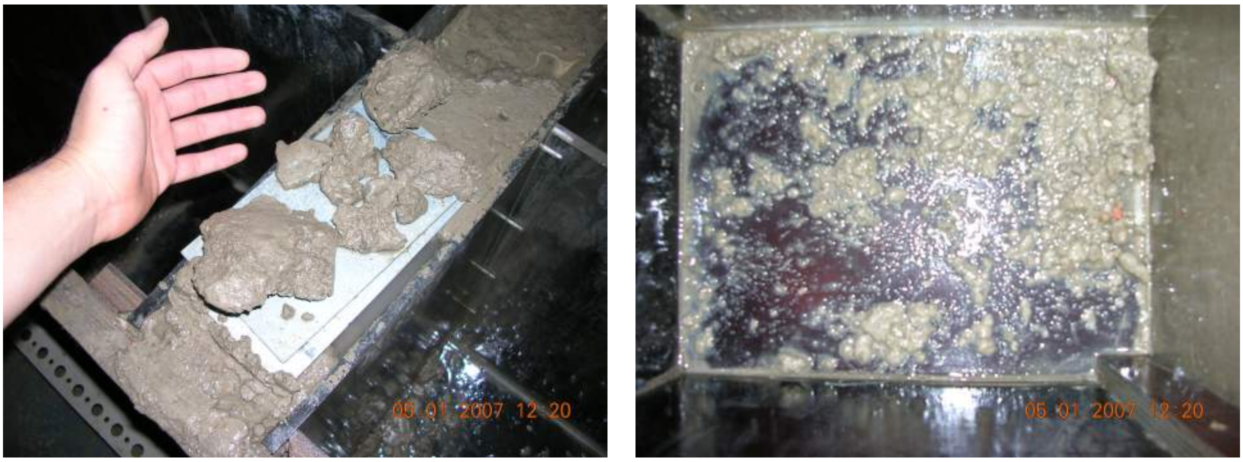



*Figure 5-23 Eroded soil mechanisms. Blue arrows represent direct pressures and red arrows are the uplift reaction pressures within the soil*


(a)


(b)

*Figure 5-24 (a) Splash water that can produce high water velocity pressure on the wall scour hole and accelerate the process of erosion (b) Lifted soil as a result internal reactions and sidewall effect*




(a) (b)

*Figure 5-25 (a) Eroded soil in the form of big blocks soil and (b) eroded soil in the form of particles and small aggregates*

From the above explanations, some factors influencing erosion on surface erosion due to impacts are identified as follows

1. Water content.
   Water content provides strength to the clay as well as its applicability in being worked

2. Soil Density.
   Loose-packed soil is less erosion resistant than the well-compacted soil

3. Aging time.
   Clay which has been remained for so long is more erosion resistant than the new moulded/constructed clay

4. Clay type.
   Type of clay which has large sand content is less erosion resistant than the clay with small sand content.

5. Magnitude of impacts.
   The bigger the impact the more vulnerable the clay layer towards erosion.

6. Number of impacts.
   Repetition of impacts weakens the strength of the soil gradually until total damage occurs after some time.

7. Water layer.
   Water layer damps some energy from the impacts. The erosions are less but more spreading due to the movement of water along the interface between clay surface and the water layer.

8. Homogeneity.
   Soil homogeneity play role in internal strength distribution of the clay. Homogenous clay has well-distributed strength. A not-homogenous clay has weak areas that can easily fail against the impacts.

## 5.3 Analysis on Failure Mechanism of Clay Cover Due to Breaking Wave Impacts (Flume Tests)

The mechanism of dike failure initiated by the breaking wave impacts can be described into several stages (**Figure 6.21a,b,c,d,e,f,g**) as follows:

**Stage 1 - Initiation**: The plunging waves break and hit the dike slope (below the MWL, or about 0.5H) with an impact that through time cause scour hole. Some overtopping is observed but does not cause serious damage to the inner slope of the dike. Run-up and run-down erode small grains of the clay cover surface.

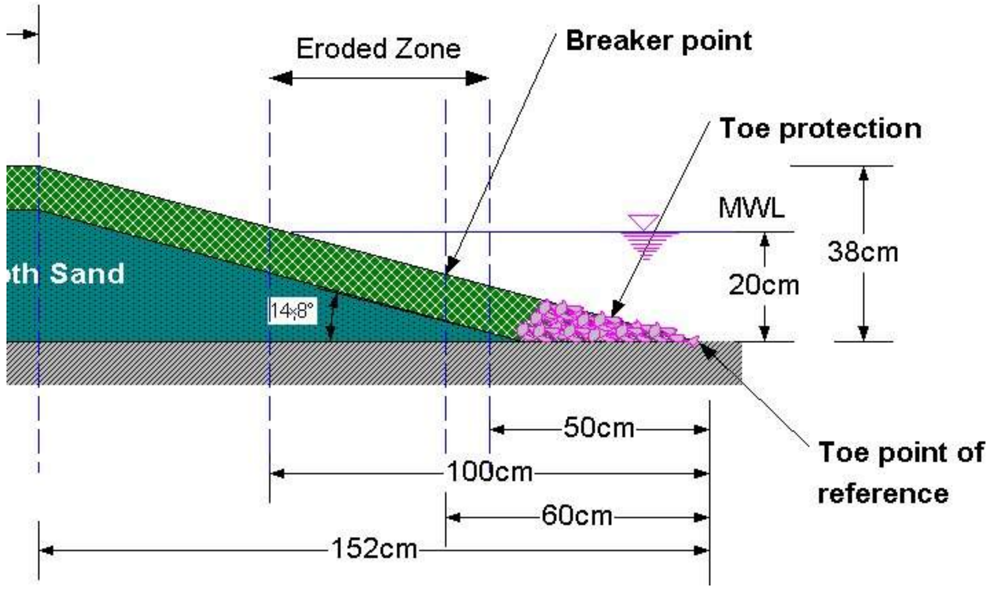



*Figure 5-26 Location of breaker point and eroded zone*

**Stage 2 - Scouring**: Once the scour hole is getting deeper, due to a developing steeper edge, run down velocity appears to be quite important by increasing the rate of erosion.

**Stage 3 - Failure**: When the scour hole reaches the whole depth of the clay cover, it breaks; seepage starts to intrude the dike core. This happens after about 12 hours of running model

**Stage 4 - Undermining**: Undermining of the sand core accelerates the erosion causing the collapse of clay cover in pieces of block. These processes continue until reaching the crest. At the same time, seepage is traveling fast and reaches its complete journey when large part of the clay cover is wide open. Once seepage completed after about 2 hours from clay cover failure, piping occurs at the bottom of the inner slope. The pieces of collapsing clay cover act as natural breakwater that blocking the incoming wave attacks until they finally turn into smaller fraction and washed away. At this stage, there is no overtopping anymore.

**Stage 5 – Slope development**: When large parts of the outer slope suffer severe damage, the slope in front of it starts to develop by itself. The abundant amount of sediments from clay particles and sands stretches from the location where the clay cover started to break up to the offshore. A bunch of sand bar is also developing at this location. The present of sand bar and the shallowing process of the foreshore change the geometry of the dike slope and the wave characteristics at the same time. There are no plunging waves anymore. Surging and collapsing waves are observed. The dike slope is forming a stable profile (s-profile) that the waves can no longer influence further changes (erosion).

**Stage 6 – Flow slide**: When all parts of the sand core become saturated, the sand starts to liquefy and the pore pressure inside the core increases pushing the clay cover of the inner side in all direction. Since the bottom of the dike receives the largest gradient of pressure, damage starts from here. The bottom of the inner slope is moving laterally followed by the movement of the clay cover on top of it. These create large and parallel cracks along the surface of the inner slope.

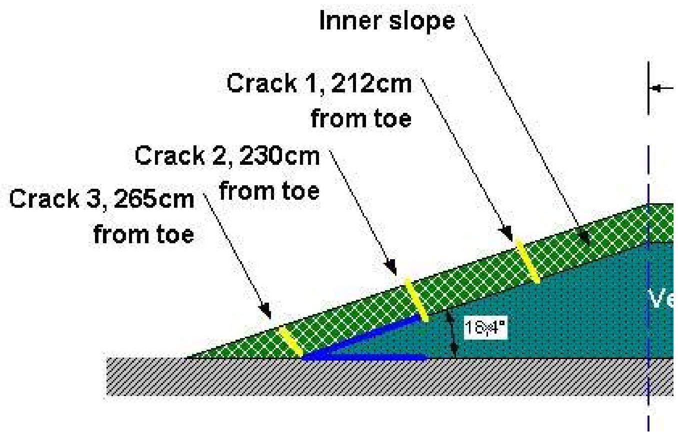



*Figure 5-27 Cracks formation due to flow slide at the inner slope*

**Stage 7 – Stable slope**: The new slope of the severely damage dike reaches its stable condition. Since there are no other factors that can cause further erosion/damage, the dike remain in place in a very fragile condition. The clay cover of the outer slope has gone. The crest level is no longer high enough. Water flows due to on going piping and from the cracks of the inner slope created by flow slide.

