Medium size shear box testing to determine the shear strength of Bremanger rock fill.
2 Abstract

This report describes the result of direct shear tests conducted on rock fill, the Bremanger sandstone, used in the construction of the project Maasvlakte 2 in the Netherlands. The tests are conducted with a medium size direct shear box (500*500*400mm). In order to investigate the possible effects of two main factors (density and particle size) on the rock fill material, a number of medium scale direct shear tests have been conducted with different low stress levels. Furthermore, two different models (Mohr-Coulomb Model and Baton’s model) are studied to determine the rockfill strength.

This report is part of a larger study (MSc-Thesis) conducted by XIAOSHAN Sun on the determination of the shear strength of Bremanger aggregate.

Figure 1 – ‘blockbuster’, the crane that will be used in the construction of the rigid sea breaker. (Olierhoek, 2010)
3 Introduction

Rockfill is generally produced at quarries and increasingly used as a fill or base material for offshore structures, dams, road embankment and foundation for buildings (Lee D. S., Kim, Oh, & Jeong, 2008; Charles, 1991).

At the port of Rotterdam the consortium of Boskalis and Van Oord (PUMA) is using rock fill for a new harbor, called the Maasvlakte 2 (MV2). The quarried rock is used to create a cobble beach and a walk way for a crane (figure 1). The cobble beach will form a transition between the soft dunes and the rigid seawater breaker protecting the MV2. It will dissipate wave energy and protect the dunes from erosion. The crane walk way will be used to put in place the 40 t armourstones of the sea water breaker. The sea defense has to match strict safety standards.

In this BSc thesis the main goal was assisting XIAOSHAN Sun with her MSc. research on the shear strength characteristics of Bremanger aggregate. This was done by conducting tests with a medium size shear box (500mm*500mm*400mm) to determine the shear properties of crushed sandstone from Dyrstad, Bremanger in Norway.

The project consists of three steps:

**Building**
- Constructing the apparatus
- Calibrating

**Testing**
- Conducting measurements on Bremanger aggregate.
- Determining the shear strength using Mohr-Coulomb and Barton’s Model

**Recommendations**
- Making recommendations to improve the medium size shear box for further research

I would like to thank XIAOSHAN Sun for her help in testing and processing the data, Dr. Ir. Dominique Ngan-Tillard for supervising my work, and Arno Mulder for all the assistance during my project. Secondly I would also like to thank all the staff at the department of geo-engineering for helping me and answering my questions.
4 Theoretical Background
Several tests can be conducted to determine the characteristics of a rock fill. In these tests we measure different parameters; Uniaxial Compressive Strength, shear stress, porosity, density, and roughness. With the help of these parameters the friction angle of the rock fill can be calculated. There are models to predict behavior of rock fill based on these parameters. When test results are obtained they can be used in the design of a construction.

4.1 Mohr Coulomb Theory
The Mohr-Coulomb equation is one way to describe the shear strength as a function of the normal stress (equation 1). The line obtained from several tests can be plotted (figure 2) and this will give an idea when failure of a rock fill will occur. When the shear stress is greater than the shear stress envelope then failure of the rock fill will occur.

\[ \tau = \sigma \tan(\phi) + c \]

In this equation the \( \tau \) is the shear strength in kPa, \( \sigma \) is the effective normal stress in kPa, \( c \) is the cohesion, and angle \( \phi \) in degrees is called the internal friction angle. In many projects concerning rock fill the cohesion is set to zero. The shear strength at low normal stress is therefore underestimated in engineering projects. This positively contributes to the safety factor.

Authors (INDRARATNA, 1994) have shown that at low confining pressures the shear strength curve for rockfill is non-linear (figure 2). They found out that for stresses lower than 500kPa the shear stress envelope is highly curved, as showed in figure 2. At increasing stresses the friction angle (\( \phi \)) is decreasing rapidly, figure 3. This supports the theory that if normal stress is increased dilation is suppressed and therefore also the friction angle (\( \phi \)) is reduced. In figure 3 there is no data below 35kPa, the area were we conducted measurements.

![Figure 2](image2.png)

**Figure 2**

![Figure 3](image3.png)

**Figure 3 – Variation of friction angle, \( \phi \) with normal stress \( \sigma \) (INDRARATNA, 1994)**
4.2 Shear strength determination of rock fill – Barton’s model

Barton’s model (Barton, Shear Strength of Rockfill, Interfaces and Rock Joints, and their Points of Contact in Rock Dump Design, 2008) can be used to estimate shear strength of a rock fill. Based on the following parameters; uniaxial strength; particle size; degree of particle roundness; and porosity following compaction. The general equation for describing the peak shear strength is;

\[ \tau = \sigma_n \tan \left( JRC \times \log \left( \frac{JCS}{\sigma_n} \right) + \Phi_r \right) \]

Equation 2 - (Barton, Shear Strength of Rockfill, Interfaces and Rock Joints, and their Points of Contact in Rock Dump Design, 2008)

in which \( \tau \) = peak shear strength in kPa; \( \sigma_n' \) = effective normal stress in kPa; JRS = joint roughness coefficient; JCS joint wall compression strength; and \( \phi \) = residual angle of friction.

JCS – Is the joint wall compression strength, which can be measured directly or estimated from the UCS value. Weathering of the rock surfaces reduces the JCS value.

JRS - The joint roughness coefficient is the ratio between the roughness amplitude and the sample length. Completely polished surfaces have a JRS equal to zero. With the help of a tilt test and equation 3 the JRS can be determined.

\[ JRS = \frac{\alpha - \Phi_r}{\log \frac{JCS}{\sigma_n'}} \]

Equation 3 - (Barton, Shear Strength of Rockfill, Interfaces and Rock Joints, and their Points of Contact in Rock Dump Design, 2008)

In which \( \alpha \) = tilt angle when sliding in degrees occurs; and \( \sigma_{n'} \) = effective normal stress in kPa when sliding occurs.

