AES/GE/11- 02  SHEAR STRENGTH OF BREMANGER SANDSTONE ROCKFILL AT LOW STRESS

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Abstract

The Bremanger Sandstone rock fill is used to form a cobble beach, to build an underwater foundation layer of a sea water breaker and to construct a run-way for a large crane in construction of Maasvlakte 2 (MV2), the extension of Rotterdam harbor. The main purpose of this work is to determine in the laboratory the strength of Bremanger rock fills under low stress. Tests are conducted on rock fills with a finer particle size distribution than that used at MV2. Results obtained cannot be directly used in the design of MV2 project. The influence of the testing equipment, for a given ratio of the equipment size to the particle size, of the degree of compaction and of the particle strength on the strength of the Bremanger rock fill are investigated. The effect of the test boundary conditions is researched by conducting tests with a triaxial cell apparatus, medium and small scale shear boxes and a tilt apparatus. The movement of particles was also studied during medium scale shear box testing to get a better insight into this effect.

Emphasis is put on determining the contribution of dilatancy to the shear resistance of the Bremanger sandstone rock fills with respect to the contribution to resistance to rolling of particles. For this purpose, dilatancy is measured during testing and the basic friction angle of the Bremanger rock fill is determined on sandblasted rock discontinuities with a Golder shear box. The basic friction angle controls the resistance to rotation of smooth particles. Test results are fitted using empirical models. The performance of these models at predicting the strength of the tested materials is assessed.
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1-INTRODUCTION

1.1 Problem statement

The Bremanger Sandstone is used as rock fill at the MV2, the extension of Rotterdam harbor
- to form a cobble beach,
- to build an underwater foundation layer of a sea water breaker and
- to construct a run-way for a large crane.

The seawater breaker will protect from the wave action the most exposed coast line of the MV2. It will form a rigid coastal protection. The cobble beach will act as a soft dynamic structure and protect the reclaimed land from the North Sea in less exposed areas and where the wave energy has already been dampened by the seawater breaker or other structures. The crane will be used to put in place the large concrete blocks that will form the seawater breaker.

The shear strength of the Bremanger sandstone is needed for predicting the resistance to erosion of the cobble beach, optimize the plough that will flatten the foundation layer of the sea water breaker and calculate the stability of the run-way under the weight of the crane. Rock fills made of blocks up to 13 to 30 cm diameter will be used. The Bremanger sandstone is a very strong rock. The stress applied by the crane or resisting the ploughing action will be low in comparison to the material strength of the Bremanger sandstone.

Testing assemblies of decimetric rock blocks requires a large test apparatus to avoid scale effects related to the size of the blocks in comparison to the dimension of the test apparatus. In practice, realising such tests with well controlled loading and boundary conditions is difficult (Marachi et al. 1969). Empirical models (Barton, Hoek & Brown) have been developed to predict the strength of rock fill from a minimum number of parameters characterizing the individual rock blocks and their packing but it is not known how reliable they are, especially in the low stress range.
1.2 Research objective

The objective of this work is to determine in the laboratory the strength of Bremanger rock fills under low stress.

Tests are conducted on rock fills with a finer particle size distribution than that used at MV2. The influence of the following factors on the strength of the Bremanger rock fill are investigated:

- the size of the aggregates with respect to the size of the testing equipment
- the size of the testing equipment, for a given ratio of the equipment size to the particle size
- the degree of compaction
- the particle strength in relation with the way the tested aggregates were produced
- the test boundary conditions.

The effect of the test boundary conditions is researched by:

- Conducting tests with a triaxial cell apparatus, medium and small scale shear boxes tests and the tilt apparatus.
- Imposing different types of boundaries during medium scale shear box testing and tracking the movement of particles.

Emphasis is put on determining the contribution of dilatancy to the shear resistance of the Bremanger sandstone rock fills with respect to the contribution to resistance to rolling of particles. For this purpose, dilatancy is measured during testing and the basic friction angle of the Bremanger rock fill is determined on flat rock discontinuities with a Golder shear box. The basic friction angle controls the resistance to rotation of smooth particles.

Test results are fitted using empirical models. The performance of these models at predicting the strength of the tested materials is assessed.
1.3 Project description and background

The port of Rotterdam will be expanding into the North Sea. Maasvlakte 2 involves the creation of a new world-class port and industrial area. Maasvlakte 2 will be situated directly adjacent to the existing Maasvlakte (Figure 1.1). Work started in September 2008 by Koninklijke Boskalis Westminster and Van Oord (PUMA), with the construction of a banana-shaped island the first part of the seawall (Newsletter_Maasvlakte_2, 2010). Behind this, sand dredged from deep sea will be sprayed on, creating new land. In order to protect the reclaimed land from the rough North Sea it is necessary to build a sound seawall.

Figure 1. 1: Maasvlakte 2 (Newsletter_Maasvlakte_2, 2010)
The largest stones to become part of the future hard seawall will be 40-ton concrete blocks. The blockbuster will be positioning the blocks in the sea at a distance of some 50m off the shore. They will serve as a breakwater for the stony dune.

A runway for the Blockbuster will be constructed. It will be made of Bremanger sandstone rock fill. The Blockbuster weighs around 1200 tonnes, has a counterweight of 360 tonnes and moves on three double sets of caterpillar treads. The blockbuster can simultaneously lift over 40 tons and stretch to 50m with this huge load it requires a well designed foundation Figure 1.2.