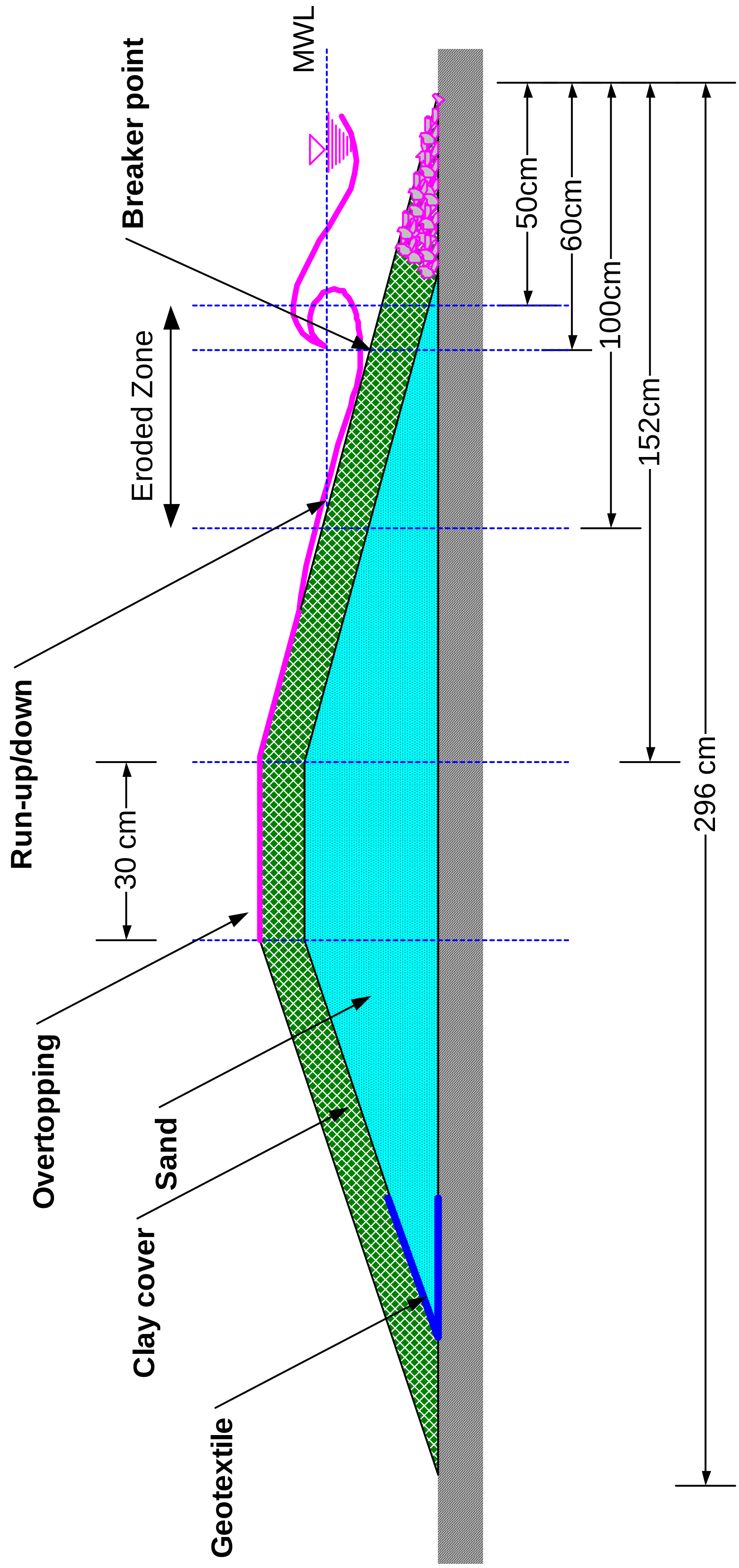



***Figure 5.28a**. Stage 1 - Initiation*

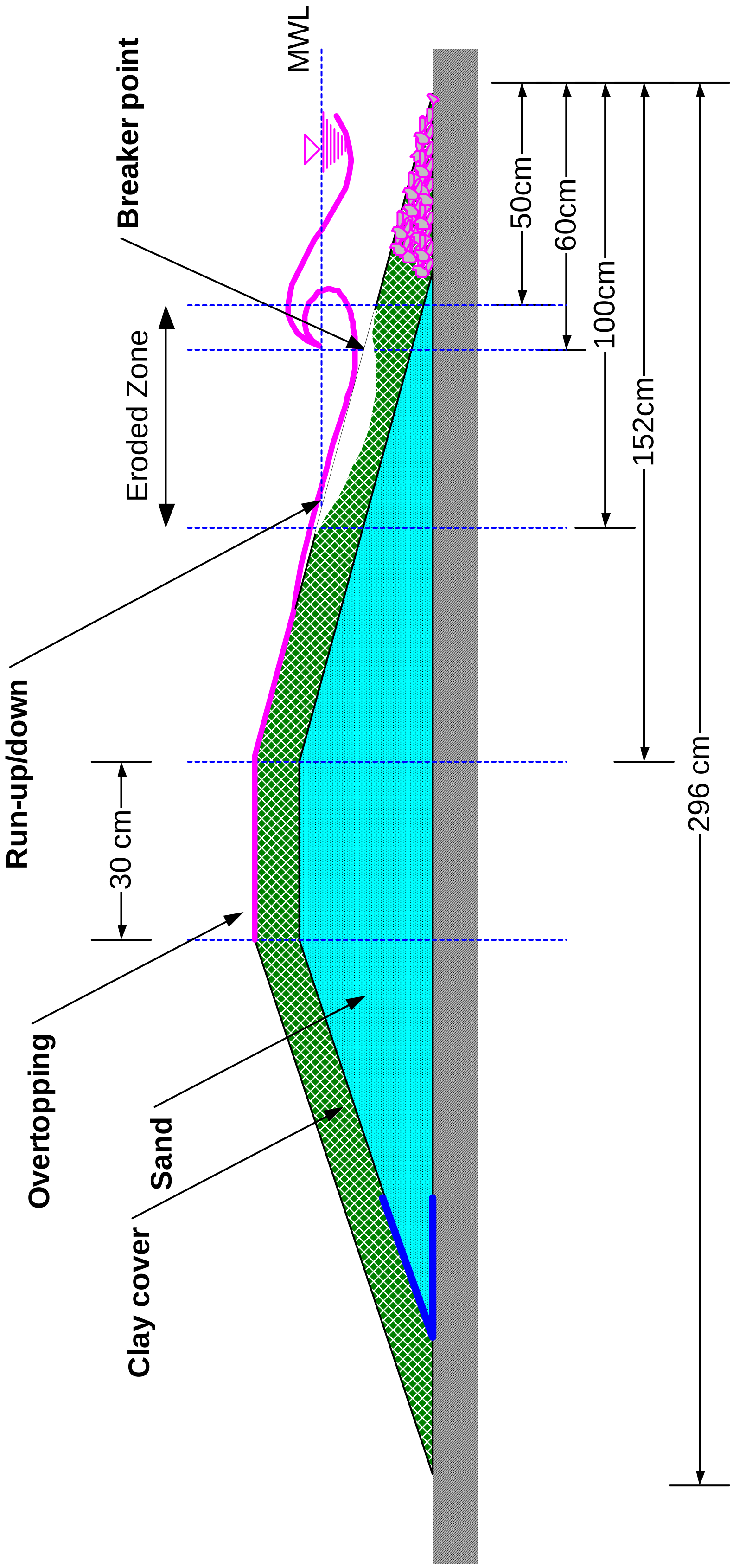



***Figure 5.28b**. Stage 2 - Scouring*

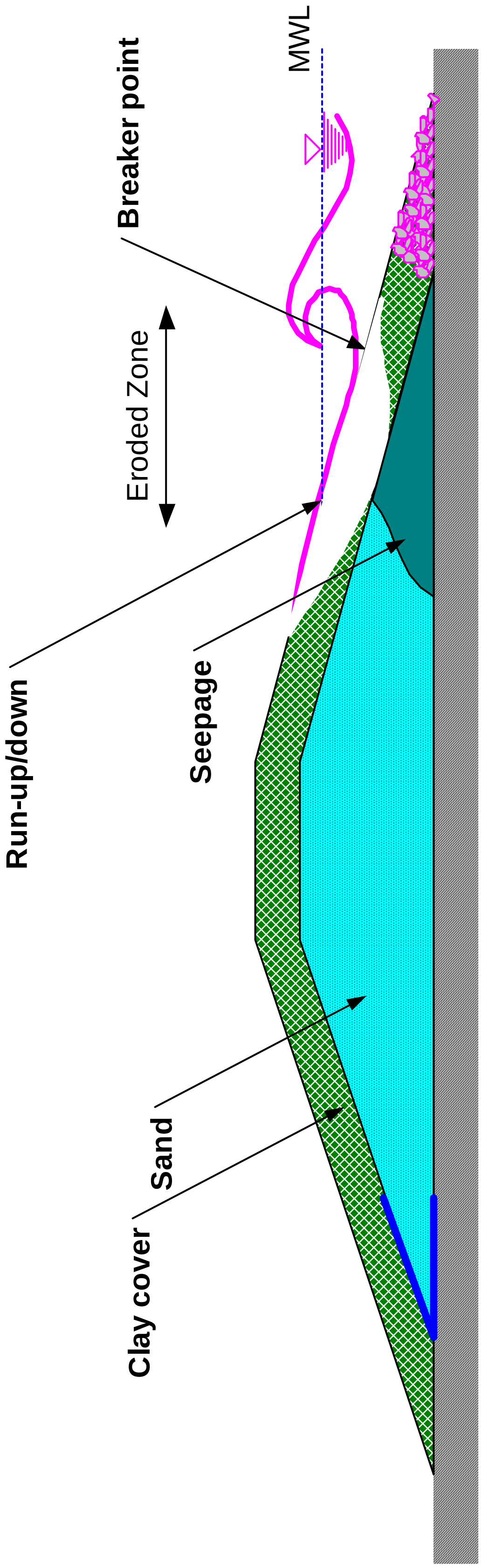



***Figure 5.28c**. Stage 3 - Clay cover failure*

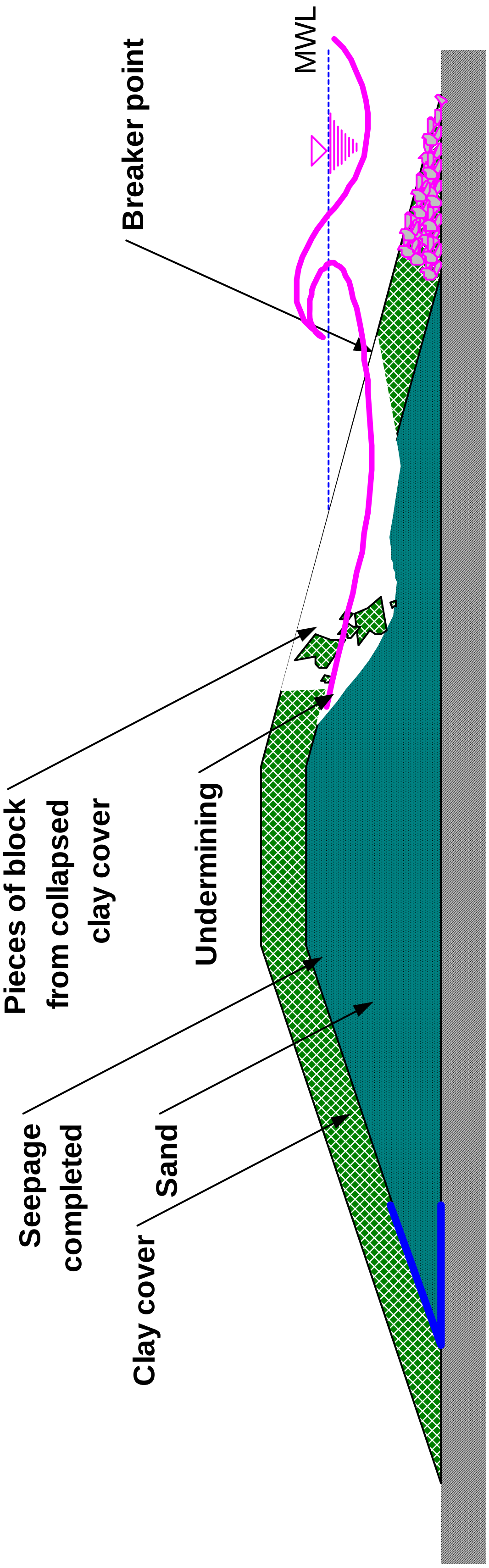



***Figure 5.28d**. Stage 4 - Sand undermining*

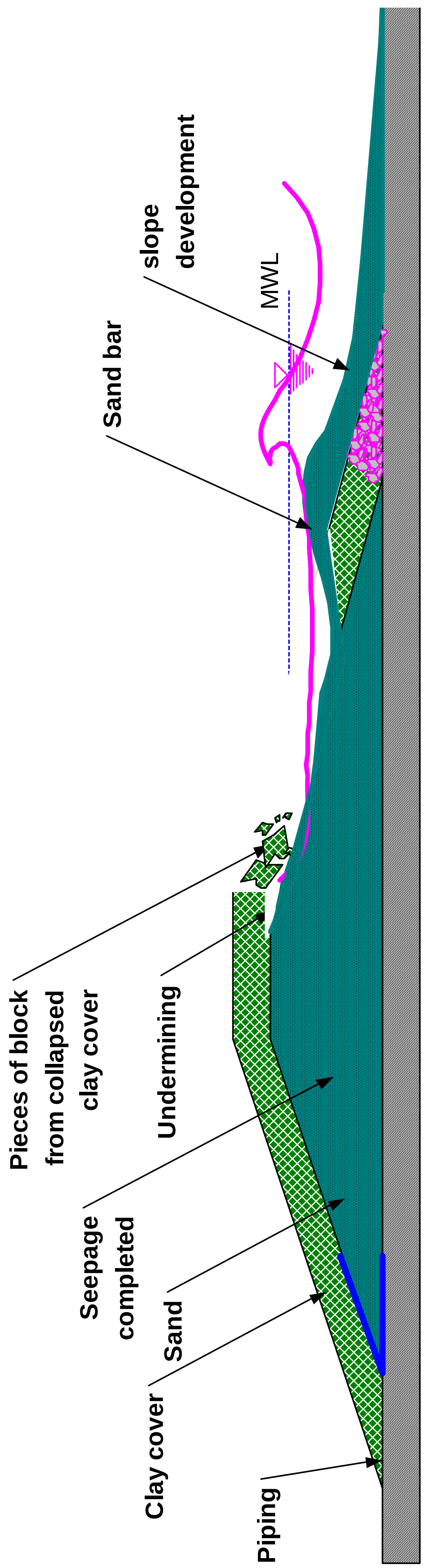



***Figure 5.28e**. Stage 5 - Front slope development*

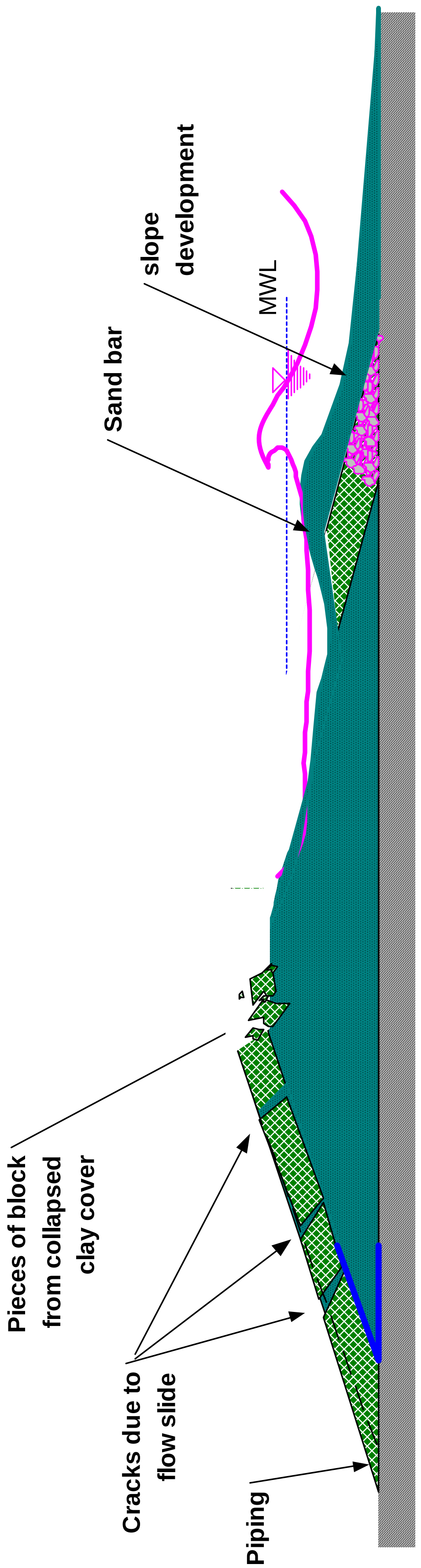



***Figure 5.28f**. Stage 6 - Flow slide at the inner slope*

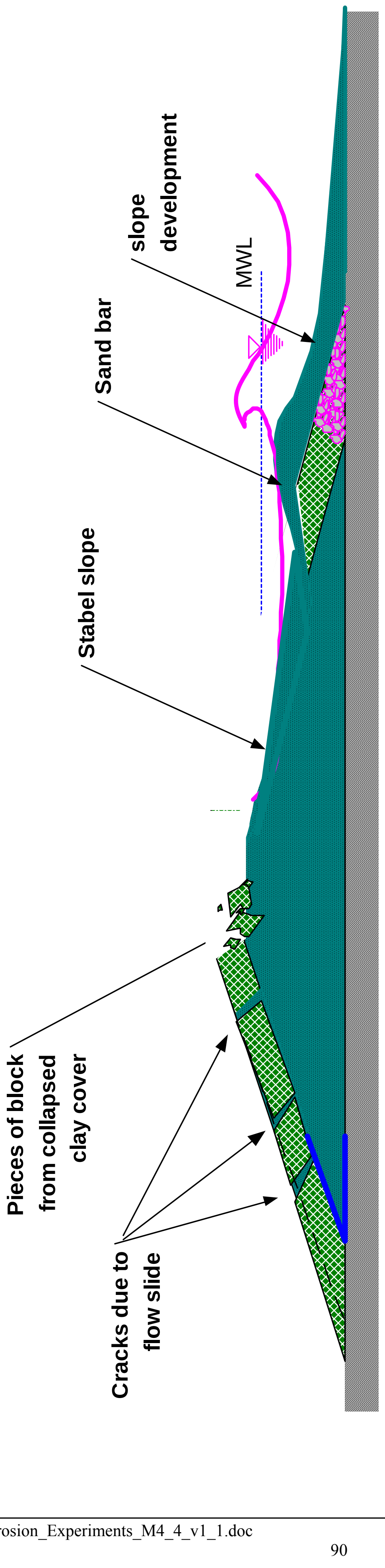



***Figure 5.28g**. Stage 7 - Stable condition*

# 6. CONCLUSIONS AND RECOMMENDATIONS

## 6.1 Erosion on water-filled cracks due to impacts

***Conclusions***

The presence of cracks due to both physical processes and biological activities weakens the stability of the clay cover of sea dikes which is subjected to the wave impacts. The experiments in this research are intended to study and analyse the erosion of compacted clay with significant water-filled cracks due to breaking wave impacts.

Erosion on clay cover with water-filled cracks has been investigated in laboratorial experiments. The crack itself was made artificially. The maximum generated impact pressure of 24.75 kPa is equivalent to the impacts pressure generated by wave height of 1.2 m. There are 2 types of clay, good and moderate clay, which are tested in different water content. The water content is limited in the range of $I_c$ value of 0.50 – 1.00 or physically, the clay is in soft – stiff condition. From the results, some aspects reveal to be very important in erosion-resistant behaviors of the water-filled cracks:

- Types of clay
- Clay homogeneity
- Magnitude of breaking wave impacts
- Water content
- Compaction
- Crack dimension

The erosion mechanism on water-filled cracks has been developed. It is divided into 2 different mechanisms based on water content of the clay (**Section 5.1.5**).

1. Clay with recommended water content (Ic>0.75) is more erosion resistant indicated by minor damage at the entrance of the water-filled cracks after being hit by the maximum impacts. The shear strength of the soil appears to be far larger than the impacts ($2c>p_{max}$) as indicated by Führböter (1966). The eroded soils are in the form of small aggregates and particles.

2. Clay with high water content (0.50<Ic<0.75) behave differently as it hits by the impacts though the soil strength is below the maximum impact pressure ($2c<p_{max}$). The samples are still strong enough to withstand the impacts. The Führböter approach clearly underestimates the strength of the soil which appears to be not only the shear strength (cohesion) but also other factors such as weight of the soil and pore pressures. The eroded soil in the form of block of soil are in agreement with Führböter (1966)

In this experiment, a new model for erosion on water-filled cracks can not be built due to limited experimental data and difficulties in model-set up. The impact magnitude from the machine is not big enough to cause failure on the samples with Ic>0.50. The change in geometry of the water-filled cracks during releasing the impacts and the presence of eroded soil that filling up the crack have changes the whole characteristics of the water-filled cracks.

***Recommendations***

Based on experimental results, some aspects should be considered for future research development (see details in **Appendix F-Recommendation**):

1. Crack Dimension
Crack dimension determines the magnitude of propagated pressure inside the water-filled cracks. Data are available for the crack width of 4mm and 6mm (see Pachnio 2005). Therefore, the crack dimension for next experiment is favourable to have crack width of 4mm or 6mm.

2. The Process of making cracks
The presence of weak lines/points at the crack walls should be avoided because they can influence erosion physical behaviours. To do that, model set-up for the samples should be modified.

3. In-situ compaction measurement
Compaction is one of key factors in erosion resistant of clay. A standard in-situ density measurement is needed in order to have global value in practices. A standard sand cone replacement method is the most suitable for this kind of experiment. It can be worked for small samples in laboratory.

4. Larger impact pressure
The maximum generated impacts are possible to be increased. The model set-up should be modified in such ways, so that the fall height of the impact machine can reach its optimum that can cause failure to the sample.

## 6.2 Surface erosion due to impacts

***Conclusions***
Sea dikes covered with clay as revetments are vulnerable to erosion due to breaking wave impacts. During storms, for example, the frequencies and the magnitudes of breaking wave impacts can be very dangerous to the stability of the dikes. Investigations on the reactions of compacted clay of sea dikes subjected by breaking wave impacts are needed to understand the erosion mechanisms and to improve design guidance of sea-side slope of sea dikes. The experiments on surface erosion of compacted clay due to impacts are intended to meet that objective. Two types of clay samples were taken from real dikes in Lower Saxony, Germany representing clay with good and moderate erosion resistances. A series of experiments was carried out using the falling water machine to generate the impacts. The compacted samples were subjected by certain number of impacts until significant erosion is observed. From experimental results and the analysis, some factors influencing erosion on surface erosion due to impacts are identified as follows:

- Types of clay
- Clay homogeneity
- Water content.
- Soil Density
- Magnitude of impacts
- Number of impacts
- Water layers

Detachment coefficient ($k_d$) can be used for determination of which recommended water content range should be applied for dike construction on the sea side. From the experiments, it concludes that the recommended water content which are more erosion resistant against the impacts are the clay with water content minimum Ic=0.75 until maximum the Plastic Limit (PL). This range of application is based on the easiness in workability and erosion resistant behaviours against the impacts. Furthermore, the detachment coefficient ($k_d$) values have been developed as a function of water content (**Table 5.3**).

A model for the erosion mechanism due to impacts for this experiment has been developed. The mechanism describes important aspects (as mentioned above) that should be taken into account in designing clay cover layer for sea dike (**Section 5.2.3**).

***Recommendations***

New approach/procedure of model set up in order to have better results, easier in operation, and more controllable is needed. The use of proctor mould could be very useful for surface erosion on compacted clay due to impacts (see details in **Appendix F-Recommendation**). By using proctor mould for placing the samples, the following advantages will follow:

1. Compaction is known and accurate and as well as the water content
2. Higher impacts are possible by lowering down the sample up to the bottom of the wooden box. Maximum fall height up to 200cm can be achieved.
3. Measurement of erosion can be done easily by measuring the maximum depth of the scour hole.
4. Effect of tongue from the opening of the valve can be avoided
5. More efficient

## 6.3 Failure Mechanism of clay covers (Flume tests)

***Conclusions***

The failure mechanisms of clay layers of sea dikes subjected by breaking wave impacts have not been well understood. Therefore, these qualitative experiments were carried out to see whether these failure mechanisms can cause severe damage to the dikes or even lead to breaching. Two models of sea dikes subjected by breaking wave impacts generated by the flume were tested.

The failure mechanism of clay cover of a dike due to breaking wave impacts for this experiment has been developed. It consists of 7 stages (**Section 5.3**):

- Stage 1 – Initiation of erosion due to impact and run-up/down
- Stage 2 – Scouring in the area of impacts
- Stage 3 - Failure of clay layer and start seepage
- Stage 4 – Undermining of sand core causing more damage to the clay layer
- Stage 5 – Slope development changes
- Stage 6 – Flow slide due to change in water content of the sand core.
- Stage 7 – Creation of stable slope

The dike did not breach, but it suffers from severe damage. The dike is loosing its functions to protect the area from flood. Though it is still standing, the clay cover of the outer slope has gone, the crest level is no longer high enough, and water flows from piping and cracks at the inner slope can not be tolerated and can cause flooding.

***Recommendations***

Total breaching due to a single factor of breaking wave impacts was not happening in these experiments but it seems possible. The dike suffers severe damage caused by the repetition of impacts. In practice, this situation can be much more dangerous because other factors such as rising water level, strong wind, and longshore currents usually work together. Breaching in this condition is not impossible.