For rock fill applications we use an empirical approach to make a shear strength estimation. The equivalent strength of particles (S) and the equivalent roughness (R) to represent the behavior of rockfill. Rock fill material can have different origins for which we can correct the \( \phi \). This is called the equivalent roundness (R) of the rockfill material. We can determine this with the help of the porosity. To determine S numerous triaxial tests were conducted. This has lead to a numerical method to determine the size effect on the particle strength.

\[ \Phi = R \log \left( \frac{S}{\sigma_n} \right) + \Phi_b \]

\[ \tau = \sigma_n \tan \left( R \log \left( \frac{S}{\sigma_n} \right) + \Phi_b \right) \]

Equation 4 - (Barton, Shear Strength of Rockfill, Interfaces and Rock Joints, and their Points of Contact in Rock Dump Design, 2008) (Barton & Kjanesnli, Shear strength of rock fill, 1981)

![Figure 4](image1)

![Figure 5](image2)

Figure 4 – Method of estimating Equivalent Roundness (R) based on porosity of rockfill, origin of material, and the degree of roundness and smoothness of particles. (Barton & Kjanesnli, Shear strength of rock fill, 1981)

Figure 5 – Method of estimating equivalent strength(S) of rockfill based on Uniaxial Compressive Strength and \( d_{50} \) particle size. (Barton & Kjanesnli, Shear strength of rock fill, 1981)
4.3 Influence of particle size

Barton’s formula depends on the $d_{50}$ of the material. It is useful to understand the consequences of this dependency. When the internal friction angle is known an estimation of the shear strength of the material (equation 4) can be made. Figure 6 shows an area where particle size has considerable effect on the $\phi$. When increasing the particle size this effect becomes less important. We can also observe that at higher normal stresses the friction angle is getting more constant according to the Barton model.

![Figure 6 – Estimating of $\phi$ with n=31%, so R=5 and S dependant on $d_{50}$ particle size according to figure 5.](image)

Determining $\phi$ at a low normal stress is important to evaluate the capability of Barton’s model to model these conditions. An example is at the toe of a rock fill dam. There is low normal stress and $\phi$ is relatively high.

**Influence of distribution of particle sizes** – The maximum particle size has a considerable effect on the shear strength of rock fill. As stated above with the Barton’s model it is generally accepted that the shear strength decreases with increasing particle size, because the friction angle is decreasing. When the ratio of maximum particle size to sample diameter is increased, the friction angle is increased, thus the shear strength increases (Douglas, The shear strength of rock mass, 2002).

4.4 Influence of packing density

Rock fill with a higher density will have a higher shear strength (Douglas, The shear strength of rock mass, 2002). Therefore the shape of the Mohr-Coulomb failure envelope is affected. Dense rock fill will show a distinct curvature. It will have a distinct drop in friction angle as it reaches higher normal stresses. Loosely packed rock fill will not show a distinct increase in shear strength at low stresses. This is consisted with the idea that loosely material will needs less dilatation (volume expansion) because particles have more freedom to move and rotate during shearing. Dense material needs more dilation under low normal stress to shear. At higher normal stresses the dilation is restrained and particle shearing will occur. The two curves, high density and low density will merge, this is consisted with the general idea there is a constant volume/void ratio at high strains. Thus dilation is the main cause of the high friction angle during shearing with low normal stresses.

$$\frac{\tau}{\sigma} + \frac{dH}{dD} = \mu$$

(5.1)

$$\tan \Phi = \mu + \tan \psi$$

(5.2)

Equation 5 – (Lee D. S., Kim, Oh, & Jeong, 2008)
Explanation of equation 5.1; $\sigma$ is the effective normal stress in kPa; $\tau$ is the shear stress in kPa, $\mu$ is the friction coefficient also called friction angle $\phi$. In equation 5.2 the first term is the external mobilized friction ($\phi$), and the second term is the dilatancy ($\psi$). When the critical state is reached there is no volume expansion more $dH/dD=0$ and $\tau/\sigma = \tan \phi$. This results in a linear dependency $\tau = \sigma \tan \phi$ of the shear stress and the normal stress. We see from the equations the fact that a dense packing will have higher dilation as a result of the higher friction angle at low normal stress.

4.5 Particle angularity and strength
Angular rockfill has a higher shear strength than rounded rockfill. This because of the interlocking particles and increased dilation. An increased friction angle results in a higher shear strength. At high normal stresses this effect has less influence but opinions in the different papers are not consistent. Some state that the effect is opposite.

The Uniaxial Compressive Strength (UCS) of the material is also important for the shear strength of the rock fill. When the UCS increases, the ratio $\sigma_n/\sigma_{UCS}$ decreases. Dilation will contribute to a higher shear strength. A lower UCS will lead to less dilation but more crushing of particles and thereby lowering the shear strength (Douglas, The shear strength of rock mass, 2002).

4.6 Summary of factors affection friction angle
This paragraph summarizes the several parameters that influence the shear strength and thus the friction angle. It may be concluded that effective normal stress, particle size and density influence the friction angle significantly. Factors as particle angularity and spreading of the particle size may have a less significant influence.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Effect on $\phi$ while increasing parameter</th>
<th>comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Effective normal stress, $\sigma_n$</td>
<td>decrease</td>
<td>With high $\sigma_n$, the $\phi$ is decreasing rapidly</td>
</tr>
<tr>
<td>UCS of rock</td>
<td>increase</td>
<td>More dilatancy, higher shear strength</td>
</tr>
<tr>
<td>Density</td>
<td>increase</td>
<td>More dilatancy, higher shear strength</td>
</tr>
<tr>
<td>Particle size, $D_{50}$</td>
<td>decrease</td>
<td></td>
</tr>
<tr>
<td>Ratio $D_{max}/D_{50}$</td>
<td>increase</td>
<td></td>
</tr>
<tr>
<td>Angularity</td>
<td>increase</td>
<td></td>
</tr>
</tbody>
</table>

4.7 Parallel gradation method
In laboratory experiments it is not possible to do full scale testing. To conduct tests on laboratory scale we want to have the same particle size distribution. When we have the same particle size distribution we can scale the results. Size effects of direct shear box tests are underestimated in engineering according to (Cerato & Lutenegger, Specimen Size and Scale Effects of Direct Shear Box Tests of Sands, Vol. 29, No. 6). The size distribution can be checked with a sieving analysis. Investigation in the size effects of equipment and scaling of tested particles is not preformed in this study. Because the sieving analysis of the MV2 crane roadway is classified. An example of using a sieving analyses to check if the distribution is the same is given in figure 7.
5 Material, apparatus and testing procedure

5.1 Basic characteristics of crushed rock aggregates

**General description** - The crushed rock tested in this study is a quarry-blasted Devonian sandstone, called the Bremanger sandstone. The Bremanger sandstone is a dark colored rock with alternating black, grey and white layers. As it has sustained some metamorphism (appendix A - mineralogical description), it is a meta-sandstone rather than a sandstone. The aggregates are produced in a quarry in Dyrstad, Norway before being shipped to the Yangtze harbor in Rotterdam for usage in the coastal defense of the Maasvlakte 2 (MV2) constructed at the moment by the PUMA consortium in the North sea. The quarried rock is used to create a cobble beach and a walk way for a crane. The cobble beach will form a transition between the soft dunes and the rigid seawater breaker protecting the MV2. It will dissipate wave energy and protect the dunes from erosion. The crane walk way will be used to put in place the 40 t armourstones of the sea water breaker.
Material properties
Laboratory testing results are available for the Bremanger aggregates. Results provided by the aggregate supplier are listed in table 2.

