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An investigation of the overlapping passive zones using a novel geotechnical centrifuge model
An investigation of the overlapping passive zones using a novel geotechnical centrifuge model

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Abstract
To improve the design of soil retaining walls in narrow trenches, a design optimization tool is
introduced by Hosseinzadeh and Joosse (2015). This design optimisation tool is an intensification
factor on the passive earth pressure coefficient $K_p$, that includes the effect of overlapping passive
zones. The authors argue that the passive earth pressure coefficient due to the overlap of passive
zones is larger compared to the passive earth pressure coefficient in case of unrestrained passive
zone development. The intensification factor $X_{K_p}$ is defined as the ratio between the unrestrained
ultimate passive capacity and the restrained ultimate passive capacity, the magnitude is dependent
on the dimensions of the narrow trench. The authors observe that the intensification factor increases
non-linear when the dimensions of the trench decreases.

The numerical study of Hosseinzadeh and Joosse (2015) is based on theoretical assumptions of the
Hardening Soil model in the Finite Element Modelling (FEM) software Plaxis 2D. To improve the
understanding of the intensification effect and to validate the numerical study, a novel physical scale
model is developed which models the unrestrained passive zone development and the effect due to
the overlap of passive zones. To simulate realistic stress levels in the model, a geotechnical
centrifuge is used where earth’s gravity is enlarged by high speed rotation of the model. Multiple
sand characterization tests are performed to obtain the parameters of the Delft Centrifuge (DC) sand.
Furthermore, interface characterization tests are performed to obtain the friction parameters playing
a role in the physical model.

Various centrifuge tests at 80g and 100g are performed in the geotechnical centrifuge facility of the
Delft University of Technology. An actuator in the strongbox is connected to a stiff wall. During
translation of the wall towards the soil body, the load and the wall displacement are measured. The
wall represents a prototype embedded length (d) of 5 m in all the tests. Restrained passive soil
behaviour is modelled symmetrically in the strongbox and the width (w) of the narrow trench is
varied in the test series. An ultimate passive load is measured by the plateau state and visible by the
full development of the shear band for w/d ratios of 6.4, 3.0, 2.0 and 1.5.

It can be concluded from the physical model tests that the intensification effect is observed in the
model results. The ultimate passive load (indicated by the plateau state) is larger in models with a
smaller w/d ratio. The intensification factor displays a non-linear increasing trend for decreasing
model dimensions w/d. The curvature of the trend line is similar to the trend observed in the
numerical study by Hosseinzadeh and Joosse (2015).

This thesis shows however discrepancies between definition and the absolute values of the
intensification factor of the numerical model and the physical model. The physical model tests show
significant larger ultimate passive capacities and intensification factors compared to the numerical
study. When small wall displacements are compared, the restrained physical model does not show
significant differences compared to the unrestrained physical model. The passive zones need to be
mobilized before they can overlap. This conclusion is not made in the numerical simulation, the
intensification effect is noticeable directly after the start of prescribed wall displacement. Limitations
of the constitutive model and the simplified shear characteristics of the soil mass are the main
reasons to explain the differences between the numerical and physical model.

It is recommended to study the effect of the overlapping passive zones on sand samples with
different grain size distributions and different packings. To increase the understanding of the
intensification effect and to increase the applicability for engineering practice, stratification and
different types of soil have to be included in further research.
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1 Introduction

Steel sheet pile walls are a very common type of construction to function as a soil retaining wall. The walls are used for both temporary constructions like building pits and long term constructions like tunnels. Depending on the type of construction, soil characteristics and the desired retaining height the geotechnical engineer creates an optimal design. This optimal design is a trade-off of multiple design choices. The geotechnical engineer can make use of several design tools like the design standards or specific design software’s.

In pipeline construction projects the geotechnical engineer often deals with relative long and narrow building pit. Some examples of this type of narrow building pits can be found in Figure 1.1 and Figure 1.2.

![Figure 1.1 Narrow sheet pile trench to construct a sewer pipeline](www.imeco.at, 2015)

![Figure 1.2 Narrow building pit with steel sheet pile walls for a pipeline jack](http://strongerchristchurch.govt.nz/, 2015)

One of the design goals of this type of construction is often to design the retaining wall with the shortest but safe length. The motivation for this design optimization is to minimize costs and nuisance to the neighbourhood. One of the measures to design the soil retaining wall shorter is the construction of struts. Another determining design aspect is the interpretation of the soil, this interpretation depends on the experience of the engineer. The stronger or stiffer the soil is assumed in the calculation, the shorter the soil retaining wall can be designed logically. This master thesis focuses on the modelling of the soil-structure behaviour, to improve future engineering practice.

1.1 Definition and Relevance

During excavation of a building pit the soil around the retaining walls shows two kinds of behaviour. At the excavation side of the sheet pile wall the soil provides a resistance force; which results in the development of a passive wedge. At the other side of the retaining wall the soil produces a force due to the unit weight of the soil; which results in an active wedge. The exact size and shape of the wedges depend on the soil characteristics. The soil behaviour in case of a single unobstructed sheet pile wall is a well-known mechanism, there are several analytical models which describe this soil behaviour and it can be found in the design standards like Eurocode 7.
1.1.1 Overlapping of the passive zones

In case of a symmetric narrow trench, the passive wedge of a sheet pile wall will cross with the passive wedge of the contrary sheet pile wall in the middle of the building pit. This phenomenon is known as the overlapping of the passive zones. This situation is made visible in Figure 1.3 for the case where the sheet pile walls move equally towards the pit; known as pure translation. Pure translation is a theoretical movement of the wall which will never occur in practice over its whole length, but it helps simplifying and understanding the phenomena of overlapping passive zones. Important variables of this thesis are the building pit width \( w \) and the embedded length of the sheet pile wall \( d \).

![Figure 1.3 Schematization of the overlap of the passive zones in a narrow building pit (pure translation)](image)

A desk study has been conducted by Joosse (2011) where this geotechnical phenomena is researched for the first time using a Finite Element Model (FEM) analysis. The goal of this desk study was to get a better understanding of this effect of overlapping and to create an easy tool to take this effect of overlapping into account in a simple 2D spring model with a single sheet pile row (i.e. D-Sheet Piling program). One of the assumption Joosse made was a horizontal translation over the whole length of the wall as can be seen in Figure 1.3.

The design tool that has been introduced is an intensification factor \( X_{kp} \). This factor is defined by Joosse as: “the ratio between the values of the passive earth pressure coefficient for a restricted and unrestricted development of the passive zone”. An example of the output (total displacement) of the numerical model created in the FEM program Plaxis 2D is shown in Figure 1.4.

![Figure 1.4 Example of the output of FEM program Plaxis 2D, model with overlapping passive zones in sand, \( \phi = 30^\circ \) (shown: total displacements [m]) (Joosse, 2011)](image)
Figure 1.5 Intensification factor (log scale) vs. the ratio of width (w) over embedded length (d), as can be seen in Figure 1.3 (sand, $\varphi = 30^\circ$) (Joosse, 2011)

The most important results of this study are shown in Figure 1.5. According to Joosse (2011) the intensification factor is a (non-linear) function of the width (w) vs. embedded length ratio (w/d). He concludes as well that there is no significant difference in the intensification factor at different building pit widths, as long as the ratio together with the embedded depth is plotted.

A more extended study by Hosseinzadeh and Joosse (2015) focuses on cohesive soils as well. The study is the first published paper about the intensification factor known by the author. According to this study, the ultimate passive stress capacity intensification factor is defined as a ratio depended on the dimensions of the trench (w, x and d), the unit weight of the soil ($\gamma$), the angle of internal friction of the soil ($\varphi'$), and wall friction ($\delta$) for the situation. The formula of the intensification factor is shown in equation 1.

$$X_{K_p}(\varphi', \varphi, \delta, w, x, d) = \frac{K_{p,w}(\varphi', \varphi, \delta, w \ll \infty, x, d)}{K_{p,\infty}(\varphi', \varphi, \delta, w = \infty, x, d)} \quad (1)$$

The formula describes the intensification factor ($X_{K_p}$) as a ratio between the passive earth pressure coefficient of a restrained (narrow) sheet pile building pit ($K_{p,w}$) and the passive earth pressure coefficient of unrestrained (wide) building pit ($K_{p,\infty}$), where the passive zones can develop without overlapping. The soil parameters ($\varphi'$, $\varphi$), wall friction ($\delta$) and the excavation depth and embedded length ($x, d$) are assumed to be the constants in the calculation of the intensification factor.

The restrained and unrestrained passive earth pressure coefficients $K_{p,w}$ and $K_{p,\infty}$ are according to Hosseinzadeh and Joosse (2015) ratios between the effective horizontal stress at failure ($\sigma'_{h,\text{ultimate}}$) and the initial vertical effective stress ($\sigma'_{v,\text{initial}}$). The passive earth pressure coefficient can be calculated by integrating the ultimate horizontal, and initial vertical effective stresses over the embedded length, see equation 2. It is important to note that the initial vertical effective stress ($\sigma'_{v,0}$) is an assumption, because due to heave of the surface this stress will increase.

$$\frac{\int_0^z (\sigma'_{h,\text{ultimate}}) \, dz}{\int_0^z (\sigma'_{v,\text{initial}}) \, dz} = K_{p,w/\infty} \quad (2)$$
The main results of this numerical study is shown in Figure 1.6 where the intensification factor is related to the w/d ratio of the narrow sheet pile trench. These results have not been subjected to validation by a physical model.

Figure 1.6 Intensification factor vs. the ratio of width over embedded length for different cohesive and cohesionless soil (Hosseinzadeh and Joosse, 2015)

The relevance of applying the intensification factor in the design is demonstrated with a case study by Hosseinzadeh and Joosse (2015) of a 4 meter wide trench. The length of the sheet pile walls was reduced with 15% and a significant reduction in the maximum bending moment of approximately 43% was achieved due to applying the intensification factor. This design optimization is not only relevant for the (paying) client but for the environment as well. It means less inconvenience due to vibration or hammering and less steel that has to be transported.

1.2 Research question and hypothesis

To improve the existing knowledge, research and the understandings of the intensification effect on the ultimate passive capacity the analytical and numerical models need to be supported by physical models. Therefore, the main research question of this thesis is defined:

Is the intensification factor of the ultimate passive stress capacity due to the effect of overlapping passive zones, introduced by the numerical and analytical approach of Hosseinzadeh & Joosse (2015), valid based on a physical model?

The corresponding hypothesis is based on the results of numerical research so far on this topic shown in Figure 1.6.

There is an intensification factor when the passive zones do overlap, this factor is increasing with a non-linear trend if the ratio w/d of trench width (w) over embedded length (d) decreases linearly.

The necessity of a physical model is described by Wood (2004). “A well designed physical model provides an important opportunity in the modelling cycle”. And Wood continues: “We can never prove a theoretical model to be true; all we can say about a successful model, or a conjecture on which that model is based, is that it has not yet been falsified or refuted.” Based on the problem described previously, there is clearly a need for a physical model since there is only theoretical and numerical research available about the intensification due to overlapping of passive zones.
1.3 Scope of research
To set the boundaries of this research, a scope is defined. The scope is very much based on the scope and assumptions in the study of Hosseinzadeh and Joosse (2015).

- This research is only about the passive zones; no attention will be given to the active zones;
- This research is restricted to a 2D plane strain situation;
- The sheet pile walls shift in pure translation (Figure 1.3), this means that elasticity of the sheet pile walls is not playing a role;
- The influence of structural elements like anchors, struts and an underwater concrete slab are not taken into account in this research.

1.4 Definition of terms
The following terms will be used in this thesis.

<table>
<thead>
<tr>
<th>Term</th>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Passive earth pressure</td>
<td>( K_p )</td>
<td>Coefficient to describe the ratio of horizontal effective stress over vertical effective stress during passive loading</td>
</tr>
<tr>
<td>Intensification factor</td>
<td>( X_{K_p} )</td>
<td>The ratio between the values of the coefficient of passive earth pressure for a restricted and unrestricted development of the passive zone (Joosse, 2011)</td>
</tr>
<tr>
<td>Wall translation</td>
<td>[-]</td>
<td>Pure and only horizontal movement of a wall</td>
</tr>
<tr>
<td>Narrow building pit</td>
<td>[-]</td>
<td>A building pit where the passive zones do overlap</td>
</tr>
<tr>
<td>Building pit width</td>
<td>( w )</td>
<td>Distance between two sheet pile walls (see Figure 1.3)</td>
</tr>
<tr>
<td>(Symbol ( b ) is used by Hosseinzadeh and Joosse (2015))</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Embedded length</td>
<td>( d )</td>
<td>Length of sheet pile wall minus excavation depth (see Figure 1.3). (Symbol ( d_2 ) is used by Hosseinzadeh and Joosse (2015))</td>
</tr>
<tr>
<td>Shear band</td>
<td>[-]</td>
<td>A thin weak shearing zone in a soil body (Stuit, 1995)</td>
</tr>
<tr>
<td>Prototype</td>
<td>[-]</td>
<td>The to be (full-scale) modelled object</td>
</tr>
<tr>
<td>Model</td>
<td>[-]</td>
<td>Small scale object which will undergo enhanced acceleration</td>
</tr>
<tr>
<td>(compared to the earth’s gravity)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Angle of internal friction</td>
<td>( \varphi' )</td>
<td>Shear strength parameter for soils</td>
</tr>
<tr>
<td>Relative density</td>
<td>( D_r )</td>
<td>Value to indicate the porosity of the grains relative to the maximum and minimum porosity</td>
</tr>
</tbody>
</table>

1.5 Outline of the thesis
This thesis is written in several sequential chapters which describe the findings of the research. Chapter 2 contains a summary of the literature review about the existing research of physical modelling which involves the passive behaviour of soil. The lessons learned from coupling of centrifuge modelling and numerical modelling can be found as well as relevant aspects in centrifuge modelling.

Chapter 3 explains the development trajectory of the novel physical model to simulate the overlap of passive zones. It describes the choices being made in the design process and the principles of centrifuge modelling. The soil characterization tests results and the interface characterization of the sand with the strongbox materials can be found in chapter 4.

The procedure of processing the physical model results and the evaluation of the test results are presented and discussed in chapter 5. The analysis of test results in this chapter lead to the intensification factor. Chapter 6 shows the differences between the restrained and unrestrained shear band development in the physical model as well as the differences in shear band shape. A
volumetric strain analysis for small wall displacements lead to an explanation about the results from
chapter 5.

In chapter 7, the numerical model and its assumptions are discussed. The numerical model is
compared with the physical model tests results from chapter 5 and chapter 6. The discussion of the
differences of the model results can be found in this chapter.

In the final chapter of this thesis, chapter 8, the general conclusions and findings are being
summarized. Recommendations can be found concerning different aspects of this thesis. Finally,
possible future research regarding the intensification effect is suggested.
2 Literature study

In the literature review mainly experimental studies regarding passive soil behaviour have been analysed and compared. The most relevant and important conclusions for this research are used for the design process, execution and interpretation of the centrifuge tests planned for the thesis. In Appendix A – literature review report, the complete study can be found. This chapter is a summary where the most important findings are highlighted.

2.1 Experiments related to passive earth pressures

Translating a wall into a soil body gives the most distinct and longest shear band in comparison with a rotating wall (Figure 2.1). This translating results therefore in the largest passive resistance of the soil because the largest soil mass is pushed. From the experimental studies, Lucia (1966), Arthur (1962) and Bransby (1968) it is not possible to deduce the dimensions and wall displacements. Hence no correlation can be found about how the type of wall movement determines the dimensions of the shape of the passive zone.

![Figure 2.1 Shear zones observed in experiments of passive mode with initially dense sand (radiographs and schematically): (a) during wall translation (Lucia, 1966) (b) during wall rotation around the top (Arthur, 1962) and (c) during wall rotation around the toe (Bransby, 1968) (O - rotation point)]

Soil density plays a major role in the amount of passive earth resistance. A dense soil tends to withstand more passive force in comparison with a loosely packed soil (Figure 2.2). Different experimental studies, for example by Stuit (1995) and Gutberlet et al. (2013), show that samples of (medium) dense sands can result the reduction of ultimate passive load at large displacements. This results in peak in the force vs. wall displacement diagram for dense sand shown in Figure 2.2. An important conclusion is that passive failure in (medium) dense sands is fully occurred when the monitored force has a decreasing trend or reaches a stable, plateau state.
The shape of the shear band will tend to be short, steep and straight in case of low wall friction. Loosely packed sand shows this behaviour as well. A (medium) dense sand with significant wall friction results in a curved and longer shear band. In case of relatively large wall displacement ($d/h^1 \geq 0.2$) a second shear band can occur due to the possible accumulation of soil close to the wall. The studies of Stuit and Gutberlet et. al learns that full passive failure of the soil body is expected between 0.05 and 0.10 $d/h$. This important finding is taken into account in the design phase of the physical model. More detailed information about the studies used in this chapter can be found in Appendix A.

![Figure 2.2 Load vs. wall displacement for homogenous loose ($R_d=0.29$) and dense ($R_d=0.99$) sand in a 1g test with a translating wall with 1.25 meter embedded length (Gutberlet et al., 2013)](image)

**2.2 Comparison numerical models and centrifuge research**

For the validation of the intensification factor it is important to control the forces and displacements in the physical model in the same way as it can be programmed in a numerical model. Failure initiation by means of forced displacement of the wall (conducted by Yang and Liu (2007)) is the best and most suitable method for the research goal in this thesis. The other option are to model the enhanced acceleration numerically (conducted by Yang et al. (2008)) is less suitable because the wall movement cannot be controlled. In chapter 3 of Appendix A more conclusions regarding the coupling of numerical and physical centrifuge models are explained.

**2.3 Other aspects of centrifuge modelling**

Several important aspects play a role during centrifugal testing of passive soil failure. Summarized lessons learned from literature are as follows:

- The shear band of a passive failure zone starts growing at the toe of the translating wall. After it reaches surface level a secondary shear band can occur (Lucia, 1966);
- The boundary effects of the side walls cause an increase in load necessary for passive failure. These 3D effects are significant and cannot be neglected because the centrifuge facility dimensions (width of the model) are limited. A possible solution by Gutberlet et al. (2013) to quantify the 3D effects of the side walls is to analyse the shear band at the top surface;

---

$^1 d/h$: wall displacement vs. wall height ratio
• The grain size diameter of the model sand has to be preferably identical as the prototype sand sizes according to Madabhushi (2014). This is mainly because of large differences in mechanical behaviour of the sand when the grain size is downscaled;

• To simulate the correct mechanical behaviour of the sand without the particle size effect it is recommended by Ovesen (1979) to have a minimum length of a model plate element of 30 times the main grain size diameter. Because this project involves loading the soil with a plate as well, this rule of thumb is valid for this project;

• A three dimensional discrete model developed by Tejchman et al. (2011) shows that 10 times larger diameter model grains will result in 30% extra passive forces in the model. The consequences for the physical centrifuge model in this thesis is that possibly a too large load will be monitored;

• The shear band thickness will develop up to 10 to 20 times the mean grain size in the sand sample of the centrifuge model (Stuit, 1995). This relationship does not change at different accelerations in the centrifuge.
3 Physical modelling

There are various categories of physical models. Full scale physical models do have the advantages that they model the situation with the correct stress levels. These type of tests have their limitations like reproducibility, costs and unknown effects. Another category of a physical model is the small scale test. In small scale testing all the parameters are more controlled and the experiment preparation time is less. This creates the opportunity to conduct a wider study to e.g. to the change of a specific parameter while all the other parameters are the same; a parametric study.

It is important to have the realistic stress levels in the model because soil is known to be a (non-linear) stress depended material. The confining stress greatly influences elasticity, the peak angle of internal friction and dilatancy. It is therefore erroneous to model with other stresses than the full-scale stresses (Bolton, 1986). With the geotechnical centrifuge facility, it is possible to have a (small scale) model with identical stress levels as the (full-scale) prototype.

Hence, it can be concluded that a small scale model in the geotechnical centrifuge is the best suitable method to help answering the main research goal. The repeatability, accuracy and the preparation times are the most decisive reasons for this choice. This chapter will explain the considerations regarding the experiment setup and the design.

3.1 Centrifuge modelling

There are beam and drum type geotechnical centrifuges all over the world with different dimensions and maximum accelerations. Beam centrifuges can have a large diameters up to about 8 meters, the TU Delft centrifuge however is a relatively small facility with a diameter of about 2.5 meter (Figure 3.1). The weakness of very large diameter beam centrifuges are the running costs and the model preparation. A drum centrifuge is often a smaller facility (see Figure 3.2). Models have to be placed in vertical position which can result in practical difficulties.