This qualitative experiment of course has a lot of drawbacks. Scaling effects are obvious during the experiments. The models are scaled down but (for example) the clay can not be scaled down. The clay properties are the same as the prototype. The waves generated by the flume are relatively too small to create hard impacts from plunging waves. The sediment from the eroded clay and sand are trapped inside the flume and causing thick sediment concentration in the water. All those problems are only can be solved by a comprehensive test. More investigations using the same method or others (such numerical models, or directly field investigations) are needed to have in depth behaviors of the dike failure initiated by the breaking wave impacts

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# A. Tests Summary for Water-filled crack experiments

| Test ID | Date | Clay type | Fall height [cm] | Impact (Pmax) [kPa] | The $n^{th}$ impacts | Total impacts [kPa] | $w_n$ [%] | Compaction [gr/ml] | Estimated cohesion ($c_u$) [kPa] | 2xc [kPa] | (Pmax-2c) [kPa] | Failure angel [deg] | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 15-Nov-06 | I | 162 | 24.75 | 1 | 24.75 | 37.5 | 1.278 | 80.32 | 160.64 | -135.89 | - | Well compacted, no damage |
| | 15-Nov-06 | I | 162 | 24.75 | 7 | 173.24 | 37.5 | | 80.32 | 160.64 | -135.89 | - | No damage, weak point observed |
| | 15-Nov-06 | I | 162 | 24.75 | 8 | 197.99 | 37.5 | | 80.32 | 160.64 | -135.89 | - | Soil around the weak poit is lifted |
| | 15-Nov-06 | I | 162 | 24.75 | 9 | 222.73 | 37.5 | | 80.32 | 160.64 | -135.89 | - | Damage is started aroung the weak point |
| | 15-Nov-06 | I | 162 | 24.75 | 10 | 247.48 | 37.5 | | 80.32 | 160.64 | -135.89 | - | All soil bodies around the weak point are removed |
| 2 | 16-Nov-06 | I | 162 | 24.75 | 1 | 24.75 | 44.9 | 1.162 | 33.05 | 66.10 | -41.35 | - | Lifted soil |
| | 16-Nov-06 | I | 162 | 24.75 | 4 | 98.99 | 44.9 | | 33.05 | 66.10 | -41.35 | - | Lifted soil is obvious, crack is widened |
| | 16-Nov-06 | I | 162 | 24.75 | 5 | 123.74 | 44.9 | | 33.05 | 66.10 | -41.35 | - | Block soil removed |
| 3 | 17-Nov-06 | I | 163 | 24.82 | 1 | 24.82 | 38.5 | - | 71.24 | 142.47 | -117.65 | - | Less compacted, no damage |
| | 17-Nov-06 | I | 162 | 24.75 | 5 | 123.74 | 38.5 | | 71.24 | 142.47 | -117.72 | - | Minor damage |
| | 17-Nov-06 | I | 162 | 24.75 | 6 | 148.49 | 38.5 | | 71.24 | 142.47 | -117.72 | - | Side crack damage observed |
| 4 | 17-Nov-06 | I | 162 | 24.75 | 1 | 24.75 | 38.2 | - | 73.85 | 147.69 | -122.95 | - | Almost no compaction, side crack damage observed |
| | 17-Nov-06 | I | 162 | 24.75 | 2 | 49.50 | 38.2 | | 73.85 | 147.69 | -122.95 | - | Almost no compaction, severe damage observed |
| | | | | | | | | | | | | | Water flows through large pores |
| 5 | 23-Nov-06 | I | 162 | 24.75 | 1 | 24.75 | 45.2 | 1.19 | 31.88 | 63.76 | -39.01 | - | No damade, crack is widening |
| | 23-Nov-06 | I | 162 | 24.75 | 3 | 74.24 | 45.2 | | 31.88 | 63.76 | -39.01 | - | Side wall started to lift, crack is widening |
| | 23-Nov-06 | I | 162 | 24.75 | 11 | 272.23 | 45.2 | | 31.88 | 63.76 | -39.01 | 27.50 | Lifted soil at its maximum, crack is widening |
| | | | | | | | | | | | | | Crack observed on the side wall |
| 6 | 23-Nov-06 | I | 162 | 24.75 | 1 | 24.75 | 48.5 | 1.08 | 21.46 | 42.91 | -18.16 | - | No damade, crack is widening |
| | 23-Nov-06 | I | 162 | 24.75 | 3 | 74.245 | 48.5 | | 21.46 | 42.91 | -18.16 | - | Side wall started to lift, crack is widening |
| | 23-Nov-06 | I | 162 | 24.75 | 11 | 272.23 | 48.5 | | 21.46 | 42.91 | -18.16 | - | Lifted soil at its maximum, crack is widening |
| | 23-Nov-06 | I | 162 | 24.75 | 12 | 296.98 | 48.5 | | 21.46 | 42.91 | -18.16 | 14.00 | Block soil removed |
| 7 | 24-Nov-06 | I | 162 | 24.75 | 1 | 24.75 | 53.2 | 1.04 | 12.21 | 24.41 | 0.33 | 21.80 | Failure occures |
| | | | | | | | | | | | | | Clay is extremely wet |
| 53 | 29-Jan-07 | II | 162 | 24.75 | 1 | 24.75 | 23.2 | 1.45 | 27.04 | 54.08 | -29.33 | - | Well compacted, no damage |
| | 29-Jan-07 | II | 162 | 24.75 | 5 | 123.74 | 23.2 | | 27.04 | 54.08 | -29.33 | - | Minor damage |
| | 29-Jan-07 | II | 162 | 24.75 | 9 | 222.73 | 23.2 | | 27.04 | 54.08 | -29.33 | 11.90 | Slightly wall damage, 6 cm removed soil |
| 54 | 29-Jan-07 | II | 162 | 24.75 | 1 | 24.75 | 27.6 | 1.38 | 11.22 | 22.43 | 2.32 | - | Well compacted, no damage |
| 54 | 29-Jan-07 | II | 162 | 24.75 | 9 | 222.73 | 27.6 | | 11.22 | 22.43 | 2.32 | 33.60 | Lifted soil at its maximum, crack is widening |

| Test ID | Date | Clay type | Fall height [cm] | Impact (Pmax) [kPa] | The $n^{th}$ impacts | Total impacts [kPa] | $w_n$ [%] | Compaction [gr/ml] | Estimated cohesion ($c_u$) [kPa] | 2xc [kPa] | (Pmax-2c) [kPa] | Failure angel [deg] | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 55 | 29-Jan-07 | II | 162 | 24.75 | 1 | 24.75 | 21.7 | 1.38 | 36.50 | 73.00 | -48.26 | - | Well compacted, no damage |
| | 29-Jan-07 | II | 162 | 24.75 | 5 | 123.74 | 21.7 | | 36.50 | 73.00 | -48.26 | - | Minor side damage from side effect |
| | 29-Jan-07 | II | 162 | 24.75 | 9 | 222.73 | 21.7 | | 36.50 | 73.00 | -48.26 | - | Minor becomes significant damage |
| | | | | | | | | | | | | | Crack at 3cm depth observed, no failure |
| 56 | 30-Jan-07 | II | 162 | 24.75 | 1 | 24.75 | 28.1 | 1.4 | 10.15 | 20.30 | 4.45 | - | Well compacted, no damage |
| | 30-Jan-07 | II | 162 | 24.75 | 2 | 49.50 | 28.1 | | 10.15 | 20.30 | 4.45 | - | Lifted soil, gap widen |
| | 30-Jan-07 | II | 162 | 24.75 | 4 | 98.99 | 28.1 | | 10.15 | 20.30 | 4.45 | 14.28 | Failure occures, cracks at the wall side of lifted soil |
| 57 | 30-Jan-07 | II | 162 | 24.75 | 1 | 24.75 | 27.4 | 1.41 | 11.67 | 23.35 | 1.40 | - | Well compacted, no damage |
| | 30-Jan-07 | II | 162 | 24.75 | 2 | 49.50 | 27.4 | | 11.67 | 23.35 | 1.40 | - | Lifted soil, gap widen |
| | 30-Jan-07 | II | 162 | 24.75 | 6 | 148.49 | 27.4 | | 11.67 | 23.35 | 1.40 | 25.5 | Failure occures, block soil removed |

# Tests Summary on Surface Erosion of Compacted Clay for Good Clay

| Test ID | Date | Clay type | Fall height [cm] | Impacts [kPa] | Number of Impacts | Total impacts [kPa] | $w_n$ [%] | Water layer [cm] | Eroded soil [cm$^3$/300cm$^2$] | kd [cm$^3$/kPa] | Compaction [gr/cm$^3$] | Compaction relative [%] | impact duration | Total time per test | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 8 | 11/27/2006 | I | 165 | 24.75 | 200 | 4950 | 38.6 | 0 | 260 | 0.05253 | 1.2861 | | 1 h 49 m | 4 h 49 m | |
| 9 | 11/28/2006 | I | 165 | 24.74 | 200 | 4948 | 36.6 | 1 | 200 | 0.04042 | | | 1 h 33 m | 4 h 33 m | |
| 10 | 11/29/2006 | I | 165 | 24.74 | 200 | 4948 | 39.6 | 2.5 | 190 | 0.0384 | | | 1 h 55 m | 4 h 55 m | |
| 11 | 12/1/2006 | I | 125 | 21.53 | 200 | 4306 | 35.6 | 0 | 20 | 0.00464 | | | 1 h 28 m | 3 h 28 m | Pump is used |
| 12 | 12/4/2006 | I | 125 | 21.53 | 200 | 4306 | 37.1 | 0 | 90 | 0.0209 | | | 1 h 25 m | 3 h 25 m | |
| 23 a | 12/13/2006 | I | 165 | 24.75 | 200 | 4950 | 34.7 | 0 | 75 | 0.01515 | 1.3041 | 89.4 | 3 h 30 m | 6 h 30 m | Optimum wc. Using the same soil. Score hole measurement was done using water in order to maintain undisturbed condition of the sample |
| b | 12/13/2006 | I | 165 | 24.75 | 250 | 6187.5 | 34.7 | 0 | 180 | 0.02909 | 1.3041 | 89.4 | | | |
| c | 12/13/2006 | I | 165 | 24.75 | 300 | 7425 | 34.7 | 0 | 305 | 0.04108 | 1.3041 | 89.4 | | | |
| d | 12/13/2006 | I | 165 | 24.75 | 350 | 8662.5 | 34.7 | 0 | 480 | 0.05541 | 1.3041 | 89.4 | | | |
| e | 12/13/2006 | I | 165 | 24.75 | 400 | 9900 | 34.7 | 0 | 650 | 0.06566 | 1.3041 | 89.4 | | | |
| 24 a | 12/14/2006 | I | 165 | 24.75 | 200 | 4950 | 37.3 | 0 | 205 | 0.04141 | 1.2861 | 88.2 | 3 h 36 m | 6 h 36 m | |
| b | 12/14/2006 | I | 165 | 24.75 | 250 | 6187.5 | 37.3 | 0 | 310 | 0.0501 | 1.2861 | 88.2 | | | |
| c | 12/14/2006 | I | 165 | 24.75 | 300 | 7425 | 37.3 | 0 | 450 | 0.06061 | 1.2861 | 88.2 | | | |
| d | 12/14/2006 | I | 165 | 24.75 | 350 | 8662.5 | 37.3 | 0 | 500 | 0.05772 | 1.2861 | 88.2 | | | |
| e | 12/14/2006 | I | 165 | 24.75 | 400 | 9900 | 37.3 | 0 | 690 | 0.0697 | 1.2861 | 88.2 | | | |
| 25 a | 1/3/2007 | I | 165 | 24.75 | 50 | 1237.5 | 42.06 | 0 | 150 | 0.12121 | 1.2804 | 87.8 | 23m | 2h 33m | |
| b | 1/3/2007 | I | 165 | 24.75 | 75 | 1856.3 | 42.06 | 0 | 310 | 0.167 | 1.2804 | 87.8 | 11m | | |
| 26 a | 1/4/2007 | I | 165 | 24.75 | 25 | 618.75 | 46.02 | 0 | 170 | 0.27475 | 1.1879 | 81.5 | 12m | 2h 22m | |
| b | 1/4/2007 | I | 165 | 24.75 | 50 | 1237.5 | 46.02 | 0 | 520 | 0.4202 | 1.1879 | 81.5 | 10m | | |
| 27 a | 1/4/2007 | I | 165 | 24.75 | 100 | 2475 | 41.35 | 0 | 25 | 0.0101 | 1.2055 | 82.7 | 4h 43m | 8h 43m | |
| b | 1/4/2007 | I | 165 | 24.75 | 150 | 3712.5 | 41.35 | 0 | 45 | 0.01212 | 1.2055 | 82.7 | | | |
| c | 1/4/2007 | I | 165 | 24.75 | 200 | 4950 | 41.35 | 0 | 65 | 0.01313 | 1.2055 | 82.7 | | | |
| d | 1/4/2007 | I | 165 | 24.75 | 250 | 6187.5 | 41.35 | 0 | 135 | 0.02182 | 1.2055 | 82.7 | | | |
| e | 1/4/2007 | I | 165 | 24.75 | 300 | 7425 | 41.35 | 0 | 275 | 0.03704 | 1.2055 | 82.7 | | | |
| f | 1/4/2007 | I | 165 | 24.75 | 350 | 8662.5 | 41.35 | 0 | 415 | 0.04791 | 1.2055 | 82.7 | | | |
| g | 1/4/2007 | I | 165 | 24.75 | 400 | 9900 | 41.35 | 0 | 570 | 0.05758 | 1.2055 | 82.7 | | | |

| Test ID | Date | Clay type | Fall height [cm] | Impacts [kPa] | Number of Impacts | Total impacts [kPa] | $w_n$ [%] | Water layer [cm] | Eroded soil [$cm^3/300cm^2$] | kd [$cm^3/kPa$] | Compaction [$gr/cm^3$] | Compaction relative [%] | impact duration | Total time per test | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| h | 1/4/2007 | I | 165 | 24.75 | 450 | 11138 | 41.35 | 0 | 740 | 0.06644 | 1.2055 | 82.7 | | | |
| i | 1/4/2007 | I | 165 | 24.75 | 500 | 12375 | 41.35 | 0 | 790 | 0.06384 | 1.2055 | 82.7 | | | |
| 28 a | 1/5/2007 | I | 165 | 24.75 | 100 | 2475 | 42.7 | 0 | 25 | 0.0101 | 1.1641 | 79.8 | 4h 37m | 8h 37m | |
| b | 1/5/2007 | I | 165 | 24.75 | 150 | 3712.5 | 42.7 | 0 | 45 | 0.01212 | 1.1641 | 79.8 | | | |
| c | 1/5/2007 | I | 165 | 24.75 | 200 | 4950 | 42.7 | 0 | 70 | 0.01414 | 1.1641 | 79.8 | | | |
| d | 1/5/2007 | I | 165 | 24.75 | 250 | 6187.5 | 42.7 | 0 | 85 | 0.01374 | 1.1641 | 79.8 | | | |
| e | 1/5/2007 | I | 165 | 24.75 | 300 | 7425 | 42.7 | 0 | 155 | 0.02088 | 1.1641 | 79.8 | | | |
| f | 1/5/2007 | I | 165 | 24.75 | 350 | 8662.5 | 42.7 | 0 | 195 | 0.02251 | 1.1641 | 79.8 | | | |
| g | 1/5/2007 | I | 165 | 24.75 | 400 | 9900 | 42.7 | 0 | 285 | 0.02879 | 1.1641 | 79.8 | | | |
| h | 1/5/2007 | I | 165 | 24.75 | 450 | 11138 | 42.7 | 0 | 365 | 0.03277 | 1.1641 | 79.8 | | | |
| i | 1/5/2007 | I | 165 | 24.75 | 500 | 12375 | 42.7 | 0 | 860 | 0.06949 | 1.1641 | 79.8 | | | |
| 29 a | 1/6/2007 | I | 165 | 24.75 | 100 | 2475 | 43 | 0 | 175 | 0.07071 | 1.1603 | 79.6 | 4h 27m | 8h 27m | |
| b | 1/6/2007 | I | 165 | 24.75 | 150 | 3712.5 | 43 | 0 | 285 | 0.07677 | 1.1603 | 79.6 | | | |
| c | 1/6/2007 | I | 165 | 24.75 | 200 | 4950 | 43 | 0 | 655 | 0.13232 | 1.1603 | 79.6 | | | |
| d | 1/6/2007 | I | 165 | 24.75 | 250 | 6187.5 | 43 | 0 | 1000 | 0.16162 | 1.1603 | 79.6 | | | |
| e | 1/6/2007 | I | 165 | 24.75 | 300 | 7425 | 43 | 0 | 1260 | 0.1697 | 1.1603 | 79.6 | | | |
| f | 1/6/2007 | I | 165 | 24.75 | 350 | 8662.5 | 43 | 0 | 1520 | 0.17547 | 1.1603 | 79.6 | | | |
| g | 1/6/2007 | I | 165 | 24.75 | 400 | 9900 | 43 | 0 | 1790 | 0.18081 | 1.1603 | 79.6 | | | |
| 30 a | 1/8/2007 | I | 165 | 24.75 | 10 | 247.5 | 48.6 | 0 | 215 | 0.86869 | 1.0756 | 73.8 | 44m | 2h 44m | |
| b | 1/8/2007 | I | 165 | 24.75 | 20 | 495 | 48.6 | 0 | 415 | 0.83838 | 1.0756 | 73.8 | | | |
| c | 1/8/2007 | I | 165 | 24.75 | 30 | 742.5 | 48.6 | 0 | 720 | 0.9697 | 1.0756 | 73.8 | | | |
| d | 1/8/2007 | I | 165 | 24.75 | 40 | 990 | 48.6 | 0 | 980 | 0.9899 | 1.0756 | 73.8 | | | |
| e | 1/8/2007 | I | 165 | 24.75 | 50 | 1237.5 | 48.6 | 0 | 1340 | 1.08283 | 1.0756 | 73.8 | | | |
| 31 a | 1/8/2007 | I | 165 | 24.75 | 10 | 247.5 | 47.64 | 0 | 390 | 1.57576 | 1.0364 | 71.1 | 25m | 2h 25m | |
| b | 1/8/2007 | I | 165 | 24.75 | 20 | 495 | 47.64 | 0 | 1020 | 2.06061 | 1.0364 | 71.1 | | | |
| c | 1/8/2007 | I | 165 | 24.75 | 30 | 742.5 | 47.64 | 0 | 1655 | 2.22896 | 1.0364 | 71.1 | | | |
| 32 a | 1/9/2007 | I | 165 | 24.75 | 10 | 247.5 | 48.32 | 0 | 190 | 0.76768 | 1.1241 | 77.1 | 38m | 2h 38m | |
| b | 1/9/2007 | I | 165 | 24.75 | 20 | 495 | 48.32 | 0 | 480 | 0.9697 | 1.1241 | 77.1 | | | |
| c | 1/9/2007 | I | 165 | 24.75 | 30 | 742.5 | 48.32 | 0 | 780 | 1.05051 | 1.1241 | 77.1 | | | |
| e | 1/9/2007 | I | 165 | 24.75 | 50 | 1237.5 | 48.32 | 0 | 1650 | 1.33333 | 1.1241 | 77.1 | | | |