Table 2 – Properties of the Bremanger sandstone. (Alnaes, Johannsson, & Myrvang, 1999)

<table>
<thead>
<tr>
<th>Bremanger aggregate</th>
<th>Average values after testing series</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus of elasticity [GPa]</td>
<td>93.4</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.310</td>
</tr>
<tr>
<td>Uniaxial compressive strength [MPa]</td>
<td>188.7</td>
</tr>
<tr>
<td>Angle of fracture [degrees]</td>
<td>156</td>
</tr>
<tr>
<td>Sonic velocity [m/s]</td>
<td>5664</td>
</tr>
<tr>
<td>Bulk density [kg/m$^3$]</td>
<td>2747</td>
</tr>
</tbody>
</table>

In our testing series we used two different fractions of the crushed aggregate, 31.5-50 mm and 31.5-80 mm. On site, the 60-100mm fractions are used on the cobble shore. Such fractions cannot be tested in the laboratory with an apparatus of reasonable size. Scaling is therefore applied.

5.2 Test apparatus

General description. The medium size direct shear test apparatus (figure 10 and figure 11) consists of a shear box that is 50 cm wide and long and 40 cm high, vertical and shearing loading units, a steel frame, force and displacement measuring devices, and a data acquisition system. The vertical load is exerted on the specimen with a loading plate with on top steel dead weight. The shearing load is applied with a electric motor with a worm wheel and reduction gear. The shearing load is applied to the lower shear box while the upper shear box is fixed with a force transducer on the steel frame. The dial gauges and the force transducers are connected to the data acquisition system mp3, a in house software developed at TU Delft.

The friction of the boxes on top of each other is 83.8 N. The calibration curves of the measuring devices are shown in Appendix B – measuring devices.

To prevent that rocks get trapped in between the edges of the boxes during shearing (figure 12), the movement of the top shear box was restricted by inserting wooden blocks between the top shear box and the steel frame, figure 10.

Figure 10 – Direct shear box at laboratory for geo-engineering at TU Delft the Netherlands.
Displacement transducers- The horizontal displacement dial gauge has a range of 20 cm. During testing the maximum allowable horizontal displacement is 10 cm maximum which corresponds to a shear strain of 20% maximum (10 cm displacement/50 cm length of shear box). The vertical gauge has a range of 2 cm, it was re-set during the test to extend the measuring range if necessary. The artificial shift in the data was removed and it was checked that the gradient of the vertical displacement versus horizontal displacement before and after correction was the same. The horizontal and vertical displacement transducers were calibrated such that -10 – 10 Volts correspond to 0 – 20 cm, and 0 – 10 Volts to 0 – 2 cm, respectively. Data acquisition was conducted with an interval of 1 second.

Load cells- The horizontal forces are measured with two 50 kN load cells. The load transducers were calibrated in a compression test machine from 0 – 10 Volts for 0 – 50kN. (appendix B - calibration). The range of the load cell (100 kN) limits the shear stress to 500 kPa (at shear strain of 20%). The vertical forces were not measured with a loading cell. Because dead weight was used it was fixed after installing the load.
The vertical loading with dead weight is limited. At the maximum, 800 kg of steel plates are applied onto the top plate, which correspond to a vertical stress of 32 kPa. The plates are secured for toppling by safety straps hanging loosely on the portal crane. Adding more dead weight was not considered safe, figure 14.

**Speed of testing** - The tests were all conducted with the same speed, 10mm/min as recommended in the handbook for testing (Mulder & Verwaal, 2006). In other publications (Lee D. S., Kim, Oh, & Jeong, 2008) this speed is also used.

**Shear box calibration** - The shear box was calibrated for friction by running the testing device several times without any rock fill inside. This resulted in a friction of 0.0838 kN. For the rest of the testing series we corrected for this friction. It was also checked whether or not the two horizontal load transducers were evenly loaded. By improving the initial alignment of the boxes, balanced loads were observed. However, depending on the re-arrangement of the gravels during shearing, unbalanced horizontal forces were observed.

### 5.3 Testing procedure

The aggregate tested was first manually sieved. The fraction used in the tests was washed to remove the fine clay layer coating the gravels. The material was weighted before being placed inside the shear box, to determine the rock fill density. Two different ways for filling the shear boxes were used to obtain different densities. For “normal density tests”, the box was filled in by pouring buckets of material in the shear box, without physically compacting. For “high density tests”, the material was divided in four parts. After pouring a part, compaction with a tamping rammer weighting 90 N took place (figure 13). Each layer was around ten centimeter high after compaction.

![Manually compacting with a tamping rammer.](image)

Then, the sample was first subjected to the designated vertical load (normal stress) and then sheared to 20% shear strain by applying horizontal loading, with a rate of 10mm/min. Tests were conducted under three different vertical loads: 1.4kPa (34kg dead weight), 20kPa (500kg dead weight), and 30kPa (800kg dead weight).

### 5.4 Test program

In the testing program, factors affecting the stress strain characteristics of Bremanger aggregates, such as density and particle size were investigated in the low effective normal stress domain. Several models (section 4, Barton, Mohr-Coulomb) have been used to describe rockfill under high normal
stresses, for example in studies performed for dam building. Few experimental results are available for low normal stresses, where the rock fill behavior is highly non-linear. The testing was performed to check if the models could be used in this low normal stress domain.

The maximum allowable particle size in the medium size shear box is determined by the dimensions of the shear box (section 5.2). Because the material which is used on site is too big for testing, smaller size material was used. To test the particle size effect of the aggregate, two different test series were conducted: tests where the largest particles were 80mm, and 50mm. Tests performed with the smaller fraction were performed on a normal and high density packing's corresponding to porosities of 42% and 48%, respectively.