The geotechnical centrifuge facility that is going to be used in this research is the centrifuge facility of the Technical University in Delft. The centrifuge is a beam centrifuge build in 1990 and can work with models up to a weight of 300 N up to an acceleration of about 300g.

The strongbox with the experiment set-up is placed on a swing platform and will swing-up during the centrifuge flight (Figure 3.3). More specific information and application examples of the TU Delft geotechnical centrifuge is described by Allersma (1994).
During a centrifuge flight the rotation causes inertial stresses in the model. These stresses are meant to be approximately the same as the stresses in the prototype. Rotational speed ($\omega$) and the distance of the model to the centrifuge axis at that moment (radius, $r$) define the inertial stress (acceleration or g-level) and therefore the scale of the model. This relationship is shown in equation 3 and Figure 3.4.

$$ N \ g = r \ \omega^2 $$ (3)

Centrifuge scaling laws (Table 3.1) are the conversion rules between the centrifuge model and the prototype. More information about the scaling laws and centrifuge modelling is written by Taylor (1995) and Madabhushi (2014). In this thesis the scaling laws for dimensions and force are relevant.

**Table 3.1 Scaling laws (based on table from Madabhushi (2014))**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Scaling law Model/prototype</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>General scaling laws</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(slow events)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Length</td>
<td>$1/N$</td>
<td>m</td>
</tr>
<tr>
<td>Area</td>
<td>$1/N^2$</td>
<td>m$^2$</td>
</tr>
<tr>
<td>Volume</td>
<td>$1/N^3$</td>
<td>m$^3$</td>
</tr>
<tr>
<td>Mass</td>
<td>$1/N^3$</td>
<td>Nm$^{-1}s^2$</td>
</tr>
<tr>
<td>Stress</td>
<td>1</td>
<td>Nm$^{-2}$</td>
</tr>
<tr>
<td>Strain</td>
<td>1</td>
<td>-</td>
</tr>
<tr>
<td>Force</td>
<td>$1/N^2$</td>
<td>N</td>
</tr>
<tr>
<td>Bending moment</td>
<td>$1/N^3$</td>
<td>Nm</td>
</tr>
<tr>
<td>Work</td>
<td>$1/N^3$</td>
<td>Nm</td>
</tr>
<tr>
<td>Energy</td>
<td>$1/N^3$</td>
<td>J</td>
</tr>
<tr>
<td>Seepage velocity</td>
<td>N</td>
<td>Ms$^{-1}$</td>
</tr>
<tr>
<td>Time (consolidation)</td>
<td>$1/N^2$</td>
<td>s</td>
</tr>
<tr>
<td><strong>Dynamic events</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Time (dynamic)</td>
<td>$1/N$</td>
<td>s</td>
</tr>
<tr>
<td>Frequency</td>
<td>N</td>
<td>s$^{-1}$</td>
</tr>
<tr>
<td>Displacement</td>
<td>$1/N$</td>
<td>m</td>
</tr>
<tr>
<td>Velocity</td>
<td>1</td>
<td>ms$^{-1}$</td>
</tr>
<tr>
<td>Acceleration/ Acceleration due to gravity (g)</td>
<td>N</td>
<td>ms$^{-2}$</td>
</tr>
</tbody>
</table>
Figure 3.4 Inertial stresses in a centrifuge model induced by rotation about a fixed axis correspond to gravitational stresses in the corresponding prototype (Taylor, 1995)

In a full scale model the earth’s gravity is uniform over the soil depth, while in a small scale model in a centrifuge there is a slight variation in acceleration through the model due to the change in radius (Taylor, 1995). Hence there is an under-stress in the top part of the model and an over-stress in the bottom part of the model depending on the nominal radius \( r_e \) and the model height \( H \). Figure 3.5 illustrates the different definitions. Equation 4 shows the formula for the ratio of maximum under-stress and maximum over-stress.

\[
R_{\text{under}} = R_{\text{over}} = \frac{H}{6 * r_e} \tag{4}
\]

If a model height \( H \) of 75 mm is considered, the nominal radius in the TU Delft centrifuge with this model is 1140 mm. This results in an over/under stress ratio of 1.1%, which is considered not to have a significant effect on the comparison with the prototype stresses.
3.2 Model approach & design

In the numerical model of Hosseinzadeh and Joosse (2015), the authors calculate $K_{p,\infty}$ and $K_{p,w}$ by the effective horizontal stresses 0.1 meter from the translating wall with an embedded length of 5 meter. This distance is chosen to eliminate pure numerical effects and to correctly capture the horizontal stresses near the wall surface. Monitoring this horizontal stress physically is hardly possible in a small-scale model. What can be monitored accurately in a physical model is the total horizontal passive load ($E_{p,h}$) necessary to push a wall into a soil body (see equation 2).

A calibrated load cell can monitor the loads accurately during a certain wall displacement, therefore the certainty about the deviation or the repeatability of the influencing parameters ($y', \varphi', \delta, x, d$) is critical for a successful physical model.

In FEM software however, there is no such thing as the natural variation of the parameters. This implies that the effect of difference in one parameter can be studied. In physical modelling practice, due to laboratory conditions the quality of the comparability and repeatability is very high in comparison with the infield conditions but never 100%. Regarding the intensification effect, it means that the soil conditions and the dimensions in equation 1 can be repeated and therefore be compared. Chapter 4 will go further in detail about the soil conditions and the uncertainties of the parameters.

A list of requirements of the physical model has been set up before the design of the physical model. This list is mainly based on previous part and the assumptions in the paper written by Hosseinzadeh and Joosse (2015).

- In the physical model both unrestrained model ($w/d \geq 6.0$) as restrained model ($w/d = 0.5-3.0$) should be able to be submitted to testing;
- The soil characteristic parameters ($y', \varphi'$) and wall friction ($\delta$) need to be equal in the unrestrained and restrained model because they will be compared;
- 3D effects should be prevented as much as possible, because a 2D plane strain numerical model is going to be validated;
- It should be possible to monitor shear band development from the side and the top;
- It is essential to have a continuously monitoring of the load during the wall displacement;
- Horizontal translation of the wall is a requirement in this study (Hosseinzadeh and Joosse, 2015);
- The user of the model should be able to choose the dimensions ($w, d$) in the model, so that the desired $w/d$ ratio can be chosen;
- To exclude the friction effect of the base plate of the sand container on the shear band, the distance between the movable wall and this base plate need to be sufficient. A value of 0.5 times the embedded length of the model wall is considered to be enough;
- The dimensions of the complete setup cannot exceed the dimensions of the basket in the TU Delft centrifuge facility, the maximum size of the strongbox is about 420x150x400mm.

It is very common in centrifuge modelling to use a so called strongbox made up of stiff Plexiglas plates in combination with aluminium base and side parts. This strongbox is the container for the soil and is a very stiff construction to minimize effects due to bending of the walls during a centrifuge flight. This standard design of a strongbox needs to be customized to be able to model the intensification factor. Two different models are designed which are described in next subchapters.
3.2.1 Design 1
Design 1 (Figure 3.7) is custom designed to work with a load frame fixed in the centrifuge basket above a conventional strongbox. The horizontal actuator of this load frame is connected with a vertical strip (A) to the model sheet pile wall (B). This model sheet pile wall moves between two guiding beams to guarantee pure horizontal wall displacement (translation). Because the actuator is limited in 1 direction, the overlap of passive zones need to be modelled symmetrically. This implies a wall without friction, that has essentially the function of a mirror or symmetrical axis. Several different restrained model ratios do need to be tested, therefore the position of this wall is adjustable. In this way the user can easily model unrestrained passive behaviour and various restrained situations.

![](image.jpg)

*Figure 3.7 Design 1, the loading frame (not visible) is connected to the vertical strip (A) and causes the model wall (B) to move into the soil in the container. The sand container can vary in width due to the adjustable symmetric wall (C)*

Due to practical considerations, the load in this setup will be measured with calibrated strain gauges placed on the vertical strip (A) instead of a load cell. The bending of the vertical strip is linear related to the load. A built-in linear variable differential transformer (LVDT) monitors the wall displacement in this setup. Some of the advantages of this setup design are: recycling of an old strongbox, using a working loading frame. A disadvantage of this design is that every test this setup and the loading frame need to be reconnected exactly the same. Another downside is that the it is not known that the system with the bending vertical strip can fully transfer the load of the frame to the wall.

3.2.2 Design 2
Design 2 is a modified design of the bulldozer test setup designed and used by Stuit (1995). This setup (Figure 3.8) consists of a horizontal actuator which can replace a wall of the strongbox. The setup has been proven to work up to 300g and has a maximum load capacity of 5000 N (Stuit, 1995). With the pulse wheel the wall displacement and velocity can be monitored and controlled accurately. A calibrated load cell in the actuator directly behind the model sheet pile wall guarantees accurate monitoring of the loads. Detailed drawings of the actuator and its parts can be found in Appendix D.
Figure 3.8 Horizontal actuator used for ‘bulldozer tests’ by Stuit (1995)

The strongbox designed for this actuator has a similar feature of design 1: the easy adjustable symmetric wall. This feature is necessary because there is only one actuator available and no space in the basket for two actuators. Because of the unique dimensions of the actuator no existing strongbox can be used. Design 2 is shown in Figure 3.9 and is made tailor fit for the actuator used for ‘bulldozer tests’. The dimensions of the strongbox are chosen based on the requirements regarding the maximum dimensions of the centrifuge.

Figure 3.9 Design 2, the horizontal actuator (Figure 3.8) can be fitted in this strongbox on the left side. The symmetric wall (A) is connected to the right wall and can be adjusted with nuts to test different unrestrained and restrained models.

3.2.3 Final design

The high capacity actuator used by Stuit made the decisive choice for design 2 as the best physical model to simulate the overlap of passive zones. According to analytical and numerical calculations a load of between 1000 and 2000 N is needed at 100g to translate the wall into the sand. The actuator of design 2 has proven in the past that it was able to generate this load (Stuit, 1995). The loading frame used in design 1 has not been used for this specific purpose and is therefore less reliable. In Figure 3.10 and Figure 3.11 the physical model is shown with the symmetric wall in its unrestrained and restrained position with and without sand.

Shown in Figure 3.11 are the camera systems which can capture images during a centrifuge test. The images of the sidewall of the model are made with the high performance industrial GigE camera (A in Figure 3.11 ) which is directly connected to the centrifuge control room. When the centrifuge is rotating, the user can monitor the wall translation via both the pulse wheel output and the camera footage. The top view camera, a GoPro Hero 4 (B in Figure 3.11) is however not directly connected to the computer because of practical considerations.
Figure 3.10 Design 2, strongbox with horizontal actuator and adjustable symmetric wall in unrestrained position (A). Visible are the electro motor (B), pulse wheel (C) and the load cell (D).

Figure 3.11 Design 2, mounted in the centrifuge basket in restrained position with sand sample, cameras are indicated with A and B. The black dots on the Plexiglas are the calibration points used for the image analysis.
3.3 GeoPIV

Monitoring the shear band development by visualization is important in this thesis to explain and to control the possible differences in the test results in the unrestrained and the restrained model. Another advantage is that the visualization of the cross-section of the model can give more information about the differences between a numerical model and a physical model.

Validation experiments by White et al. (2003) shows that a deformation measurement system based on particle image velocimetry (PIV) and close-range photogrammetry gives a high precision in comparison with previous image-base deformation methods like target markers or grid lines. Hence a PIV logarithm which is called geoPIV and developed by White and Take (2002) will be used as a tool to visualize deformations and strains. Figure 3.12 shows the steps of a PIV analysis clearly explained by Stamhuis (2006). Every resultant vector is defined in a matrix with its coordinates and its magnitude.

![Figure 3.12](image.png)

Figure 3.12 “Diagram of the steps in PIV analysis of successively recorded particle patterns in a flow: two sub-images from the same location of two frames are compared in a cross-correlation procedure, resulting in a 2D probability density distribution which shows a peak at the most probable displacement. The peak is located with high precision and a velocity vector, representing the average displacement of the particles in sub-image 1 compared to sub-image 2 is calculated.” (Stamhuis, 2006)

With calibration points on the Plexiglas of the physical model, the image space pixel coordinates can be converted to object space coordinates (Figure 3.13). With this conversion deformations and shear strains are changed to real world dimensions and can therefore be compared to i.e. the results of a numerical study.
3.4 Limitations and discussion

Every physical model has its limitations, so does the model described in this chapter. The most outstanding limitation is the fact that modelling a 2D plane strain situation is hardly possible. The friction effect of the Plexiglas will influence the load monitored in the load cell. A correction for this friction load will therefore be necessary before the results can be analysed. The Plexiglas friction load also has the consequence that shear band development just behind the wall is not exactly representing the shear band in the 2D plane strain model. The grains at the Plexiglas wall will move less in comparison with the grains in the centre of the model due to the friction force.

The ‘human factor’ during the preparation, performing and processing of a test is almost impossible to eliminate. For the planned series of test in this thesis a checklist has been setup to reduce the risk of the human factor which can result in large differences in the execution steps of tests. The full checklist can be found in Appendix E.
4 Sand and interface characterization
4.1 Soil characteristics
Good control of the soil characteristics is one of the largest advantages of physical modelling in a laboratory environment. It gives the opportunity to better study specific geotechnical phenomena with less concern about the uncertainties of the soil conditions outside the laboratory in the field. This subchapter contains the characterization of the sand used for all the physical modelling in this thesis.

4.1.1 Soil type & Grain size distribution
The type of soil used for all the experiments in this thesis is fine sand. A small desk study (Table 7, Appendix B) shows that in the western part of the Netherlands the sand is relatively fine grained at the depth of interest between 5 and 10 meter under surface. The average grain size is about 250 \( \mu \text{m} \), and the uniformity coefficient about 2. This implies a relatively narrow (uniform) grain size distribution.

A batch of naturally rounded river sand is composed, which matches the average grain size and uniformity in the western part of the Netherlands. The amount of quartz in this batch is more than 96%. The sand is called fine Delft Centrifuge sand (DC-sand). As argued in chapter 2, the dimensions of the grains will not be subjected to the scaling laws because it will cause a significant change in behaviour.

The grain size distribution curve is shown in Figure 4.1, it is the average of three sieve tests conducted on the batch according to BS:410 and the geo-engineering lab manual (Verwaal, 2006). There are more different sieve sizes used than according to the standard, this to increase the accuracy of the grain size distribution chart. The particle diameter plotted on the x-axis is as usual on a logarithmic scale.

![Figure 4.1 Grain size distribution chart of the DC-sand (fine) used for sample preparation in this thesis](image)

From the grain size distribution chart the grain size characteristics can be identified. They are shown in Table 4.1. The results of the sieve tests on the fine DC-sand can be found in table 1, 2 and 3 of Appendix B.
Table 4.1 Grain size characteristics

<table>
<thead>
<tr>
<th>$D_{50}$ [μm]</th>
<th>280</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_u (D_{60}/D_{10})$ [-]</td>
<td>1.7</td>
</tr>
</tbody>
</table>

4.1.2 Density

The minimum and maximum density of the DC fine sand batch is determined with the Japanese method (Japanese Geotechnical Society, 1996) which is elaborated in the geo-engineering lab manual (Verwaal, 2006). In Table 4.2, the average values of the dry unit weight and the porosity are displayed. In total 10 density tests are conducted with a circular metal mold with a volume of 112.3 cm$^3$, the full results are shown in Table 4 and 5 of Appendix B.

Table 4.2 Density parameters fine sand

<table>
<thead>
<tr>
<th></th>
<th>Minimum density</th>
<th>Maximum density</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry unit weight, $\gamma$ [kN/m$^3$]</td>
<td>14.13</td>
<td>16.63</td>
</tr>
<tr>
<td>Porosity [%]</td>
<td>46.7</td>
<td>37.2</td>
</tr>
</tbody>
</table>

Following the scope of this thesis a relative density of about 50% (medium dense sand) is preferred. The medium dense sand corresponds with a unit weight of 15.38 kN/m$^3$. A known to be consistent and convenient method to prepare homogeneous samples is by dropping the grains from a fixed height, known as dry pluviation. Multiple falling height tests are conducted with four different pipe lengths. The test data is shown in Figure 4.2 together with the maximum and minimum densities from the density tests previously described. The graph shows that a relative density of maximum 60-70% can be reached by increasing the falling height of the grains. Larger relative densities of the sand sample can be achieved due to vibrating. 65 centimetres dry pluviation is chosen for all tests in this thesis. The variation in relative density is about 10% for the a mold with a volume of 112.3 cm$^3$.

![Figure 4.2 Measured densities (y-axis) plotted with their corresponding falling height (x-axis). Shown are the maximum and minimum density test results (from Table 4.2), and the trend line through the data.](image-url)
4.1.3 Shear strength tests

Soil shear strength is a crucial strength parameter in engineering practice, because it indicates the capacity of the soil against shearing. Because shearing is the dominant failing mechanism within this thesis it is studied extensively in a laboratory environment.

A common method to test the shear strength of dry sand is the direct shear box. Figure 4.3 shows an impression of the cross section of the direct shear box. During the direct shear test the normal force (N) is a constant load acting as a confining stress on the sample. While the shearing is induced by an actuator, both horizontal and vertical displacements are measured together with the shear force (T). Equations 5 and 6 show the formulas for calculating the shear stress and the normal stress. Because the area of shearing is decreasing during a test, the corrected area (A_c) need to be taken into account.

\[
\text{Shear stress; } \tau = \frac{T}{A_c} \quad (5)
\]

\[
\text{Normal stress; } \sigma_n = \frac{N}{A_c} \quad (6)
\]

The expected results of the direct shear tests for loose, medium and dense sands are shown in Figure 4.4 and Figure 4.5. Despite the shortcomings of the direct shear test such as a predefined (horizontal) shear plane and a non-uniform stress distribution in the sand sample, the strength parameters can be obtained. It is trivial that a dense grain skeleton can resist more shear stress in comparison with a loosely packed sand. This is the so called effective ‘peak’ shear strength (\(\phi'_{\text{peak}}\)).

However, both Figure 4.4 and literature (Bolton (1986)) argue that for dense and loose sand the shear stress will reach the same ultimate shear strength (\(\phi'_{\text{residual}}\)) at relatively large shear displacements. This is the result of full failure of the (dense) grain skeleton. When a shear band is fully developed, it characterized by a thin weak zone which is normative for the complete soil strength. The occurrence of this softening phenomena is supported by the positive volumetric change (dilatancy) of the sample. Considering the large shear displacements at passive failure due to the actuator in the physical model, the residual shear strength (\(\phi'_{\text{residual}}\)) is normative for all densities.

The direct shear tests conducted at the batch of sand for this thesis are done at normal stresses between about 30 and 110 kPa. This stress range correspond with the to be expected confining stresses in the physical model, 5 meter of medium dense DC-sand is about 75 kPa. All the tests are
conducted according to British Standards (1990) procedures. Three different densities are tested: loose, medium dense and dense sand. The medium dense samples are prepared identical to the density tests: dry pluviation of 65 centimetres. The loose and dense samples are prepared according to the Japanese method (Japanese Geotechnical Society, 1996), explained in previous subchapter.

The shear rate of all the direct shear tests with sand is set to 1 mm/min according to the geotechnical lab manual (Verwaal, 2006). Some tests are repeated to check the repeatability and its deviation. Other tests are conducted at lower shear rate (0.1 mm/min) to study if there is a difference. From these control tests at lower speeds it can be concluded that there is no significant difference in ultimate shear stress (see figure 2, Appendix B). A list of all the direct shear tests and the shear stress vs. displacement results graphs are listed in Appendix B.

In Figure 4.6 the peak shear stress points are plotted against the corresponding normal stress points. The shear stress and normal stress are not corrected for the changing shearing area during a test, because in the Mohr-Coulomb formula the correction factor drops out. This is because it is applied in both normal stress and shear stress (Bardet, 1997).