![Image of Blockbuster](image1)

Figure 1.2: The Blockbuster (Newsletter Maasvlakte 2, 2010)

For the stony dune of the hard sea defences, different sizes of stone are used (0.3-35 mm, 20-135 mm, 5-70 kg, 150-800 kg, 1 - 10 tonnes). On the sand core of the sea defences, a stone dumping vessel will construct a slope, with increasingly large stones as it gets higher. Plough boat will profile the slope and the stone applied.

![Diagram of stone dumping vessel and plough boat](image2)

Figure 1.3: Stone dumping vessel and plough boat to profile the slope
1.4 Organization of Thesis

This thesis comprises seven chapters. The first chapter presents the problem statement and research objectives. In Chapter 2, literature on shear strength of rock fills is reviewed and the different shear strength tests are explained. Chapter 3, 4 and 5 deal with the tests conducted on the Bremanger rockfill. In chapter 3, the material and the test procedures used are explained. In chapter 4, the results of the laboratory tests are presented and analyzed. They are discussed and compared to each other and as well as with previous works in Chapter 5. In Chapter 6, the determination of the basic friction angle is explained and in chapter 7 the conclusions and recommendations are given.
2. LITERATURE REVIEW

2.1 Introduction

Rockfill may be defined as a coarse-grained material which consists primarily of angular to sub-angular blocks particles obtained by blasting rocks or rounded to sub rounded particles extracted from river beds. To design rockfill embankments, offshore structures and subgrade for roads, low shear strength parameters of the rock fill are needed. However, understanding of rockfill behavior in general has lagged far behind knowledge of the stress-strain behavior of soils due to the cost of large-scale testing equipment capable of handling prototype-sized pieces of rocks (Parkin, 1991).

In recent decades, construction of increasingly high rockfill dams has led to the development of large-scale testing equipment for rockfill (Ghanbari et al, 2008). For the measurement of shear strength in laboratory, the tri-axial compression test apparatus, direct shear box test apparatus, and/or ring shear test apparatuses have been used. Among these apparatuses, the direct shear box test apparatus and the triaxial compression test apparatus were used in this study.

2.2 Shear Strength of Rockfill

The shear strength of a rockfill is its resistance to deformation by continuous shear displacement of the rockfill particles upon the action of shear stress. When the maximum shear stress is reached, the rockfill is regarded to have failed. The failure conditions of a rockfill may be expressed in terms of limiting shear stress, called shear strength, or as a function of principal stresses. The shear strength of granular soil is frequently characterized by the angle of internal friction ($\varphi$) and cohesion (C). Friction angle is the measure of the resistance of the particles to shear force when normal stress on the shear plane is not zero.

Apparent cohesion in rockfills can be due to the interlocking of particles when the material is dense. Two different friction angle concepts are used in this study, secant friction angle and internal friction angle. Secant friction angle ($\phi_{sec}$) is the arctangent of
the shear strength over the normal stress at failure ($\phi_{sec} = \tan^{-1}\left(\frac{\tau_{peak}}{\sigma_{peak}}\right)$). Each specimen has a different secant friction angle. The internal friction angle ($\varphi$) is determined from Mohr-Coulomb linear failure envelopes constructed as best-fit lines.

### 2.3 Shear Strength Testing

#### 2.3.1 Direct Shear Box Test

Direct shear testing, introduced by Coulomb in 1776, has long been used to estimate the shear strength parameters. Many alternative testing machines have been developed since then.

Direct shear testing has many advantages over alternative test methods (Takada, 1993; Terzaghi, et al, 1996) including:

- The sample can be made to shear in a prescribed plane or zone;
- If appropriate sample dimensions are used, the shear deformation is approximately plane strain, and deformation occurs mostly by simple shear, which is often the design assumption of earth structures;
- The test sample required can be relatively small or very large;
- The structure of the apparatus, and the testing method are simple.

Disadvantages of the direct shear test include changes in the area of the shear surface during shearing, and uncertainty in interpretation of the results due to non-uniformity of the stresses and strains that occur across the shear surface, and throughout the sample thickness.

None the less the test remains popular in practice owing to its simplicity, and the results obtained from it are consistent with the results from other more sophisticated tests (Potts, 1987).

Figure 2.1 shows three different types of shear box currently in use worldwide. The following summaries are from (Shibuya et al. 1997); the references are those quoted by the author.
(a) **type A**: the top platen and the upper part of the box are each independently allowed freedom to move vertically and to rotate (Skempton & Bishop, 1950).

(b) **type B**: the top platen is rigidly fixed to the upper box so that the two move vertically or rotate together (Jewell & Wroth, 1987)

(c) **type C**: the upper box is prevented from moving vertically or rotating; the top platen moves independently, but it can also be prevented from rotating (Mikasa, 1960; Takada, 1993).

The important feature to be considered is the degree of freedom for the specimen boundary to relocate during shear. This decreases from type A to type C. In type C, if the walls are not lubricated, then friction forces develop and counteract partially then normal load applied.