To check the reproducibility of the test results, the tests conducted on the largest fraction were repeated once.

### 5.5 Overview of the tests conducted

Table 3 – Overview of the test conducted on Bremanger aggregate.

<table>
<thead>
<tr>
<th>Medium Size Shear Box</th>
<th>Category Full Name</th>
<th>Category Symbol</th>
<th>Tests</th>
<th>Density (Mg/m³)</th>
<th>Porosity (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Normal density (31.5mm&lt;P&lt;50mm)</td>
<td>M1</td>
<td>Three tests (34.12 kg, 495.97 kg, 806.9 kg)</td>
<td>1.40</td>
<td>48.15</td>
</tr>
<tr>
<td></td>
<td>High density (31.5mm&lt;P&lt;50mm)</td>
<td>M2</td>
<td>Three tests (34.12 kg, 495.97 kg, 806.9 kg)</td>
<td>1.56</td>
<td>42.22</td>
</tr>
<tr>
<td></td>
<td>Normal density (31.5mm&lt;P&lt;80mm)</td>
<td>M3</td>
<td>Six tests (two 34.12 kg, two 466.27 kg, two 777.2 kg)</td>
<td>1.40</td>
<td>48.15</td>
</tr>
</tbody>
</table>

Figure 14 – Shear box after testing, clearly visible vertical expansion (dilatancy) after 20% shear strain applied.
6 Results

6.1 General trends
As expected from previous testing on rock aggregates, the stress strain behavior of rock aggregate (figure 15) is nonlinear and is normal-stress dependant in this low stress area. The shear stress – horizontal strain curves (general trend) are similar for normal density and high density, only shear stress values are different, denser samples showing, as expected, a higher strength. All curves, except those obtained for the lowest normal stress, show stress softening, i.e., a peak followed by stress reduction towards a residual level, that is probably not yet reached during the tests. The peak occurs at strains around 8%. Up the peak stress, interlocking of particles contributes to the resistance of the rock fill. When the residual stress is reached, resistance is provided by friction between particles.

![Horizontal strain vs. Shear stress for M1 and M2 samples.](image1)

6.2 Dilatancy
All tests show dilation (vertical expansion) (figure 16, figure 17). Before shearing compaction of the rock fill under the applied normal load was observed. With the measuring setup we couldn’t measure the exact compaction.

Higher effective normal stresses reduce the dilatancy. This correlation between normal stress applied and dilatancy is clearly visible in picture 17. In figure 16 the difference between the two series of tests is visible, M1 1400kg/m3 and M2 1560kg/m3. High density packing ensures a higher dilatancy angle (ψ). As mentioned in equation 5 there is a correlation between the friction angle and the dilatancy angle. When the density of the sample is higher than the dilatancy higher thus also a higher friction angle. A higher friction angle will give a higher shear strength. We can conclude that increasing the density of a rock fill will increase the shear strength of the rock fill.
Figure 16 – Horizontal displacement vs. vertical displacement for M1 and M2 samples. Dilatancy angle M1 between 19-21 degrees, M2 between 28-31 degrees.

Figure 17 – Horizontal displacement vs. vertical displacement for M3 samples. Dilatancy angle between 19-21 degrees.

6.3 Mohr-Coulomb model

**Method** – Interpretation of the result using the Mohr-Coulomb model. From each test we use the data at max stress level. We use two different ways of analyzing the data with cohesion and without cohesion.

\[
\tau = c + \sigma_n \tan \phi \quad (c \neq 0)
\]

\[
\tau = \sigma_n \tan \phi \quad (c = 0)
\]

Equation 6

The peak shear strength is \(\tau\); the peak normal stress is \(\sigma_n\); the angle of friction is \(\phi\); and \(c\) is the cohesion. The friction angle can be obtained by conducting a few calculations.
Step 1: Plotting the peak shear strength for different normal stresses for each experimental category, the peak shear strength is plot on the y axis while the normal stress is plot on the x axis;

Step 2: Add a liner trend \((\text{intercept } \neq 0)\) for points of each category because the experimental points generally follow along lines, got an equation of this liner trend \((y=ax+b)\), so \(a\) is equal to \(\tan \varphi\), \(b\) is equal to the cohesive strength;

Step 3: Add a liner trend \((\text{intercept } =0)\) for points of each category, got an equation of the liner trend \((y=ax)\), so \(a\) is equal to \(\tan \varphi\).

**Results** – In Appendix C – graphs from tests M1, M2, M3 the results of all the test are published.

<table>
<thead>
<tr>
<th>Category Full Name</th>
<th>Category Symbol</th>
<th>Hypothesis 1 (without cohesion)</th>
<th>Hypothesis 2 (with cohesion)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Cohesion [kPa]</td>
<td>Friction Angle [degrees]</td>
</tr>
<tr>
<td>Normal density</td>
<td>M1</td>
<td>0</td>
<td>74.02</td>
</tr>
<tr>
<td>(31.5mm&lt;P&lt;50mm)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>High density</td>
<td>M2</td>
<td>0</td>
<td>79.10</td>
</tr>
<tr>
<td>(31.5mm&lt;P&lt;50mm)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Normal density</td>
<td>M3</td>
<td>0</td>
<td>77.58</td>
</tr>
<tr>
<td>(31.5mm&lt;P&lt;80mm)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Interpretation of the Mohr-Coulomb model friction angles** – With the results obtained the two different hypothesis can be compared.

-First can be observed that the \(R^2\) value is very low. This means that the data point don’t fit a linear trend line very accurate. The non linearity in the low normal stress domain may be cause of this.

-Secondly the friction angles are very high. Normally you would assume this would be around 50° (INDRARATNA, 1994) (Lee D. S., Kim, Oh, & Jeong, 2008). There may be several causes for this phenomenon. The low normal stress level and the nonlinearity of the shear stress vs. the effective normal stress, high gradient of shear envelope, \(\tan \varphi\). The side effects caused by the particle size \((d_{50})\) may be influencing the results. The high strength of the material, other authors (Lee D. S., Kim, Oh, & Jeong, 2008) have conducted test with rock fill with a lower UCS (60-136MPa). The UCS of the Bremanger is considerable higher (section 5.1; 188.7MPa av.) this may cause a higher friction angle thus a higher shear strength. Another way of determine the friction angle is given by (Lee D. S., Kim, Oh, & Jeong, 2008), \(\phi= 0.09 \text{ UCS } + 35.2 = 52^\circ\). Lee used material with al lower UCS, we may assume that this increasing strength would have effect on the friction angle (Douglas, determine the shear strength of rock masses, 2002).