![Figure 4.6 Shear failure points (peak values) of the direct shear tests for different sand densities](image)

The angle and the start value of the trend lines through the data points of loose sand tests is used to obtain the effective residual angle of internal friction ($\varphi^{\prime}_{\text{residual}}$) and the effective cohesion ($c^{'}$) according to the Mohr Coulomb failure criterion theorem (equation 7).

$$\tau_f = c^{'} + \sigma_f^{\prime} \cdot \tan \varphi^{'}$$  \hspace{1cm} (7)

The maximum shear stress ($\tau_f$) and the corresponding effective normal stress ($\sigma_f^{\prime}$) for medium and dense sand is used to define the peak strengths at their specific normal stresses. The peak strengths of sand cannot be derived from the results without naming the confining stress (Bolton, 1986). Direct
shear results of loose sand at various normal stresses however, define the residual failure plane in
the Mohr Coulomb failure criterion theorem.

According to Thiel (2009), the interpretation of the effective friction angle ($\phi'$) should be only based
on the measured values. Knowing that dry sand has no cohesion and that most shear strength
envelopes are truly curved (nonlinear) (Thiel, 2009), only the angle of the linear trend line through
the loose sample points defines the effective residual friction angle ($\phi'_{residual}$). The shear strength
values obtained from the direct shear tests can be found in Table 4.3.

Table 4.3 Strength parameters sand obtained from direct shear tests

<table>
<thead>
<tr>
<th></th>
<th>$c'$ [kPa]</th>
<th>$\phi'_{peak}$ (30.4 kPa) [degrees]</th>
<th>$\phi'_{peak}$ (76.5 kPa) [degrees]</th>
<th>$\phi'_{peak}$ (107.3 kPa) [degrees]</th>
<th>$\phi'_{residual}$ [degrees]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose sand (RD 0%)</td>
<td>[-]</td>
<td>[-]</td>
<td>[-]</td>
<td>[-]</td>
<td>31.4</td>
</tr>
<tr>
<td>Medium dense sand (RD 50%)</td>
<td>[-]</td>
<td>40.5</td>
<td>37.9</td>
<td>37.2</td>
<td>31.4</td>
</tr>
<tr>
<td>Dense sand (RD 100%)</td>
<td>[-]</td>
<td>51.7</td>
<td>47.3</td>
<td>46.5</td>
<td>31.4</td>
</tr>
</tbody>
</table>

The effective angle of internal friction is derived from triaxial test results as well. Although the main
goal of the triaxial tests was to obtain the $E_{50}$ the results can be used to validate the direct shear
results. Due to practical reasons a confining pressure of 100 kPa is used for the triaxial tests. With
the assumption that the cohesion is zero, the Mohr-Coulomb failure criteria results in a peak angle of
internal friction of 36.2 degrees for 5 triaxial tests with medium dense sand. Because the shear
behaviour in the physical model is more like in the direct shear test (plane strain) than in the triaxial
test, the values from Table 4.3 are used in this thesis.

4.1.4 Dilatancy

Dilatancy is the result of the positive volume change in the shear zone due to the change from
(medium) dense packing to loose packing. Shearing is a key element in this research to passive soil
capacity and therefore dilatancy plays a role in the physical model of this thesis.

The maximum angle of dilation occurs at the point where the peak shear stress changes to the
residual shear stress (see Figure 4.4 and Figure 4.5). The maximum dilatancy angle can be directly
obtained from the direct shear results. In Figure 4.7 the vertical displacement is plotted against the
horizontal displacement. The average (maximum) dilatancy angle of six different tests with medium
dense sand is determined to be 8.7 degrees.
4.1.5 Young’s modulus

The Young’s modulus at 50% of the failure stress at the reference confining pressure of 100 kPa, the $E_{50}$ value is determined with triaxial testing. Triaxial testing is the most common test to obtain the elasticity modulus of soil. The vacuum triaxial test is used for this series of tests, the sample preparation results in a homogeneous dry sand sample. The capacity of the vacuum pump creates a steady confining pressure of nearly 100 kPa. The axial stress is applied with a UCS apparatus.

The complete triaxial test results can be found in Appendix B. In the stress-strain graph to obtain $E_{50}$ (figure 6, appendix B) several test results are corrected for the (already applied) pre-pressure while setting up the UCS apparatus. The axial stresses are not corrected for the increase in area during the triaxial tests because the change in peak stress will be little and will not affect the $E_{50}$ value significantly. The derived values from the stress-strain graph can be found in Table 4.4, the average elasticity ($E_{50}$) of medium dense sand is 30.3 MPa.

<table>
<thead>
<tr>
<th>Test</th>
<th>Peak stress [GPa]</th>
<th>stress at 50% [GPa]</th>
<th>m strain at 50% [-]</th>
<th>$E_{50}$ [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.30</td>
<td>0.15</td>
<td>4.5</td>
<td>32.8</td>
</tr>
<tr>
<td>2</td>
<td>0.30</td>
<td>0.15</td>
<td>5.6</td>
<td>26.4</td>
</tr>
<tr>
<td>3</td>
<td>0.28</td>
<td>0.14</td>
<td>4.4</td>
<td>31.3</td>
</tr>
<tr>
<td>4</td>
<td>0.28</td>
<td>0.14</td>
<td>5.0</td>
<td>28.0</td>
</tr>
<tr>
<td>5</td>
<td>0.30</td>
<td>0.15</td>
<td>4.5</td>
<td>33.3</td>
</tr>
<tr>
<td></td>
<td><strong>Average</strong></td>
<td></td>
<td><strong>30.3</strong></td>
<td></td>
</tr>
</tbody>
</table>

4.2 Interface characteristics

In the novel physical model in this thesis there is interaction between several interfaces and the soil. It is important to quantify those effects to conclude on the magnitude of the side effects. In a typical strongbox made up for centrifuge testing two walls are made of glass or Plexiglas and side walls and the base are often made up of aluminium. The roughness and the scratch hardness value of those
materials causes the friction between the sand and the material in case of shear displacement. This subchapter is about the quantification of the unwanted friction and the designed wall friction, $\delta$.

4.2.1 Soil - steel friction

There are two goals with testing the friction between steel and sand. The first goal is to obtain the roughness which matches with the relationship of steel sheet pile walls and sand so that the model wall friction represents reality. According to the literature study the ratio between the peak angle of internal friction of the sand ($\phi_{\text{peak}}$) and the angle of friction of the wall ($\delta$) is around 2/3. The second goal of the tests will be to quantify the lowest friction angle possible with steel with a very smooth surface. This will be applicable to the desired frictionless wall in the experiment which represents the symmetrical line of the model.

The experiment setup for the sand steel interaction is almost the same as for a standard direct shear test. The lower half of the sample container in the direct shear box is replaced by a steel plate with a certain roughness value. The roughness is applied on the steel plate with electrical discharge machining, so a very accurate and constant roughness can be created.

Because of the scope of this project the medium dense DC-sand sample is used for all the tests. The list of all the direct shear tests with steel is included in Appendix C, as well as all the shear stress vs horizontal displacement graphs. Figure 4.8 shows the residual shear stress points with the corresponding normal stresses. Generally, there is no peak value in these direct shear tests results with steel. Only in the test series with the steel roughness of 8.6 $\mu$m there are peak values visible at small displacements. Like the direct shear tests in sand, no area correction factor is used for the stresses, this because the area is not reduced at all during these tests series.

![Figure 4.8 Residual failure points direct shear tests sand for different steel roughness values](image-url)
The angle of the trend line gives the (wall) friction parameters according to the Mohr-Coulomb failure criterion. In the Table 4.5 the wall friction angles corresponding to their steel roughness are shown. Figure 4.9 shows the relation between the steel roughness value and the wall friction ratio.

Table 4.5 Results direct shear tests steel

<table>
<thead>
<tr>
<th>steel roughness [μm]</th>
<th>Wall friction (δ) [degrees]</th>
<th>δ/φ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.27</td>
<td>11.5</td>
<td>0.32</td>
</tr>
<tr>
<td>2.4</td>
<td>15.2</td>
<td>0.43</td>
</tr>
<tr>
<td>5.3</td>
<td>22.9</td>
<td>0.64</td>
</tr>
<tr>
<td>8.6</td>
<td>30.7</td>
<td>0.86</td>
</tr>
</tbody>
</table>

Figure 4.9 Relation between steel roughness (type B) and wall friction

In Figure 4.9 it is shown that the desired ratio δ/φ = 2/3 is obtained with a steel roughness of about 6 μm. Because of practical reasons the roughness type of the steel blocks previously described and tested was not possible to apply in the physical model. Therefore, a new testing block is created with a different type of roughness. A validation test has been done with a horizontal alignment of the roughness of about 6 μm (Type A, Figure 4.10) instead of the ‘random’ roughness in the tests mentioned previously (Type B, Figure 4.10).

Figure 4.10 Steel roughness type A (left) and type B (right)

These validation test did not show a significant different in friction angle for both types. A wall friction angle (°) of 22.7 degrees is derived from the direct shear results. This type of steel roughness and its corresponding friction angle will be used in the physical model of the sheet pile wall. It has to be noted that a different soil batch with other grain sizes could lead to significant different wall friction, this because the shape and size of the grains are of major influence of the shearing characteristics.
The other conclusion from these test series is that a wall roughness of 1.27 μm and a polished surface will not give the desired low friction angle. Their friction angles are respectively 11.5 and 12.3 degrees. All direct shear result to test the sand-soil interaction can be found in Appendix C.

Tatsuoka et al. (1984) suggest a layer of silicone grease and on top a latex membrane as a measure against friction, shown in Figure 4.11. Because of the viscosity of the silicone grease, even under high confining stresses the granular material slides over the metal surface. Two direct shear tests are conducted with respectively 338 kPa and 676 kPa normal stress. Both tests showed a very low shear stress of between 2-3 kPa. This implies a friction angle of between 0.25 and 0.34 degrees for the friction of sand vs. latex-silicone grease for those confined pressures.

Figure 4.11 Schematization of layers of the frictionless wall

To show the effectiveness of the combination of layers of latex and silicone grease, an evaluation is made between the shear band shapes of the physical model without (test E,3.0,100g) and with (test H,3.0,100g) the frictionless solution. The incremental deviatoric strain distribution from 7 to 8 mm wall displacement is shown in both tests in Figure 4.12. The characteristics of the tests can be found in Table 5.1 and the explanation about the deviatoric strains in Chapter 6.1.

The friction of (polished) steel of the symmetric wall (δ = 12.3 degrees) causes the shear band from interacting with the wall. In the model with the frictionless solution, the shear band reflects on the symmetric wall. This reflection indicates that this frictionless solution is effective and that the symmetric wall truly works as a mirror.

Figure 4.12 Comparison of the shear band shape in case of 12.3 degrees friction on the symmetric wall or no wall friction on this wall

4.2.2 Soil - Plexiglas friction

The friction between sand and Plexiglas is a more complex phenomenon in comparison with the friction of steel. This has two reasons, firstly the hardness of Plexiglas and secondly the already existing scratches on the Plexiglas. The hardness of the Plexiglas may cause an issue because at the
higher confining stresses the sand might penetrate more into the Plexiglas, which gives a relatively higher peak shear stress in comparison with the lower normal stresses. The second reason is that the more the Plexiglas is used the higher friction it will give due to increase of scratches.

The direct shear test apparatus is used again to measure the friction; the lower half of the sand container is filled with a sample of Plexiglas. For all the tests the Plexiglas is orientated the same, the scratches are therefore in the similar direction, which will be the occasion in the physical model. The testing starts with a smooth, non-scratched piece of Plexiglas.

Because of the scope in this project the medium dense fine DC-sand is used for all the tests. The list of all the test with Plexiglas is included in Appendix C. The test in the list are consecutive in time, so the higher the number the higher the effect of scratches should be.

In Figure 4.13 the failure shear stress points and their corresponding normal stresses are plotted based on 11 direct shear tests. A distinction is made in Figure 4.13 between the peak failure shear stress points and the (average) plateau failure points at large shear displacements. In the physical model of the overlapping of passive zones the shear displacements will be relatively large. Hence the lower residual shear stresses will be taken into account during the derivation of the friction value.

![Figure 4.13 Shear failure points direct shear tests sand and Plexiglas](image)

The angle of the trend line through the residual shear stress points gives an angle of wall friction of 9.5 degrees. The effect of the increase of scratches during the shear tests is not significantly. At a normal stress of 76.5 and 122.9 kPa the increase of shear stress after each test is about 0.5 kPa and 1.3 kPa. After 11 shear tests with the Plexiglas sample the scratches can be seen on the surface but are small and not deep. Hence the increase of friction after each test will be neglected during this study.
5 Physical model results

In total 20 centrifuge tests have been performed in the test series of this thesis. All the tests characteristics can be found in Table 5.1. All tests have been repeated at least once to check the repeatability of the physical model.

Test C,4.0,100g until J,3.0,100g have been performed to study the effect of wall friction on the symmetric wall. Test H,3.0,100g showed the lowest load results, therefore this type of membrane on the symmetric wall is used for all further test. Some tests have been rejected because the electro motor of the actuator stopped working at 100g before the desired ultimate passive failure state has been reached. Hence, the last three centrifuge tests are performed at 80g. The following test results will be discussed in this chapter:

- 2 unrestrained model tests (A,6.4,100g & B,6.4,100g);
- 2 restrained model (w/d=3.0) tests (H,3.0,100g & J,3.0,100g);
- 3 restrained model (w/d=2.0) tests (K,2.0,100g & L,2.0,100g & T,2.0,80g);
- 2 restrained model (w/d=1.5) tests (R,1.5,80g & S,1.5,80g);
- 2 restrained model (w/d=1.0) tests (N,1.0,100g & O,1.0,100g).

Table 5.1 Overview experiments and their characteristics

<table>
<thead>
<tr>
<th>No.</th>
<th>Test name</th>
<th>Date</th>
<th>Time</th>
<th>Acceleration</th>
<th>w/d</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>A,6.4,100g</td>
<td>16-9-2015</td>
<td>9:30</td>
<td>100g</td>
<td>6.4</td>
<td>66% Unrestrained test</td>
</tr>
<tr>
<td>2</td>
<td>B,6.4,100g</td>
<td>16-9-2015</td>
<td>14:30</td>
<td>100g</td>
<td>6.4</td>
<td>65% Unrestrained test</td>
</tr>
<tr>
<td>3</td>
<td>C,4.0,100g</td>
<td>17-9-2015</td>
<td>14:30</td>
<td>100g</td>
<td>4.0</td>
<td>59% Without frictionless ‘symmetric wall’</td>
</tr>
<tr>
<td>4</td>
<td>D,4.0,100g</td>
<td>21-9-2015</td>
<td>11:30</td>
<td>100g</td>
<td>4.0</td>
<td>64% Without frictionless ‘symmetric wall’</td>
</tr>
<tr>
<td>5</td>
<td>E,3.0,100g</td>
<td>21-9-2015</td>
<td>14:45</td>
<td>100g</td>
<td>3.0</td>
<td>58% Without frictionless ‘symmetric wall’</td>
</tr>
<tr>
<td>6</td>
<td>F,3.0,100g</td>
<td>22-9-2015</td>
<td>15:30</td>
<td>100g</td>
<td>3.0</td>
<td>58% Without frictionless ‘symmetric wall’</td>
</tr>
<tr>
<td>7</td>
<td>G,3.0,100g</td>
<td>23-9-2015</td>
<td>12:30</td>
<td>100g</td>
<td>3.0</td>
<td>60% Frictionless ‘symmetric wall’ type 1²</td>
</tr>
<tr>
<td>8</td>
<td>H,3.0,100g</td>
<td>24-9-2015</td>
<td>10:00</td>
<td>100g</td>
<td>3.0</td>
<td>57% Frictionless ‘symmetric wall’ type 2³</td>
</tr>
<tr>
<td>9</td>
<td>I,3.0,100g</td>
<td>24-9-2015</td>
<td>14:30</td>
<td>100g</td>
<td>3.0</td>
<td>63% Frictionless ‘symmetric wall’ type 3⁴</td>
</tr>
<tr>
<td>10</td>
<td>J,3.0,100g</td>
<td>28-9-2015</td>
<td>11:15</td>
<td>100g</td>
<td>3.0</td>
<td>70% Frictionless ‘symmetric wall’ type 4⁵</td>
</tr>
<tr>
<td>11</td>
<td>K,2.0,100g</td>
<td>28-9-2015</td>
<td>15:45</td>
<td>100g</td>
<td>2.0</td>
<td>47%</td>
</tr>
<tr>
<td>12</td>
<td>L,2.0,100g</td>
<td>29-9-2015</td>
<td>10:30</td>
<td>100g</td>
<td>2.0</td>
<td>52%</td>
</tr>
<tr>
<td>13</td>
<td>M,2.0,100g</td>
<td>30-9-2015</td>
<td>10:00</td>
<td>100g</td>
<td>2.0</td>
<td>50% Motor stops before passive failure</td>
</tr>
<tr>
<td>14</td>
<td>N,1.0,100g</td>
<td>30-9-2015</td>
<td>13:45</td>
<td>100g</td>
<td>1.0</td>
<td>52% Motor stops before passive failure</td>
</tr>
<tr>
<td>15</td>
<td>O,1.0,100g</td>
<td>1-10-2015</td>
<td>9:30</td>
<td>100g</td>
<td>1.0</td>
<td>41% Motor stops before passive failure</td>
</tr>
<tr>
<td>16</td>
<td>P,1.5,100g</td>
<td>1-10-2015</td>
<td>12:00</td>
<td>100g</td>
<td>1.5</td>
<td>57% Motor stops before passive failure</td>
</tr>
<tr>
<td>17</td>
<td>Q,1.5,100g</td>
<td>1-10-2015</td>
<td>15:45</td>
<td>100g</td>
<td>1.5</td>
<td>65% Motor stops before passive failure</td>
</tr>
<tr>
<td>18</td>
<td>R,1.5,80g</td>
<td>2-10-2015</td>
<td>11:00</td>
<td>80g</td>
<td>1.5</td>
<td>54%</td>
</tr>
<tr>
<td>19</td>
<td>S,1.5,80g</td>
<td>2-10-2015</td>
<td>12:30</td>
<td>80g</td>
<td>1.5</td>
<td>52%</td>
</tr>
<tr>
<td>20</td>
<td>T,2.0,80g</td>
<td>2-10-2015</td>
<td>14:15</td>
<td>80g</td>
<td>2.0</td>
<td>69%</td>
</tr>
</tbody>
</table>

² Latex membrane, cut into two separate membranes
³ Latex membrane, uncut sheet
⁴ Latex membrane, same as type 1 but with overlapping membranes
⁵ From test J, type 2 has been used for all following tests
5.1 Processing the physical model results

The calibrated load cell and the pulse wheel in the actuator are connected to the computer via a wireless data transfer system. The load results cannot be used without an offset correction and a friction correction. The wall position number, measured by the pulse wheel, need to be corrected as well because of mechanical reasons. This sub-chapter explains the offset corrections and friction corrections on the measured data.

5.1.1 Load offset

The first phase is to convert the measured load during wall translation into positive, this is called the 'sign correction'. Secondly, bending of the load cell due to the acceleration forces results in a negative load. Because this load is constant and acts during the complete test it can be added to all results. The load offset is monitored just before the load cell makes full contact with the model wall. For the majority of the experiments an offset correction interpreted of +50 N. Table 1 in appendix F shows a list with load offset corrections for each test. Some tests do not require load offset correction, because the load cell already did have full contact with the model wall before the test.

An example of the sign correction (red dashed line) and the offset correction (red line) is given in Figure 5.1 where the load vs. wall position of test A,6.4,100g is shown. The first 800 μm wall displacement of the test is displayed. In the plot the wall moves from position right to left, indicated with the arrow.

![Figure 5.1 Unprocessed & processed load results test A,6.4,100g](image)

5.1.2 Wall position offset

The wall position offset is applied in three separate stages. Firstly the data without significant load increment is discarded by changing the position of first significant measured load (50N) to 0 μm (red line in Figure 5.1 to red line in Figure 5.2). The free space between the load cell and the model wall is the reason why the load does not change the first 500 μm wall displacement in Figure 5.1.

Secondly the sign of the wall position data is changed to positive. Thirdly the position increment between about 40 and 150 μm is discarded, because there is no load increment there (green solid line, Figure 5.2). This position offset correction at a load of about 150 N varies per test and can be found in the overview of all corrections in appendix F, table 1. The reason for this offset is most likely because the actuator in the setup has some free space in the mechanical parts when the load in increasing from 100 N to 200 N.
5.1.3 Correction for Plexiglas vs. wall (PW) friction

The stresses acting on the Plexiglas during a centrifuge test is causing bending in this wall. Even though the bending appears to be very little and the distance between the Plexiglas and the translating wall is very small, grains will intrude in the gap of this interface. In Figure 5.3 this interface is marked with a blue square, some of the intruded grains are indicated with arrows.