| Test ID | Date | Clay type | Fall height [cm] | Impacts [kPa] | Number of Impacts | Total impacts [kPa] | $w_n$ [%] | Water layer [cm] | Eroded soil [cm$^3$/300cm$^2$] | kd [cm$^3$/kPa] | Compaction [gr/cm$^3$] | Compaction relative [%] | impact duration | Total time per test | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 33 a | 1/9/2007 | I | 165 | 24.75 | 10 | 247.5 | 50.2 | 0 | 310 | 1.25253 | 1.0587 | 72.6 | 34m | 2h 34m | |
| b | 1/9/2007 | I | 165 | 24.75 | 20 | 495 | 50.2 | 0 | 660 | 1.33333 | 1.0587 | 72.6 | | | |
| c | 1/9/2007 | I | 165 | 24.75 | 30 | 742.5 | 50.2 | 0 | 1110 | 1.49495 | 1.0587 | 72.6 | | | |
| d | 1/9/2007 | I | 165 | 24.75 | 40 | 990 | 50.2 | 0 | 1480 | 1.49495 | 1.0587 | 72.6 | | | |
| e | 1/9/2007 | I | 165 | 24.75 | 50 | 1237.5 | 50.2 | 0 | 2100 | 1.69697 | 1.0587 | 72.6 | | | |
| 34 a | 1/9/2007 | I | 165 | 24.75 | 10 | 247.5 | 50.73 | 0 | 740 | 2.9899 | 1.1288 | 77.4 | 22m | 2h 22m | |
| b | 1/9/2007 | I | 165 | 24.75 | 20 | 495 | 50.73 | 0 | 1490 | 3.0101 | 1.1288 | 77.4 | | | |
| c | 1/9/2007 | I | 165 | 24.75 | 30 | 742.5 | 50.73 | 0 | 2050 | 2.76094 | 1.1288 | 77.4 | | | |
| 35 a | 1/10/2007 | I | 165 | 24.75 | 100 | 2475 | 35.17 | 0 | 0 | 0 | 1.2637 | 86.7 | 4h 45m | 8h 45m | Computer error occure |
| b | 1/10/2007 | I | 165 | 24.75 | 150 | 3712.5 | 35.17 | 0 | 50 | 0.01347 | 1.2637 | 86.7 | | | |
| c | 1/10/2007 | I | 165 | 24.75 | 200 | 4950 | 35.17 | 0 | 90 | 0.01818 | 1.2637 | 86.7 | | | |
| d | 1/10/2007 | I | 165 | 24.75 | 250 | 6187.5 | 35.17 | 0 | 145 | 0.02343 | 1.2637 | 86.7 | | | |
| e | 1/10/2007 | I | 165 | 24.75 | 300 | 7425 | 35.17 | 0 | 235 | 0.03165 | 1.2637 | 86.7 | | | |
| f | 1/10/2007 | I | 165 | 24.75 | 350 | 8662.5 | 35.17 | 0 | 300 | 0.03463 | 1.2637 | 86.7 | | | |
| g | 1/10/2007 | I | 165 | 24.75 | 400 | 9900 | 35.17 | 0 | 360 | 0.03636 | 1.2637 | 86.7 | | | |
| h | 1/10/2007 | I | 165 | 24.75 | 450 | 11138 | 35.17 | 0 | 420 | 0.03771 | 1.2637 | 86.7 | | | |
| i | 1/10/2007 | I | 165 | 24.75 | 500 | 12375 | 35.17 | 0 | 485 | 0.03919 | 1.2637 | 86.7 | | | |
| 36 a | 1/11/2007 | I | 165 | 24.75 | 50 | 1237.5 | 27.71 | 0 | 500 | 0.40404 | 1.2084 | 82.9 | 30m | 2h 30m | erosion started from side wall, assymetric |
| b | 1/11/2007 | I | 165 | 24.75 | 90 | 2227.5 | 27.71 | 0 | 2810 | 1.2615 | 1.2084 | 82.9 | | | |
| 37 a | 1/11/2007 | I | 165 | 24.75 | 50 | 1237.5 | 27 | 0 | 90 | 0.07273 | 1.2607 | 86.5 | 1h 45m | 3h 45m | erosion started from side wall, asymmetric |
| b | 1/11/2007 | I | 165 | 24.75 | 100 | 2475 | 27 | 0 | 180 | 0.07273 | 1.2607 | 86.5 | | | |
| c | 1/11/2007 | I | 165 | 24.75 | 150 | 3712.5 | 27 | 0 | 360 | 0.09697 | 1.2607 | 86.5 | | | |
| d | 1/11/2007 | I | 165 | 24.75 | 200 | 4950 | 27 | 0 | 1300 | 0.26263 | 1.2607 | 86.5 | | | |
| e | 1/11/2007 | I | 165 | 24.75 | 238 | 5890.5 | 27 | 0 | 4100 | 0.69604 | 1.2607 | 86.5 | | | |

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# Tests Summary on Surface Erosion of Compacted Clay for Moderate Clay

| Test ID | | Date | Clay Type | Fall height [cm] | Impacts [kPa] | Number of Impacts | $w_c$ [%] | Water layer [cm] | Eroded soil [$cm^3/300cm^2$] | kd ($cm^3$/kPa) | scour depth [cm] | Compaction [$gr/cm^3$] | Compaction relative (%) | impact duration | Total time per test | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 38 | a | 1/16/2007 | II | 162 | 24.75 | 50 | 21.14 | 0 | 250 | 0.20202 | 2.4 | 1.3806 | 84.0 | 1h 40m | 3h 40m | side erosion |
| | b | 1/16/2007 | II | 162 | 24.75 | 75 | 21.14 | 0 | 420 | 0.226263 | 3.7 | 1.3806 | 84.0 | | | asymmetric |
| | c | 1/16/2007 | II | 162 | 24.75 | 100 | 21.14 | 0 | 900 | 0.363636 | 6.3 | 1.3806 | 84.0 | | | maybe compaction problem |
| | d | 1/16/2007 | II | 162 | 24.75 | 125 | 21.14 | 0 | 1570 | 0.507475 | 9 | 1.3806 | 84.0 | | | |
| | e | 1/16/2007 | II | 162 | 24.75 | 150 | 21.14 | 0 | 2920 | 0.786532 | 10.5 | 1.3806 | 84.0 | | | |
| 39 | a | 1/16/2007 | II | 162 | 24.75 | 25 | 26.02 | 0 | 270 | 0.436364 | 3.3 | 1.3923 | 84.7 | 58m | 2h 58m | 5cm eroded soil observed |
| | b | 1/16/2007 | II | 162 | 24.75 | 50 | 26.02 | 0 | 640 | 0.517172 | 5 | 1.3923 | 84.7 | | | |
| | c | 1/16/2007 | II | 162 | 24.75 | 75 | 26.02 | 0 | 1020 | 0.549495 | 7.4 | 1.3923 | 84.7 | | | no cracks observed |
| | d | 1/16/2007 | II | 162 | 24.75 | 100 | 26.02 | 0 | 1660 | 0.670707 | 9.5 | 1.3923 | 84.7 | | | |
| 40 | a | 1/17/2007 | II | 162 | 24.75 | 25 | 24.2 | 0 | 40 | 0.064646 | 1 | 1.4316 | 87.1 | 2h | 4h | Lateral crack |
| | b | 1/17/2007 | II | 162 | 24.75 | 50 | 24.2 | 0 | 220 | 0.177778 | 2.5 | 1.4316 | 87.1 | | | |
| | c | 1/17/2007 | II | 162 | 24.75 | 75 | 24.2 | 0 | 370 | 0.199327 | 3.5 | 1.4316 | 87.1 | | | |
| | d | 1/17/2007 | II | 162 | 24.75 | 100 | 24.2 | 0 | 600 | 0.242424 | 4.6 | 1.4316 | 87.1 | | | |
| | e | 1/17/2007 | II | 162 | 24.75 | 125 | 24.2 | 0 | 770 | 0.248889 | 5.6 | 1.4316 | 87.1 | | | |
| | f | 1/17/2007 | II | 162 | 24.75 | 150 | 24.2 | 0 | 1140 | 0.307071 | 6.3 | 1.4316 | 87.1 | | | |
| | g | 1/17/2007 | II | 162 | 24.75 | 175 | 24.2 | 0 | 1370 | 0.316306 | 8.3 | 1.4316 | 87.1 | | | |
| | h | 1/17/2007 | II | 162 | 24.75 | 200 | 24.2 | 0 | 1680 | 0.339394 | 10.4 | 1.4316 | 87.1 | | | |
| 41 | a | 1/17/2007 | II | 162 | 24.75 | 25 | 22.01 | 0 | 100 | 0.161616 | 2 | 1.4248 | 86.7 | 1h 37m | 3h 37m | side erosion |
| | b | 1/17/2007 | II | 162 | 24.75 | 50 | 22.01 | 0 | 170 | 0.137374 | 2.8 | 1.4248 | 86.7 | | | comp error |
| | c | 1/17/2007 | II | 162 | 24.75 | 75 | 22.01 | 0 | 470 | 0.253199 | 3.5 | 1.4248 | 86.7 | | | valve error |
| | d | 1/17/2007 | II | 162 | 24.75 | 100 | 22.01 | 0 | 870 | 0.351515 | 5 | 1.4248 | 86.7 | | | leftside big erosion |
| | e | 1/17/2007 | II | 162 | 24.75 | 125 | 22.01 | 0 | 1780 | 0.575354 | 8 | 1.4248 | 86.7 | | | |
| | f | 1/17/2007 | II | 162 | 24.75 | 150 | 22.01 | 0 | 3800 | 1.023569 | 13 | 1.4248 | 86.7 | | | |
| 42 | a | 1/18/2007 | II | 162 | 24.75 | 25 | 20.56 | 0 | 240 | 0.387879 | 2.5 | 1.3006 | 79.2 | 38m | 2h 38m | lefttside erosion |
| | b | 1/18/2007 | II | 162 | 24.75 | 50 | 20.56 | 0 | 1440 | 1.163636 | 9.5 | 1.3006 | 79.2 | | | |
| | c | 1/18/2007 | II | 162 | 24.75 | 60 | 20.56 | 0 | 2530 | 1.703704 | 13.5 | 1.3006 | 79.2 | | | |

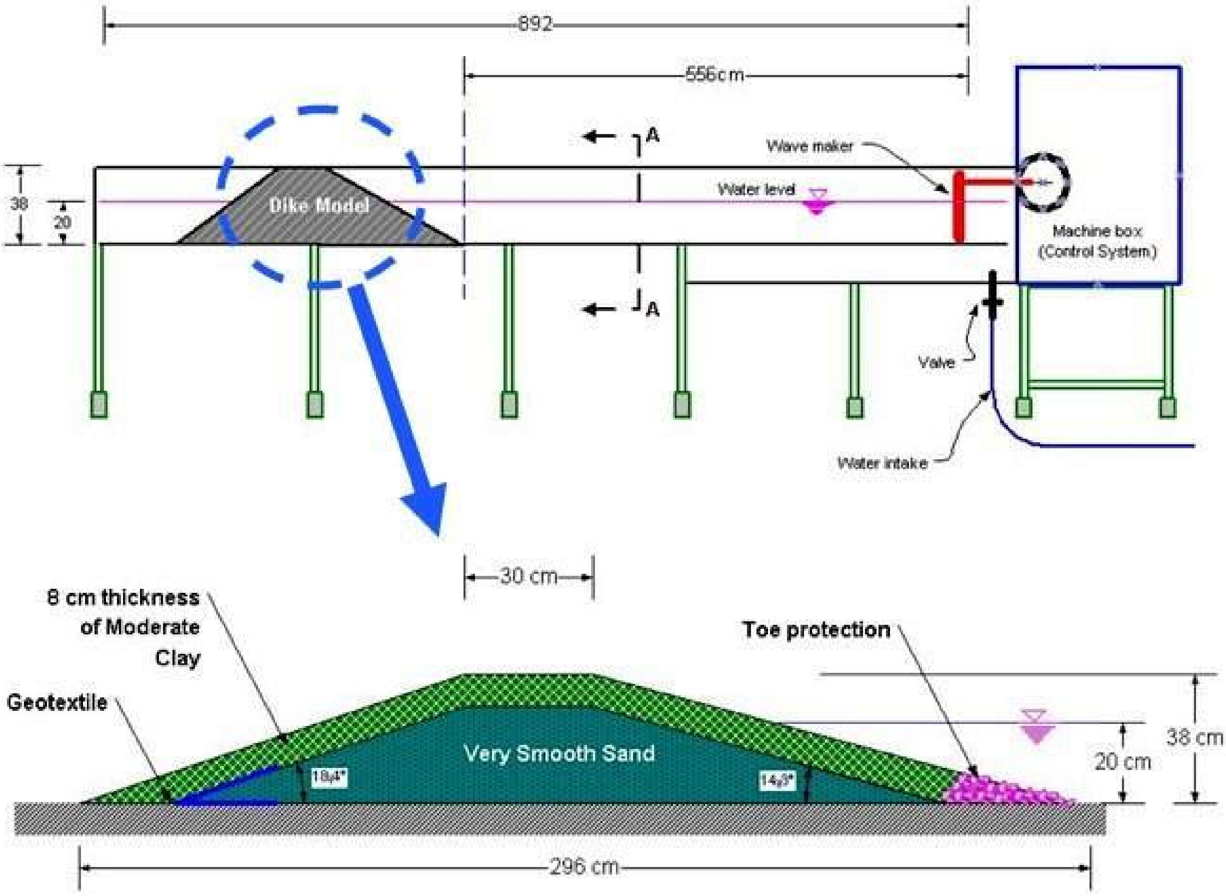
| Test ID | | Date | Clay Type | Fall height [cm] | Impacts [kPa] | Number of Impacts | $w_c$ [%] | Water layer [cm] | Eroded soil [$cm^3/300cm^2$] | kd ($cm^3$/kPa) | scour depth [cm] | Compaction [$gr/cm^3$] | Compaction relative (%) | impact duration | Total time per test | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 43 | a | 1/18/2007 | II | 162 | 24.75 | 10 | 27.59 | 0 | 130 | 0.525253 | 1.5 | 1.4284 | 86.9 | 43m | 2h 43m | lifted |
| | b | 1/18/2007 | II | 162 | 24.75 | 20 | 27.59 | 0 | 360 | 0.727273 | 3 | 1.4284 | 86.9 | | | cracks |
| | c | 1/18/2007 | II | 162 | 24.75 | 30 | 27.59 | 0 | 850 | 1.144781 | 5.5 | 1.4284 | 86.9 | | | tongue |
| | d | 1/18/2007 | II | 162 | 24.75 | 40 | 27.59 | 0 | 1490 | 1.505051 | 7.7 | 1.4284 | 86.9 | | | 10cm block soil |
| | e | 1/18/2007 | II | 162 | 24.75 | 50 | 27.59 | 0 | 2370 | 1.915152 | 10.5 | 1.4284 | 86.9 | | | |
| 44 | a | 1/18/2007 | II | 162 | 24.75 | 10 | 26.31 | 0 | 40 | 0.161616 | 1.5 | 1.4745 | 89.7 | 1h 5m | 3h 5m | lateral cracks |
| | b | 1/18/2007 | II | 162 | 24.75 | 20 | 26.31 | 0 | 90 | 0.181818 | 1.8 | 1.4745 | 89.7 | | | 6cm eroded soil |
| | c | 1/18/2007 | II | 162 | 24.75 | 30 | 26.31 | 0 | 230 | 0.309764 | 3.5 | 1.4745 | 89.7 | | | |
| | d | 1/18/2007 | II | 162 | 24.75 | 40 | 26.31 | 0 | 445 | 0.449495 | 4.1 | 1.4745 | 89.7 | | | |
| | e | 1/18/2007 | II | 162 | 24.75 | 50 | 26.31 | 0 | 680 | 0.549495 | 5.7 | 1.4745 | 89.7 | | | |
| | f | 1/18/2007 | II | 162 | 24.75 | 75 | 26.31 | 0 | 1290 | 0.694949 | 8.8 | 1.4745 | 89.7 | | | |
| | g | 1/18/2007 | II | 162 | 24.75 | 100 | 26.31 | 0 | 2020 | 0.816162 | 11.5 | 1.4745 | 89.7 | | | |
| 45 | a | 1/19/2007 | II | 162 | 24.75 | 20 | 19.5 | 0 | 410 | 0.828283 | 4 | 1.219 | 74.2 | 25m | 2h 25m | left erosion |
| | b | 1/19/2007 | II | 162 | 24.75 | 30 | 19.5 | 0 | 960 | 1.292929 | 7.5 | 1.219 | 74.2 | | | |
| | c | 1/19/2007 | II | 162 | 24.75 | 40 | 19.5 | 0 | 1970 | 1.989899 | 11.3 | 1.219 | 74.2 | | | |
| 46 | a | 1/19/2007 | II | 162 | 24.75 | 25 | 24.07 | 0 | 40 | 0.064646 | 0.3 | 1.512 | 92.0 | 1h 48m | 3h 48m | |
| | b | 1/19/2007 | II | 162 | 24.75 | 50 | 24.07 | 0 | 70 | 0.056566 | 1.2 | 1.512 | 92.0 | | | |
| | c | 1/19/2007 | II | 162 | 24.75 | 75 | 24.07 | 0 | 195 | 0.105051 | 2 | 1.512 | 92.0 | | | |
| | d | 1/19/2007 | II | 162 | 24.75 | 100 | 24.07 | 0 | 325 | 0.131313 | 2.7 | 1.512 | 92.0 | | | |
| | e | 1/19/2007 | II | 162 | 24.75 | 125 | 24.07 | 0 | 480 | 0.155152 | 3 | 1.512 | 92.0 | | | |
| | f | 1/19/2007 | II | 162 | 24.75 | 150 | 24.07 | 0 | 560 | 0.150842 | 4 | 1.512 | 92.0 | | | |
| | g | 1/19/2007 | II | 162 | 24.75 | 200 | 24.07 | 0 | 970 | 0.19596 | 5.7 | 1.512 | 92.0 | | | |
| 47 | a | 1/20/2007 | II | 162 | 24.75 | 25 | 24.14 | 0 | 25 | 0.040404 | 0.5 | 1.466 | 89.2 | 1h 38m | 3h 38m | |
| | b | 1/20/2007 | II | 162 | 24.75 | 50 | 24.14 | 0 | 55 | 0.044444 | 1.1 | 1.466 | 89.2 | | | |
| | c | 1/20/2007 | II | 162 | 24.75 | 100 | 24.14 | 0 | 220 | 0.088889 | 1.9 | 1.466 | 89.2 | | | |
| | d | 1/20/2007 | II | 162 | 24.75 | 150 | 24.14 | 0 | 385 | 0.103704 | 2.7 | 1.466 | 89.2 | | | |
| | e | 1/20/2007 | II | 162 | 24.75 | 200 | 24.14 | 0 | 610 | 0.123232 | 4 | 1.466 | 89.2 | | | |