Figure 2. 1: Direct shear boxes currently in use (Shibuya, 1997)
2.3.2 Triaxial Compression Tests

The triaxial test offers one of the best ways to determine the shear strength of granular materials in the laboratory. The specimen is usually subjected to a constant all-around confining pressure ($\sigma_3$) simulating the lateral stress caused by the overburden pressure. In triaxial test, the top and sides of the specimen are assumed not to be subjected to shear stress. Since these planes do not have shear stresses, by definition, they are principal planes (Richard et al, 1997).

In this study a Vacuum triaxial apparatus was used. Unlike conventional triaxial chamber, the confining pressure is applied by reducing the pore pressure on the inside of the specimen by applying a vacuum of known magnitude. The important advantages of the vacuum triaxial test are:

- Cylindrical pressure chamber is not placed around the specimen
- Also unobstructed access to the specimen is possible which is not true when the specimen is enclosed by the conventional triaxial chamber.

The disadvantages of the vacuum triaxial test are:

- The maximum confining pressure is limited to less than 100kpa
- Specimen cannot be tested in a saturated condition, and the problems of moisture migration could occur particularly at high degrees of saturation and high vacuum levels.

Nevertheless, the vacuum triaxial test offers an attractive alternative to the conventional triaxial test. This is particularly true if it has come along with digital photographs to capture the axial deformation and circumferential deformation (Adam et al, 2009). Figure 2.2 shows the behavior of soil tested using triaxial apparatus. The results may show either hardening or both hardening and softening behavior.
In the shear box tests the position of the failure plane is defined by the apparatus, whereas in triaxial tests the position of the failure plane is much less well defined. Large-scale triaxial compression testing is an accurate estimation of shear strength. The friction angle of rockfill materials in large scale direct shear is 3 to 4 degrees more than in triaxial test. According to this, it is necessary to consider a higher safety factor in using the friction angle obtained from direct shear test (Ghanbari et al, 2008).

2.4 Factors Controlling the Shear Strength of Rockfill

The following paragraph has been summarized from (McLemore et al., 2009); the references are those quoted by the authors.

The angle of internal friction angle is a function of the following parameters (Hawley, 2001; Holtz and Kovacs, 2003):

- Particle shape and roughness of grain surface (friction angle typically increases with increasing angularity and surface roughness)

- Grain quality (weak rock materials such as shale have lower friction angles compared to strong rock materials such as granite)

- Grain size (friction angle increases or decreases with increase in grain size)
• Grain size distribution (friction angle typically increases with decreasing coefficient of uniformity, Cu)

• Specific gravity (related to mineralogy)

• State of compaction or packing (friction angle typically increases with increasing density or decreasing void ratio)

• Applied stress level (friction angle decreases with increasing confining stress, resulting in a curved strength envelope passing through the origin instead of the classical straight line)

• Degree of saturation.

These factors compete with each other, complicating their effect on friction angle. In the following sub sections, the influence of specific material characteristics on the shear strength of rock fills is described.

2.4.1 Confining pressure/Normal stress

The effect of confining pressure in triaxial tests or normal load in the case of direct shear tests on shear strength and behavior of materials have been studied for years. (Marachi et al., 1969; Leps, 1970; Bertacchi & Bellotti, 1970; Penman et al., 1982; Indraratna et al., 1993; Anagnosti & Popovic, 1982; Al-Hussaini, 1983).

The strength and stress-strain-dilatancy behavior of rockfill varies remarkably with confining stress $\sigma_3$. The friction angle of a rockfill specimen decreases with the normal stress; the rate of decrease diminishes as the normal stress becomes greater. This could be explained that, at very low confining stresses, the rockfill particles are relatively free to move with respect to each other and dilatancy effect can cause a significant increase in internal friction angle (Indraratna, 1994). As the confining stress increases, dilatancy effects gradually disappear due to particle crushing, causing a notable reduction of internal friction angle. This curved strength envelop of rockfill has a large impact on the stability analysis of rockfill dam due to the fact that a lower safety factor will be produced for the shallow slip surface when using constant friction angle (Indraratna,
In other words, the high friction angles associated to low normal stresses are favorable to resistance against ravelling (Barton & Kjaernsli, 1981), at the slope toe and close to the downstream face of the slope (Figure 2.3).

### 2.4.2 Maximum Particle size

There is no common agreement on the effect of particle size on shear strength after evaluating the literature on this topic. Different views are presented with some indicating that the shear strength decreases with increasing particle size (Marachi et al., 1972; Marsal, 1973), while some have opposite views indicating that an increase in the particle size increases the load per particle, and hence crushing begins at a smaller confining stress, and causes a reduction in the friction angle (Anagnosti & Popovic, 1982) and Barton (1981) shows that for materials compacted to the same density with geometrically similar grading, the smaller the elements are, the higher the friction angle of the material is. No effect at all has been observed by Charles & Watts, (1980).

![Figure 2.3: Estimated variation of $\Phi'$ under toe and beneath downstream slope of dam (Barton & Kjaernsli, 1981).](image)
2.4.3 Density

It is generally accepted that the shear strength of rockfill increases with a higher relative density (Leps, 1970; Marsal, 1973). The effect of relative density on the friction angle can be explained by the phenomenon of interlocking the denser the rockfill; the greater the interlocking, and so the greater the value of friction angle. Douglas (2002) also indicated that the shape of failure envelope is affected by this factor. The dense rockfill specimens show a marked curvature on the stress-strain curve, with a distinct drop in the friction angle while the loose rockfill specimens shows minimal curvature and drop in friction because the loose material require less dilation as particle have more freedom to move or rotate during shearing. The two curves tend to merge at very high confining pressures.