The amount of this PW friction can be monitored by retracting the wall without sand in the strongbox at 100 or 80g depending on the test. If the sand is not removed, active earth pressure will be measured. The friction caused by the intruded grains and the friction between all the moving parts are measured by the load cell. This means that the friction of the system itself is included in this value.

The friction value of the actuator and the system is measured by monitoring the load before the first tests with sand at 100g. Therefore absolutely no influence of sand grains is measured, the results are shown in Figure 5.4. The graph shows the loads of three cycles of 5 mm retraction and pushing of the wall. Between 150 and 200 N is measured for the friction of the system itself only for both pushing and retraction.
In Figure 5.5 results of PW friction tests are shown where grain intrusion is included. Please note that the wall moves virtually from right to left in this graph during the retraction test. The peaks visible are caused by static friction or reorientation of the grains. Therefore, the PW friction will be taken as an average of the residual friction loads. Given the variation in results and the uncertainty in the load offset, it is decided by the author to set this friction load to 250 N for all 100g tests. There is no clear correlation between the sequence of the tests and a possible increase of the PW friction.

Because there is only one PW retraction test done at 80g, the uncertainty is much higher. The results of this test appears to be quite different compared to the tests at 100g, it does not reach a residual friction (stable value) for example. The scratching history of the Plexiglas and the lower acceleration during this test can be a reason of this difference. At a lower acceleration more grains can intrude at the interface below the translating wall. Given these test results it is decided to set this friction load to 350 N for all 80g tests.

After every single test performed in this thesis, the interface between the Plexiglas and the wall is cleaned from grains. Therefore, it can be assumed that at position 0 there is no PW friction active. Another assumption is that the friction reaches the maximum load when full passive failure is occurred in the unrestrained model. It is assumed that the friction increases linearly together with the wall displacement. Figure 5.7 visualizes how the PW friction decreases the model load of test A,6,4,100g. At 6000 μm wall displacement the PW friction correction is a constant value of 250 N.
5.1.4 Correction for Plexiglas vs. Soil (PS) friction

In the physical model, the soil shears over the Plexiglas interface during the wall displacement. As a result of this, the total load monitored by the load cell in the setup has to be corrected for the PS friction as well. This friction will cause a difference in load-displacement results if the 2D model has to be studied. Therefore, correct elimination of this PS friction is important.

The maximal horizontal stress against the Plexiglas occurs during full passive failure of the soil. The horizontal force of the actuator on the translating wall is causing a passive reaction of the soil which results in a horizontal stress ($\sigma'_h$) in the wall direction. However the perpendicular horizontal stress in the direction of the Plexiglas ($\sigma'_{h,\text{pl}}$) acts at rest ($K_0$), because the Plexiglas does not move relative to the soil body and active ($K_a$), because in the centre of the Plexiglas there is some bending. In this approximation the at rest coefficient is normative because it results in a larger horizontal stress. This stress interpretation on the Plexiglas interface is important in the PS friction calculation. It is shown schematically for a soil element making contact to the Plexiglas in Figure 5.6.
Assuming the simple stress assumptions, a friction calculation with the wall friction ($\delta$) of the PS interface, the known surface ($A$) and depth ($z$) can be performed based on the equation 8.

$$F_{PS\ friction\ one\ side} = \int_0^z \sigma'_{h;pl} \cdot \tan(\delta) \, dA \ (8)$$

The approximation of this integral results in a PS friction of 50 N for two sides in the unrestrained model, assuming a triangular shear surface of 0.05 m by 0.15 m. See Appendix F for the full explanation of the friction approximation.

In the restrained models the shear surface will reduce, but the horizontal effective stress is larger compared to the unrestrained model. Combining this with the fact that the increase in horizontal stress is maybe not applied on the whole Plexiglas shear surface makes its rather complex. It is assumed by the author that the restrained model ($w/d=3.0$) has no increase in PS friction, and that the restrained models ($w/d=2.0$) and ($w/d=1.5$) do have a PS friction load of respectively 75 N and 100 N.

The consequences of the PW and PS friction for the measured model loads in test A,6.4,100g, is shown in Figure 5.7. In the following sub-chapters, the friction corrections, offset load and offset position are taken into account when the physical model results are shown and discussed.

The uncertainty of the assumptions in the friction calculation causes variation in the final results. It is expected that the estimated load has an uncertainty bandwidth of about 100 N compared to about 2000 N total load. Based on this and on the observations it is concluded that the variation regarding the friction is about 5%.
5.2 Results unrestrained model tests (w/d=6.4)

Two unrestrained model tests are conducted in the test series of this thesis, test A,6.4,100g and test B,6.4,100g. The test characteristics can be found in Table 5.1. A w/d ratio of 6.4 is considered to be unrestrained, because the passive zone can develop without obstruction. As can be seen in Figure 5.8 and Figure 5.9, the observed shear bands do not interfere with the side wall opposing the translating wall. Hence it is assumed that the soil body in this test reacts truly unrestrained.

Figure 5.7 The development of uncorrected (red), to partly corrected (orange), to fully corrected model loads (green) of test A,6.4,100g

Figure 5.8 Side view of test B,6.4,100g. Indicated is the initial embedded length, direction of the wall movement and the observed shear band

Figure 5.9 Top view of test B,6.4,100g. Indicated is the direction of the wall movement and the outcrop of the shear band
The centrifuge scaling laws are applied on the (corrected) measured model loads and model displacements. As stated in the scope of this thesis, a 2D model is going to be analysed. Hence the total model load (N) will be converted to prototype load per meter depth (kN/m). Two calculation examples for N=100g with the scaling laws are shown.

\[
\text{Force}_{\text{model}} = \text{Force}_{\text{prototype}} \times \frac{1}{N^2} = 2000N \text{ (model)} \rightarrow 20.000kN(\text{prototype})
\]

\[
\text{Wall displ}_{\text{model}} = \text{Wall displ}_{\text{prototype}} \times \frac{1}{N} = 6000\mu m \text{ (model)} \rightarrow 0.6 m(\text{prototype})
\]

In Figure 5.7 the green curve represents the (corrected) model loads for one of the unrestrained models. At about 1000 \(\mu m\) wall displacement a peak in the load can be identified. It is assumed that this peak is caused by a mechanical defect in the actuator also appearing in test B,6.4,100g and test H,3.0,100g. This peak cannot be identified in other tests like K,2.0,100g where the wall displacement starts beyond this position.

The point when the ultimate passive load is reached is a combination of the development of the shear band in the soil and the plateau state in the load – displacement curve (Hosseinzadeh and Joosse, 2015). In this chapter the plateau state definition will be used to determine the ultimate passive load of the physical model.

Figure 5.10 shows the load – displacement graph after applying the scaling laws and conversion to the 2D plane strain load per meter depth. A red dashed line indicates the ultimate passive load, interpreted by the author. From the test results it can be concluded that an ultimate passive load of about 1440 kN/m is reached at 0.6 meter wall displacement.

![Figure 5.10 Prototype load vs. wall displacement (unrestrained model tests)](image-url)
5.3 Results restrained model tests
In a restrained model, the passive zones overlap. Dimension ratios w/d of 3.0, 2.0, 1.5 and 1.0 are being subjected to centrifuge testing and are reported separately. The restrained test results are always compared with the unrestrained test results because the differences between those are of specific interest in this thesis.

5.3.1 Restrained model (w/d = 3.0)
Test H,3.0,100g and J,3.0,100g have an initial distance of 0.075 m between the translating wall and the symmetric wall. The initial embedded length of the translating wall is 0.05 m, therefore w/d = 3.0 in these tests.

In Figure 5.11, the results of the unrestrained model and the restrained model (w/d=3.0) tests are shown. The red dashed line indicates the average ultimate passive load for both tests, interpreted by the author. From the results it can be concluded that the average ultimate passive load for the restrained model w/d = 3 is about 1510 kN/m. That is a difference of 70 kN/m or 4.9% in comparison with the unrestrained model results.

![Figure 5.11 Prototype load vs. wall displacement (unrestrained model tests & restrained model (w/d=3.0) tests)](image)

5.3.2 Restrained model (w/d = 2.0)
Test K,2.0,100g, L,2.0,100g and T,2.0,80g have w/d ratios of 2.0. The tests at 100g have initial distances of 0.05 m between the translating wall and the symmetric wall and an initial embedded length of 0.05 m. The test at 80g has an initial distance of 0.0625 m and an initial embedded depth of 0.0625 m. These tests are comparable because the prototype dimensions are the same according to the geometrical centrifuge scaling laws.

In Figure 5.12 the unrestrained model test results and the restrained model (w/d=2.0) results are shown. The average ultimate failure load is indicated with a red dashed line. The ultimate passive load is reached between 0.4 and 0.5 meter wall displacement. From the results it can be concluded
that the average ultimate passive load for the restrained model w/d = 2.0 is about 1830 kN/m. That implies a difference of 390 kN/m or 27.1% in comparison with the unrestrained model results.

![Ultimate passive load w/d = 2.0](image)

**Figure 5.12** Prototype load vs. wall displacement (unrestrained model tests & restrained model w/d=2.0 tests)

5.3.3 Restrained model (w/d = 1.5)
Test R,1.5,80g and S,1.5,80g have dimension ratios of w/d = 1.5. The initial distance between the translating wall and the symmetric wall is 0.047 m and the initial embedded length is 0.0625 m. Hence the prototype embedded depth of 5 meter is the same as previously described tests.

In Figure 5.13 the prototype loads vs. wall displacement results of the unrestrained model tests and the restrained model w/d = 1.5 tests are displayed. The definition of the ultimate passive load cannot be applied on these results because there is no distinctive plateau. However, at about 0.3 meter wall displacement there appears to be a smaller load increment. Hence, the author identifies this point as the location of the ultimate passive load.

The average ultimate passive load for the restrained model w/d = 1.5 is about 2250 kN/m. Compared with the unrestrained model this results in a difference of 810 kN/m, which is about 56%.
5.3.4  Restrained model (w/d = 1.0)

Test N,1.0,100g and O,1.0,100g are conducted with the smallest w/d ratio of the test series. The initial model width of this test is 0.025 m with an initial embedded length of 0.05 m. Because of mechanical defects, the actuator was only able to translate the wall into the soil for about 0.2 meter. As can be seen in the test results (Figure 5.14) this quantity of wall displacement does not result in a clear or distinctive ultimate passive load at all. Therefore, this test series will not be used in the conclusion about the intensification factor of this w/d ratio.
5.4 Conclusions and discussion

The physical model test results shown in the load – displacement curves indicate when the soil body reaches a state of ultimate passive failure according to the plateau criterion. The author has based the definition of the ultimate passive load on the load plateau criterion. This criterion does not provide satisfying ultimate passive load results for the restrained model (w/d=1.5) and (w/d=1.0) because the load does not reach this plateau state.

An overview of the test series and the ultimate passive loads is shown in Table 5.2. The table shows the ultimate passive loads, the intensification factors and the amount of wall displacement at the ultimate passive load. The intensification factor is based on the ratio between the ultimate passive load in the specific restrained model and the average ultimate passive load of the unrestrained model tests.

<table>
<thead>
<tr>
<th>Test name</th>
<th>R_d</th>
<th>Ultimate passive load [kN/m]</th>
<th>Intensification factor [-]</th>
<th>Wall displacement at ultimate passive load</th>
</tr>
</thead>
<tbody>
<tr>
<td>A,6.4,100g</td>
<td>66%</td>
<td>1422</td>
<td>1.00</td>
<td>60 cm</td>
</tr>
<tr>
<td>B,6.4,100g</td>
<td>65%</td>
<td>1442</td>
<td>1.00</td>
<td>60 cm</td>
</tr>
<tr>
<td>H,3.0,100g</td>
<td>57%</td>
<td>1471</td>
<td>1.03</td>
<td>50 cm</td>
</tr>
<tr>
<td>J,3.0,100g</td>
<td>70%</td>
<td>1542</td>
<td>1.08</td>
<td>55 cm</td>
</tr>
<tr>
<td>K,2.0,100g</td>
<td>47%</td>
<td>1777</td>
<td>1.24</td>
<td>40 cm</td>
</tr>
<tr>
<td>L,2.0,100g</td>
<td>52%</td>
<td>1959</td>
<td>1.23</td>
<td>42 cm</td>
</tr>
<tr>
<td>T,2.0,80g</td>
<td>69%</td>
<td>1759</td>
<td>1.37</td>
<td>48 cm</td>
</tr>
<tr>
<td>R,1.5,80g</td>
<td>54%</td>
<td>2211</td>
<td>1.54</td>
<td>32 cm</td>
</tr>
<tr>
<td>S,1.5,80g</td>
<td>52%</td>
<td>2288</td>
<td>1.60</td>
<td>33 cm</td>
</tr>
</tbody>
</table>

The ultimate passive load results in Table 5.2 are plotted in Figure 5.15 against the w/d ratio. The variation in results is larger in the restrained test compared to the unrestrained tests. If the relative densities of the tests are compared, there is no relation between ultimate passive loads and relative densities. For example, Test T,2.0,80g shows about 20% difference in relative density with test K, but about the identical ultimate passive loads. This implies that there is another reason for the variation in results. The most obvious reason could be heterogeneity of the sand sample; a local weak or strong zone can influence the monitored load significantly.

The average ultimate passive loads show a clear non-linear trend line in Figure 5.14. The trend line of the intensification factor follows the trend of the ultimate passive loads because they are directly related. Therefore it can be concluded that the intensification factor shows the same trend (Figure 5.16) and that the hypotheses in this thesis is confirmed: the intensification effect is increasing non-linear, when the width of the trench is decreasing.
In Figure 5.17 all the discussed test results are shown, except the restrained model (w/d=1.0) results, because they are not used in the conclusion of the intensification factor. From 0 m up to 0.05 m wall displacement the differences between the curves are not significant. After 0.05 m wall displacement the load curves start to differ more from each other, depending on the w/d ratio. This implies that up to 0.05 m wall displacement the effect of the overlapping passive zones is not been noticeable in the measurements of the load cell. The visualization results of the shear strain with geoPIV in the models can give more information about this observation, this can be seen in Chapter 6.

The amount of wall displacement, to reach a passive failure state, decreases when the w/d ratio in the model is smaller. In the restrained model (w/d=2.0) it appeared that a clear plateau state can be identified at about 0.4 m wall displacement. The results of restrained model (w/d=1.5) tests are more questionable then the tests with larger w/d ratios because there is no clear plateau state in this curve. Shear strain visualization of the restrained model (w/d=1.5) can provide clarity about the point of ultimate passive load.
Heave of the surface level results in an increase of passive capacity of the sand because of the increase in vertical stress. As stated previously, the soil cannot gain more strength when the ultimate passive load is reached, because the shear band is fully developed. However, the soil can change to a denser state due to compaction. The direct shear test results a larger peak friction angle if the sand density increases. Hence the increase of load of the soil beyond this point of ultimate passive load occurs most likely because of heave and compaction. Figure 5.18 shows the physical model results of all previously discussed tests at larger wall displacements. Smaller w/d ratio models do show a larger increase of load per meter wall displacement due to heave or compaction.

Soil deformation visualisation techniques (geoPIV) can be used to study the volumetric change and the quantity of (vertical) heave, \( u_r \), per model dimension ratio. In this way it can be studied if heave of the surface level or compaction of the soil body is the main cause of the load increase. There are possibly more unknown reasons for the increase of load after 0.3 – 0.5 m wall displacement such as the friction effect of the side walls. According to Figure 5.19, the heave after 0.5 wall displacement in the unrestrained model (B,6.4,100g) is about 0.25 m, this is 5% of the total wall embedded length of the wall (d). For the restrained model (w/d =2.0, T,2.0,80g), heave in the centre of the model trench is about 0.5 m, which is about 10% increase of vertical stress. It can be concluded therefore that heave plays a minor role in the load increase after ultimate passive failure. Because this is not the main subject of this thesis, no further conclusions about post failure behaviour will be made.

The series of tests with the physical model used in this thesis provides a first possibility to conclude on the intensification factor. Additional tests are necessary to determine the intensification factor for even more narrow sheet pile trenches. Because the amount of repetition in the test series is rather low, more of the same tests should lead to a conclusion about the variation of the intensification factor and the accuracy of the physical model. Knowing the uncertainties of different known and unknown factors like and homogeneity of the sample, the maximum error bandwidth of the model is
estimated about 10%. This is mainly because the test results of w/d=2.0 show the largest variation of about 200 kN/m on a total load of about 2000 kN/m.

Figure 5.18 Prototype loads vs. wall displacement (various w/d ratios) results after ultimate passive loads

Figure 5.19 Prototype heave vs. wall displacement comparison
6 Shear band characteristics

The tool to visualize the shear band development in the physical model, is the Matlab algorithm geoPIV. A description and explanation of this tool can be found in chapter 3.3. In this analysis the algorithm is used to calculate the volumetric (hydrostatic) and the shear (deviatoric) strain distribution between two specific images. There is a difference between hydrostatic and deviatoric strains in continuum mechanics according to continuummechanics.org (2015): “The two are subsets of any given strain tensor, which, when added together, give the original strain tensor back. The hydrostatic strain is closely related to volume change, while the deviatoric strain ($\varepsilon_s$) is related to deformation at constant volume”, not to be confused with shear strain. The reason to analyse the shear band development is to obtain a better understanding of the load - wall displacement curves. Furthermore, it is important to compare the unrestrained and restrained model test visually by the shear strains.

Firstly, the shear band development from the start of an unrestrained and restrained test will be analysed with the total or cumulative deviatoric strains. This means that the images during the test are always compared with the initial image. Secondly the shapes of the shear bands are compared with the incremental deviatoric strains. This means for example that the image with 6 mm model wall displacement is compared with the image of 5 mm model wall displacement.

6.1 Unrestrained shear band development

The side view images of test B,6,4,100g are used in the shear band development study of the unrestrained model. The initial model and the deformed soil body after 6 mm of wall displacement is shown in Figure 6.1 as an example. The wall movement is from left to right. After 6 mm wall displacement, a curve in the surface level can be observed in the figure.

![Figure 6.1 Side view images of test B,6,4,100g](image)

A mesh of interrogation patches of 64 by 64 pixels is found to be most optimal for displacement up to 7 mm for the total shear strain approach. This patches size corresponds with a patch of 4 by 4 mm and about 14 by 14 grains based on the D50 of the sand sample. Smaller patch size results in an unacceptable quantity of wild vectors on the shear band. Wild vectors occurs when the algorithm determines an erroneous vector because there is insufficient texture in the image for example. Larger patch size results in loss of detail on the shear band. The final mesh with interrogation patches used in this analysis of total strains is visible in Figure 6.2.

The load – wall displacement curve of prototype loads and wall displacement corresponding to the images are shown in Figure 6.3. Indicated with the letters A-G are locations of the images which are subjected to the strain analysis. To make the distributions comparable, a fixed deviatoric strain scale
from 0 to 60% is used in all the images with the deviatoric strain distributions. Figure 6.4 shows the deviatoric strain distribution between 0 and 0.7 m wall displacement.

**Figure 6.2** Mesh of interrogation patches used for test B,6.4,100g to visualize the total deviatoric strains with geoPIV

**Figure 6.3** Prototype load vs. wall displacement restrained model test B,6.4,100g
From the development in the total deviatoric strains of the unrestrained model, it can be concluded that the shear band starts at the toe of the wall and that it develops from the toe towards the soil surface over 0.6 or 0.7 m wall displacement. At 0.7 m wall displacement, half of the shear band shows total deviatoric strains of 60% and half shows strains of about 30%.

According to Figure 6.3, strain distribution C represents the shear band just before the ultimate passive failure state is indicated by the plateau state. Shown in Figure 6.4 C, is a partially developed shear band with almost zero deviatoric strains near the soil surface. Figure 6.4 D and E correspond with the plateau state in the load – wall displacement curve (Figure 6.3), the images show that the strain values near the soil surface have reached 10 – 15% deviatoric strain.