| Test ID | | Date | Clay Type | Fall height [cm] | Impacts [kPa] | Number of Impacts | $w_c$ [%] | Water layer [cm] | Eroded soil [cm$^3$/300cm$^2$] | kd (cm$^3$/kPa) | scour depth [cm] | Compaction [gr/cm$^3$] | Compaction relative (%) | impact duration | Total time per test | Notes |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 48 | a | 1/24/2007 | II | 162 | 24.75 | 50 | 24.85 | 0 | 50 | 0.040404 | 0.5 | 1.417 | 86.2 | 1h 37m | 3h 37m | Hit by the impacts after 4 days |
| | b | 1/24/2007 | II | 162 | 24.75 | 100 | 24.85 | 0 | 95 | 0.038384 | 1.8 | 1.417 | 86.2 | | | |
| | c | 1/24/2007 | II | 162 | 24.75 | 150 | 24.85 | 0 | 185 | 0.049832 | 2.5 | 1.417 | 86.2 | | | |
| | d | 1/24/2007 | II | 162 | 24.75 | 200 | 24.85 | 0 | 310 | 0.062626 | 3.8 | 1.417 | 86.2 | | | |
| 49 | a | 1/26/2007 | II | 162 | 24.75 | 10 | 16.75 | 0 | 180 | 0.727273 | 2.4 | 1.354 | 82.4 | 44m | 2h 44m | side erosion |
| | b | 1/26/2007 | II | 162 | 24.75 | 20 | 16.75 | 0 | 420 | 0.848485 | 3.3 | 1.354 | 82.4 | | | side erosion |
| | c | 1/26/2007 | II | 162 | 24.75 | 30 | 16.75 | 0 | 835 | 1.124579 | 4.5 | 1.354 | 82.4 | | | side erosion |
| | d | 1/26/2007 | II | 162 | 24.75 | 40 | 16.75 | 0 | 1420 | 1.434343 | 6.1 | 1.354 | 82.4 | | | |
| | e | 1/26/2007 | II | 162 | 24.75 | 50 | 16.75 | 0 | 2300 | 1.858586 | 9.8 | 1.354 | 82.4 | | | leftside erosion |
| 50 | a | 1/26/2007 | II | 162 | 24.75 | 10 | 29.27 | 0 | 270 | 1.090909 | 2.4 | 1.370 | 83.4 | 35m | 2h 35m | lifted |
| | b | 1/26/2007 | II | 162 | 24.75 | 20 | 29.27 | 0 | 560 | 1.131313 | 4.1 | 1.370 | 83.4 | | | 10cm block soil |
| | c | 1/26/2007 | II | 162 | 24.75 | 30 | 29.27 | 0 | 1850 | 2.491582 | 7.7 | 1.370 | 83.4 | | | removed |
| | d | 1/26/2007 | II | 162 | 24.75 | 40 | 29.27 | 0 | 2540 | 2.565657 | 9.3 | 1.370 | 83.4 | | | |
| 51 | a | 1/27/2007 | II | 162 | 24.75 | 10 | 28.48 | 0 | 405 | 1.636364 | 3.3 | 1.404 | 85.4 | 30m | 2h 30m | cracks observed |
| | b | 1/27/2007 | II | 162 | 24.75 | 20 | 28.48 | 0 | 1015 | 2.050505 | 6.5 | 1.404 | 85.4 | | | lifted |
| | c | 1/27/2007 | II | 162 | 24.75 | 30 | 28.48 | 0 | 1725 | 2.323232 | 9.5 | 1.404 | 85.4 | | | 4-7cm block soils removed |
| | d | 1/27/2007 | II | 162 | 24.75 | 40 | 28.48 | 0 | 2320 | 2.343434 | 11.5 | 1.404 | 85.4 | | | |
| 52 | a | 1/29/2007 | II | 162 | 24.75 | 50 | 23.8 | 0 | 110 | 0.088889 | 1.2 | 1.481 | 90.1 | 1h 38m | 3h 38m | |
| | b | 1/29/2007 | II | 162 | 24.75 | 100 | 23.8 | 0 | 305 | 0.123232 | 2.7 | 1.481 | 90.1 | | | |
| | c | 1/29/2007 | II | 162 | 24.75 | 150 | 23.8 | 0 | 590 | 0.158923 | 4.1 | 1.481 | 90.1 | | | |
| | d | 1/29/2007 | II | 162 | 24.75 | 200 | 23.8 | 0 | 910 | 0.183838 | 5 | 1.481 | 90.1 | | | |

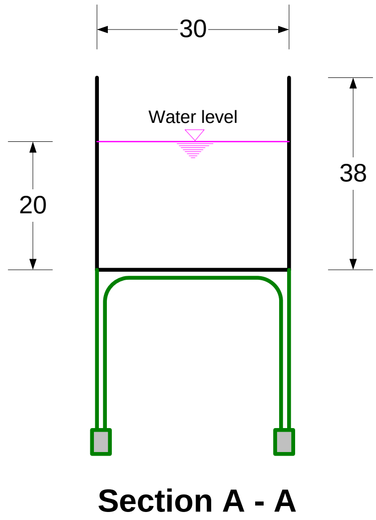
# B. Qualitative Experiment on Breaching Mechanism Initiated by Wave Impacts of Sea Dike with Clay Cover

## *a. (Experiment with Flume)*

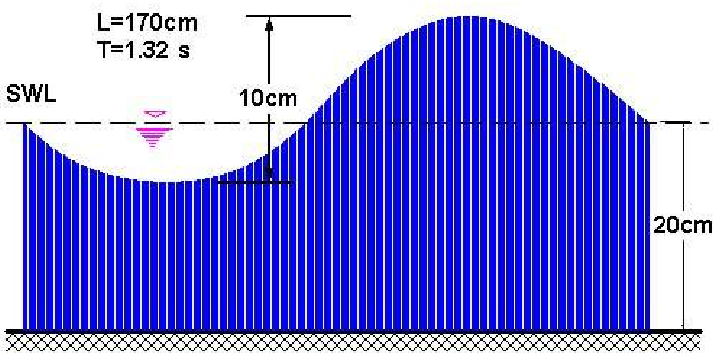
## A. Main Features




**<u>Cross Section of Dike Model and its location inside the flume</u>**




**Flume Cross Section (in centrimetre)**




**Waves Characteristics Generated by the Flume**

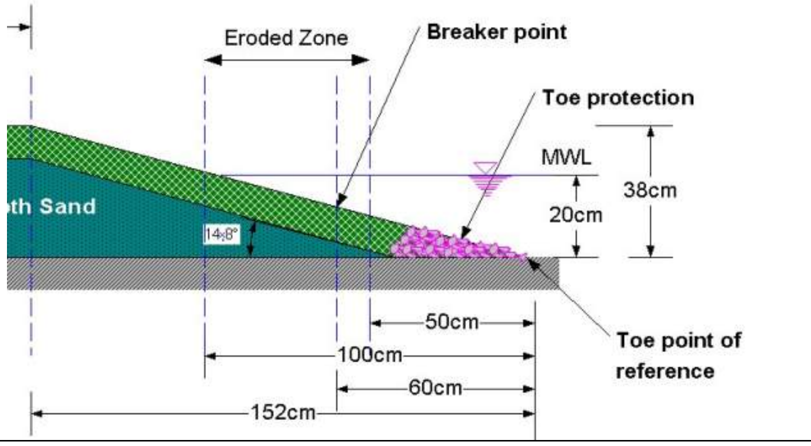
**Main Materials for model:**

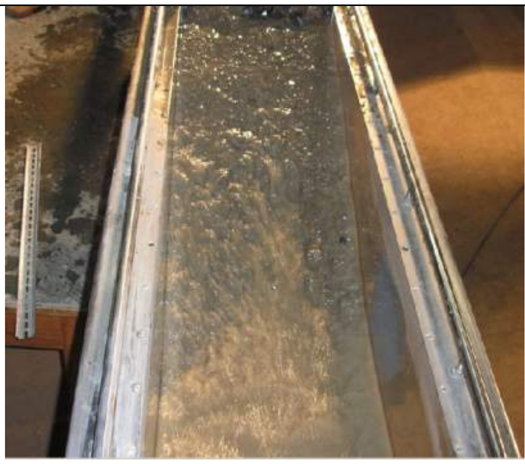
1. Moderate clay from Elisabethgroden km-9.0 for dike cover
2. Sand with $d_{50}$=0.103mm for dike core
3. Geotextile for piping protection
4. Stones with $d_{50}$=1.5cm for toe protection

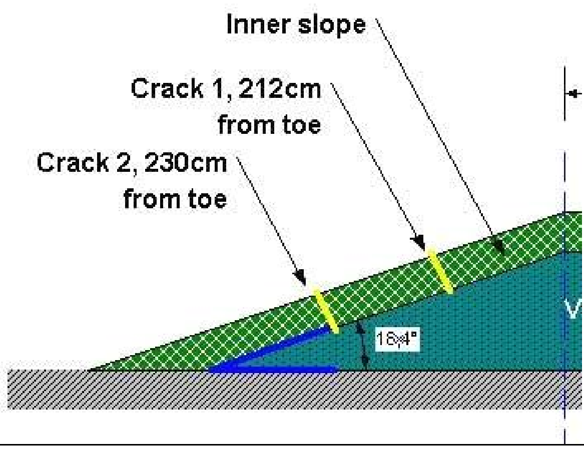
# B. Model 1

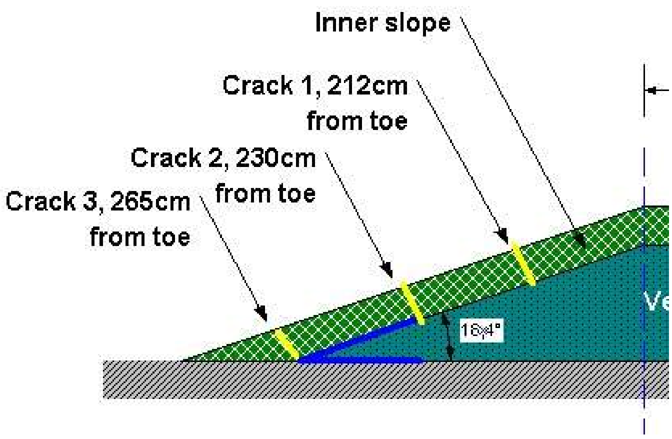
Starting time : Monday, 22-January-2007 at 11.30
Halted : Monday, 22-January-2007 at 20.00 – 21.15 (1 hour, 15 minutes)
Monday, 22-January-2007 at 22.30 – 23.15 (45 minutes)
Tuesday, 23-January-2007 at 13.30 until
Wednesday, 24-January-2007 at 08.00 (18 hours, 30 minutes)
Wednesday, 24-January-2007 at 13.00-15.00 (2 hours)
Finish : Friday, 26-January-2007 at 15.00
Total running time: ~78 hours

**Time table**

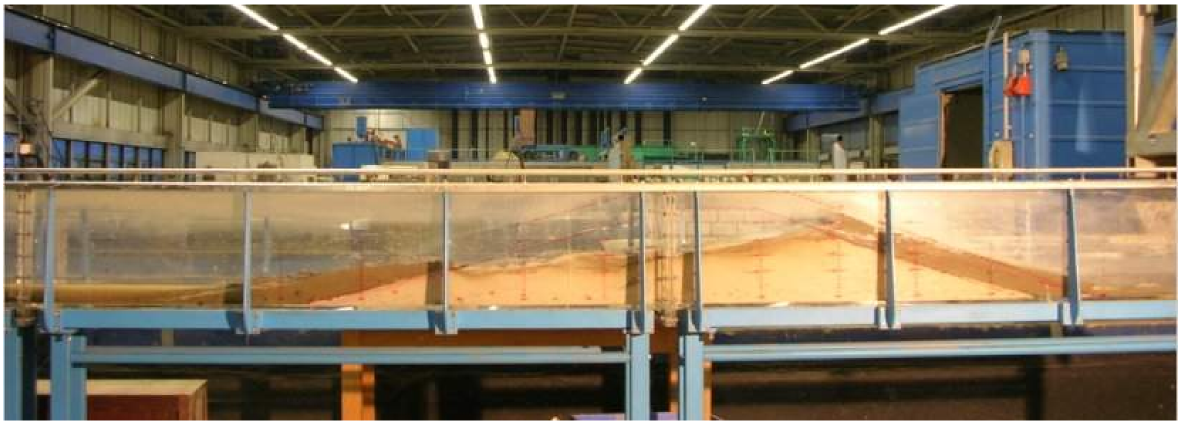
<table>
<tr><th>Date</th><th>Time</th><th>Notes</th><th>Movie</th></tr>
<tr><td>21/01/07</td><td>09.30</td><td>- Start building the dike</td><td></td></tr>
<tr><td></td><td>16.00</td><td>- The dike was in place</td><td></td></tr>
<tr><td>22/01/07</td><td>11.30</td><td>- Start the test using H=10cm and T=1.3s</td><td></td></tr>
<tr><td></td><td>12.00</td><td>- Smooth particles on the clay surface started to erode<br>- Small scour at the breaker point was observed<br>- Some overtopping observed as predicted<br>- Placing the metal plate at the top to avoid splash from the overtopping</td><td>Y</td></tr>
<tr><td></td><td>13.30</td><td>- Rough surface observed<br>- Eroded soil at the breaker point is getting deeper<br>- Scour depth reached 0.5 cm (at the edge)<br>- Some overtopping</td><td>Y</td></tr>
<tr><td></td><td>15.00</td><td>- Breaker point at 60cm from the toe. Due to reflection, this point sometimes change in the range of: 50-90 cm from the toe<br>- Scour depth at the breaker zone reached 1 cm<br>- Some overtopping</td><td>Y</td></tr>
<tr><td></td><td>16.00</td><td>- Scour depth is 2 cm at the edge and >2cm in the middle<br>- Eroded zone is about 50-100 cm form the toe<br>- Deeper in the middle possibly caused by less compaction, side effect, rundown<br>
</td><td>Y</td></tr>
<tr><td></td><td>17.00</td><td>- Scour depth 4.5 cm<br>- Eroded zone is slightly larger than 50-100cm</td><td>Y</td></tr>
<tr><td></td><td>18.00</td><td>- Scour depth 6cm<br>- Rundown velocity larger<br>- Overtopping</td><td>Y</td></tr>
</table>

| Date | Time | Notes | Movie |
|---|---|---|---|
| | |  | |
| | 19.00 | - Scour depth 7cm<br>- Overtopping | Y |
| | 20.00 | - Dinner break | |
| | 21.15 | - Start again | Y |
| | 22.30 | - The water was too muddy, start replacing the water | |
| | 23.15 | - Start again | Y |
| | 24.00 | - Clay cover failure in the middle of the dike at the breaker point with a lot of cracks around it<br>- Scour depth reach the limit (8cm) | |
| | 24.15 | - Side collapsed towards the scour clay<br>- Scour hole wider | |
| | 24.30 | - Increase f=32 Hz and increase the elevation<br>- The undermined sand cause the clay cover on top of it collapsed<br>- No Overtopping | Y |
| | 24.35 | - Remove the plates<br>- Seepage reaches 195cm from the toe<br>- Collapsing clay moves forward beyond the breaking point<br>- Breaking type changes, more severe due to development of sand bar<br>- Block of clay cover collapsed after the sand was undermined for some time<br>- These block clays then act as an absorber against the incoming waves until they broke apart into small particles and wash away<br>- Undermining sand goes quickly | |

| Date | Time | Notes | Movie |
|---|---|---|---|
| | |  | |
| 23/01/07 | 00.50 | - Seepage reach 200cm | |
| | 01.13 | - Seepage completed<br>- Dike crest start to collapse<br>- Eroded soil (particles) was taken away by the rundown and deposited around the toe | |
| | 01.42 | - Movie stopped | |
| | 02.33 | - A berm was created at the water level and acts as a damper, makes the process slow<br>- Undermining sand goes slow<br>- The berm can last long enough, because it has very thick water pad that decreases the impact energy | |
| | 03.00 | - Part of 5cm crest layer collapses | Y |
| | 03.22 | - The whole 5cm crest layer collapses | |
| | 03.34 | - 13cm crest layer collapses | |
| | 03.50 | - 18cm crest layer collapses<br>- The berm now has a slope<br>- Up till now, no erosion et all at the land side<br>- Small scour created at the breaker point in front of the berm<br>- It takes long time to break the eroded soil in to small particles | Y |
| | 04.13 | - Big cracks at the land side observed at 212cm (crack1) and 230 cm (crack 2) from the toe or at 25 and 20cm high.<br>- Possibly caused by sliding, pore pressure, and the geotextile<br>- The big crack (212 cm from toe) filled by water, water comes out from the smaller one<br> | |

| Date | Time | Notes | Movie |
|---|---|---|---|
| | |  | |
| | 04.30 | - The berm is moving landward<br>- The edge of the berm is eroded below the waterline few centimetres up to 110cm from the toe<br>(pic) | |
| | 05.10 | - Piping observed<br>- The water now looks like choco-milk (too much sediment on water)<br>- Toe become higher, about 9cm higher<br>- 230cm crack located at the edge of the geotextile | |
| | 05.45 | - The whole crest collapses | Y |
| | 07.00 | - Breakfast break | |
| | 08.15 | - The slope in front of the broken dike is getting stable | Y |
| | 09.00 | - Undermined sand reaches 175cm from toe | |
| | 10.24 | - Increase the water level up to 25cm (disturb)<br>- Sand bar developed at 90 cm from the toe<br>- Eroded clay layer reaches 180cm from the toe | Y |
| | 11.08 | - The 3rd crack appears 265 cm from the toe<br><br>- | |
| | 11.31 | (Disturb)<br>- The collapse of the clay layer goes gradually, started by the formation of cracks due to undermined sand | Y |

| Date | Time | Notes | Movie |
|---|---|---|---|
| | | - The 2nd crack slide down and close, the 1st crack getting wider<br>- Eroded clay reaches 200cm from the toe<br>- Sliding that caused the cracks clearly visible form the land side toe | |
| | 13.30 | - The experiment stopped | |
| 24/01/07 | 08.00 | - Started again | Y |
| | 09.00 | - No progress | Y |
| | 12.30 | - Hardly any progress | Y |
| | 13.00 | - Change the water | |
| | 15.00 | - Start again | Y |
| | 19.30 | - No progress | Y |
| 26/01/07 | 15.00 | -The test was stopped, no breaching, the damage dike is developing a natural beach with a very gentle slope. The slope starts from the end of the 2nd section of the flume. The dike damages up to 200cm from the toe | |