2.4.4 Effect of Gradation

A number of researchers investigated the gradation effect on the shear strength by varying the coefficient of uniformity ($C_u$) of rockfills. Douglas (2002) findings are a confirmation of Marachi et al (1969) views, that a better graded rockfill, has a larger friction angle compared to uniform rockfill, due to a better interlocking and less particle breakage in the former, the less breakage arising from the fact that in a well graded rockfill there are more interparticle contacts and the load per contact is thus less than in a uniform rockfill. The impact of type of grading on the friction angle is about 2 to 3 degrees (Ghanbari et al., 2008).

2.5 Rockfill Modeling, Scale effect, Determination of the Maximum Particle size

2.5.1 Rockfill Modeling

The most difficult condition to simulate is testing on particle size distribution similar to in-situ particle size distribution. Most laboratory devices are not capable of handling large sizes and therefore require that the material is scaled down to obtain smaller particle-sizes for a particular test. In their research, Marachi et al. (1969) found that by reducing the particle size distribution parallel to the original sample size, it was possible to achieve similar results by ensuring that the same contact points among particles were maintained. However, this hypothesis is too simple, since scale effects also affect
the behavioral features. For these reasons, the extents to which laboratory tests can be relied upon to predict field behavior has always been a rather open question and probably remain so (Parkin, 1991). Therefore, it is necessary to eliminate these uncertainties as far as possible.

The following paragraph has been summarized by (Parkin, 1991); the references are those quoted by the author.

Fumigalli et al. 1970 have formulated principles for preparing a laboratory model rockfill. 
- Equality of strength in all fractions
- Geometrical similar grading curves
- Equality of void ratios
- Similarity of particle shape

Whilst all these conditions are logical, not all are capable of being met in all cases, nor has it been proven that this is necessary.

### 2.5.2 Scale Effect

Cerato & Luteneger (2006) conducted Direct Shear Test (DST) using three different sizes of boxes (60 x 60 mm; 101.6 x 101.6 mm, and; 304.8 x 304.8 mm). In their study, they reported decrease in the friction angles with an increase of the shear box. They also studied the scale ratio of the specimen: height to diameter ratio (H/D) and width to maximum particle size ratio (W/Dmax). They reported an increase of the friction angle with a decrease of the H/D ratio. Palmeira and Milligan (1989) found that the thickness of the shear zone at the sample mid-height was significantly affected by the scale of the test as well as the post-peak behavior presented by the sample.

### 2.5.3 Allowable Maximum Particle Size

The maximum allowable particle size to be tested d, is determined by the smallest dimension of the triaxial apparatus, D. In terms of the D/d ratio, Penman (1971) states that the lower limit is 4 for a broad grading, or 6 for a narrow grading (as used by Marsal, 1973) after performing triaxial tests on rock fill dam material. Holtz and Gibbs (1956) were able to obtain a consistent Mohr envelop for D/d>=6, but obtained higher values of friction angle φ for D/d =4.
There are several different standard systems to determine the maximum allowable particle size for a shear box test.

- The ASTM D 3080-90 standard, where the maximum allowable particle size for a large shear box test is 1/10 of the box width and 1/6 of the box height. A minimum specimen width to thickness ratio 2:1 is required. Square boxes are used.

- The Japanese standard, where the maximum allowable particle size for a large shear box test is 1/10-1/5 of the box length, 1/7-1/5 of the box height and 1/9-1/5 of the smaller of the box length or height (Lee et al., 2009).

The Japanese standard allows smaller shear box size with respect to the maximum particle size than the ASTM standard. Both recommendations were considered in selecting the maximum particle size for the tests and discussed in Chapter Three.

### 2.6 Shear Strength Models

There are a number of different models for describing the strength of rockfill materials. Table 2.1 shows the shear strength model for rockfill as summarized by Douglas (2002).

In this study, the four most widely used and accepted strength models for soil and rock (Mohr-Coulomb model, Power Curve strength model, Hoek & Brown model and Barton model) are used to analyze the direct shear tests data. The basic concept of these models is explained below.

#### 2.6.1 Mohr-Coulomb Model

A failure criterion governing the shear failure of soils was first put forward in 1776 by Coulomb and later modified by Mohr in the form of equation 2.1. It is the most common failure criterion. The shear strength on a given surface depends linearly on the normal stress ($\sigma_1$, $\sigma_2$, and $\sigma_3$) acting on that surface (Figure 2.3).

$$\tau = c + \sigma \tan \phi$$  \hspace{1cm} Equation 2.1
Where:

\( \tau \) = Shear strength

\( c \) = Cohesion

\( \sigma \) = Effective normal stress on the failure plane

\( \varphi \) = Friction angle

Mohr and Coulomb defined the strength of soils by two parameters: cohesion and the internal friction angle of the soil particles to resist failure.