After 0.4 m wall displacement, the strain values on the shear band are increasing and a second, more scattered, shear band appears between the top of the wall and the main shear band. This appears to be a secondary effect, because it starts after the plateau state is reached.
6.2 Restrained shear band development

The side images of the restrained model w/d = 2.0 (test T,2.0,80g) are used to analyse the shear band development. Figure 6.5 shows the images at 0 mm, 2 mm, 4 mm and 6 mm wall displacement in the model. Since the test is subjected to 80g it defines prototype wall displacements of 0, 0.16, 0.32 and 0.48 meter. The mesh with interrogation patches in the restrained model is visible in Figure 6.6. A mesh of interrogation patches of 64 by 64 pixels is used again for obtaining the total deviatoric strains.

Figure 6.5 Side view images of the test T,2.0,80g. F.l.t.r: 0 mm (initial), 2 mm, 4 mm and 6 mm (model) wall displacement

Figure 6.6 Mesh of interrogation patches used for test T,2.0,80g to visualize the total strains with geoPIV
The prototype load vs. wall displacement curve of test T,2,0,80g is shown in Figure 6.7. The total deviatoric strains distributions corresponding to the letters in the graph can be found in Figure 6.8. To make the distributions comparable, a fixed deviatoric strain scale from 0 to 60% is used in all the images with the deviatoric strain distributions.
The development in the total deviatoric strain distributions in Figure 6.8 shows that the shear band starts to develop at the toe of the wall. At 0.16 m wall displacement (Figure 6.8 C), the shear band reaches to the symmetric wall. In Figure 6.8 D and E strain values between 10 and 20% appear near the surface, this indicates significant shearing all over the band. However according to Figure 6.7 the plateau state is certainly not reached in Figure 6.8 D. This has consequences for linking the ultimate passive load state with the observation shear band development of the restrained model. Specifically, that the shear band criterion and the plateau state criterion cannot be linked.

6.3 Shear band shape

It is important to study the shear band shape, because it is characterized by the soil body and wall parameters. If any variation in the sand samples of the test series occurs, this should be visible in the shear band shape. In order to study the shear band shape, it is more accurate to analyse the incremental deviatoric strains. The relative differences between these images are smaller, therefore a mesh with smaller patch sizes can be used without the occurrence of many erroneous wild vectors. The advantage of smaller interrogation patch sizes is that more details of the shear band are visible. An example is given in Figure 6.9, where the incremental deviatoric strain distribution of 6 to 7 mm wall displacement of a 64 x 64 pixels mesh (used in the total deviatoric shear strain analyses) and a 30 x 30 pixels mesh is compared. In this fine mesh patches of about 6 by 6 pixels are tracked by the algorithm.

![Figure 6.9 Comparison between coarse mesh (left) and fine mesh (right) of interrogation patches used in geoPIV and the corresponding distributions of incremental deviatoric strains (0.6-0.7 m, B,6.4,100g)](image)

As can be seen in the incremental deviatoric strain distribution of the fine mesh, the shear band is having lower values near the surface compared to the band near the toe. This is similar to the observation of Figure 6.4, where the total deviatoric strains are visible. This implies that at passive failure, shear strain values are not the same and that the grain skeleton is relatively intact near the surface.

To analyze the shear band, it is more accurate to compare more distinct and narrow shear bands. Therefore, the shear band shape of test A,6.4,100g and B,6.4,100g is analyzed between 0.7 and 0.8 m
wall displacement. Because there is a certain error range with selecting the pictures of the test, the absolute values of incremental deviatoric strains cannot be compared and are therefore not shown. The shape, outcrop distance and inclination angles of the shear bands are relevant and will be compared.

![Figure 6.10 Comparison of the distribution of the incremental deviatoric strains between 7 and 8 mm wall displacement for the unrestrained models tests A,6.4,100g and B,6.4,100g](image)

Both images compared in Figure 6.10 have the same w/d dimensions, however the shear band shape dimensions are different. The shear band of test A,6.4,100g shows a 4 degrees larger inclination angle $\theta_1$ compared to test B,6.4,100g. The outcrop distance is about 95 mm from the translating wall compared to an outcrop distance of 110 mm of test B,6.4,100g. The differences in shear band shape of test A,6.4,100g and test B,6.4,100g does not result in large differences in the load – wall displacement curves of these specific tests (Figure 5.10). It can be concluded therefore that the differences in shapes are not significant or that the observed shear band (next the Plexiglas wall) does not affect the shear behavior in the whole soil body significantly.

![Figure 6.11, Figure 6.12 and Figure 6.13 show the shear bands of all the restrained model tests w/d=3.0, w/d=2.0 and w/d=1.5. Because of the inaccuray of incremental wall displacement values, the absolute values of the incremental deviatoric strains are not compared. In all strains visualization results, a mesh with interrogation patches of 30 x 30 pixels is used. These meshes used in this analysis can be found in Appendix G.](image)

The deviatoric strain distributions of the restrained models show that the shear band starts near the wall toe, hits the symmetric axis and then reaches the surface. The inclination angle of the shear band at the translating wall $\theta_1$, and the inclination angle $\theta_2$, at the symmetric wall of the restrained model w/d=3.0 appears to be smaller compared to the $\theta_1$ and $\theta_2$ of the restrained model w/d=2.0 and w/d=1.5. In all the images secondary, narrow and lower value shear bands appear perpendicular to the main shear band. Deformation of the whole soil body because of the large strains can be a reason for the appearance of these secondary shear bands. In the distributions of restrained model w/d=1.5, the secondary shear bands have similar shear values as the primary shear band.
Figure 6.11 Comparison of the distribution of the incremental deviatoric strains between 7 and 8 mm wall displacement for the restrained model tests H,3.0,100g and J,3.0,100g

Figure 6.12 Comparison of the distribution of the incremental deviatoric strains between 5 and 6 mm wall displacement for the restrained model tests K,2.0,100g, L,2.0,100g and T,2.0,80g
6.4 Volumetric strain

As stated in Chapter 5, the force measured in both unrestrained and restrained model do show similar curves for small wall displacements. A hypothesis for this observation is that up to about 0.06 m wall displacement, the soil deformation between the unrestrained and restrained model is not significantly different. A volumetric strain ($\varepsilon_v$) analysis for the small wall displacements provides information about the location of change in volume of the soil body. During the wall displacement, the soil body is densifying up to the point where it actually fails by significant shearing. The volumetric strain distributions of the unrestrained and restrained model (w/d = 2.0) are shown in Figure 6.14, the strain scale values are fixed between minus 10% (expansion) and 10% (compaction).

The red values in the distribution indicate a decrease of volume, which indicates compaction or compression. The compression zones or ‘waves’ in the soil body start near the translating wall after 0.02 m wall displacement and continue to move into the soil body with further wall displacements. At 0.06 m wall displacement the compression zone is close to the symmetric wall. The next distribution, at 0.08 m displacement, shows interaction with the symmetric wall. This is the point in the load curve where the difference between the models is noticeable in the measured load.

The restrained model shows larger volumetric strain values over all, this indicates that there is relatively more compaction in the model compared to the unrestrained model. Possible reasons can be the presence of the symmetric wall or small differences between the images because there is only a limited amount of pictures of this small wall displacement available. This can possibly cause a small variation in wall displacement between the unrestrained and restrained model.
Figure 6.14 Total volumetric strain distribution of the unrestrained and restrained model between 0.02 and 0.08 mm wall displacement.
6.5 Conclusions and discussion

It can be concluded from the shear band development analysis of the unrestrained model, that the plateau state in the load curve is correlated to the observation of a full shear band from the wall toe to the surface. However, the restrained model shows a fully developed shear band before the plateau state is reached. This does not match with the assumption that the plateau criterion and the shear band criterion are linked for all models.

The values of the deviatoric strains are not similar over the length of the band during the increase of wall displacement. Higher strain values appear in the shear band part near the toe of the translating wall and lower values near the surface of the soil body. From this observation it can be concluded that the shear strength is not the similar in the complete soil body during the observed wall displacement. It can be concluded that near the surface the shear strength is larger compared to the area near the wall toe. This matches with the conclusion from the direct shear tests in Chapter 4; at lower confining stresses the peak shear strength value is larger for medium dense sand compared with high confining stresses.

The load–wall displacement curves in Chapter 5 shows that the restrained model reaches a plateau state over less wall displacement compared to the unrestrained model. From the shear band development in this chapter it can be concluded that the shear band develops with less wall displacement as well, compared to the unrestrained model. The conclusions from both measured and observed results do match therefore.

The significant differences in shear band shape of the unrestrained tests do not result in significant differences in the load–wall displacement curves. This implies that the shear behaviour in the complete soil body of these tests overrules the visible results next to the Plexiglas wall. For the restrained models, this conclusion is not been made because the shear band shapes are more difficult to compare because of the distortions in the images.

Another conclusion is that the differences between the restrained models tests with smaller w/d ratios are mainly characterized by larger angles of shear band inclination $\theta_1$ and $\theta_2$. The smaller the w/d ratio, the more scattered or smeared the shear band appears to be compared to the relative distinct shear band in the unrestrained model (Figure 6.10).

Further research regarding the volumetric strain is recommended to quantify more specifically the value of densification and the change of void ratio. If these results are compared for loosely packed sands and denser sands, the exact influence of compaction can be linked to the intensification effect.
7 Comparison of physical and numerical model results

The primary motivation of the physical model tests described in this thesis, is the validation of the numerical study of Hosseinzadeh and Joosse (2015). From the physical model tests results (Chapter 5) it can be concluded that there is certainly an intensification effect of the ultimate passive load. This is interpreted by the author to a certain intensification factor corresponding to a w/d ratio. This intensification factor of the passive capacity appears to increase non-linear when w/d decreases.

The numerical model in the study of Hosseinzadeh and Joosse (2015) does show the intensification effect and corresponding factors as well. This chapter compares these two types of models and show the similarities and differences between them. Firstly, the numerical model will be explained and the results are shown. Secondly the actual comparison about the different aspects in the models is conducted.

7.1 Numerical modelling and results

The numerical model of Hosseinzadeh and Joosse (2015) is used by the author as a basis to set up a numerical model simulation with the dimensions of the prototype. The soil density, residual shear strength parameter and Young’s moduli parameters obtained from the soil characterization in Chapter 4.1 are included in the numerical model. The specific interface characterization parameters of the physical model studied in Chapter 4.2 has been included as well in the numerical model.

The constitutive model in Plaxis 2D used by Hosseinzadeh and Joosse (2015) the model the overlap of passive zones in sand, is the Hardening Soil (HS) model. The authors argue that the HS-model is suitable for this study because of its stress dependency of the elasticity parameters. The soil characteristics of DC-sand, the wall friction of the physical model and the numerical parameters assumed and used in the HS-model are shown in Table 7.1. The soil type ‘cohesionless’ and the corresponding parameters are directly obtained from Hosseinzadeh and Joosse (2015) because this model is compared.

A cohesion of 0.01 kN/m² is used in both models in order to avoid erroneous numerical results. The parameter ‘power m’ in the HS model relates the stiffness dependency to the stress level, a value of 0.5 is used in both models to create identical results for the numerical studies. This is important because the assumptions of numerical parameters will be subjected to the comparison with the physical model. All characteristics are for the DC-sand with a relative density of about 50%.

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Dry unit weight</th>
<th>Stiffness moduli</th>
<th>Power</th>
<th>Cohesion</th>
<th>Angle of internal friction</th>
<th>Wall friction angle</th>
<th>Strength reduction factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>[-]</td>
<td>γuns</td>
<td>E₅₀mf</td>
<td>E₅₀mf</td>
<td>m</td>
<td>c</td>
<td>δ</td>
<td>Rₘₐₜ</td>
</tr>
<tr>
<td>DC-Sand (fine)</td>
<td>15.4</td>
<td>30,300</td>
<td>30,300</td>
<td>90,900</td>
<td>0.5</td>
<td>31.4</td>
<td>22.7</td>
</tr>
<tr>
<td>Cohesionless (sand)</td>
<td>18.0</td>
<td>28,300</td>
<td>14,400</td>
<td>84,800</td>
<td>0.5</td>
<td>30.0</td>
<td>20.0</td>
</tr>
</tbody>
</table>

The final design of the unrestrained numerical model (w/d = 6.4) and the restrained numerical model (w/d = 2.0) are shown in Figure 7.1. The prototype dimensions of the centrifuge model are used in the calculation similar to the numerical study which is going to be validated.

It is determined by Hosseinzadeh and Joosse (2015), that there is no significant difference in obtaining the intensification factor in a symmetric or full model of the narrow trench. To save
calculation time it is decided to model the symmetric restrained model (Figure 7.1, right). Shown are the elements, boundary conditions, dimensions and the location of the prescribed displacement on the wall. The symmetric wall is fixed in x-direction, there is no friction in y-direction.

Because the mesh in the numerical model undergoes relatively large displacements, the option ‘updated mesh’ is therefore used in the calculation options. A mesh refinement sensitivity study by Hosseinzadeh and Joosse (2015) results in the conclusion that a medium mesh has sufficient accuracy in modelling and does not result in differences in the load – wall displacements curves with a fine mesh. Hence a medium mesh will be used in the numerical modelling of this thesis. The possible effect of the mesh size on the shear band thickness is not studied in this thesis because it is not the main focus of this research.

Figure 7.1 Mesh, prescribed displacement and boundary conditions in the FEM software Plaxis 2D, unrestrained model (left) and symmetric restrained model (right)

It is argued by Hosseinzadeh and Joosse (2015) that the passive failure state in the numerical model is indicated by a combination of the plastic failure points appearing at the location of the shear band, the relative shear stress distribution and the plateau state in the force – displacement curve. For the unrestrained model with cohesionless sand a failure state is reached at about 0.045 m. In the restrained model w/d = 0.4 no plateau state is reached (Figure 7.2, left), therefore only the shear band and the occurrence of plastic points in the model are indicators of ultimate passive failure for these dimensions. Figure 7.2 (b) shows the calculation results based on the parameters of the DC-sand and the dimension of the strongbox used in this thesis. The trend of the curve is similar, however the loads are slightly greater, this is probably caused by the slightly larger strength parameters (Table 7.1).

Figure 7.2 Comparison of the load vs. displacement results between the numerical model by Hosseinzadeh and Joosse (2015) (a) and the numerical simulation of the physical model used in this thesis (b)
The calculation results of the load development between 0 and 0.4 meter wall displacement in the unrestrained numerical model and the various restrained models are shown in Figure 7.3. Due to numerical instabilities with the interface on the moving wall in the model not all models have reached 0.4 meter wall displacement. To compare the numerical model with the physical model results the soil parameters of the DC sand are used in the calculation.

![Figure 7.3 Load vs. wall displacement numerical results of the unrestrained model and various restrained models with DC sand parameters](image)

At about 0.2 meter the passive load results in the unrestrained model w/d = 6.4 and the restrained model w/d = 3.0 and w/d = 2.0 are rather similar. This indicates that the intensification effect of the overlapping passive zones is faded out at those displacements for the restrained model w/d = 3.0 and w/d = 2.0.

All curves in Figure 7.3 show a linear upward trend of the loads after the ultimate passive load has been reached. Like Hosseinzadeh and Joosse (2015) argue, the ultimate state has to be determined not only by the plateau state criterion, but by the development of a full shear band as well. Since the numerical model has been explained the differences with the physical model can be summed:

- Side wall friction, this influences the load and the observed shear band in the physical model;
- Homogeneity of the sand sample, there is no significant variation in numerical model results of the same dimension ratio value;
- Characteristics of the soil body, the numerical model is always a simplification compared to reality;
7.2 Unrestrained model comparison

The load curve of the unrestrained physical model and the numerical model with wall displacements up to 0.05 meter are plotted in Figure 7.4. The differences in load curves are small and show a similar shape and load value. At larger wall displacements more differences are visible in the curves, as can be seen in Figure 7.5. It is concluded in chapter 5 that the ultimate passive load of the unrestrained physical model is considered at 0.6 meter wall displacement according to the plateau criterion. However, in the numerical results there is no distinctive maximum of the load (Figure 7.5) but a constant increase in load. Possible reasons for the differences in results are discussed in chapter 7.8.

Figure 7.4 Comparison load vs. wall displacement of unrestrained physical model tests and unrestrained numerical model (small displacements)

Figure 7.5 Comparison load vs. wall displacement of unrestrained physical model tests and unrestrained numerical model (large displacements)
7.3 Restrained model (w/d=3.0) comparison

The restrained model tests are compared firstly up to 0.05 m wall displacement in Figure 7.6 and secondly in Figure 7.7 for large wall displacements. The calculated numerical loads up to 0.10 m wall displacement are slightly larger than the measured loads. After 0.14 m wall displacement however, the numerical model does appear to have a lower passive resistance compared with the physical model. Both physical model and numerical model do show a minor increase in load after 0.6 m wall displacement. Conclusions and discussion about this comparison can be found in Chapter 7.8.
7.4 Restrained model (w/d=2.0) comparison

The comparison of results for the restrained model (w/d=2.0) are shown in Figure 7.8. It shows a difference in load of about 100 N from 0.01 m to 0.05 m. The numerical load-displacements curve for larger displacements (Figure 7.9) crosses the curve of the physical model at 0.10 m wall displacements and shows a lower passive resistance of the soil compared to the physical model. Conclusions and discussion about this comparison can be found in Chapter 7.8.

Figure 7.8 Comparison load vs. wall displacement of restrained physical model (w/d=2.0) tests and restrained numerical model (small displacements)

Figure 7.9 Comparison load vs. wall displacement of restrained physical model (w/d=2.0) tests and restrained numerical model (large displacements)
7.5 Restrained model (w/d=1.5) comparison

The comparison between the physical model and the numerical calculation show even larger differences if the w/d ratio is lowered to 1.5. In the first 0.05 meter of wall displacement, a significant greater passive resistance (200 N) in the numerical model is shown in Figure 7.10. The quantity of wall displacement where the physical model shows larger passive resistance is for this w/d ratio 0.09 m (Figure 7.11). Conclusions and discussion about the differences in comparisons can be found in Chapter 7.8.

Figure 7.10 Comparison load vs. wall displacement of restrained physical model (w/d=1.5) tests and restrained numerical model (small displacements)

Figure 7.11 Comparison load vs. wall displacement of restrained physical model (w/d=1.5) tests and restrained numerical model (large displacements)
7.6 Comparison of the intensification factor

The outcomes of the physical model tests (Chapter 5) resulted in intensification factors for various restrained model dimensions between 6.4 and 1.5. The factors derived from the physical tests cannot be directly compared with the intensification factors from the numerical study by Hosseinzadeh and Joosse (2015) because the definitions are different. Apart from the difference in definition, the soil parameter set of the numerical study and the physical study is slightly different as well.

In this thesis the ratio between the ultimate passive (total) loads restrained \( F_{h,w} \) and unrestrained \( F_{h,\infty} \) defines the intensification factor, shown in equation 9. Equation 10 shows the definition of the intensification factor according to Hosseinzadeh and Joosse (2015) as explained in Chapter 1. The definition of \( K_p \) in equation 10 assumes a constant initial vertical effect stress and assumes \( K_p \) as a parameter over the complete depth (instead of a single small soil element), therefore it is significantly different from the reality and physical model definition and cannot be compared.

\[
X_{KP} = \frac{F_{h,w}}{F_{h,\infty}} \quad (9)
\]

\[
X_{KP} = \frac{K_{p,w}}{K_{p,\infty}} = \frac{\int_0^z (\sigma_{h,\text{ultimate},w}) \, dz}{\int_0^z (\sigma_{v,\text{initial}}) \, dz} \quad (10)
\]

7.7 Comparison of shear band development

7.7.1 Unrestrained model

The shear band development up to 0.5 m wall displacement in the numerical and physical unrestrained model is shown in Figure 7.12 with steps of 0.1 m wall displacement. Shown is the total deviatoric strain \( \varepsilon_s \) distribution, the prototype dimensions and the amount of wall translation per image. The shear strain distribution in the physical model are introduced and described in Chapter 6. The color scales of the deviatoric strains are identical for both model types. Strains are unitless, therefore they do not need to be scaled with centrifuge scaling laws.

The numerical shear band development (Figure 7.12, left) and the shear band development in the physical model (Figure 7.12, right) show several differences. In both physical and numerical model, the shear band starts developing from the toe of the wall. A shear band with values between 10% and 20% deviatoric strains can be observed in the numerical model after 0.30 m wall displacement. The shear band in the physical model however, at the same amount of wall displacement, shows larger variety in strain values, namely between 0% and 30%. The large variety in shear strain values is a significant difference compared with the numerical model.
Figure 7.12 Distribution of the numerical total deviatoric strains (left) and physical total deviatoric strains (right) in unrestrained model (w/d=6.4) for various wall displacements between 0.1 m and 0.5 m
7.7.2 Restrained model (w/d=2.0)

The total deviatoric strain distribution development in the restrained numerical model (w/d=2.0) is shown in Figure 7.13 and is being compared with the total deviatoric strain results of the physical model (Chapter 6), shown in Figure 7.14. The (prototype) wall displacements and dimensions in both type of models is identical and is indicated in the images.