**With/without cohesion?** – When comparing the two hypothesis the model with cohesion would be more accurate, by the higher \(R^2\) value. But more appropriate would be to question the testing conditions, given that we are approaching a nonlinear assumed relationship (INDRARATNA, 1994) with a linear approximation.

When conducting test at low normal stress and determining the friction angle with Mohr-Coulomb model there are some limitations.

- The number of tested conducted is limited, and they are all in a small normal stress range. Extending the normal stress range and the number of test will improve the accuracy.
- The material’s strength is causing high dilatancy and tipping of the dead weight, it is not clear if all the weight is causing vertical stress on the packing (this will be discussed in section 7).

6.4 Barton model

**Method** – Secondly the results will be interpret with the Barton’s model. For explanation of these procedure the formula’s used are explained.

\[ \Phi = R \log \left( \frac{S}{\sigma_n} \right) + \Phi_b \]  
(7.1)

\[ \tau = \sigma_n \tan \left[ R \log \left( \frac{S}{\sigma_n} \right) + \Phi_b \right] \]  
(7.2)

\[ i = R \log \left( \frac{S}{\sigma_n} \right) \]  
(7.3)

\[ \tau = \sigma_n \tan [i + \Phi_b] \]  
(7.4)  (7.3) in (7.2)

\[ \tan^{-1} \left( \frac{\tau}{\sigma_n} \right) = i + \Phi_b \]  
(7.5) \( \text{ATAN}[\tau/\sigma_n] \)

\[ \tan^{-1} \left( \frac{\tau}{\sigma_n} \right) = R \log S - R \log \sigma_n + \Phi_b \]  
(7.6) \( \text{ATAN}[\tau/\sigma_n] \)

Equation 7 – (7.1,7.2) (Barton, Shear Strength of Rockfill, Interfaces and Rock Joints, and their Points of Contact in Rock Dump Design, 2008) (Barton & Kjanernsli, Shear strength of rock fill, 1981), (7.3 -7.6) based on Barton.

where \( \tau \) is peak shear strength;  
\( \sigma_n \) is peak normal load;  
\( \Phi \) is the basic friction angle;  
\( R \) is equivalent roughness of rockfill;  
\( S \) is equivalent strength of rockfill particles;  
i is the structural component of strength (equal to dilatancy angle, \( \psi \), equation 5);

Step 1: Estimate the basic friction angle \( \Phi_b \). Assume i is the dilatancy angle. i can be calculated by the slope of horizontal displacement-vertical displacement curve at max. stress ratio. \( \tan^{-1}(\tau/\sigma_n) \) can be calculated by test data. As a result;

\[ \Phi_b = \tan^{-1} \left( \frac{\tau}{\sigma_n} \right) - i \]  
Equation 8

<table>
<thead>
<tr>
<th>Category Full Name</th>
<th>Category Symbol</th>
<th>Average basic friction angle ( \Phi_b ) [degrees]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal density (31.5mm&lt;P&lt;50mm)</td>
<td>M1</td>
<td>56.1</td>
</tr>
<tr>
<td>High density (31.5mm&lt;P&lt;50mm)</td>
<td>M2</td>
<td>52.7</td>
</tr>
<tr>
<td>Normal density (31.5mm&lt;P&lt;80mm)</td>
<td>M3</td>
<td>53.5</td>
</tr>
</tbody>
</table>

Table 5 – Average basic friction angle
Step 2: From the changed Barton’s equation (7.6), \( \tan^{-1}(\tau/\phi_n) \) can be treated as \( y \) while \( \log \sigma_n \) is treated as \( x \) because the \( \tau \) and \( \phi_n \) are known from the experiment data;

Calculate \( \tan^{-1}(\tau/\phi_n) \) value and \( \log \sigma_n \) value for each specimen and plot \( \tan^{-1}(\tau/\phi_n) \) value (as \( Y \)) and \( \log \sigma_n \) (as \( X \)) value for each experimental category;

Step 3: Add a liner trend (intercept \( \neq 0 \)) for points of each category, got an equation of this liner trend line (\( y = -ax + b \)). And \( a \) is equal to \( R \), so the \( R \) value is known. \( S \) value can be calculated by equation 9.3.

\[
a = R \tag{9.1}
\]
\[
b = R \log S + \Phi_b \tag{9.2}
\]
\[
S = 10^{\frac{b-\Phi_b}{R}} \tag{9.3}
\]

Equation 9 – equation 9.3 shows the calculation preformed to derive the \( S \) from the trend line equation

With the help of equation 9, the equivalent strength can be calculated. But when analyzing equation 9.2 for category M1, one would notice that the \( R \) is more important in the formula than the \( \Phi_b \), and series M1 had a relative high \( R \) causing it to deviate from the two other categories. The results of this calculation are in table 6. In Barton’s model (figure 5) graphically the \( S \) value could be determined. This would be \( S = 0.7 \times \text{UCS} = 140 \text{MPa} \). With back calculating we can obtain the basic friction angle associated with \( S=140 \text{MPa} \) (table 6, in green).

**Results** – In Appendix C – graphs from tests M1, M2, M3 the results of all the test are published.

<table>
<thead>
<tr>
<th>Category Full Name</th>
<th>Category Symbol</th>
<th>R</th>
<th>b</th>
<th>( R^2 )</th>
<th>( \Phi_b )</th>
<th>S(MPa)</th>
<th>( \Phi_b )</th>
<th>S(MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal density</td>
<td>M1</td>
<td>9.19</td>
<td>87.86</td>
<td>0.838</td>
<td>56.1</td>
<td>2.869</td>
<td>40.59</td>
<td>140.099</td>
</tr>
<tr>
<td>High density</td>
<td>M2</td>
<td>6.52</td>
<td>88.78</td>
<td>0.938</td>
<td>52.3</td>
<td>345.135</td>
<td>55.25</td>
<td>140.048</td>
</tr>
<tr>
<td>Normal density</td>
<td>M3</td>
<td>6.10</td>
<td>87.17</td>
<td>0.911</td>
<td>53.5</td>
<td>328.136</td>
<td>55.76</td>
<td>140.002</td>
</tr>
</tbody>
</table>

**Interpretation** – The determination of the parameters for the Barton Model is not giving the desired results. The nonlinearity of the range were we conducted test may be the cause of this. In this low stress domain the basic friction angle is the main contribute to the friction angle (equation 7.1).
The $R^2$ value is not got for the trend lines, in combination with the low amount of data point would be the resound for this.