### 2.6.2 Power Curve Strength Model

The common criterion used for analyzing failure of geomaterials is the linear Mohr-Coulomb equation as indicated above. On the other hand, some authors report that a nonlinear failure criterion fit their data more closely. Charles and Watts (1980) developed a Mohr resistance envelope with equation 2.2 where \( A \) and \( b \) are material constants.

\[
\tau = A\sigma_n^b
\]

Equation 2.2
2.6.3 Hoek -Brown Model

Hoek-Brown model is an empirical failure criterion that establishes the strength of rock in terms of major and minor principal stresses. It predicts strength envelopes that agree well with values determined from experimental triaxial test of intact rock, and from observed failures in jointed rock masses (Rocscience Inc., 2004). The generalized Hoek-Brown failure criterion is expressed as:

\[ \sigma'_1 = \sigma'_3 + \sigma_{ci}(m_b \frac{\sigma'_3}{\sigma_{ci}} + s)^a \]

Equation 2.3

Where \( \sigma'_1 \) and \( \sigma'_3 \) are the major and minor effective principal stresses at failure; \( \sigma_{ci} \) is the uniaxial compressive strength of the intact rock material; \( s \) and \( a \) are constants for the rock mass given by the following relationships:

\[ s = \exp \left( \frac{GSI - 100}{9 - 3D} \right) \]

Equation 2.4

\[ a = \frac{1}{2} + \frac{1}{6} (e^{-GSI/15} - e^{-20/3}) \]

Equation 2.5

\( s \) is the strength reduction factor, i.e. the ratio of the uniaxial compressive strength of the rock mass and rock material.
Table 2.1: Various shear strength model for rockfill (Douglas, 2002). In addition to Douglas’ models, the Hoek Brown model and Lee’s model are also included (Sun, 2010).

<table>
<thead>
<tr>
<th>Reference</th>
<th>Equation</th>
<th>Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>De Mello (1977)</td>
<td>$\tau = A + \sigma_n^p$</td>
<td>$A,B=4.4,0.75$ (Sandy gravel); $A,B=4.4,0.75$ (Soft rockfill); $A,B=1.4,0.90$ (Soft rockfill);</td>
</tr>
<tr>
<td>Charles &amp; Watts (1980)</td>
<td>$\tau = a (\sigma_n/\sigma_c)^b$</td>
<td>$a,b=0.25,0.83$ (lower bound, $\sigma_n=0.1-1$ MPa); $a,b=0.71,0.84$ (upper bound, $\sigma_n=0.1-1$ MPa); $a,b=0.75,0.98$ (lower bound, $\sigma_n=1-7$ MPa); $a,b=1.80,0.99$ (upper bound, $\sigma_n=1-7$ MPa);</td>
</tr>
<tr>
<td>Indraratna et al (1993)</td>
<td>$\tau = A (\sigma_n/\sigma_c)^B$</td>
<td>A increase with $\sigma_n$, $\tau$; $\gamma$, $\delta_{50} \approx 0.7 - 1.5$; B increase with $\sigma_c$, $\tau$, $\gamma \approx 0.419 - 0.911$; $\sigma_0 = 1$ MPa</td>
</tr>
<tr>
<td>Indraratna (1994)</td>
<td>$\sigma_1/\sigma_c = a (\sigma_3/\sigma_c)^b$</td>
<td>$a,b=0.4,0.62$ (lower bound, 0.1 to 1 MPa); $a,b=0.78,0.65$ (upper bound, 0.1 to 1 MPa); $a,b=2.71,0.96$ (lower bound, 1 to 7 MPa); $a,b=3.58,0.99$ (upper bound, 1 to 7 MPa);</td>
</tr>
<tr>
<td>Indraratna et al (1998)</td>
<td>$\sigma_1/\sigma_3 = a\sigma_3^b$</td>
<td>$a,b=54.98, -0.49$ (gradation A); $a,b=125.17, -0.56$ (gradation B);</td>
</tr>
<tr>
<td>Doruk (1991)</td>
<td>$\sigma_1' = \sigma_3' + \eta \left( \frac{m_0}{\sigma_c^2} \right)^a$</td>
<td>$m, a$</td>
</tr>
<tr>
<td>Barton &amp; Kjærnsli (1981)</td>
<td>$\phi' = R \log \left( \frac{S}{\sigma_n} \right) + \phi_b$</td>
<td>$\phi' = \arctan \frac{\tau}{\sigma_n}$; $\phi_b$ - basic friction angle; $R$ - equivalent roughness; $S$ - equivalent strength;</td>
</tr>
<tr>
<td>Charles (1991)</td>
<td>$\phi' = C_1 \log \left( \frac{C_2}{\sigma_3} \right) + \phi_b$</td>
<td>$\phi' = \arctan \frac{\tau}{\sigma_n}$; $\phi_b$ - basic friction angle; $C_1, C_2$ - constants;</td>
</tr>
<tr>
<td>Gonzalez (1985)</td>
<td>$\phi' = \phi_0 - j \log_{10} \left( \frac{\sigma_{fr}}{\sigma_{cr}} \right) \phi_0, \sigma_{fr}, \sigma_{cr}$</td>
<td>$\phi' = \arctan \frac{\tau}{\sigma_n}$; $\phi_0, \sigma_{fr}, \sigma_{cr}$ - standard crushing grain strength;</td>
</tr>
<tr>
<td>Hoek Brown (1997, 2002)</td>
<td>$\sigma_1' = \sigma_3' + \sigma_c \left( \frac{m_b \sigma_3}{\sigma_c} \right) \sigma_s^a$</td>
<td>$\sigma_1'$ and $\sigma_3'$ are the major and minor effective principal stresses at failure; $\sigma_c$ is the uniaxial compressive strength of the intact rock material; $s$ and $a$ are constants for the rock mass;</td>
</tr>
<tr>
<td>Lee (2009)</td>
<td>$\phi = 0.09 \text{UCS} + 35.2$</td>
<td>UCS is the uniaxial compressive strength of the parent rock in MPa; $\phi$ is the internal friction angle in degrees</td>
</tr>
</tbody>
</table>
\( \sigma_n \) = Normal stress;
\( \sigma_c \) = Unconfined compressive strength of the intact rock pieces;
\( C_u \) = Coefficient of uniformity;
\( \gamma \) = Unit weight;
\( d_{50} \) = Particle diameter at which 50% of the material is finer;