The shear band development in the numerical model is characterized by the increase of deviatoric strains over the entire length of the band. Like in previous comparison of the unrestrained models, the numerical model shows less variety on the shear band of shear values compared to the physical model. Furthermore, the final shape of the shear band in the numerical model can already be recognized at 0.08 m wall displacement, which is not the case in the physical model because of the relative large patch size used in geoPIV.
7.8 Conclusions and discussion

7.8.1 Small wall displacements
The load – wall displacement curves of the numerical and physical unrestrained model show similar results for displacements up to 0.05 m (Figure 7.4). This implies a decent simulation with the numerical model of unrestrained passive soil behaviour up to this wall displacement. However, if the load – wall displacement curves of the restrained models are compared, larger differences between the curves occur (see Figure 7.6, Figure 7.8 and Figure 7.10). It is observed that smaller w/d ratios of the soil body result in larger differences between the numerical and the physical model load-wall displacement curves. The numerical loads are significantly larger than the physical loads for the same amount of wall displacement.

It can be concluded from Figure 7.2 that the intensification effect in the numerical model is observed immediately after the start of the wall displacement. This is however not observed in the corresponding load – wall displacement curves of the physical model tests. The load curves of different w/d ratios hardly show an intensification effect in the physical model load between 0 and 0.05 m wall displacement. The volumetric strain analysis (Figure 6.14) shows that the compression zone of the restrained and unrestrained model are similar for small displacements and that they do not extend more than 5 meter into the soil body at 0.06 m wall displacement. This analysis explains that the soil deformation is starting from the model sheet pile wall. There is no difference in the restrained model compared to the unrestrained model up to the point where the compression zones make contact in the centre of the trench.

The compression wave cannot be seen in the numerical analysis for small wall displacements. This implies that modelling the change of soil body density and mobilization of the passive zones as a result of wall displacement is different in the compared models and that this causes the difference in the load curve shape.

7.8.2 Large wall displacements
The analysis of the load – wall displacement curves for larger wall displacements (≥0.05 m) result in new differences between the models. In the unrestrained numerical model the plateau state occurs after 0.3 m wall displacement, while the unrestrained physical model needs about 0.6 m to reach the plateau state (Figure 7.5).

The comparison of the curves with a large wall displacement range show that both unrestrained and restrained physical models have larger ultimate passive capacities compared to the numerical models. An explanation of this difference can be given by the complex behaviour of sand. From the direct shear test results (Chapter 4), two shear strength parameters can be identified and derived.

- The effective peak shear strength parameter, which occurs at small shear strains. This value depends on the relative density of the sand and the acting confining stress;
- The effective residual shear strength parameter, which occurs at large shear strains. This strength parameter is not dependent on the relative density nor the confining stress.

In the initial state of the soil body, the effective peak shear strength parameter defines the true strength of the sand against shearing, because the complete soil body has a uniform medium density and has not been sheared. After the shear band development, the effective residual shear strength is normative, because a thin weak shearing zone now defines the strength of the entire soil body. From the analysis of shear band development, it is shown that there is a sequential growth of the shear band, which implies different shear strength parameter values in the soil body during wall displacement like in the direct shear test results.
In the Hardening Soil model a single shear strength value is used as a model parameter instead of both peak and residual strength. It is erroneous to use the peak shear strength in the numerical model, because the parameter is depended on the confining stresses. The result of using the residual shear strength parameter however, is that up to the point of ultimate passive failure, the passive capacity of the soil in the numerical model is an underestimation compared to the real passive capacity. This theory is not applicable to loads at a small wall displacement scale, in previous subchapter it was concluded that the physical model shows similar or an underestimation of the numerical model loads. It is a possibility that for small wall displacements Young’s modulus is governing in the value of passive resistance force, after 0.05 m the soil starts to shear which results in the fact that the lower shear parameters are governing.

It can be concluded that the HS model provides a good first approximation of the passive capacity and the effect of the overlapping passive zone. The HS model is however limited to simulate the true ultimate shear strength for medium dense sand and the strength development of the soil between 0 m and large wall displacement (0.4-0.8 m) because of the simplification of the shear characteristics.

7.8.3 Shear band

The shear bands of the numerical and physical model tend to develop faster in the restrained dimensions compared to the unrestrained model. This is shown in the comparison of absolute deviatoric strain values (Figure 7.12, Figure 7.13 and Figure 7.14). This observation correlates with the observation that the ultimate passive load occurs at a smaller wall displacement in the restrained model curves.

The total deviatoric strain values of the numerical model show relative low variation in values over the entire length of the shear band during wall displacement. This is in contrast to the large variation in strain values at the shear band which is observed in the total deviatoric strain distribution of the physical model. Very small strain values at the shear band near the surface of the unrestrained model do correspond with a plateau state in the load – wall displacement curve of this particular model. In the restrained model, this correlation cannot be made, the plateau state is not being reached while the shear band seems to have reached the surface.

7.8.4 Intensification factor

The intensification factors derived from the physical model results are larger compared to the intensification factor of the numerical study by Hosseinzadeh and Joosse (2015). The graphs with the numerical and physical intensification factors show however a similar curvature in the non-linear increasing trend line. The differences between the values can be partly explained because a slightly different soil parameter set is used in the physical model. But primarily the differences occur because of the difference in definition of the ultimate passive load. In this thesis the factor is based on measured values from the physical model tests, while in the numerical study it is defined by theoretical and therefore less realistic assumptions.

It can be concluded as well that the different subjective criteria to determine the ultimate passive capacity such as the plateau criterion can cause variation in the determination of the intensification factor. When similar assumptions are made in determining the intensification factor for different w/d ratios, the trend of this factor can be compared for different soil types.

The lack of load increase in the restrained physical model tests for the first 0.05 m wall displacement has consequences for the interpretation of the intensification effect. Based on the measured load – displacement results of this thesis and the results of the volumetric strain analysis, it can be discussed that the intensification effect does not occur at small wall displacements (<0.05 m). It can
be concluded that the effect of the overlapping passive zones is only noticeable in the load curve when the passive zones are mobilized.

To make final conclusions about the displacement dependency of the intensification factor, the effect of the grains size on this factor need to be studied more extensively. Although in Chapter 2 it is argued that the grain size effect is not significant, the effect of the grain size distribution could be significant. Larger variation in grain sizes could lead to different mechanical behaviour, for example the Young’s modulus and the shear strength parameters will be affected when large and small grain sizes form a denser skeleton compared to a grain skeleton of a uniform distribution.
8 Conclusions & recommendations

The physical model used in this thesis provides helpful and new information about the effect of the overlapping passive zones in narrow sheet pile trenches. The measured results, like the load–displacement curves, as well as the visual results, like the soil deformation images, are useful in providing a better understanding of the intensification effect. The main research question of this thesis can now be answered.

Is the intensification factor of the ultimate passive stress capacity due to the effect of overlapping passive zones, introduced by the numerical and analytical approach of Hosseinzadeh & Joosse (2015), valid based on a physical model?

Yes, the intensification effect which results in an intensification factor of the ultimate passive stress capacity due to the overlapping passive zones can be observed and measured in a physical model. The subsequent conclusions in this chapter explain further findings of this research in more detail. The hypothesis of this thesis is based on the conclusions of the numerical study on this topic (Figure 1.6) and is defined as follows:

There is an intensification factor when the passive zones do overlap, this factor is increasing with a non-linear trend if the ratio w/d of trench width (w) over embedded length (d) decreases.

Based on the research in this thesis it can be concluded that the hypothesis about the intensification effect points in the correct direction. But the different definitions for the intensification factor between the physical and numerical make it impossible to answer the hypothesis completely.

8.1 Conclusions

The following conclusions can be made from the physical modelling part in this thesis and the part about the comparison between the physical and numerical model of the overlapping passive zones:

- The physical model designed in this thesis can model the overlap of passive zones 2D plane strain. According to the scaling laws and the requirements for centrifuge modelling, realistic mechanical behaviour of sand is being modelled;
- The accuracy and repeatability of the physical model is shown by satisfying similarity of results in the physical model tests (Figure 5.16). The unrestrained model tests show very similar load results (≤2% variation), narrow model simulations do show medium similar load results (10% variation). The maximum variation in relative density towards the mean value of the sand in the tests series is 17% (Table 5.1). Based on this it can be concluded that deviation in ultimate passive loads is expected to be due to the sand sample preparation method, a local weak or strong zone in the sample is the most obvious reason for difference in results. The uncertainty in the friction value increases the error bandwidth of the physical model test results by approximately 5%;
- The intensification effect due to the overlapping passive zones can be clearly observed in the physical model test results (Figure 5.16). The definition of the intensification factor derived from the physical model is different compared to the definition of the intensification factor of the numerical study by Hosseinzadeh & Joosse (2015). Hence, the absolute values of the intensification factors cannot be compared. Nevertheless, identical curvature in the trend lines of the increase of the intensification factor over the decrease of model width are shown;
- Visualization of the deviatoric strain in the physical model results in a high variation of strain values at the shear band compared to the lower variety in shear values on the shear band in
the numerical model. Simplified shear behaviour in the constitutive model of the numerical analysis is one of the reasons for this difference;

- The development of the shear band in the restrained physical model occurs with less wall displacement compared to the unrestrained model. This is in line with the measured load – wall displacement curves, where the restrained model appears to reach plateau state with less wall displacement as well;

- The intensification effect due to the overlap of passive zones in the physical model is not noticeable by differences in unrestrained and restrained load curves, up to about 0.05 m wall displacement (Figure 5.16). The visualization of the volumetric strain in the restrained model is in line with the load curves, because it shows that the passive zone is not mobilized before 0.05 m and therefore no overlapping effect can occur at all. The numerical model however, shows a significant intensification effect from the beginning of wall translation into the soil body and no volumetric strain change development as in the physical model. Overestimation of the Young’s modulus in the numerical model is the most probable cause for this difference between the models;

- The peak shear strength in the medium dense DC sand causes the physical model to have a larger passive capacity compared to the numerical model. Because the stress dependency of the peak shear strength parameter is not included in the HS model, the residual shear strength parameter is used. The residual shear strength parameter is lower compared to the peak value, hence the numerical model calculates a lower ultimate passive capacity of the soil at large wall displacements compared to the real ultimate passive capacity of the soil.

8.2 Recommendations
The recommendations are subdivided in the different aspects of this thesis, they are listed below.

8.2.1 Recommendations concerning physical modelling
- The variation in relative density in combination with possible heterogeneity of the sand sample is expected to be one of the reasons for the variation in test results of the same model dimensions. It is therefore recommended to improve the sample preparation method to create a more homogeneous sample. It is suggested to make use of a sand raining machine to exclude the human error in the sample preparation;

- The effect of the interface friction on the Plexiglas sidewalls is causing an unwanted friction load which disturbs the correct modelling of a 2D plane strain situation. Hence it is recommended to study more possibilities to eliminate the friction on the sidewalls and to find a more accurate approximation of the side wall friction load.

8.2.2 Recommendations concerning the numerical modelling
- The Plaxis 2D constitutive model ‘Hardening Soil’ is a good first approximation of the passive capacity of a soil body. It is recommended however, to use a more advanced constitutive model which can model the different peak shear strengths of the sand under varying confining stresses and shear strains. An example of such a model is the hypoplastic model with intergranular strain;

- A 3D FE-analysis will increase the understanding of the side effects in the physical model like the Plexiglas and the corner effects of the strongbox. It is therefore recommended to simulate the physical model in a 3D FE-model;

8.2.3 Recommendations concerning engineering practice
- The main goal of the numerical study by Hosseinzadeh & Joosse (2015) was to find an easy design optimization tool to take into account the effect of the overlapping passive zones. This
The intensification effect is proven to be valid for the ultimate passive load in a physical model at large wall displacements. Based on the observations of the load curves and volumetric strain for small wall displacements, it can be reasoned to doubt on the use of the intensification factor in the design when the wall deformations are expected to be less than about 0.05 m. It is therefore recommended to zoom in on this aspect by for example more pictures during this wall displacement range;

- Because of the centrifuge scaling laws, the sand grains used in the physical model are relative large compared to reality. This characteristic of the grains could have an effect on how the loads at small wall displacements develop in engineering practice with small grains. It is therefore highly recommended to study the effect of the grain size and different packings on the development of passive capacity.

8.2.4 Future research

- The influence of relative density, stratification and different types of the soil like clay or silt on the intensification effect is recommended to study in further research. Physical model test with different w/d ratios and variation in embedded length (d) will improve the understandings of the intensification effect and the influencing parameters;

- In engineering practice a heterogeneous soil consisting of various soil types is often reality. The relevance and effectiveness of modelling homogenous samples to simulate the situation in reality should be studied more elaborative to come to a conclusion about the application of the outcome of this and future studies in engineering practice.
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Appendix A - Literature review report

Master thesis project:

Physical modelling of the overlapping of passive zones

Version: final
Date: 1-9-2015

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1 Introduction
This literature review will focus on the understanding of lateral earth pressures in combination with physical modelling. No specific study has been found about the main object of this thesis: physical modelling of the overlapping of passive zones. Many studies are although helpful and relevant to this thesis.

The goal of this literature review is to give a structured overview of all the relevant research available to this thesis topic. This literature overview forms the foundation of this master thesis. In the thesis work plan the problem description, relevance, research goal and the scope of this thesis are introduced and explained. The research goal is as follows:

“Design a physical model to validate the ultimate passive stress capacity intensification factors due to the overlap of passive zones in narrow trenches.”

The scope of the master thesis is defined in the following definitions:

- A narrow trench is defined by a trench where the passive zones do overlap;
- This research is restricted to a 2D plane strain situation;
- The sheet pile walls are assumed to be infinite stiff;
- The sheet pile walls shift in pure translation;
- The influence of structural elements like anchors, struts and underwater concrete are not taken into account in this research;
- Uplifting of the building pit is not regarded;
- Sand will be considered, dry medium dense sand will be used for the experiments.

![Figure 1-1 Overlap of the passive zones in a narrow trench (wall movement: pure translation)](image)

A summary is given in chapter 1.1 about different types of lateral earth pressure theories, analysis and research. This exposition is fundamental for better understanding the models for passive soil behaviour and to create an overview, although this is not the main topic of this literature research.

1.1 Lateral earth pressure
The ultimate passive resistance is described by the lateral earth pressure coefficient $K_{p,h}$. There is no international or uniform categorization of all the models and methods. For this exposition three categories of lateral earth pressure theories and calculations are shortly highlighted and explained.

---

1. Analytical theories;
2. Numerical analysis;
3. Experimental research.

1.1.1 Analytical theories
Lateral earth pressures on soil retaining walls are being studied analytically for the first time in the 18th century by the French scientist Coulomb (1776). He considered the soil behind the wall as a free body with acting forces which are in equilibrium, known as the commonly used limit equilibrium theory. Rankine’s theory (1857) is very similar to Coulomb’s theory but considers acting stresses on the free body. Coulomb’s theory is extended by Mayniel (1808) with wall friction and by Müller-Breslau (1906) with a non-horizontal backfill and non-vertical wall.

The greatest limitation of the limit equilibrium theories mentioned above is that they will give an underestimation for the passive resistance due to the assumption of a planar slip plane. Due to wall friction the slip planes are certainly not planar and will mobilize a larger quantity of soil. Methods of this more realistic approach are given by for example Streck (1966) regarding a broken slip plane and Ohde (1938) regarding a log-spiral based slip plane, see figure below.

![Figure 1-2 (a) ‘Broken’ slip surface according to Streck (1966); and (b) ‘Log spiral’ slip surface according to Ohde (1938)](image)

1.1.2 Numerical analysis
Due to computer technology numerical analysis e.g. finite element models are more widely used as a method to model lateral earth pressures. This category can be divided in two parts according to Hosseinzadeh and Joosse (2015): finite element limit analysis and displacement-based finite element analysis e.g., Plaxis 2D finite element code (Brinkgreve et al., 2015). The first analysis combines the bound theorem with finite element techniques. The second analysis uses a cinematically admissible strain field which is connected to the displacements. Stresses are a secondary quantity in this analysis. More information about the displacement-based finite element method is written by Zdravkovic (1999).

1.1.3 Experimental research
The third category includes all the experimental research that has been conducted to model the lateral earth pressures e.g. by Stuit (1995), Gutberlet et al. (2013) and Lucia (1966). This thesis fits in this category because it will validate a numerical analysis with designing and performing physical experiments.
2 Experiments related to passive earth pressures

Since the beginning of 20th century, experiments are being conducted to better understand and to validate the passive earth pressures theories. Terzaghi (1920) started with experimenting on large movable walls and is followed by many others as can be found in this literature review.

In this chapter several experiments regarding the lateral earth pressures will be evaluated and relevant conclusions will be used for this thesis. These experiments will be categorized and discussed by the four relevant factors defined by Duncan and Mokwa (2001) that control passive earth pressures:

- Movement of the structure;
- Soil strength and stiffness;
- Interface friction and adhesion2;
- Structure shape.

2.1 Movement of the structure

There are basically three different kinds of wall movement categorized in literature: wall translation, wall rotation around the top and wall rotation around the toe. Various low stress experimental studies done at the university of Cambridge in the ’60s and ’70s about passive and active behaviour of soil are collected by Leśniewska (2000). The different shear band shapes with various wall movements are visible in Figure 2-1.

![Figure 2-1](image)

*Figure 2-1 Shear zones observed in experiments of passive mode with initially dense sand (radiographs and schematically): (a) during wall translation (Lucia, 1966), (b) during wall rotation around the top (Arthur, 1962) and (c) during wall rotation around the toe (Bransby, 1968) (O - rotation point)*

2 Adhesion is not applicable for sand and will therefore not be discussed
The shape and geometry of the shear zones is clearly related to the type of movement because every type of movement results in a different shear band. Therefore also the ultimate passive resistance is highly dependent to the wall movement type. According to Leśniewska (2000) the largest ultimate passive resistance of the soil is obtained during translating, followed by rotation around the toe and rotation around the top.

Because the scale in Figure 2-1 is unknown no conclusions can be drawn of the difference in soil mass and difference in length of the shear bands. Also no experimental studies have been found which give a rule of thumb for the shear band length.

### 2.2 Soil strength and stiffness

#### 2.2.1 Relation to the maximum passive earth pressure

Soil strength is related to the porosity and relative density of sand, because these parameters indicate the packing of the grain skeleton. In loosely packed sand the strength parameters are relatively low in comparison with densely packed sand. This is mainly because there is more quartz material in a certain volume and the grains have less space for movements.

Several experiments with the aim to check the effect of different relative densities on passive soil capacity have been conducted in the last decades e.g. by James and Bransby (1970). The conclusions are pointing in the same trivial direction: denser sand results in a higher passive resistance of the soil.

One of the most recent series of test are done in the small scale model at 1g of Gutberlet et al. (2013). In these experiments a wall of 12.5 cm. is translated into the soil body to generate passive shear zones. Further characteristics of these specific tests and the sand used can be found in Table 2.1 and Table 2.2.