### 6.5 Comparison with results from other studies

The test results of the medium shear box are compared with the results that were obtained with the small scale shear box tests. In the comparison we only use the low normal stress data from the small scale shear box. The tests conducted with the small shear box have more data in the range 100kPa - 850kPa, but the comparison with the low stress data obtained with the normal shear box isn’t accurate. In the next table the friction angle can be compared (table 7).

Table 7 – Use the 0-113 kPa normal stress of the small shear box test to compare with medium size shear box data

<table>
<thead>
<tr>
<th>Category Full Name</th>
<th>Category Symbol</th>
<th>Hypothesis 1 (without cohesion)</th>
<th>Hypothesis 2 (with cohesion)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cohesion (kPa)</td>
<td>Friction Angle (deg.)</td>
<td>$R^2$</td>
<td>Cohesion (kPa)</td>
</tr>
<tr>
<td>Normal density</td>
<td>NS</td>
<td>0</td>
<td>53.51</td>
</tr>
<tr>
<td>(1.18mm&lt;P&lt;3.35mm)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Normal density</td>
<td>NB</td>
<td>0</td>
<td>60.93</td>
</tr>
<tr>
<td>(3.35mm&lt;P&lt;6.30mm)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>High density</td>
<td>HS</td>
<td>0</td>
<td>57.83</td>
</tr>
<tr>
<td>(1.18mm&lt;P&lt;3.35mm)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>High density</td>
<td>HB</td>
<td>0</td>
<td>62.54</td>
</tr>
<tr>
<td>(3.35mm&lt;P&lt;6.30mm)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mixture (70%small, 30% big)</td>
<td>MS</td>
<td>0</td>
<td>57.51</td>
</tr>
<tr>
<td>Mixture (30%small, 70% big)</td>
<td>MB</td>
<td>0</td>
<td>58.36</td>
</tr>
<tr>
<td>Normal density</td>
<td>M1</td>
<td>0</td>
<td>74.02</td>
</tr>
<tr>
<td>(31.5mm&lt;P&lt;50mm)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>High density</td>
<td>M2</td>
<td>0</td>
<td>79.10</td>
</tr>
<tr>
<td>(31.5mm&lt;P&lt;50mm)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Normal density</td>
<td>M3</td>
<td>0</td>
<td>77.58</td>
</tr>
<tr>
<td>(31.5mm&lt;P&lt;80mm)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The results from the small scale shear box have a higher $R^2$ value than the medium size shear box data. The friction angle is lower for the small scale tests. In the results there is a dependence between the particle size increase and the increase in friction angle. When the results from the medium size shear box are compared with the results from the small shear box over a wider normal stress range we see more differences.

When comparing to (Asadzadeh & Soroush, 2009) we see that the result of our study are in the same range. With incorporating the recommendation stated in the next section it would be possible to make a better comparison between the data.

### 7 Recommendations for further studies

**Set up limitations** – the set up has considerable limitations, because the normal stress is applied with dead weight there is a limitation on the amount of normal stress which can be applied on the material. Therefore the testing has been conducted in a small range of low normal stresses. The
current models to describe shear strength of rock fill are based on higher normal stresses. It is interesting to test in this area but the absence of the possibility to test under higher stress is problematic. The test results can’t be checked with other studies because they are in a different stress domain.

For testing with higher normal stresses there is an proposal for altering the testing setup. The idea is to change the way of applying normal stress. Instead of using dead weight, air pressure cushions are used. These cushions apply a constant force onto the shear box. By regulating the air cushions with a pressure relive valve this can stay constant. The force is monitored by a pressure gauge on the air cushion, and by a load cell between the air cushion and the shear box.

![Figure 19 – Sketch of the altered testing setup with air cushions to apply normal stress.](image)

In the current setup the vertical displacement transducer is installed after loading. The result is that the compaction of the material is not captured in the data from the tests. An solution need to found to measure the initial compaction of the rockfill as a result of the loading (normal stress). And measure this accurately.

The material’s strength is causing high dilatancy and tipping of the dead weight, it is not clear if all the weight is causing vertical stress on the packing. With this tipping the loading plate directly on top of the rockfill is touching the side of the shear box. It is not clear if the load of the dead weight is partly transferred to the shear box instead of the rock fill. When the load is tipping the force is divided in a component pressuring the rock fill and a sliding component. With the alteration of the setup this problem is not relevant any more. The new setup has a fixed loading system attached to the frame so tipping can’t occur anymore.

**Accuracy of the test results** – The problem with shear box testing is the accuracy of the results. As referred in literature (Cerato & Lutenegger, Specimen size and scale effects of direct shear box tests of sands, 2006) it is hard to reproduce the same result with a shear box. Therefore more tests need to be conducted to determine the parameters more accurately. This in combination with testing in a wider range of normal stress would certainly improve the test data.

Another point of interest is investigation in the shear bed, is the whole shear box shearing? This wasn’t researched in our study but for further studies this would be a improvement.
**Particle size** – For more accurate test results it would be interesting to test smaller fractions in the shear box. These results could then be compared to the small scale shear box to see if they match. This would delete the influence of scaling between the two shear boxes.

### 8 Conclusion

Three series of medium size shear box tests were carried out in an attempt to determine the shear behavior of the Bremanger aggregate used in the cobble beach and crane roadway on the Maasvlakte 2.

The findings of this study are as follows.

- The stress-strain behavior of the rock fill is nonlinear and stress-dependent. The test conducted at low normal stress shows strain softening. The density dependence of the rock fill is observed. High density packing ensures higher dilatancy and shear strength.

- The obtained friction angles are in the range 69° – 77° according to the test conditions. These values are in a wide range caused by the small range of normal stress applied and the small number of tests conducted. We see an increase in friction angle with an increase in density. About the dependence of the particle size on the friction angle this is not clearly observed, due to the fact there were only two different fractions tested and they were overlapping each other.

- The modeling using Mohr-Coulomb and Barton was performed but the linear models did not represent the nonlinear behavior in the low stress domain as preferred. They capture the higher stress domain, more linear behavior better.