\( m_b \) is a reduced value of the material constant. It represents the degree of interlocking of the rock mass and quality of discontinuity walls and is given by:

\[
m_b = m_i \exp \left( \frac{GSI - 100}{28 - 14D} \right)
\]

Equation 2.6

\( mb \) and \( s \) are rock constants while \( mi \) is a material constant for intact rock that plays the role of friction angle for a curved failure envelop.

GSI, known as the Geological Strength Index, relates the failure criterion to visual geological observations in the field. Its value ranges from 100 for fully intact rock down to 0 for very poor and laminated / sheared rock sections. The GSI parameter can be selected on the basis of the well-known charts as depicted in Figure 2.5.

\( D \) is a factor which depends upon the degree of disturbance to which the rock mass has been subjected by blast damage and stress relaxation. It varies from 0 for undisturbed in situ rock masses to 1 for very disturbed rock masses.

Figure 2.5: Estimate of Geological Index GSI based on geological descriptions (Hoek & Brown, 1997).
The rock fill can be treated as a rock mass that is highly damaged (D=1), poorly interlocked (low GSI), without any significant uniaxial compressive strength and tensile strength (s=0). However, contrary to the Barton model, the Hoek and Brown model does not capture the dilatancy behavior observed during shearing. It allows for volume expansion due to tensile stresses rather than volume expansion due to shearing (Brinkgreve, 2010).

### 2.6.4 Barton Model

The Barton failure criterion (Barton, 1973; Barton, 1976; Barton & Choubey, 1977) is an empirical relationship widely used in modelling the shear strength of rock discontinuities (Rocscience Inc., 2004). Barton and Kjærnsli (1981) compared the behaviour of natural rock joint with that of rockfill (Barton & Kjærnsli, 1981; Barton, 2008). They found that rockfill and rock joints have several features in common, including dilatancy behaviour under low effective normal stress, and significant crushing of contact points with reduced dilation under high normal stress. The Barton failure criterion has the non-linear form:
\[ \tau = \sigma_n \tan[R \times \log(S/\sigma_n) + \phi_b] \]
\[ i = R \times \log(S/\sigma_n) \]

Equation 2.7

Where:
\( \tau \) is peak shear stress;
\( \sigma_n \) is peak normal load;
\( \phi_b \) 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;

\( \phi_b \) reflects the surface texture and mineralogy of the rock material; it depends on the mineralogy and grain size of the rock material. \( \phi_b \) can be obtained by carrying out direct shear box tests or tilt tests on smooth saw cut or sand blasted discontinuities. Alternatively, it can be derived from a table in which Barton summarized values of the basic friction angle values published in the 1960’s and early 1970’s for a number of rock types (Barton, 1973). It is probable that values derived from sand blasted surface are low estimates of the basic friction angle. Damage cracks have been found at a distance of 0.1 mm behind the sand blasted surface (Verhoef, 1987). These cracks are likely to cause an early grain failure during shearing.

**Estimation of S (JCS) and R (JRC)**

The techniques available for estimating the input parameters to Barton’s criterion are discussed below. \( R \) and \( S \) values can be estimated using empirical charts. \( R \) is a function of porosity of the rockfill and particle origin, roundedness and smoothness (Figure 2.6). \( S \) can be estimated empirically by the \( S/\)UCS reduction factors once the mean particle size is known (Figure 2.6). Barton and Kjaernsli (1981) explains that the \( S \)-shape of the curve is probably due to the fact that large rock samples contain many micro-cracks and sand grains, none. The reduction of particle strength with decreasing mean particle size is stronger in triaxial than in plane strain conditions. It reaches up to 70% when the mean particle diameter varies between 5 and 20 mm in triaxial conditions and, up to only 30 % when the mean particle diameter varies between 3 and 15 mm in plane strain conditions. According to Barton model, an increase in mean particle size will result in a decrease in rock fill strength. Any difference in grading is captured in the achieved porosity, and therefore in the equivalent roughness parameter.
Figure 2.7: An empirical method for estimating the equivalent roughness $R$ of rockfill as a function of porosity and particle origin, roundedness and smoothness. Barton and Kjærnsli (1981).
Figure 2.8: Method of estimating equivalent strength (S) of rockfill based on uniaxial compression strength and d50 particle size (Barton & Kjaernsli, 1981)

Figure 2.9: Principle of tilt test for rockfill (Barton, 2008).
Note the expression of $R$, the equivalent roughness for the rock fill in 5 (Figure 2.9) derived by applying the Barton’s model to the tilt test conditions. $\alpha$ is the tilt angle and $\sigma'_{n}$ the normal stress on the sliding surface.