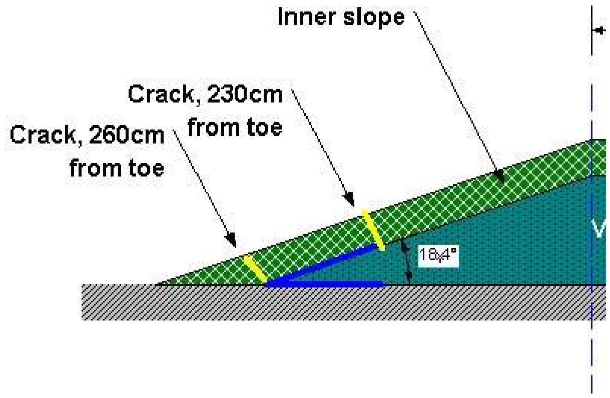
*Final Profile of the failured dike. No breaching occurred due to single factor of breaking wave impacts*

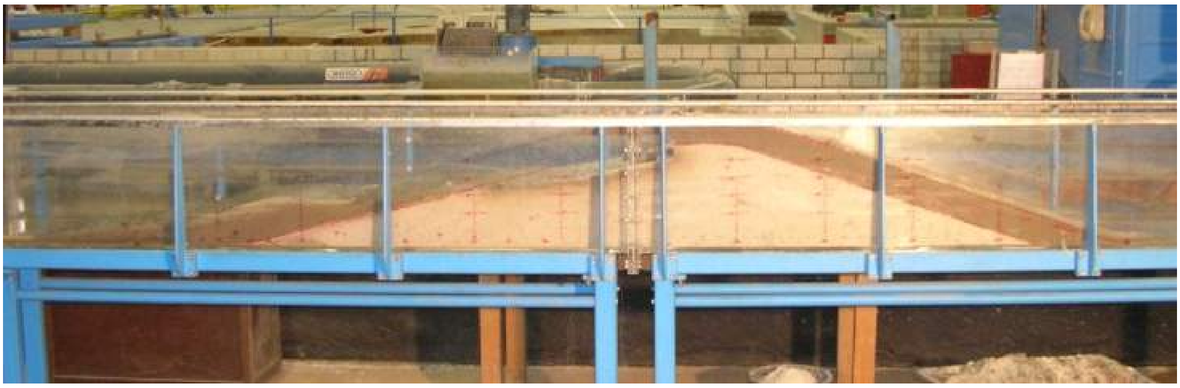
# C. Model 2

Starting time : Sunday, 28-January-2007 at 17.40
Halted : Monday, 29-January-2007 at 08.00 – 10.00 (2 hours)
Tuesday, 30-January-2007 at 09.00 – 10.00 (1 hours)
Finish : Tuesday, 30-January-2007 at 16.30
Total running time: ~ 45 hours

| Date | Time | Notes | Movie |
|---|---|---|---|
| 28/01/07 | 17.00 | - Dike Model is ready | |
| | 17.30 | - Start filling the flume by water | |
| | 17.40 | - Start the test using H=10cm and T=0.75s<br>- Plunging waves<br>- Breaker point at 60cm from toe<br>- Some overtopping | Y |
| | 19.30 | - Scour hole was observed due to impacts but not well distributed on the slope surface, only at the left side | Y |
| 29/01/07 | 07.00 | - Clay cover fails already with undistributed damage, only at the left side.<br>- Seepage has been completed | Y |

<table>
<tr><th>Date</th><th>Time</th><th>Notes</th><th>Movie</th></tr>
<tr><td></td><td></td><td>- No more overtopping<br><br></td><td></td></tr>
<tr><td></td><td>08.00</td><td>- Change the water</td><td></td></tr>
<tr><td></td><td>10.00</td><td>- Test resume<br>- Damage reaches the crest and gets wider (causing the more distributed erosion)<br>- Right side of the slope collapses gradually and faster than the already-broken clay cover at the left side<br><br>- 5cm clay cover from the crest collapses<br>- 2 cracks observed at the inner slope of the dike. Water comes out from these cracks.<br>- Breaker point is still at 60cm from toe</td><td>Y</td></tr>
</table>

| Date | Time | Notes | Movie |
|---|---|---|---|
| | |   - | |
| | 13.00 | | Y |
| | 16.45 | - Half of the crest collapses | Y |
| 30/01/07 | 08.00 | - Hardly any progress | Y |
| | 09.00 | - Change the water | |
| | 10.00 | - Test resume | Y |
| | 13.30 | - Increase the water level up to 25cm | |
| | 16.30 | - The test is stopped<br>- Damage up to 180cm from toe<br>- Cracks at the inner slope are wider<br>- Piping<br>- The waves break at the sand bar | Y |



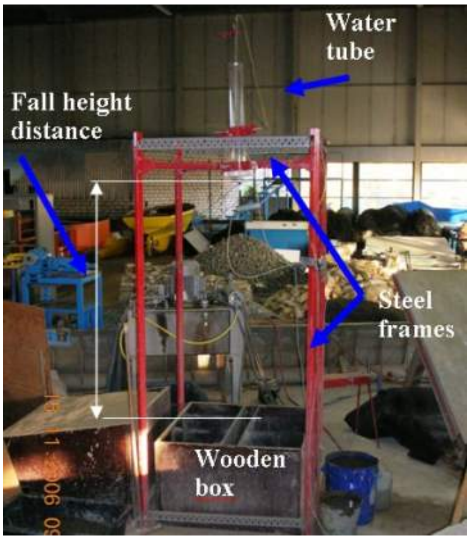
*Final Profile of model 2 after being hit by breaking wave impacts for 3 days.*

# C. Experimental procedure

## a. Procedure for water-filled crack experiment

After the clay with desired water content is ready, the following procedure can be started

1. Position the water tube into the required fall height. The maximum impact pressure that can be generated by the machine is 24.75 kPa for the fall height ($f_h$) of 1.62 m, water column ($f_w$) of 0.25 m, and impact duration of 0.115 s (Pachnio, 2005).



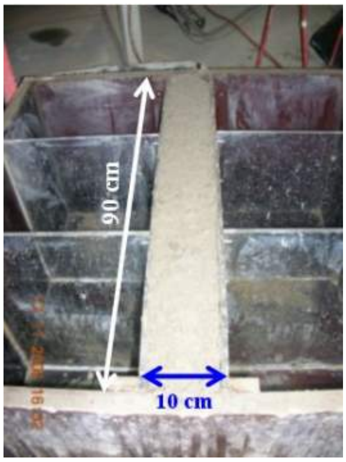

*Impact machine*

*Water tube showing its steel frames*

2. The transparent box is filled in by the clay and compacted in six layers (every 10 cm thickness) using a hammer which has dimension of 9.9cm x 17.4cm and weight of about 2.258 N.

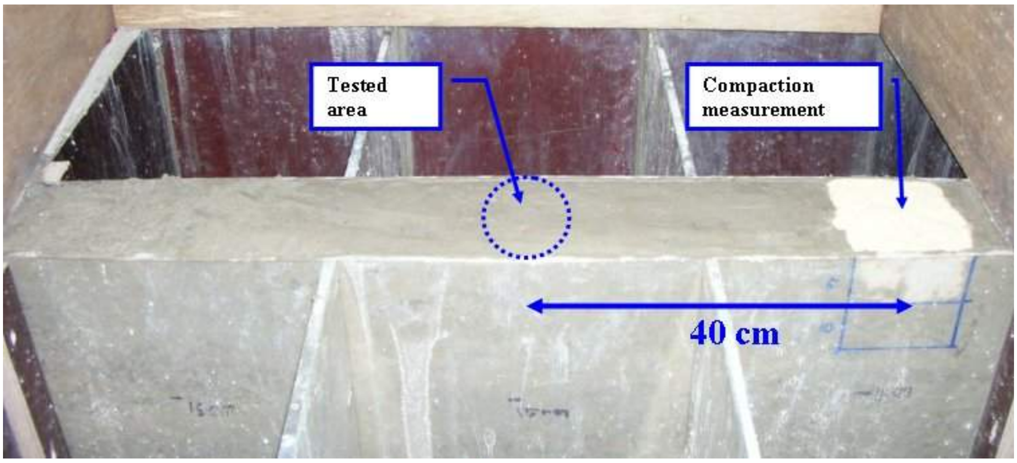
*The Hammer*


*Compaction Processes*




*Compacted sample*

3. Measure the soil density using sand replacement method about 30 cm from the artificial crack (details in **appendix**).



*Location of density measurement*


*Sand replacement method for density measurement*

4. Position the crack location so that the water mass will fall exactly at the crack.



*Crack location positioning by using weighted rope*

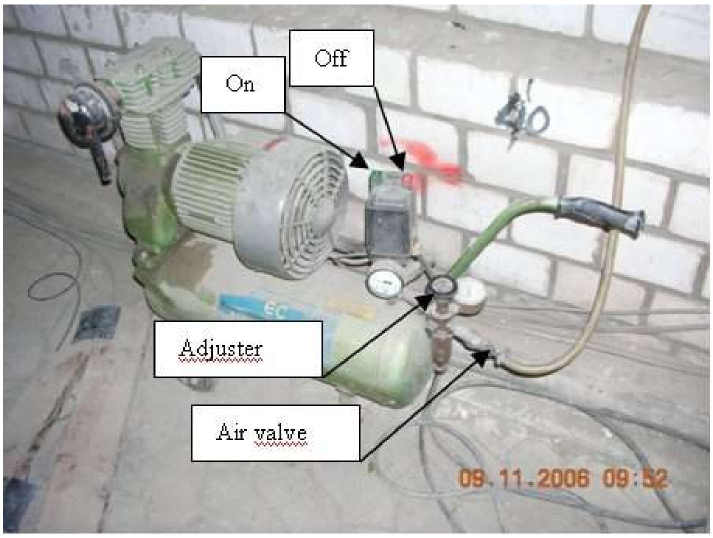
5. An artificial crack can be made by inserting 2 plates into the clay sample. The soil between the plates then is removed. The crack dimension is 15 cm deep, 1 cm wide and 10 cm long.

*The making of artificial crack.*
*The soil between 2 inserted thin plates is dig out*

6. The artificial crack is filled up by water.

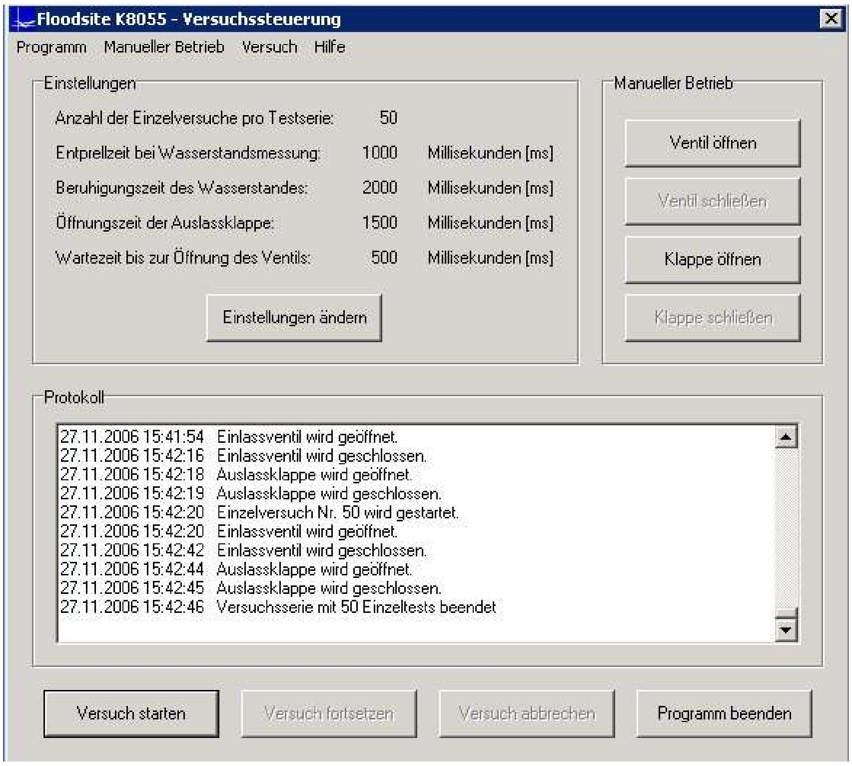
*Water-filled crack*

7. Open up the water tap to let the water flow and switch on the compressor to maintain the water tube in stable pressure




*Compressor to provide suitable pressure inside the water tube and prevent it from leaking*

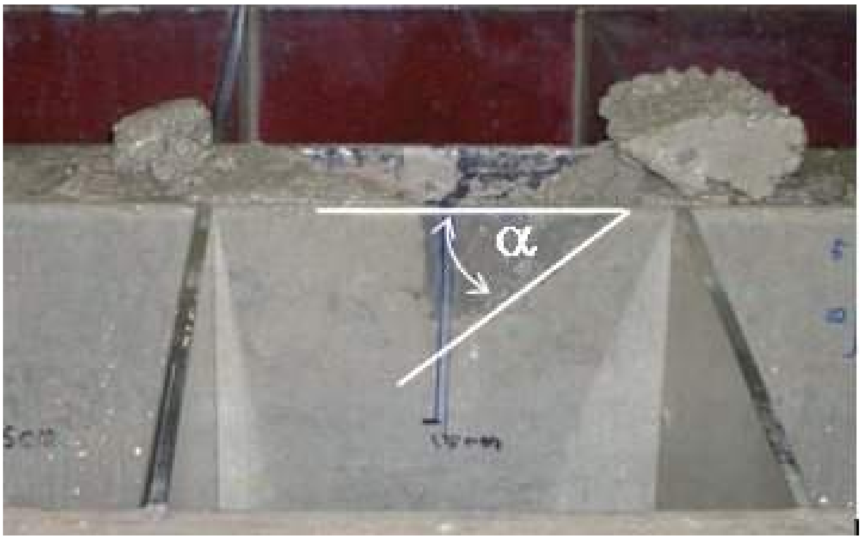
8. Start the test by automatically filling and releasing the water mass from the tube and hit the water-filled crack. A computer program is available to control setting up parameters needed.




*Windows interface for releasing automatically the water from the tube*

*Impact of the falling water mass*

9. After the impact. If there is a failure, a picture is taken and the shear angle failure ($\alpha$) is measured. If there are no any failures, the impacts will be carried on until significant failure is observed.



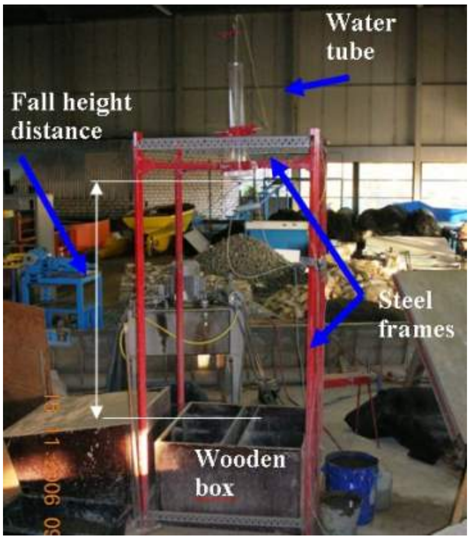

*Shear angel failure measurement*

10. Measure the actual water content from the side of the crack for shear strength estimation.

### *b. Procedure for Surface Erosion experiment*

After the clay with desired water content is ready, the following procedure can be started

1. Position the water tube into the required fall height. The maximum impact pressure that can be generated by the machine is 24.75 kPa for the fall height ($f_h$) of 1.62 m, water column ($f_w$) of 0.25 m, and impact duration of 0.115 s (Pachnio, 2005).



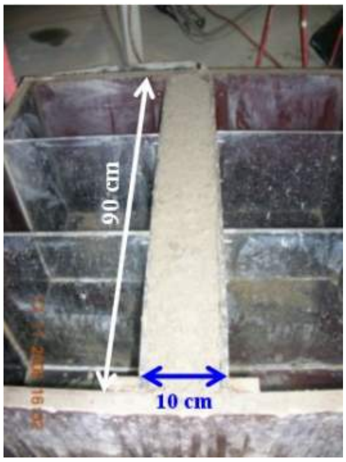
Impact machine


Water tube showing its steel frames

2. The transparent box is filled in by the clay and compacted in six layers (every 10 cm thickness) using a hammer which has dimension of 9.9cm x 17.4cm and weight of about 2.258 N.

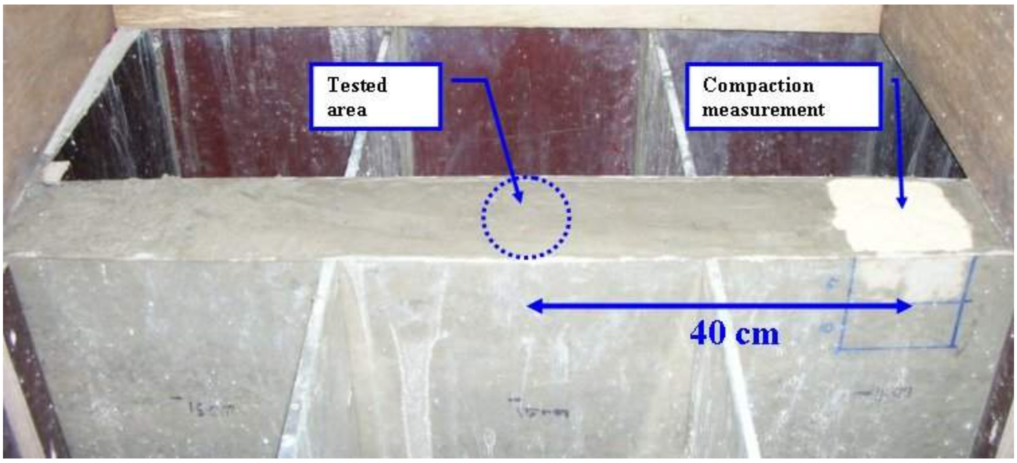

*The Hammer*

*Compaction Processes*



*Compacted sample*

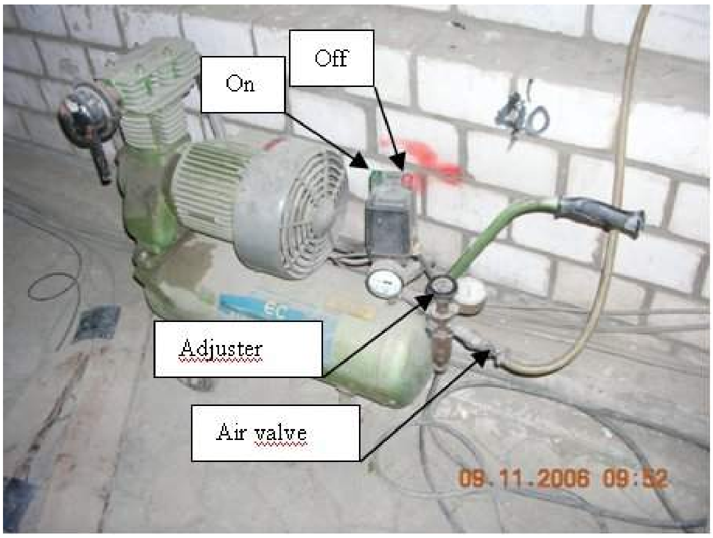
3. Measure the soil density using sand replacement method about 30 cm from the artificial crack (details in **appendix**).