- Recommendations for improving the test set can be found in section 7. This will certainly result in better quality data. And give the opportunity to perform tests in a wider range. This will also improve the modeling using Barton and Mohr-Coulomb significantly.
9 Bibliography


10 Appendices
Appendix A – mineralogical description
Appendix B – measuring devices calibration
Appendix C – graphs from tests M1, M2, M3
Appendix D – Plaxis road way simulation
Appendix E – Formulas
Appendix F – Porosity of rockfill
Appendix G – Testing procedure
10.1 Appendix A – mineralogical description

**Description of Bremanger sandstone** (Bootsma, 2010)

The Bremanger Sandstone has a sedimentary structure. Macroscopic, in big pieces of rock a certain layering is clearly visible. Since cleavage does not occur along this layering a microscopic analysis was performed on a thin section of the specimen perpendicular to the visible layering.

**Minerals:**

The following minerals were observed in the thin sections (Picture X.1). An estimation of their contributing percentage to the mineralogy is given as well.

- **Quartz:** 70%
- **Feldspars (Plagioclase, Microcline, Orthoclase):** 10-15%
- **Muscovite:** 5%
- **Calcite:** 10%
- **Others (Chlorite, Titanite, Epidote, Klinozoisite):** 0-5%

![Picture X.1: Microscopic overview; normal and crossed polarizers (Microscopic width of images: 3mm)](image1)

**Microscopic description:**

The texture of the rock seems to be sedimentary but the typical structure (matrix and cement) is not clearly visible. Grain size varies from less than 0.6 mm to 1.3 mm for the largest grains. The individual minerals are tightly compressed to and into one another (Picture X.2) The presence of calcite indicates it as a probable former cement but is now recrystallized into large crystals. This is an indication of metamorphism. Another interesting observation is the typical shape and orientation of the Muscovite crystals. All Muscovite crystals are elongated crystals and are orientated in the same direction. This is likely to be an indication of the stress direction during metamorphism of the sandstone. The orientation of the Muscovite crystals is shown in Picture X.3. The picture is taken with crossed polarizers which gives the Muscovite crystals a very orange-purple color.

![Picture X.3: Orientation of Muscovite crystals.](image2)

**Conclusion:**

The Bremanger sandstone is a sedimentary rock. Metamorphism changed the mineralogy of the matrix and cement. Macroscopically the texture of the sandstone hasn’t changed, the layering is still visible. Therefore the rock type is a Metasandstone (Metamorphic Sandstone).
Bremanger sandstone (Tooren, 2010)

Macroscopic description:
Dark colored rock. Black and grey layers alternate. What is it, a foliation or a sedimentary layering? Black and grey grains are visible.

Microscopic description:
The macroscopic visible foliation/sedimentary layering is not visible within the thin section. The texture of the rock sample seems sedimentary: many angular grains are visible. However, matrix and cement are not visible. Matrix and cement seem to have recrystallized. At several places the grains are pressed into each other (lobate boundaries).
The grain size of grains varies. The length of the largest grain is 1.3 mm. Many grains have a diameter less than 0.6 mm.
The cause of the macroscopic black color is not clear.

Mineralogical composition of the grains:
Quartz: dominant mineral, around 70%
Feldspars: plagioclase, microcline, orthoclase
Calcite
Mica’s: muscovite, biotite (→ chlorite), chlorite
Opaque minerals
Lithic fragments: chert, quartzite, mylonite, volcanic rock

Mineralogical composition of matrix and cement:
In a sandstone matrix and cement are visible in between the grains. In this sample several minerals are visible in between the grains. Calcite is one of them. The calcite shows large crystals, these might result recrystallisation of a calcitic cement. Other surprising minerals include epidote, clinozoisite and titanite (sphene). Suggesting a metamorphic overprinting by which the probably clayish matrix of the sandstone disappeared.

Rock name:
Obviously this was originally a sedimentary rock, a sandstone. But metamorphism changed the mineralogy of matrix and cement, though not the rock texture. This rock type may be called a metasandstone (metamorphic sandstone).

Maaike van Tooren
2010/01/20
10.2 Appendix B – measuring devices calibration

From this data we can conclude that the load cells are both calibrated that 5KN=1V

The friction between the two shear boxes is small in comparison with the total stresses applied. The mean of the friction is 83N. The main horizontal displacement transducer is calibrated; 1 volt - 1 cm.
10.3 Appendix C – Results from tests M1, M2, M3

Horizontal Displacement vs. shear stress from tests M1, M2, M3
Appendix C – Horizontal Displacement vs. Vertical Displacement from tests M1, M2, M3
Appendix C – Dilation vs. Stress ratio from tests M1, M2, M3
Appendix C – Corrected normal stress vs. shear stress from tests M1, M2, M3
10.4 Appendix D – Plaxis road way simulation
### 10.5 Appendix E – Formulas (Douglas, The shear strength of rock mass, 2002)

#### Table 4.5. Various shear strength criteria for rockfill

<table>
<thead>
<tr>
<th>Ref.</th>
<th>Equation</th>
<th>Parameters</th>
<th>Development</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.8</td>
<td>( t = A \sigma_y^b )</td>
<td>( \sigma_y, b )</td>
<td>Empirical curve fitting.</td>
</tr>
<tr>
<td>4.9</td>
<td>( \tau = \frac{t}{\sigma_z} = \left( \frac{\sigma_x}{\sigma_y} \right)^b )</td>
<td>( \sigma_x, \sigma_y, b )</td>
<td>Non-dimensionalized form of (4.8), Note that ( A \sigma_y^b ) is independent of ( \sigma_y ) but ( \sigma_x ) is not independent of ( \tau ).</td>
</tr>
<tr>
<td>4.10</td>
<td>( \tau_{sw} = A \sigma_z^a / \sigma_y^b )</td>
<td>( \sigma_z, A, a, b )</td>
<td>Developed from base-scale data and shear tests up to ( \sigma_y = 780 ) kPa, shear on rockfill and rough slopes. Tested at laboratory scales (Figure 4.17).</td>
</tr>
<tr>
<td>4.11</td>
<td>( \frac{\tau}{\sigma_y} = \left( \frac{\sigma_x}{\sigma_y} \right)^{\alpha} )</td>
<td>( \alpha, C, \sigma_y )</td>
<td>Alternative of (4.8) for normal stresses.</td>
</tr>
<tr>
<td>4.12</td>
<td>( \frac{\tau_{sw}}{\sigma_y} = \frac{m \sigma_z}{\sigma_y} )</td>
<td>( m, C, \sigma_y )</td>
<td>Developed specifically for rockfill and low gradients in fill. Unconfined at ( \sigma_y = 6 ).</td>
</tr>
<tr>
<td>4.13</td>
<td>( \sigma_y - \sigma_i = c \tau )</td>
<td>( c )</td>
<td>Developed from field tests by setting the minimum composite strength to zero (i.e. ( c = 0 )).</td>
</tr>
</tbody>
</table>