**Table 2.1 Sand characteristics experiments Stuit & Gutberlet et al.**

<table>
<thead>
<tr>
<th>Study</th>
<th>Sand type</th>
<th>Angle of internal friction, $\varphi$ [°]</th>
<th>Unit weight, $\gamma$ [kN/m$^3$]</th>
<th>Dilatancy, $\psi$ [°]</th>
<th>Mean grain size, $D_{50}$ [mm]</th>
<th>Uniformity coefficient, $C_u$ [-]</th>
<th>Relative density [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gutberlet et al. (2013)</td>
<td>Darmstadt Sand</td>
<td>32.3</td>
<td>15.5; 17.8</td>
<td>Not known</td>
<td>0.35</td>
<td>3.1</td>
<td>0.29; 0.99</td>
</tr>
<tr>
<td>Stuit (1995)</td>
<td>Silver sand</td>
<td>39.9</td>
<td>Not known</td>
<td>14.9</td>
<td>0.259</td>
<td>1.58</td>
<td>Not known</td>
</tr>
</tbody>
</table>

**Table 2.2 Experiment setup characteristics of Stuit & Gutberlet et al.**

<table>
<thead>
<tr>
<th>Study</th>
<th>Wall height [mm]</th>
<th>Wall width [mm]</th>
<th>Acceleration [g]</th>
<th>Wall friction, $\delta$ [°]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gutberlet et al. (2013)</td>
<td>125</td>
<td>720</td>
<td>1</td>
<td>Not known</td>
</tr>
<tr>
<td>Stuit (1995)</td>
<td>56.9; 41.8; 36.8; 31.5; 23.4</td>
<td>120</td>
<td>35; 48; 54; 64; 85</td>
<td>0</td>
</tr>
</tbody>
</table>
From the Gutberlet et al. experiment results it can be concluded that the difference in density gives a clear and significant change in the shape of the force-displacement graphs (Figure 2-2). The dense sand shows a significant peak before the plateau state is reached, while the loose sand does not show a peak continuously build up the pressure during 40 mm wall displacement. The lines tend to reach the same earth pressure, this can be explained because at the shear band the soil particles are dilated and therefore changed into a loose state.

![Earth pressure E [kN]](image)

*Figure 2-2 Mobilisation curves for homogenous sand with different packing (Gutberlet et al., 2013)*

The peak force in Figure 2-2 is very likely an overestimation if it is compared to larger walls with higher stresses in the soil. This is according to Gutberlet et al. (2013) due to the fact that interlocking of the grains does have a larger effect on the soil behaviour at low stress levels than at high stress levels (Bolton and Lau, 1989), (Lirer et al., 2011).

A similar type of experiment with a translating wall pushing into a relatively loose sand is done by Stuit (1995). In this PhD thesis this type of test is called the ‘bulldozer test’ and it is performed in the TU Delft centrifuge. The major difference with the experiments of Gutberlet et al. (2013) are the stresses in the sand. Stuit did use a centrifuge for his small-scale model to scale up the stresses. More characteristics of the setup and the sand can be found in Table 2.1 and Table 2.2. The results of the ‘Suit’ tests are visible in Figure 2-3, the graph does not show a clear peak force and no distinct plateau state at first sight, the load is generally increasing during the wall displacement. All the test results show more or less the same prototype load during first part of the wall displacements.
There are some clear differences between the graphs of Figure 2-2 and Figure 2-3. From the studies of Stuit (1995) and Gutberlet et al. (2013) cross-section images (see: Figure 2-4, Figure 2-5, Figure 2-6) shows that there are differences in quantity of sand accumulation in front of the wall. Because of the lack of wall friction in the study of Stuit, the shear band inclination angle ($\theta$) is relatively large. This means that the sand will accumulate closer to the wall in comparison with the ‘Gutberlet et al.’ experiments. This heap of sand close to the wall adds extra material which results in a continually growing passive resistance during the wall displacement. This effect is emphasized by the scaling due to the centrifugal acceleration in the experiments of Stuit. These reasons are most likely the main cause of the difference in force-displacement evaluation between Figure 2-2 and Figure 2-3 and why there is no plateau state in Figure 2-3.

For this thesis displacements up to the first failure (which is identified as a peak force) are relevant because the ultimate passive capacity is then reached. From this subchapter it can be concluded that a $d/h$ ratio$^3$ between 0.05 - 0.1 is sufficient to reach the failure state of the passive wedge for a sand material. These ratios can be distracted from both Stuit (1995) and Gutberlet et al. (2013) and will be used in the design of the experiment setup.

2.2.2 Relation to the shear band shape

In Figure 2-4 and Figure 2-5 the shear bands of the Gutberlet et al. tests with dense and loose sand are visible after maximum passive force has been reached. The geometry of the shear band of the dense sand is in agreement with Figure 2-1a which is conducted in dense sand as well, they are both not planar.

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3 Wall displacement (d) / wall height (h), a uniform ratio to compare different test results
The curvature of the shear band is more pronounced for soils with higher strength, i.e. higher densities. For loose soils, the propagations of shear bands are nearly in a straight line according Gutberlet et al. (2013). A reason for the difference in shape can be the difference in interface friction due to the difference in friction angle of the sand. Next subchapter about interface friction will show and explain this direct relationship.

With the techniques available in 1995 a bit less clear picture can be visualized of the shear band, but the overall shape can be distinguished. The PhD thesis of Stuit shows a different image of the shear band shape as can be seen in Figure 2-6. After the first failure of the soil body (point 3), the shear band inclination angle ($\theta$) is larger (36 degrees) in comparison with the inclination angle during the post-failure displacements (e.g. point 9, 25 degrees). This change of the inclination angle during one test is because of the same reasons as discussed before: the accumulation of sand close to the wall surface due to large wall displacement. This phenomena of relocation of the shear band is not visible in the research of Gutberlet et al. because there the wall displacements where relatively small.
Stuit and Gutberlet et al. both show that the shear band is not visible at the peak strength but at the plateau state or lower state. This indicates that the grain skeleton stays intact during building up the pressures. When the soil body has failed passively, the grain skeleton is broken and the shear band is visible.

### 2.3 Wall interface friction

The wall interface friction plays a role only when it is mobilized. This means a specific shear displacement across the interface is required. Typically relatively small values of no more than 2.5 – 6 mm. are required to reach the maximum mobilized friction angle according to Duncan and Mokwa (2001). Within the assumptions of this thesis (only horizontal translation of the wall) the shear force is directly related to the mobilized interface friction. In Figure 2-7 this is shown by means of the force polygon on the soil wedge. Limitation of the mobilized interface friction due to a limited weight of the structure is not concerned because according to the scope of the thesis there is no vertical movement of the wall. No experimental studies have been found which prove that an increase of wall friction results in an increase in passive force.
Because in this thesis only the interaction between soil and sheet pile steel is of relevant no other materials will be discussed. Several experiments to study the wall interface friction angle have been conducted (e.g. Potyondy (1961), Neuffer and Leibnitz (1964), Brumund and Leonards (1973), Acar et al. (1982) and Yoshimi and Kishida (1983). They are collected by Grondmechanica Delft (1988) and are plotted in Figure 2-8 for a smooth and coarse steel surface. The steel roughness is not defined by quantitatively. Each of the tests data point can be categorized by the friction angle of the soil used, $\varphi$ (x-axis) and by the results of the wall friction, $\delta$ (y-axis).

The friction of the smooth steel surface graph tend to be equal or smaller than the line $\delta = 2/3 \varphi$. While the results with the coarse steel surface tend to be equal or larger than the line $\delta = 2/3 \varphi$. Overall is the variation approximately plus or minus 20% to the line of $\delta = 2/3 \varphi$. Therefore this value is advised by Rijkswaterstaat Bouwdienst (CURnet, 2012) for the interface friction angle between sand and steel.
or prefab concrete sheet pile walls. This value is also advised and prescribed in Eurocode 7: geotechnical design (2012).

2.4 Structure shape
The three factors of influence on the passive capacity discussed in chapter 2.1 – 2.3 are assuming a 2D situation. In real practice this 2D assumption is only valid in a limited amount of situations. Almost always 3D effects (shape effects) play a role e.g. at the ends of wall structures where passive failure is reached. Ovesen (1964) conducted an extensive amount of experiments focusing on the passive earth pressure against short structures like anchor plates. Brinch Hansen (1966) created a method to correct the 2D passive pressure theories for shape effects.

It is stated in the scope of this thesis that we model the overlapping of passive zones 2D, therefore this topic about 3D effects will not be discussed more extensively.
3 Comparison numerical models and centrifuge research

Experimental works e.g. centrifuge tests are often used as a validation tool for computer models e.g. numerical models. This chapter will focus on the studies done to couple numerical model results and centrifuge test results for validation purposes. Although no such study has been found about the passive soil behaviour, cases about, or related to, the active or at rest behaviour are relevant as well to learn from and are therefore being disused in this chapter. Two studies are discussed which differ in triggering the soil body to fail both numerically and experimentally.

3.1 Soil body failure initiation by forced displacement

Yang and Liu (2007) conducted an extensive comparison about a narrow retaining wall case. Narrow in this study means less than the conventional building width between the rock face and a retaining wall (L, Figure 3-1) which results in reduction of K₀ and Kₐ. In this study the authors compare the horizontal stresses of a Plaxis 2D calculation of a narrow retaining wall with the theoretical equations and with both Frydman’s centrifuge tests (1987) and Take’s centrifuge tests (2001). Yang and Liu use the already existing centrifuge results of this case to validate a numerical model to study the sensitivity of the calculated earth pressures with varying walls aspect ratios.

![Figure 3-1 Simulated narrow retaining wall with centrifuge (left) and finite element model for active case (right) (Yang and Liu, 2007)](image)

The horizontal stress in the numerical model shows large similarities with the horizontal stresses in centrifuge tests. Lessons learned from this study are as follows.

A prescribed displacement in Plaxis 2D initiates active failure of the model and it can simulate active failure in the centrifuge tests properly. This means that in the numerical model of overlapping passive zones the prescribed displacements most likely models the passive failure of the soil in the same way as the centrifuge model.

Another aspect of the study is that the Mohr-Coulomb soil model is chosen for the sake of simplicity and the lack of stress-strain curves from the centrifuge experiments. The decision for a more advanced soil constitutive model can be an improvement only unless the parameters are known and have a high
certainty. The authors advises to take a small cohesion into account to prevent premature soil yielding in locally low confining pressure zones and to improve the numerical calculation.

Wall faces do not play a primary role in the study to horizontal stresses of Yang & Liu, therefore they are modelled with very high bending and normal stiffness to eliminate their effect. Interface elements with a Mohr-Coulomb criterion are applied in the Plaxis model to simulate the interface behaviour in the centrifuge model. This interface behaviour can possibly be improved if a more advanced interface model is used.

It can be concluded that a numerical calculation with a relatively simple soil and interface model (Mohr-Coulomb) can be used to simulate the stresses that have been measured in small-scale tests. Together with the fact that a forced displacement can initiate a realistic soil failure it will be used in the design process of the experiment setup in this thesis about the overlap of passive zones.

### 3.2 Soil body failure initiation by acceleration increase

A study of Yang et al. (2008) shows many similarities with the study of Yang and Liu (2007) of previous subchapter. This paper is also about a validation of a numerical model for narrow retaining walls but a different type: Mechanically Stabilized Earth (MSE) walls. The comprehensive centrifuge model tests of these reinforced soil walls by Woodruff (2003) are reviewed and used for the validation. Figure 3-2 shows both numerical and experimental model in the initial stage for one of the validated wall aspect ratios.

![Finite element setup and initial mesh](image)

One of the differences with the approach of Yang & Liu is that failure of the wall is induced by increasing the centrifugal force in the physical model until failure occurs. This is modelled in the numerical model by means of a body force equal to 10g (working stresses). After applying the body force, the failure was reached by reducing the strength parameters of the soil, the so-called Phi-C reduction function.

This technique produces more representative results because it follows a realistic loading path until the working stresses (10g). Another benefit is that the global factor of safety can be obtained by the approach with the Phi-C reduction function.
The more advanced soil model Hardening Soil is chosen by Yang et al. because it was believed that this model had better ability to match the stress-strain curves of a granular soil at working stress conditions in comparison to the Mohr-Coulomb model.

The importance of interface elements in the numerical model which corresponds to the physical interface characteristics is emphasized by Yang et al. The aluminium back wall of the centrifuge strong box was given the numerical parameter $R_{\text{inter}}$ of 0.3 and the soil-wall face (geotextile) and soil-reinforcement (geotextile) elements were given a $R_{\text{inter}}$ of 0.9.

The comparison of displacements in the centrifuge model and the numerical model at working stresses do show a good agreement according to the authors. The comparison of the failure condition (Figure 3-3) show a good agreement as well, the stationary triangular part and the sliding shape correspond.

![Figure 3-3 Comparison of displacement: (a) displacement contour from finite element simulation; (b) Image from centrifuge at failure condition (Yang et al., 2008)](image)

However the good and satisfying results of the study described above, this failure initiation of the soil body is not preferred to be used in the tests of this thesis to study the intensification factor due to the overlapping of passive zones. This is mainly because of the lack of load monitoring and control of the wall during a test. If the stress situation in the soil body is not of interest but for example the shear band shape, this failure initiation is a good option.
4 Visible and non-visible aspects during centrifugal testing

There are more aspects of interest in the preparation of the experiments. For example what to expect to see during wall displacements? Or what will be the influence of the boundary effects or the particle size effect. This chapter will focus on those aspects. The outcomes can be used in both designing the experiments as interpreting the experiments.

4.1 Development stages of the shear band

The development stages of the shear band during translating of the wall is observed qualitative by Lucia (1966) in dense sand. There are 12 radiographic pictures captured during the displacement increment of the wall. A shear band is a thin weak zone occurred by dilatancy, it consists of a loosely packed sand. This difference in density can be visualized by the radiographic method. No information is available about the scale or the amount of wall displacement.

The onset of the shear band is clearly visible in picture 3 and it reaches the surface in picture 6. From picture 6 on a secondary shear band is developing. Unfortunately no force-displacement diagram has be found to connect to the pictures. Because of the lack of information it is not fully clear under what circumstances the secondary shear band occurs.

Figure 4-1 Rigid vertical wall translated into a dense sand (Lucia, 1966)
This information about the development of the shear band can be used to compare with the footage available from the experiments in this thesis. It only gives qualitative information about the shear band like the shape.

### 4.2 Boundary effects

Because physical modelling in a laboratory always takes place in a limited space, boundary effects cannot be neglected. From many experimental studies only pictures from the sides are available, and no information is given about the friction of the glass. Therefore the boundary effects of the glass are hard to quantify or qualify. Only the study of Gutberlet et al. (2013) shows the effect of the glass walls of the strongbox due to a top view picture.

![Wall movement](image)

*Figure 4-2 Photograph of the model test set-up with PIV camera. The dashed white lines show the 'submergence' and 'emergence' of the failing surface, cylindrical in shape (Gutberlet et al., 2013)*

From the observed top trace of the shear band in Figure 4-2, Gutberlet et al. (2013) conclude that the frictional resistance of the glass is negligible because the difference between the glass and the centre of the model is approximately 5% in average compared to the total distance from the wall. No arguments are given for this conclusion.

The ratio of wall width to wall height for this study is 5.8. Because the wall width and height ratio of the experiment in this thesis are significantly lower, namely 2.4, the conclusions of the study of Gutberlet et al. (2013) cannot be directly assumed. It gives a good indication that the wall friction effect will be more than 5% and therefore possibly cannot be neglected.

The curvature of the shear zone at the surface near the glass shows that the wall friction will cause an increase in the force necessary to move the wall. The friction prevents the sand from moving near the glass surface. This looks trivial, but Figure 4-2 confirms this theory.

### 4.3 Particle size effects

During a centrifuge flight the scaling laws prescribe an increase in dimensions, forces and stresses. This also affects the grain size of the sand. In a preferred situation the average grain size is N times smaller for an experiment during a centrifuge flight but the mechanical behaviour need to be identical. A series
of test have been done by Bolton and Lau (1988) to see the difference in behaviour between ‘normal’ sand versus its (smaller) crushed version. From these tests it is concluded that problems related to ruptures or permeability give unwanted differences in behaviour. This conclusion is shared by Madabhushi (2014), he argues that most often scaling down the grains gives different and undesirable behaviour e.g. in the stress-strain relationship because of the differences in mineralogy.

On the other hand: if the sand is not downscaled, it will result for example in a 20 mm rock aggregate instead of the 0.2 mm fine sand in the scale model at 100 g. Madabhushi (2014) argues that it is necessary to use the same soil in the centrifuge model as in reality to capture the true behaviour of the soil. The most important argument he gives is that it enables us to observe the correct deformations while the soil mobilises appropriate stiffness for the strains induced.

For this thesis it means that a scaled down model sand is not desirable and therefore the prototype sand will be used in the experiments. This conclusion has some consequences for other aspects like the increase in shear band thickness, this is described in next subchapter.

4.4 Shear band thickness

The width of the shear band depends on the grains affected by dilatancy and is rather wide directly after the failure of the sand and will gradually decrease during post failure deformation according to Mühlhaus and Vardoulakis (1987) (see Figure 4-3). From this study it is concluded by Stuit (1995) that the width of the final shear band is expected between 10 to 20 times the $d_{50}$.

![Figure 4-3: Shear band thickness for various post failure states (Mühlhaus and Vardoulakis, 1987)](image)

Stuit did further research to the shear band width in 1995. His conclusions are that the shear band width is linear depended on the mean particle diameter and that this linearity is not affected by the acceleration. This conclusions can be drawn from the data points of the graph in Figure 4-4.

For this thesis it means that the model shear band width in the experiment is $N$ times larger than the prototype shear band width. The decrease in shear band width will be show up probably but this is not 100% sure because the type of test in this thesis is different.
No experimental study has been found where the particle size (and therefore the shear band width) is changed at different experiments, and where the measured earth pressure versus displacement is monitored. Such a study is very important to understand the effect of the particles who are in fact too large (Chapter 4.3). Therefore the other type of studies are being consulted and a very useful numerical one is found.

A numerical study has been published by Tejchman et al. (2011) where the particle size effect on the earth pressure forces is studied. A three dimensional discrete model YADE developed at University of Grenoble was used to do the comparison. The models show large similarities with the experiments in this thesis: the wall is translating and passive zone can develop without constrains. This model correlates the opinion of Stuit earlier in this subchapter about the linearity in grain size and shear band width: the larger the grains, the larger the shear band width.
Figure 4-6 shows the earth pressure versus wall displacement of the models in Figure 4-5 with various grain size diameters (a) $d_{50} = 5$ mm, (b) $d_{50} = 1$ mm and (c) $d_{50} = 0.5$ mm. The shape of the force-displacements graphs are about the same but there is a significant difference in peak force. Larger diameter sand will give a 30% larger passive resistance according to this model. The scale difference in diameter between a and c is 10 times.

Although the mean grain diameter difference and the dimensions of the boundaries are different in the experiment in this thesis it gives an indication that the effect cannot be neglected. For the comparison of the centrifuge model and the prototype model this indicates that the centrifuge model will give an overestimation of probably more than 30% of the real prototype forces because of the difference in grain size diameter of more than 100 times.
5 Summary literature review
In the literature review report mainly experimental studies regarding passive soil behaviour have been analysed and compared. The most relevant and important conclusions for this research are used for the design process, execution and interpretation of the centrifuge tests planned for the thesis about the physical modelling of overlapping passive zones.

5.1 Experiments related to passive earth pressures
Translating a wall into a soil body gives the most distinct and longest shear band in comparison with a rotating wall (Figure 5-1). This translating results therefore in the largest passive resistance of the soil because the largest soil mass is pushed. From the experimental studies it is not possible to draw an uniform conclusion about the dimensions of the shear band, only about the shape.

Soil density plays a major role in the amount of passive resistance. A dense soil tends to withstand more passive force in comparison with a loosely packed soil (Figure 5-2). Different experimental studies show that dilatancy in medium dense soils results in lower soil strength after the grain skeleton is ‘broken’. Therefore there is a peak visible in the force vs. wall displacement diagram. An important conclusion is that passive failure in medium dense soil has fully occurred when the monitored force has a decreasing trend and reaches a plateau state.

The shape of the shear band will tend to be short, steep and planer in case of low wall friction. Loosely packed sand shows the same behaviour. A denser sand and wall friction gives a curved and longer shear band. In case of post failure wall displacement ($d/h^4 \geq 0.2$) a second shear band can occur due to the possible accumulation of soil close to the wall. The literature also shows that full passive failure of the soil body is expected between 0.05 and 0.10 $d/h$. To have at least this amount of movement in the physical model is a very important design aspect.

$^4 d/h$: wall displacement vs. wall height ratio
5.2 Comparison numerical models and centrifuge research

For the validation of the intensification factor it is important to control the forces and displacements in the physical model in the same way as it can be done in a numerical model. Failure initiation by means of forced displacement of the wall is the best and most suitable method for this thesis. Another conclusion from the literature review is that a more complex soil model does not always result in more satisfying validation results.