*Location of density measurement*

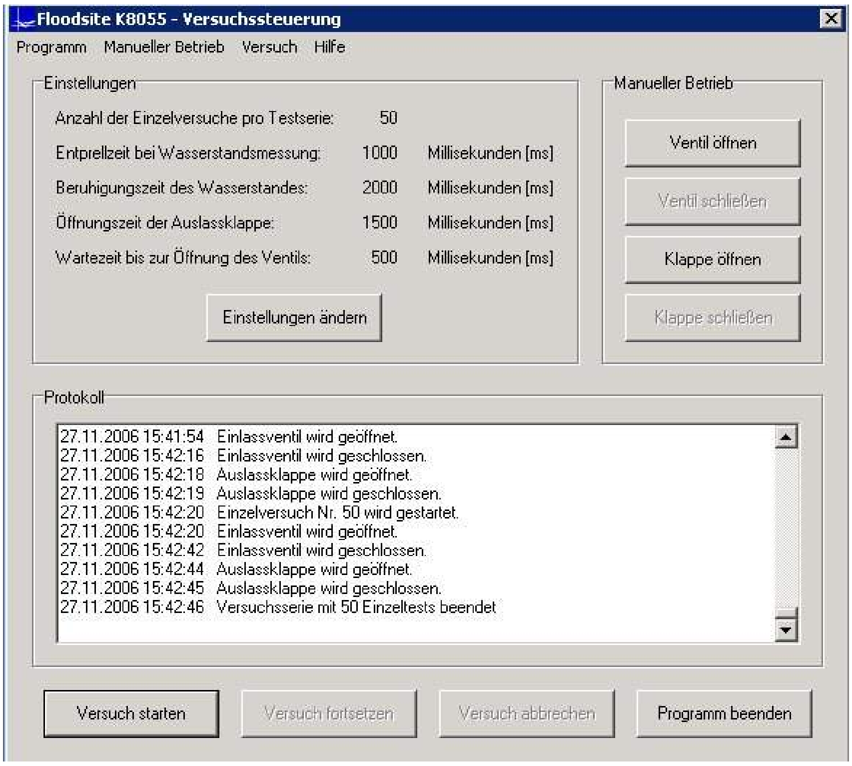
*Sand replacement method for density measurement*

4. Open up the water tap to let the water flow and switch on the compressor to maintain the water tube in stable pressure




*Compressor to provide suitable pressure inside the water tube and prevent it from leaking*

5. Start the test by automatically filling and releasing the water mass with desired number of impacts. The number of impacts can be set automatically via computer control.



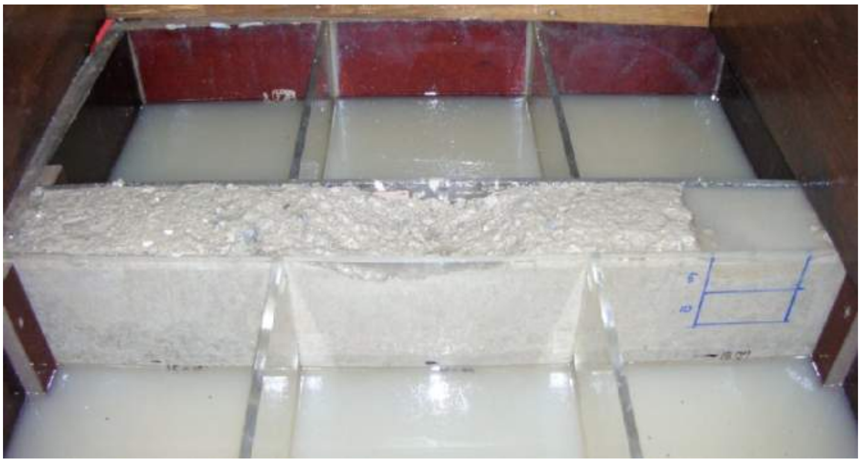
*Windows interface for releasing automatically the water from the tube*


*Impact of the falling water mass*

6. Remove the remaining water (water pad), if there are any. This water can play as water layer that damp some energy of the impacts


*Removing water from the scour hole to avoid water layer*

7. During the process of releasing impacts, measurements of eroded clay were carried out to see the progress of erosion, for example, every 50 impacts for impact number of 500 until finish. The measurement of eroded clay are divided into 3 (see **appendix** for details):
   - Eroded soil measurement by gypsum cast
   - Eroded soil measurement by filling water into the scour hole.
   - Eroded soil measurement by using a ruler to measure only the maximum depth of scour hole


*Scour hole after releasing some impacts need to be measured*

(a) (b)

(c)

*Method of eroded soil measurements: (a) measurement the scour depth by a ruler (b) Scour volume measurement by filling water (c) Scour volume measurement by gypsum cast*

8. Picture is taken for every progress of eroded soil. Important moments also should be documented such as development of cracks or removal of big block soils.


*Removal of big block soil (example of important event)*

9. If there is doubt due to certain circumstances, measurement of water content after completing the impacts is worth to do.

10. Pump out the water from the wooden box using the pump when it gets full

*Pumping out the water from the wooden box compartment*

# D. Eroded Soil Measurement - (Experiment for Surface Erosion on Compacted Clay)

## a. Eroded soil measurement by gypsum cast

1. After scour hole has been formed, prepare amount of gypsum in a bowl, pour some water into it, and mix them for few minutes. The amount of water should be enough to make the gypsum looks like thick milk.

2. Immediately after the mixed water-gypsum is ready, pour it into the scour hole. Partition is suggested to make it easier during removal of the cast, especially for large scour hole.

*Pouring the gypsum into the scour hole and dividing it into 3 sections by partitions*

3. After about 15 minutes, the gypsum cast is ready to be removed by digging the soil below the gypsum and by the help of the partition as well.


*Gypsum cast*

4. Prepare some water for volume measurement in a measurement bucket. Put the gypsum cast into the measurement bucket. The different water level read before after putting in the gypsum cast is the gypsum volume or the volume of eroded soil (*Archimedes law*).


*Volume measurement by applying Archimedes law*

### b. Eroded soil measurement by water

The need to measure the eroded soil many times after certain number of impacts (for example every 50 impacts) on the same sample required new method to get it. The method should require the following conditions:

1. The method should reliable enough to represent the amount of eroded soil.
2. The method does not disturb the current sample (scour hole), because the test will be carried on after the measurement.

Taking account that the compacted clay has very small permeability and the sample has been in constant wetting process (saturated) for certain duration (depends on number of impacts). Measurement using water appears to be applicable to satisfy those conditions. Compare to measurement using gypsum, water does not disturb the samples. Gypsum cast can bring some soil in the surface with it during removal of the cast.

To do measurement by water, trials should be made first to make sure that this method is really reliable. Before measurement, make sure that the samples are in saturated (All pores on the surface of scour hole have been filled by water). To do that, first pour some water into the scour hole and wait for ±5 minutes. After that, dry out all the water and start real measurement using measurement glass (or measurement beaker). The figures below show how the measurement by water works. Comparison results with measurement by gypsum cast showed no big differences.


*0 minute*


*1 minute later*


*2 minutes later*


*3 minutes later*


*4 minutes later*


*5 minutes later*


*6 minutes later*


*7 minutes later*

*A glass beaker for eroded soil measurement.*
*The difference reading between before and after pouring water into the scour hole is the eroded soil volume.*

### c. Eroded soil measurement by Ruler

It is simply measure the deepest part of the scour hole using a ruler. This method is suitable for soil with high permeability or sample with less compacted efforts. Surface erosion experiment using moderate clay used this method for comparison results from the measurement using water.


*Eroded soil measurement by a ruler to measure the deepest part of scour hole*

# E. In-Situ Density Measurement - by Non-standard Sand Replacement Method

Due to technical problems, it is not possible to control the compaction of the clay sample. In-situ density measurement is needed to get the information about how well the clay was compacted. There are many standard methods to do in-situ density measurement for compaction such as; Sand cone replacement, Rubber Balloon method, and Nuclear Gauge method. By looking at the dimension of the soil sample which has 10 cm wide and 100cm long, sand cone replacement method appears to be the most appropriate ones.

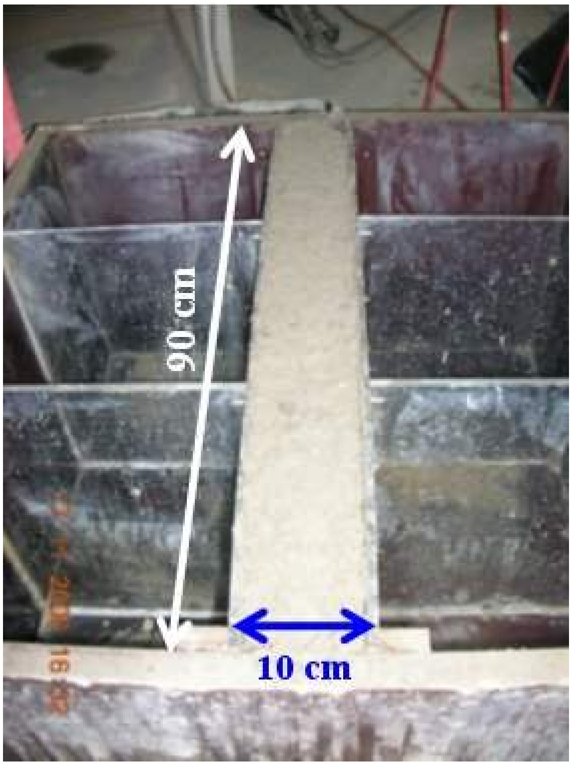



***Figure E-1**. Soil sample dimension*

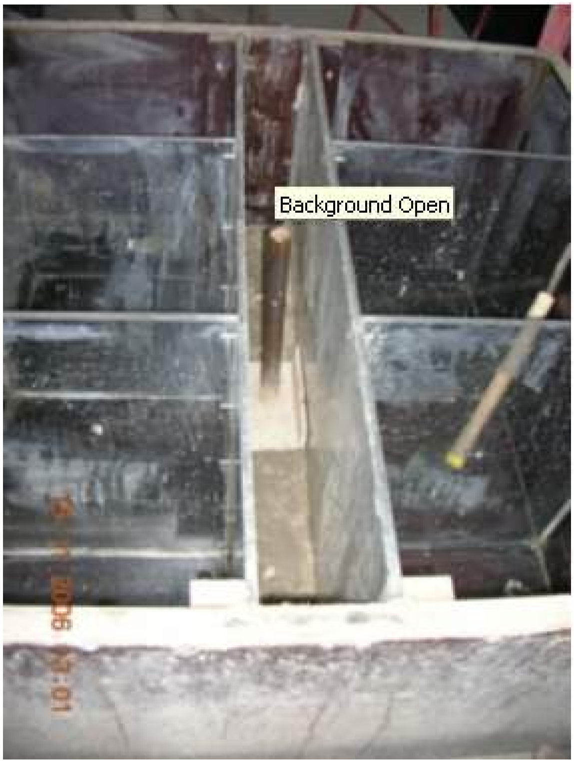



***Figure E-2**. Clay sample was compacted using a hammer which has head dimension of 9,9 cm wide and 17,4 cm long*

The standard apparatus of sand cone replacement method are:
1. Sand cone apparatus which consists of a plastic bottle with a metal cone attached to it.
2. Balance sensitive to 1 g.
3. Base plate
4. Tools for excavating a hole in the ground
5. Clean, dry and uniformly graded sand, passing the 1 mm sieve and retained on the 0.6 mm sieve.
6. Proctor compaction mold without attached extension (used for calibration)
7. Evaporating cups for soil samples
8. Oven with temperature kept at about 105-110$^{o}$C

The apparatus at number 1 (**Figure E-3**) is not available in LWI. Therefore, instead of using sand cone apparatus, a glass beaker is used. The consequence of this, the procedure will be slightly different from the standard method. **Figure E-4** shows the apparatus used for sand replacement method (Not Sand Cone Replacement, the standard one)

***Figure E-3***. *Standard Sand Cone apparatus, not available at LWI (From geotechnical laboratory, University of Texas at Arlington)*

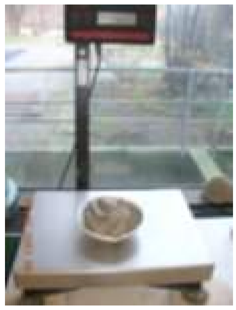
*Sieves with 1mm and 0.6mm mesh*

*Proctor compaction mold without attached extension and excavating tool*

*A glass beaker for sand container*

*Oven*

*Evaporating cups*



*Balance*

***Fig E-4***. *Sand Replacement apparatus*

The procedure to measure the soil density of the clay sample using sand replacement method is almost similar the standard sand cone replacement method. The different between these two methods is only on the use of sand cone apparatus which has been replaced by the glass beaker. The procedures of this method are as follow:

1. Collect the dry sand by sieving the sand from the quarry using the 1mm and 0.6mm mesh


2. Weigh the proctor compaction mold

3. Weigh the proctor compaction mold with sand inside

4. Calculate the density of the sand

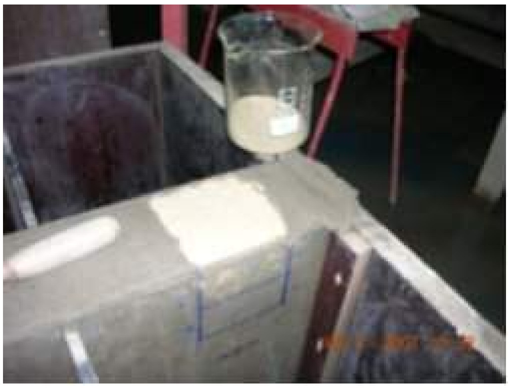
$$\rho_{sand} = \frac{W_{sand}}{V_{proctormold}}$$

5. Put the dry sand in the glass beaker and weigh it ($W_{sand-1}$)

6. Dig a hole on the clay sample with 10cm wide, 10cm long and at least 5cm deep (for clay with high water content).

7. Put the removed clay on the evaporating cup, weigh it ($W_{wetsoil}$) and put it into the oven for 24 hours with 105$^{o}$C

8. Pour the sand into the dig hole up to the surface. This stage should be done carefully and make sure that there are no overtopped sand.