Once $S$ and $\varphi_b$ are known, it is possible to back-calculate $R$ from a tilt test as explained in Figure 2.9. $R$ and $S$ are used to estimate $i$, the structural component of strength. $i$ corresponds to an increase of strength due to interlocking of rock fill particles. It is the equivalent for rock discontinuities of the contribution of interlocked asperities. For discontinuities, the structural component of strength is related to discontinuity dilatancy. When shearing causes a slight damage of discontinuity walls, they are equal (Barton & Choubey, 1977). In a similar way, for rock fills made of strong rocks with respect to applied stresses, one can expect that the structural component of strength is equal to the dilatancy at failure.

$$i = \Psi$$

With $\Psi$, the rockfill dilatancy at failure.

Dilatancy is strongly stress dependent which explains partly the non-linearity of the failure envelop of rock fills. As stress increases towards the equivalent particle strength, $\varphi_b$ crushing at contact points between particles becomes dominant and $i$ decreases.

By measuring shear strength and dilatancy during direct shear box shearing under different normal stresses, the parameters of the Barton’s model might be derived. The obtained values (in case of a reasonable fit) can be compared to values either derived using the table proposed by Barton for the basic friction angle and the charts developed to estimate the equivalent particle strength and roughness or back-calculated from tilt test results.

Barton (1973) indicated that at low values of normal stress, a maximum value of secant friction angle of 70 degree seems to occur with some frequency on rock joint although it is quite possible for rough, continuous joints to have friction angle up to 80 degree at extremely low normal stresses.

Leps (1970) assembled a significant number of large-scale triaxial shear test data for rockfills of various types. Barton (1981) used these data to fit in the Barton’s model and suggest that $R$ ranges from 5 to 10 and $S$ ranges from 10 to 100 MPa.
2.7 Summary of Previous Studies on Bremanger Sandstone Rockfill

Some studies for determining the shear strength of the Bremanger rockfill have been performed at TU Delft. Sun (2010), Linden (2010) and Boostma (2010) performed geotechnical characterization of the Bremanger sandstones. The studies included determination of shear strength by direct shear tests using two types of shear boxes, a smaller direct shear box of size (100*100*40mm) and a medium shear boxes (500*500*400mm). And Boostma (2010) investigated the basic friction angle using Golden shear box.

(Sun 2010; Linden 2010) investigated the Bremanger rockfill using small and medium shear box in the laboratory to determine the effects of particle size, packing density and uniformity, specimen size, and normal stress on the shear strength. Sun (2010) concluded that the shear strength of Bremanger rockfill is affected by the particle size, density, specimen size and normal stress. (Sun, 2010) found that, The Hoek-Brown model, initially developed for rock masses, can be used to predict shear strength of Bremanger rockfill.

Boostma (2010) performed shear tests using the Golder shear box to determine the basic friction angle of the Bremanger sandstone. The tests were done on a flat saw-cut surfaces and tensile cracked rock discontinuities. The study included the influence of different rock properties such as roughness, rock strength and visible layering. Boostma (2010) found out that the basic friction angles were not measured correctly using flat saw-cut surfaces. Basic friction angles measured on tensile cracked samples sheared up to their residual strength were found to be higher than published values for sandstone.


3 LABORATORY TEST PROCEDURES

This chapter begins with the description of the type of material used in this research. A detailed description of the test program and procedure are presented in the following sections.

3.1 Materials Used

The material tested in this study is Bremanger sandstone, which is used to create a cobble beach as well as a walk way for a crane and the foundation layer of the sea water breaker in the Maasvlakte 2 project (MV2). This sandstone is mined in an open pit operation situated on a mountain plateau 400-500 meters at Dyrstad, in Norway. The rockfill material was obtained by blasting and shipped to the Yangtze harbor in Rotterdam. The rock is of sedimentary origin but had undergone through some degree of metamorphism. The different field and laboratory investigations indicate that the sandstone at Dyrstad is very homogeneous both in visual appearance and rock properties (Myrvang, 1998). It can be described as strong to very strong rock with an average 189 MPa Uniaxial compressive strength. The basic physical properties of the materials are given in Table 3.1.

Table 3.1 Characteristics of Bremanger sandstone (Myrvang, 1998).

<table>
<thead>
<tr>
<th>Property</th>
<th>Unit</th>
<th>Average value</th>
<th>Minimum</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulk density</td>
<td>kg/m³</td>
<td>2749</td>
<td>2713</td>
<td>2775</td>
</tr>
<tr>
<td>Sonic velocity</td>
<td>m/s</td>
<td>5665</td>
<td>5586</td>
<td>5705</td>
</tr>
<tr>
<td>Uniaxial compressive strength</td>
<td>MPa</td>
<td>189</td>
<td>149</td>
<td>255</td>
</tr>
<tr>
<td>Elastic modulus</td>
<td>GPa</td>
<td>93.4</td>
<td>84.9</td>
<td>105.1</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>-</td>
<td>0.310</td>
<td>0.222</td>
<td>0.354</td>
</tr>
<tr>
<td>Point load index</td>
<td>-</td>
<td>10.6</td>
<td>7.5</td>
<td>13.2</td>
</tr>
<tr>
<td>Normalized Point Load index IS50</td>
<td>MPa</td>
<td>10.6</td>
<td>7.7</td>
<td>13.2</td>
</tr>
<tr>
<td>Tensile strength</td>
<td>MPa</td>
<td>13</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