---

<table>
<thead>
<tr>
<th>Eq.</th>
<th>Reference</th>
<th>Equation</th>
<th>Parameters</th>
<th>Development</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.14</td>
<td>Barton &amp; Kjellund (1981)</td>
<td>( \phi' = 30 \log \left( \frac{S}{\sigma_y} \right) + \phi )</td>
<td>( \phi, S, \sigma_y )</td>
<td>Extension of empirical direct strength relation of Barton (1977). Parameters determined based on “at site” and “at site” testing.</td>
</tr>
<tr>
<td>4.15</td>
<td>Charlock (1991)</td>
<td>( \phi' = C \log \left( \frac{\sigma_y}{\sigma_i} \right) + \phi )</td>
<td>( C, \phi, \sigma_y, \sigma_i )</td>
<td>Similar approach to Barton &amp; Kjellund (1981) for principal stresses.</td>
</tr>
<tr>
<td>4.16</td>
<td>Germaine (1983)</td>
<td>( \phi' = \phi_{0} - j \log \left( \frac{\sigma_y}{\sigma_{0}} \right) )</td>
<td>( \phi_{0}, j, \sigma_y, \sigma_{0} )</td>
<td>Logarithmic shear strength based on results from Kjellund. Note this approach requires a minimum logarithmic strength to be performed.</td>
</tr>
</tbody>
</table>

\( \sigma_y \) = Normal stress
\( \phi \) = Shear strength of intact rock
\( C \) = Coefficient of confinement
\( \sigma_i \) = Unit weight
\( \sigma_{0} \) = Particle diameter at which 50% of the material is finer
\[ \phi = \phi_0 - j \log_{10} \left( \frac{\sigma_{\text{cr}}}{\sigma_{\text{u}}} \right) \]

\[ \phi_0 = \phi_\Delta + j \log_{10} \sigma_{\text{cr}} \]

\[ j = j_0 - \Omega \log_{10} N_R \]

where:

- \( j_0 \) = experimental parameters, constant for specific material and condition
- \( \Omega \) = experimental parameters, constant for specific material and condition

\[ N_R = \left( \frac{\mu_k}{\pi \rho} \right)^{1/3} \left( \frac{\mu_k}{\pi \rho} \right)^{2/3} \]

- \( \mu_k \) = coefficient of uniformity = \( C_u \)
- \( \pi \) = crushing strength number = \( S_c \sigma_m \)
- \( x \) = volume concentration of solids = \( V_s / V \)
- \( x \) = shape factor = \( 1/r_c \)

\[ \mathcal{N}_R = \left( \frac{S_c \sigma_m}{\rho} \right)^{1/3} \rho \left( \frac{1}{1 + e} \right)^2 \]

\[ S_c = \frac{6 \sum \Delta P_k}{\pi \sum \frac{d_k}{d_{\text{max}}}} \]

\[ \sigma_m = \frac{P_{\text{max}}}{d_{\text{max}}} \]

- \( V_s \) = Volume of solids
- \( V \) = Total volume
- \( c \) = void ratio
- \( \rho \) = particle density = \( G_s \rho_s \)
- \( \Delta P_k \) = percentage of material of fraction \( k \) with mean size \( d_k \)
- \( d_k \) = mean particle diameter for fraction \( k \) = \( \sqrt{d_{\text{min}} d_{\text{max}}} \)
- \( d_j \) = sieve size
- \( P_{\text{max}} \) = crushing load of particle of nominal size \( d_{\text{max}} \)
- \( d_{\text{max}} \) = maximum particle size

**Figure 4.16. Gonzalez (1985) shear strength of rockfill equations**

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10.6 Appendix F – Porosity of rockfill

\[ e = \frac{(1 + w) \gamma_s}{\gamma_t} - 1 \]
\[ n = \frac{e}{1+e} \]

where \( n \) is the porosity;
\( e \) is the pore index;
\( \gamma_s \) is the specific weight of the rock mass material;
\( \gamma_t \) is the specific weight of the “in situ” rockfill;
\( W \) is the moisture content of the rockfill.

Calculate steps:

Step 1: The specific weight of the Bremanger sandstone is 2.7 Mg/m^3, the specific weight of the “in situ” rockfill is known. The moisture content of the rockfill is assumed to 0%;

Step 2: The pore index can be calculated, then the porosity can be calculated. The porosity of the Bremanger sandstone is between 36.67-39.63%;

Step 3: According to Barton’s figure, \( R \) is between 5.5-6.
10.7 Appendix G – Testing procedure

Testing procedure for performing a test with the medium size shear box at the laboratory of geoengineering at the TU Delft.

- Check if the motor is in the starting position, otherwise return it in the starting position with the controls of the motor. (That is on the motor switch the side where the is NOT a green mark.)
- Check if the top shearing box is aligned with the bottom shear box on the inside.
- Check if the horizontal displacement transducer is not connected (when connected there is a risk of damaging it)
- Check if the shear load cells are free of the shear box.
- Put the wooden vertical movement restrictor blocks in place.

- Fill the box as desired in the test, weigh before filling to calculate the density of the box. The box is 0.5x0.5xheight of the filling.
- Load first the plate which fits inside of the box to ensure it only rests on the material and not on the shear box. Check if this first plane is lying horizontal. Apply the desired normal load with the crane. Use photos to see different stacks of dead weight. Ensure the stack is stable and use straps of crane loosely as safety for sliding.
• Attach horizontal displacement transducer. It needs to get larger in test so screw it in the top box or it wouldn’t measure anything 😊. Check if it is aligned correctly.

• Attach vertical displacement transducer in the center of the top plate. See picture for two of the configurations

• Check if the data acquisition is correctly for the displacement transducers.
• Loosely rotate the disks of the load cells to touch the top shear box. Look at the amplifier to make it as close to zero as possible.
• Check if the motor is in the right speed (we used 2)
• Zero everything on MP3 data program
• Start new data record
• Start motor
• Stop before the motor it is touching the top shear box, when this is the case the force of the motor will directly be transferred to the load cells and they will be destroyed.

• Save the test results immediately, export as .csv file