9. Weigh the glass beaker with the remaining sand ($W_{sand\text{-}2}$) and calculate the sand volume of the filled hole

$$V_{sand} = \frac{W_{sand}}{\rho_{sand}}$$

Where $W_{sand} = W_{sand-1} - W_{sand-2}$

10. Measure the weight of dry clay after 24 hours baking process ($W_{drysoil}$).
11. Calculate the dry density of the clay sample

$$\rho_{dry} = \frac{W_{dry\,soil}}{V_{sand}}$$

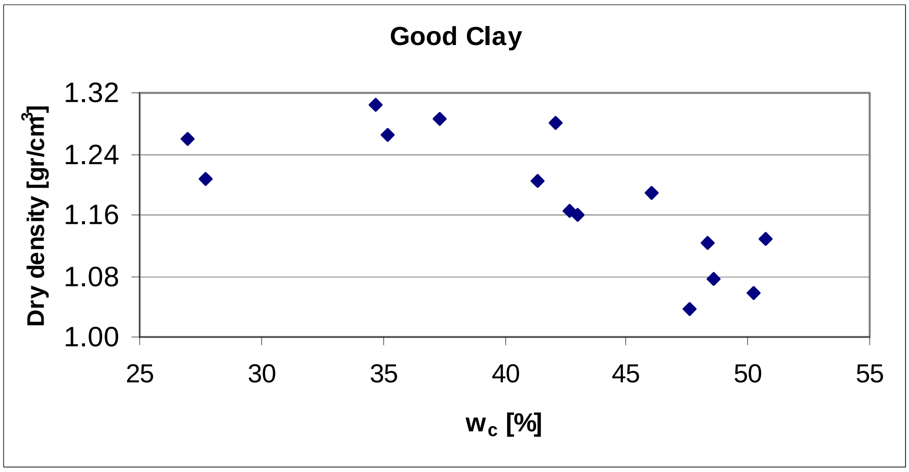
***Tabel 1.*** *Example of dry density calculation of the clay sample*

| Parameters | Values | Units | Notes |
|---|---|---|---|
| $W_{mold}$ | 6591 | gr | |
| $W_{sand+mold}$ | 7936 | gr | |
| $Vol_{mold}$ | 950 | ml | |
| $W_{sand}$ | 1345 | gr | |
| Sand Density | 1.4158 | gr/cm$^3$ | |
| $W_{glass}$ | 231 | gr | |
| $W_{glass+sand1}$ | 1578 | gr | before filing the hole |
| $W_{glass+sand2}$ | 805 | gr | after filling the hole |
| $W_{sand}$ | 773 | gr | for filling the hole |
| $W_{cup(1)}$ | 191 | gr | 17A |
| $W_{cup(2)}$ | 126 | gr | 17B |
| $W_{cups}$ | 317 | gr | 17A+B |
| $W_{cup+wetsoil}$ | 724 | gr | 17A |
| $W_{cup+wetsoil}$ | 549 | gr | 17B |
| $W_{cups+wetsoils}$ | 1273 | gr | 17A+B |
| $W_{wetsoil}$ | 956 | gr | 17A+B |
| $W_{cup+drysoil}$ | 588 | gr | 17A |
| $W_{cup+drysoil}$ | 441 | gr | 17B |
| $W_{cups+drysoils}$ | 1029 | gr | 17A+B |
| $W_{drysoil}$ | 712 | gr | 17A+B |
| $W_c$ | 34.27 | % | |
| Sand Volume | 545.99 | cm$^3$ | or hole volume |
| Wet density | 1.7510 | gr/ cm$^3$ | |
| Dry density | 1.3041 | gr/ cm$^3$ | |

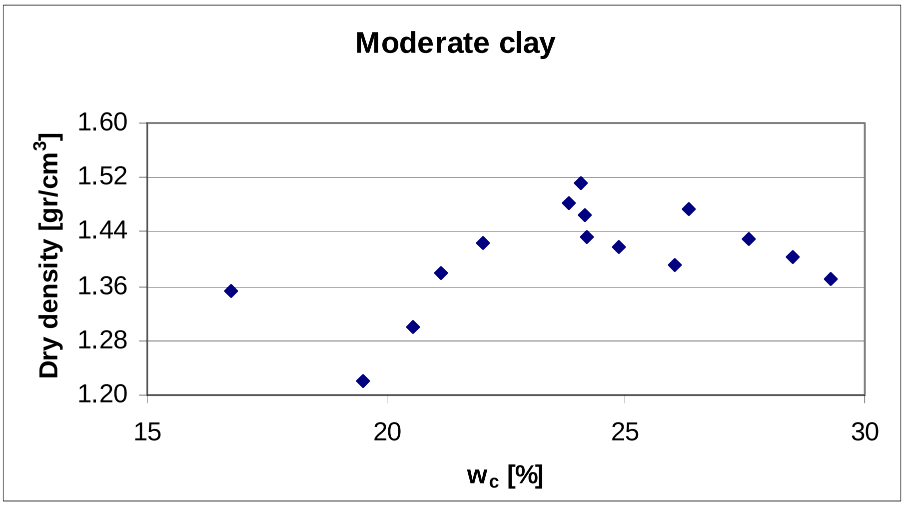
# Results

| Good clay | | | | Moderate clay | | | |
|---|---|---|---|---|---|---|---|
| Test ID | $w_n$ (%) | Compaction [gr/cm$^{3]}$ | Relative Density (%) | Test ID | $w_n$ (%) | Compaction [gr/cm$^{3]}$ | Relative Density (%) |
| 23 | 34.7 | 1.304 | 89.4 | 38 | 21.14 | 1.381 | 84.0 |
| 24 | 37.3 | 1.286 | 88.2 | 39 | 26.02 | 1.392 | 84.7 |
| 25 | 42.06 | 1.280 | 87.8 | 40 | 24.2 | 1.432 | 87.1 |
| 26 | 46.02 | 1.188 | 81.5 | 41 | 22.01 | 1.425 | 86.7 |
| 27 | 41.35 | 1.206 | 82.7 | 42 | 20.56 | 1.301 | 79.2 |
| 28 | 42.7 | 1.164 | 79.8 | 43 | 27.59 | 1.428 | 86.9 |
| 29 | 43 | 1.160 | 79.6 | 44 | 26.31 | 1.475 | 89.7 |
| 30 | 48.6 | 1.076 | 73.8 | 45 | 19.5 | 1.219 | 74.2 |
| 31 | 47.64 | 1.036 | 71.1 | 46 | 24.07 | 1.512 | 92.0 |
| 32 | 48.32 | 1.124 | 77.1 | 47 | 24.14 | 1.466 | 89.2 |
| 33 | 50.2 | 1.059 | 72.6 | 48 | 24.85 | 1.417 | 86.2 |
| 34 | 50.73 | 1.129 | 77.4 | 49 | 16.75 | 1.354 | 82.4 |
| 35 | 35.17 | 1.264 | 86.7 | 50 | 29.27 | 1.370 | 83.4 |
| 36 | 27.71 | 1.208 | 82.9 | 51 | 28.48 | 1.404 | 85.4 |
| 37 | 27 | 1.261 | 86.5 | 52 | 23.8 | 1.481 | 90.1 |
| Average | | 1.183 | 81.1 | | Average | 1.404 | 85.4 |
| Max | | 1.304 | 89.4 | | Max | 1.512 | 92.0 |
| Min | | 1.036 | 71.1 | | Min | 1.219 | 74.2 |

# Cäciliengroden




# Elisabethgroden km-9.0



# RECOMMENDATIONS
## (Experimental Set-up)

### A. Experimental set-up for erosion tests on compacted clay with water-filled cracks

#### *1. Soil Preparations*

Clay samples with moderate erosion resistant (Clay category II according to TAW (1996) or Class 2 according to Weissman (2003)) are the most suitable for these experiments. The estimated strength of this type of clay (see **Figure 3.3**, **Chapter 3**) is highly possible to be lower than the maximum generated impact pressures from the machine.

To look at possible failure on compacted clay with water-filled cracks, the water content should be maintained in the range of 0.50< Ic<0.75. By applying this range of water content, theoretically, the strength of the clay (two times its cohesion) is smaller than the maximum impact ($2c<p_{max}$) (see **Section 2.5.1**).

Homogeneity of samples is also very important. The samples should be free from the presence of hard lumps. Crushing hard lumps either using crushing machine or manually using hands is better to be done when they are still in dry conditions. After that, sieving process by using a mesh which has diameter 4.75mm can be done to get homogeneous samples for the test.

The clay samples usually need to be moistured to get the desired water content. Let the samples to hydrate for about 48 hours before doing the tests.

#### *2. Fall height*

Fall height of the impact machine is related to generation of impact magnitudes. From the current situation, the maximum fall height is 162cm. The water tube can not be higher than what it is now but the sample can be lowered until it reaches its optimum fall height. Current height of the sample from the bottom is 60cm. For the test, only 20 cm deep is considered to have effect on the samples because the maximum depth of artificial crack is only 15cm. Therefore, by lowering down the sample position, the fall height can be increased up to 200cm or equivalent to the generated impact pressure of about 27kPa. **Table F.1** below shows estimation of failure by applying impact pressure of 27kPa and samples with water content in the range of 0.50< Ic<0.75.

***Table F.1*** *Estimation of soil strength of moderate clay and maximum impact pressure*

| Ic[-] | wc[-] | c[kPa] | 2c[kPa] | $p_{max}$[kPa] | Note |
|---|---|---|---|---|---|
| 0.74 | 0.255 | 17.07 | 34.14 | 27 | $p_{max}<2c$ |
| 0.62 | 0.280 | 10.35 | 20.71 | 27 | $p_{max}>2c$ |
| 0.50 | 0.305 | 6.28 | 12.56 | 27 | $p_{max}>2c$ |

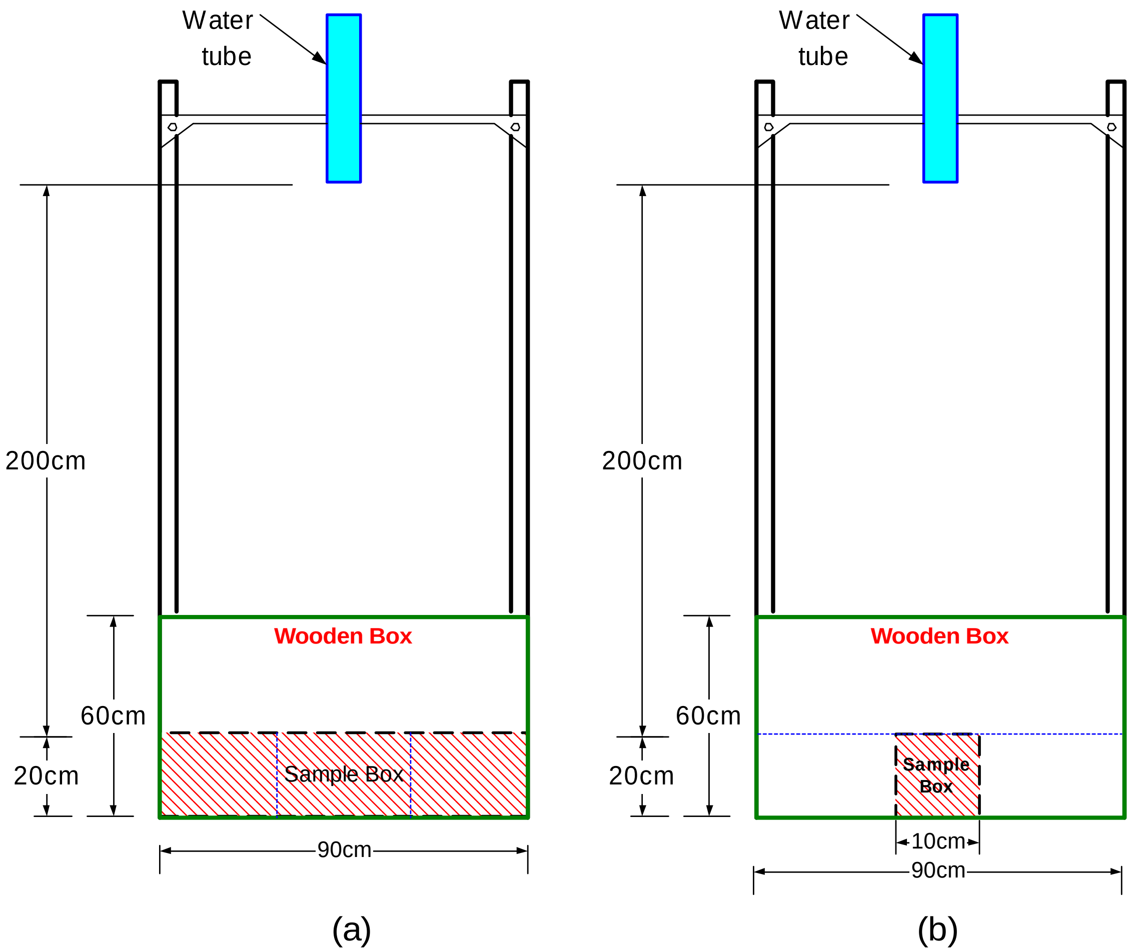



***Figure F.1** Lowering down the sample box to get higher fall height (a) Front view (b) Side view*

### 3. *Making an artificial crack and its dimension*

To avoid too much disturbance to the soil around the crack, the process of making an artificial crack should be changed. Punching 2 steel plates into compacted clay causing some problems to the samples (see **Section 5.1.3**). The steel plates can be replaced by a solid plate. This solid plate can have a dimension of 15cm long (not included extra length for a handle), 10cm wide and 0.4cm or 0.6cm wide. The width of 4mm or 6mm is chosen as 1$^{st}$ step because the propagated impact pressures inside the crack from those dimensions had been measured (see Pachnio, 2005).

To have an artificial crack, put the sold plate together with the soil into the sample box. Then, compaction can be done simultaneously around the crack (**Figure F.3**) using the available hammer. To avoid difficulties during removal of the solid plate, lubrication using water prior to compaction can be very helpful.

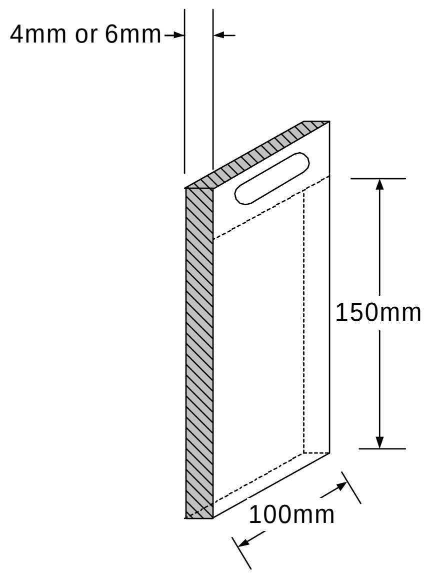



***Figure F.2*** *A solid plate for making an artificial crack*

Once the clay has been compacted, the solid plate can be removed by pulling it up. Putting a pressure to avoid damage on top of the sample when pulling up the plate can be done by any necessary means.

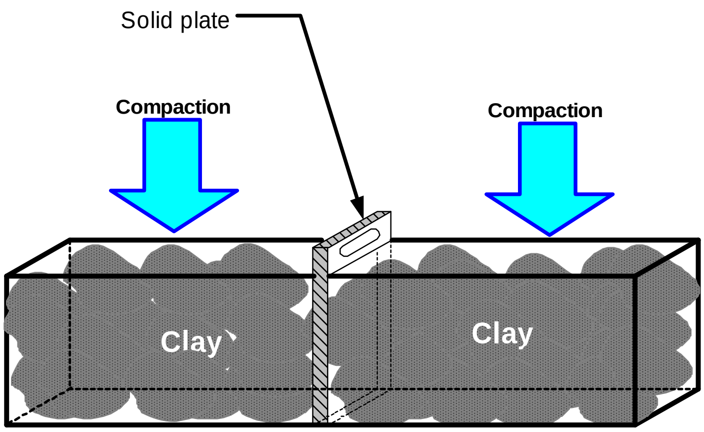



***Figure F.3*** *Process of making an artificial crack*

### *4. Density measurement*

The in-situ density measurement can be done the same as it is described in **Appendix E** in this report.

## B. Experimental set-up for surface erosion test of compacted clay without cracks

### *1. Soil preparation*

Three types of soil can be prepared for investigation of surface erosion on compacted clay:

- Good clay,
- Moderate clay and
- Mix of good-moderate clay to see erosion behaviour in between.

Just like soil preparation for the test for erosion on water-filled crack, soil homogeneity and hydration process for about 48 hours should be taken into account.

Range of water content for the samples can follow the one as described in **Section 3.2.1**, especially for the clay with dry water content ($w_c$<PL) which were not fully covered in this report

### *2. Proctor mould*

The use of proctor mould could be very useful for surface erosion on compacted clay due to impacts. Proctor mould has dimension of 116.43mm high and a diameter of 101.6mm. By using proctor mould for placing the clay samples, the following advantages will follow:

- Compaction is known and accurate and as well as the water content (How to do compaction, see standard procedures released by the apparatus supplier)

- Higher impacts are possible by lowering down the sample up to the bottom of the wooden box. Maximum fall height up to 200cm (27kPa) can be achieved.

- Measurement of erosion can be done easily by measuring the maximum depth of the scour hole.

- Effect of tongue from the opening of the valve can be avoided

- More efficient (not required bulky samples and more straight forward)

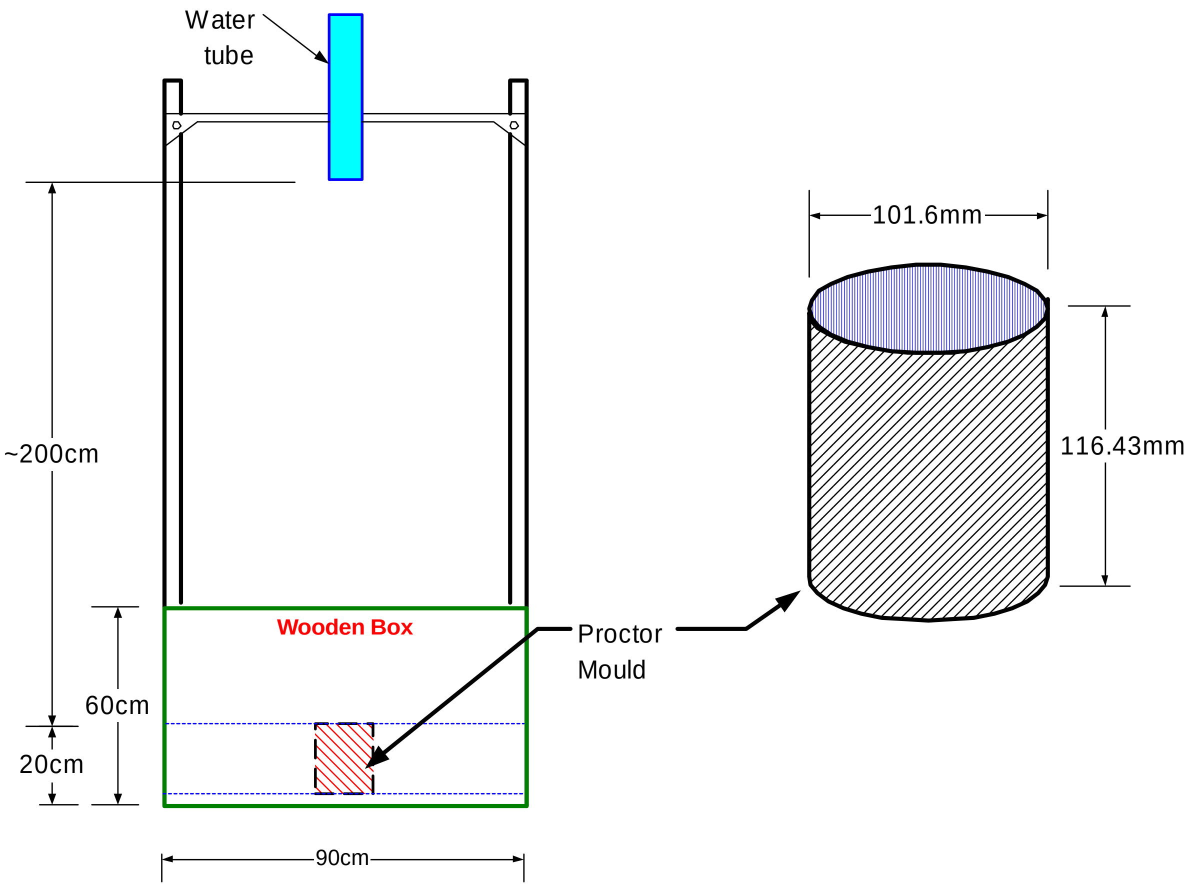



***Figure F.4*** *Proctor mould for placing the samples*

### *3. Fall height*

Fall height can be varied from ~160cm up to ~200cm. Variation in fall height is needed to see the influence of impact magnitude changes to the surface erosion processes.