5.3 Visible and non-visible aspects during centrifugal testing

Several important aspects play a role during centrifugal testing of passive soil failure. Summarized lessons learned from literature are as follows:

- The shear band of a passive failure zone starts growing at the toe of the translating wall. After it reaches surface level a secondary shear band can occur;
- The boundary effects of the side glass causes an increase in load necessary for passive failure. The boundary effect are significant and cannot be neglected, but have to be taken into account. Due to monitoring the top surface the boundary effects can be quantified;
- The grain size diameter of the model sand will be the same as the prototype sand because there are too many complications when the grain size is downscaled together with the model;
- Working with larger model grains than prototype grains will result in larger passive forces in the model;
- Shear band thickness is linearly depended on the mean grain size of the model. Therefore the conventional scaling laws can be applied on the shear band after the shear band is corrected for the grain size diameter.
6 Bibliography


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Appendix B: Sand characterisation

Sieve test results

Table 1 Sieve test results 1

<table>
<thead>
<tr>
<th>Sieve size [mm]</th>
<th>Amount [gr.]</th>
<th>Amount [%]</th>
<th>sum</th>
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Table 2 Sieve test results 2

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<th>Sieve size [mm]</th>
<th>Amount [gr.]</th>
<th>Amount [%]</th>
<th>sum</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>0</td>
<td>0.0</td>
<td>100.0</td>
</tr>
<tr>
<td>0.425</td>
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<td>85.0</td>
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<tr>
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<td>57.9</td>
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<tr>
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<td>33</td>
<td>24.8</td>
<td>33.1</td>
</tr>
<tr>
<td>0.212</td>
<td>24</td>
<td>18.0</td>
<td>15.0</td>
</tr>
<tr>
<td>0.15</td>
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</tbody>
</table>

Table 3 Sieve test results 3

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<tr>
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<th>Amount [gr.]</th>
<th>Amount [%]</th>
<th>sum</th>
</tr>
</thead>
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<tr>
<td>0.5</td>
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<tr>
<td>0.425</td>
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<td>86.8</td>
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Density test results

Table 4 Density test results: maximum density

<table>
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<tr>
<th>Test</th>
<th>Weight [gr.]</th>
<th>dry unit weight. $\gamma$ [kN/m$^3$]</th>
<th>Porosity [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>189.02</td>
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</tr>
<tr>
<td>2</td>
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</tr>
<tr>
<td>3</td>
<td>191.06</td>
<td>16.68</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>191.23</td>
<td>16.70</td>
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<tr>
<td>5</td>
<td>190.68</td>
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<td>Average</td>
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<td>0.372</td>
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</table>

Table 5 Density test results: minimum density

<table>
<thead>
<tr>
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<th>Weight [gr.]</th>
<th>dry unit weight. $\gamma$ [kN/m$^3$]</th>
<th>Porosity [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
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<td>3</td>
<td>161.76</td>
<td>14.12</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>161.66</td>
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<td>5</td>
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Direct shear test results

Table 6 List of direct shear tests sand

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<th>Test no.</th>
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<th>Normal stress [kPa]</th>
<th>Shear stress [kPa]</th>
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<td>Loose</td>
<td>30.4</td>
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<td>Loose</td>
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<td>31.0</td>
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<td>5</td>
<td>Loose</td>
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<td>67.5</td>
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<td>Loose</td>
<td>107.3</td>
<td>68.0</td>
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<td>12</td>
<td>Dense</td>
<td>30.4</td>
<td>36.0</td>
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<td>Dense</td>
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<td>83.0</td>
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<td>Dense</td>
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</table>
Figure 1: Test data from direct shear test at different normal stresses (sand RD = 0%).

Figure 2: Test data from direct shear test at different normal stresses (sand RD = 50%).
Figure 3 Test data from direct shear test at different normal stresses (sand RD = 100%)

Dilatancy

Figure 4 Vertical vs. horizontal displacement of direct shear tests with medium dense sand (RD=50%)
Triaxial test results

**Figure 5** Triaxial test results (full)

**Figure 6** Triaxial test results (zoom to obtain E<sub>50</sub>)
Table 7 Desk study of average grain sizes in western part of the Netherlands

<table>
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<tr>
<th>Project</th>
<th>Location</th>
<th>Depth sample</th>
<th>D60</th>
<th>D10</th>
<th>D60/D10</th>
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</thead>
<tbody>
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<td>359</td>
<td>172</td>
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</tr>
<tr>
<td>2</td>
<td>1016 Diemen</td>
<td>7.9</td>
<td>331</td>
<td>149</td>
<td>2.2</td>
</tr>
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<td>3</td>
<td>1016 Diemen</td>
<td>9.9</td>
<td>391</td>
<td>188</td>
<td>2.1</td>
</tr>
<tr>
<td>4</td>
<td>1098 Wateringen-Bleiswijk</td>
<td>12.0</td>
<td>150</td>
<td>63</td>
<td>2.4</td>
</tr>
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<td>120</td>
<td>63</td>
<td>1.9</td>
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<td>6</td>
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<td>150</td>
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</tr>
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<td>125</td>
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<td>1.8</td>
</tr>
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<td>1.0</td>
<td>400</td>
<td>110</td>
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</tr>
<tr>
<td>9</td>
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<td>4.0</td>
<td>180</td>
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</tr>
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<td>1233 Rotterdam</td>
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<td>200</td>
<td>80</td>
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<td>220</td>
<td>90</td>
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<td>Spijkenisse</td>
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</tr>
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</tr>
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<td>663 Voorburg</td>
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<td>210</td>
<td>135</td>
<td>1.6</td>
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<td>18</td>
<td>806 Rijswijk</td>
<td>6.5</td>
<td>160</td>
<td>105</td>
<td>1.5</td>
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<tr>
<td>19</td>
<td>806 Rijswijk</td>
<td>9.5</td>
<td>110</td>
<td>70</td>
<td>1.6</td>
</tr>
<tr>
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<td>806 Rijswijk</td>
<td>7.5</td>
<td>170</td>
<td>90</td>
<td>1.9</td>
</tr>
<tr>
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<td>806 Rijswijk</td>
<td>8.5</td>
<td>110</td>
<td>63</td>
<td>1.7</td>
</tr>
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<td>22</td>
<td>861 Binckhorst, Den Haag</td>
<td>6.1</td>
<td>150</td>
<td>95</td>
<td>1.6</td>
</tr>
<tr>
<td>23</td>
<td>1243 Rozenburg-Rotterdam</td>
<td>0.3</td>
<td>180</td>
<td>95</td>
<td>1.9</td>
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<tr>
<td>24</td>
<td>1243 Rozenburg-Rotterdam</td>
<td>8.3</td>
<td>190</td>
<td>95</td>
<td>2.0</td>
</tr>
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</table>
Appendix C – Interface characterization

Steel friction – type B

Table 1 List of direct shear test sand vs. steel type B

<table>
<thead>
<tr>
<th>Test no.</th>
<th>Density</th>
<th>Steel roughness [μm]</th>
<th>Normal stress [kPa]</th>
<th>Shear stress [kPa]</th>
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</thead>
<tbody>
<tr>
<td>1.</td>
<td>Medium dense</td>
<td>1.3</td>
<td>15.1</td>
<td>4.0</td>
</tr>
<tr>
<td>2.</td>
<td>Medium dense</td>
<td>1.3</td>
<td>30.4</td>
<td>7.5</td>
</tr>
<tr>
<td>3.</td>
<td>Medium dense</td>
<td>1.3</td>
<td>76.5</td>
<td>16.5</td>
</tr>
<tr>
<td>4.</td>
<td>Medium dense</td>
<td>1.3</td>
<td>107.3</td>
<td>23.0</td>
</tr>
<tr>
<td>5.</td>
<td>Medium dense</td>
<td>2.4</td>
<td>30.</td>
<td>8.5</td>
</tr>
<tr>
<td>6.</td>
<td>Medium dense</td>
<td>2.4</td>
<td>76.5</td>
<td>20.5</td>
</tr>
<tr>
<td>7.</td>
<td>Medium dense</td>
<td>2.4</td>
<td>107.3</td>
<td>26.5</td>
</tr>
<tr>
<td>8.</td>
<td>Medium dense</td>
<td>5.3</td>
<td>30.4</td>
<td>14.0</td>
</tr>
<tr>
<td>9.</td>
<td>Medium dense</td>
<td>5.3</td>
<td>76.5</td>
<td>34.0</td>
</tr>
<tr>
<td>10.</td>
<td>Medium dense</td>
<td>5.3</td>
<td>107.3</td>
<td>46.5</td>
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<td>8.6</td>
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</tr>
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<td>Medium dense</td>
<td>8.6</td>
<td>76.5</td>
<td>42.0</td>
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<td>Medium dense</td>
<td>8.6</td>
<td>107.3</td>
<td>57.0</td>
</tr>
</tbody>
</table>

Figure 1 Test data from direct shear test at different normal stresses with steel (1.27 μm. type B)
Figure 2 Test data from direct shear test at different normal stresses with steel (2.4 μm. type B)

Figure 3 Test data from direct shear test at different normal stresses with steel (5.3 μm. type B)
Figure 4 Test data from direct shear tests at different normal stresses with steel (8.6 μm, type B)

Figure 5 Failure points direct shear tests sand vs. different steel roughness, shown are the residual shear stress points
Steel friction – Polished surface/Type A

Table 2 List of direct shear test sand vs. steel polished surface/type A

<table>
<thead>
<tr>
<th>Test no.</th>
<th>Density</th>
<th>Steel roughness [μm]</th>
<th>Normal stress [kPa]</th>
<th>Shear stress [kPa]</th>
</tr>
</thead>
<tbody>
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<td>1.</td>
<td>Medium dense</td>
<td>Polished surface</td>
<td>15.1</td>
<td>3.0</td>
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<td>2.</td>
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<td>Polished surface</td>
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<td>5.5</td>
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<tr>
<td>3.</td>
<td>Medium dense</td>
<td>Polished surface</td>
<td>76.5</td>
<td>17.0</td>
</tr>
<tr>
<td>4.</td>
<td>Medium dense</td>
<td>Polished surface</td>
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<td>22.5</td>
</tr>
<tr>
<td>5.</td>
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<td>30.4</td>
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<td>7.</td>
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<td>107.3</td>
<td>44.5</td>
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Figure 6 Test data from direct shear tests at different normal stresses with steel (polished surface)

Figure 7 Test data from direct shear tests at different normal stresses with steel (6 μm. type A)
Figure 8 Failure points direct shear tests sand vs. steel polished/type A, shown are the residual shear stress points

Direct shear test with ‘frictionless’ wall

Figure 9 Test data from direct shear tests at different normal stresses with steel and lubrication layer
Plexiglas

Table 3 List of direct shear test sand vs. Plexiglas

<table>
<thead>
<tr>
<th>Test no.</th>
<th>Density</th>
<th>Normal stress [kPa]</th>
<th>Shear stress [kPa]</th>
<th>(Peak) Shear stress [kPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Medium dense</td>
<td>30.4</td>
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<td>30.4</td>
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<tr>
<td>4.</td>
<td>Medium dense</td>
<td>76.5</td>
<td>13.9</td>
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Figure 10 Test data from direct shear tests at different normal stresses with Plexiglas
Stuk D/P is een vaste verbinding!
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<th>Benaming</th>
<th>Materiaal</th>
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<td>RVS</td>
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<td>½</td>
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<td>RVS</td>
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<tr>
<td>T</td>
<td>8</td>
<td>h</td>
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Schaal : 21/7
Datum : BLE MÅTEN IN MY
Getekend : O.V. MAARS 1992
Geconstr. : BEN. + TOL. IN OVERLEG
Gezien :

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**Informatie:**
Joop van Leeuwen 3427
Herke Stuit 3325
Stuk D/0 is een vaste verbinding!
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Schaal: 1:1  Datum: ALLE MATEN IN MM
Getekend: J. van Leeuwen  MAART 92  BEN + TDL. IN OVERLEG
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Gezien:  

AANVULLING BULLDOZER 1990

TU Delft  Faculteit der Civiele Techniek
Technische Universiteit Delft  Vakgroep Waterbouwkunde
Joop van Leeuwen 3427  Herke Stuit 3325

Informatie:
VERZINKEN AAN DEZE KANT!

MET SCHERP
Appendix E - Checklist centrifuge modelling

This checklist is set up to model the passive resistance of a horizontally moving, stiff wall who can be used to model the overlap of passive zones physically. The checklist is divided in three subchapters:

- Preparation of the model;
- Performing the centrifuge test;
- Processing the experiment.

This checklist must be used together with the logbook made for these tests.

1.1 Preparation of the model

1. Read this checklist **completely** before going to the next point;
2. Bring the strongbox to the preparation room, don’t prepare the soil in the centrifuge room because fine materials can damage the electronics;
3. Make sure no grains are in between the Plexiglas and the aluminium (see picture below), if necessary dismount the Plexiglas box, clean the surfaces and mount it again. This step is necessary to keep the scratching of the sand grains to the minimum;

![Figure 0.1: Empty strongbox before test](image)

4. In case of modelling the (possible) overlapping of passive zones it is important to prepare the symmetric wall. Apply the silicone grease on the metal surface and place the latex membrane (see picture).
5. In case of a simulation of the overlapping passive zones, locate the symmetric wall on its desired position. In case of the unrestrained experiments locate the wall as far as possible;
6. Select the right bar which hold the symmetric wall on its place, keep the length sticking out through the back wall to a maximum of 100 mm;
7. Place two aluminium strips on the bottom of the strongbox behind the symmetric wall to keep it vertical during the test. Make use of the small blocks to vary the length of the strips necessary for the specific w/d ratio;
8. Check if the bulldozer wall has a gap of at least 2 mm with its own frame, this is due to unexplainable loads at about 1 mm displacement (see test A – H);
9. Make sure all the nuts and washers are tight and correctly mounted;
10. Place the empty strongbox in the centrifuge and change the position of the wall with MP3 so that the width is exactly as desired. Make sure that the last 500 micron the wall is being pushed so that the load cell makes contact with the wall;

11. Weight the empty box every single tests because of the different bars each test etc.;
12. Bring the strongbox to the preparation room and fill it with the falling height chosen by the user (funnel, 65 cm PVC pipe = rd 50%). Keep the pipe vertical during pouring the sand;
13. Flatten the sand surface with the strip and remove the unnecessary soil. Don’t dig too much in the soil because you will disturb the sand;
14. Weigh the complete setup on a scale for the density check;
15. Do a check for the density, if it is not as expected: remove the sand and fill it again;
16. Check if the counter weight is sufficient (± 1 kg);
17. Bring the strongbox with the sand to the centrifuge room and place it on the platform parallel to the radius **(without vibrating)**. Make sure the pushing part of the strongbox is at the side which is the closest to the centre of the centrifuge (see picture below);
18. Connect the load cell plug and the motor plug to the corresponding sockets on the centrifuge;
19. Tighten all the loose cables with tie-raps to the basket, make sure that around the hinge there is sufficient space for the cables to move;
20. Check if no objects are in the centrifuge, close and lock the tool boxes and remove light objects from the edge of the centrifuge. Once in a while if it is necessary you have to vacuum clean the centrifuge. Heavy winds will blow stuff away. It is mandatory to ask someone to check this step.
1.2 Running the centrifuge test

1. **Always** perform the test with a technician;
2. Make sure that everything is fixed properly and that there are no loose parts;
3. Switch on the bulldozer machine, so that the electric motor works (see picture below);

![Figure 0.6: Switch on the bulldozer machine (off position)](image)

4. Start the live stream with the program ‘Xplit’, if necessary login with the youtube account [A.Askarinejad@tudelft.nl](mailto:A.Askarinejad@tudelft.nl) (password: Thalandra7&). The live stream is always on: [https://www.youtube.com/channel/UCUdbtPZAVsBTxRfUb2RKi_g/live](https://www.youtube.com/channel/UCUdbtPZAVsBTxRfUb2RKi_g/live), check if it is online.
5. Startup the webcamviewer and select the webcam in front of the centrifuge room door;
6. Switch on the two TP link routers and start the program ‘IC capture’ the right settings can be loaded (test_f). Check if the model is completely visible on the screen. Higher FPS than 2 are possible but then there is a risk of losing the signal during a flight;
7. Mount the GoPro if it is not already there and select the right settings with the GoPro app (time lapse, 5 seconds interval). Start making pictures with the go-pro
8. Start-up the centrifuge motor with the key, **lock** the door of the centrifuge room;
9. Restart the program MP3, make sure the BullDozer 2 settings are loaded (see picture below);
10. Check the starting load, the position of the sheet pile wall and the camera positions;
11. Position the 3 windows with the camera’s on one screen and the program MP3 on the other so you have a good overview;
12. Start the data acquisition (red box, see picture below), the file name should be: **date_tests#**, the data interval (Delta T) should be 0.5 second;
13. Increase the RPM up to 280 RPM (100g) or 250 RPM (80g). See green box in picture above;
14. Fill in the desired wall movement speed (μm/s) and the desired amount of wall movement (μm) The numbers can be filled in in the blue box in the picture above;
15. Look at the actual position of the loading frame (PosX[μm]). Check that this position represents the real(observed)wall position, then fill in the end position. This should not be larger than 14 mm from the starting position.
16. Start the ‘sequence timer’ in IC capture and make sure that the pictures are saved in the destination folder every 5 seconds;
17. Press only start when you have double checked the numbers (RPM, wall speed, wall end position);
18. Check immediately if the software shows an increase of movement and the load [N] and that the position changes. If not press the start button again;
19. It is possible to switch to the second screen with the arrows, see picture below;
20. If the timer in IC capture turns out to be negative the connection is lost, restore the connection as soon as possible by reloading the settings ‘test_f’;
21. Stop the wall movement immediately if the load cell exceeds 5000 N (purple box, picture above);
22. If the wall is moved to the predefined location the movement stops automatically;
23. Press ‘stop timer’ in the program IC capture so that the time laps stops;
24. You can now slow down the centrifuge by filling in 0 in the green box;
25. Stop the data acquisition (button in yellow box) as soon as the centrifuge is stopped;
26. Unlock the centrifuge room and switch off the centrifuge motor (Han);
27. Switch off the bulldozer machine;

1.3 Processing the experiment
1. Inspect the sample;
2. Make pictures from different angles to determine i.e. the grains between the wall and the Plexiglas;
3. Dismount the cables and plugs, and bring the setup to the preparation room;
4. Empty to sand into the bucket carefully, don’t scratch the Plexiglas or waste sand
5. Possible check for friction;
6. Check if there is sand intruded in between the Plexiglas and the steel. If so: remove this;
### Appendix F – Physical model results

Table 1: Overview offset corrections and friction corrections

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<th>No.</th>
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<th>Load offset at 0 μm wall displacement</th>
<th>Correction for PS friction</th>
<th>Correction for PW friction¹</th>
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<td>1</td>
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<td>-110 μm</td>
<td>+50 N</td>
<td>-50 N</td>
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<td>2</td>
<td>B</td>
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<td>+50 N</td>
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<tr>
<td>3</td>
<td>C</td>
<td>-</td>
<td>+50 N</td>
<td>-</td>
<td>-250 N</td>
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<tr>
<td>4</td>
<td>D</td>
<td>-</td>
<td>+50 N</td>
<td>-</td>
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<td>+50 N</td>
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¹ Friction applied linearly from 0 – 0.6 m (prototype) wall displacement
Figure 1 PW friction test results

Plexiglas-Soil (PS) friction
The relevant parameters in the PS friction calculation are:

- Residual shear strength DC sand, $\varphi' = 31.4$ degrees
- Wall friction parameter of medium dense DC sand to the Plexiglas, $\delta = 9.5$ degrees

The formulas used to calculate the neutral and passive earth pressure coefficient are:

$$K_0 = 1 - \sin \varphi' = 0.48$$

$$K_P = \frac{1 + \sin \varphi'}{1 - \sin \varphi'} = 3.18$$

The shear surface is assumed a triangle of $15 \times 5$ cm and divided over 5 parts. The values for the horizontal stresses on each part of the plexiglas is calculated bases on the vertical stress and the earth pressure coefficients. With the horizontal stress, the shear area and the wall friction angle the friction load is calculated.
Appendix G – Meshes of interrogation patches

Figure 1 Mesh of interrogation patches of 30 x 30 pixels, used in test H,3.0,100g and J,3.0,100g

Figure 2 Mesh of interrogation patches of 30 x 30 pixels, used in test K,2.0,100g and K,2.0,100g
Figure 3 Mesh of interrogation patches of 30 x 30 pixels, used in test $T_{2.0,80g}$

Figure 4 Mesh of interrogation patches of 30 x 30 pixels, used in test $R_{1.5,80g}$ and $S_{1.5,80g}$