The material sent by Boskalis contained gravels (60-200mm) and exceeds the specimen dimension limits designed for a medium-scale shear apparatus. Therefore, the particle sizes of the actual material have to be scalped in order to fit a medium scale shear apparatus. First, the sample was sieved to retain all particles less than 37.5mm and greater than 6.3mm. Then, the retained sample was divided into five categories 37.5-
28mm, 28-20mm, 20-14mm, 14-10mm and 10-6.3mm. Following a selected gradation the samples were weighted and mixed. A second gradation curve was derived using parallel gradation modeling technique. The sizes of the scaled aggregates ranged from 0.3mm to 3.35mm, enabling the fitting of aggregates into the small scale shear apparatus Figure 3.1. The preparation of the material used in the small scale shear box is explained in section 3.4.

![Size distribution curve](image)

**Figure 3.1:** Particle size distribution curves of the Bremanger sandstone rockfill used in this study

In Table 3.2 the \(D_{10}\), \(D_{30}\), \(D_{50}\), \(D_{60}\) of the materials used are given together with their uniformity coefficient \((C_u)\) and Coefficient of gradation \((C_c)\).

<table>
<thead>
<tr>
<th>Sample</th>
<th>(D_{10})</th>
<th>(D_{30})</th>
<th>(D_{50})</th>
<th>(D_{60})</th>
<th>(C_u)</th>
<th>(C_c)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material used for Medium scale &amp; triaxial test</td>
<td>10.5</td>
<td>15</td>
<td>20</td>
<td>23</td>
<td>2.2</td>
<td>0.93</td>
</tr>
<tr>
<td>Material used for Small-scale test</td>
<td>0.5</td>
<td>1.0</td>
<td>1.4</td>
<td>1.8</td>
<td>3.6</td>
<td>1.1</td>
</tr>
</tbody>
</table>

\(D_{10}\) = particle diameter at 10% finer; \(D_{30}\) = particle diameter at 30% finer; \(D_{50}\) = particle diameter at 50% finer; \(D_{60}\) = particle diameter at 60% finer; 
\(C_u\) = uniformity coefficient; \(C_c\) = Coefficient of gradation
For the large scale triaxial compression and medium scale direct shear tests, the same material was used. The individual particles are isotropic and very strong, as it is very difficult to break with a hammer. Figure 3.1 shows the individual coarse particles of different sizes of the tested material (Figure 3.1a&b) along with a guide for the particle shape identification (Figure 3.1c). From comparison of these figures, the materials are found to be subangular.

Figure 3. 2: a- aggregates used for large scale triaxial and medium size shear box testing, b- aggregates used for small scale shear box testing, c- guideline for determining sphericity and angularity of aggregates and sands.
3.2 Large-scale triaxial compression

3.2.1 Introduction

Large-scale triaxial compression tests have been carried out using an available compression machine (Figure 3.3). In this section, the large scale triaxial compression apparatus, sample preparation and testing procedure are presented.

3.2.2 Large-scale triaxial compression apparatus

The large-scale triaxial shear apparatus used in this study was designed for testing rock fill sample a diameter of 300mm and a height of 600mm. The confining pressure was realized by means of vacuum and was kept constant during a particular test. The vertical stress was applied by means of a hydraulic actuator. The axial (vertical) as well as the radial (horizontal) displacements were measured over the middle third part of the specimen.

The overview of the testing system is shown in Figure 3.3. The testing system includes four parts: the triaxial cell; the hydraulic loading system with control box; the data acquisition system; and a desktop computer for visualizing the testing data. The details of the parts are as follows:

- The hydraulic loading system consists of a loading frame, a hydraulic actuator, a load cell and a controller for application and measurement of displacement. The hydraulic actuator has a capacity of 150KN.

- The system used for applying a vacuum consists of a vacuum regulator, analogue vacuum gauges and electronic vacuum transducers for application and measurement of the internal vacuum, which was applied to the specimen. The vacuum level can be adjusted by means of a vacuum regulator.

- The Control and Data Acquisition System consists of a PC and a multi-programmer. On the PC a tailored software generates the required loading signals and stores the acquired data signals to the hard disk. A total of 13 channels were used for: the load cell and the actuator LVDT, the two electronic pressure transducers and the 6 radial and 3 axial LVDT-s.
3.2.2 Sample preparation

Sieving

In this study, the maximum particle size used was 37.5mm. The specimen diameter to maximum particle size ratio is 8 and respects the requirement of having a ratio greater than 6. Six sieves (6.3mm, 10mm, 14mm, 20mm, 28mm) were used to separate the Bremanger sandstone into five categories (6.3mm<P<10mm, 10mm<P<14mm, 14mm<P<20mm, 20mm<P<28mm, 28mm<P<37.5mm ). Then, it was flushed with water to remove dust and oven dried. The material was weighed following the selected grain size distribution curve of the test material as shown in Figure 3.1 and was mixed thoroughly. 75kg of material was used to prepare the triaxial sample.

Preparation of Specimens

The sample was prepared as follows:

1. For a single test, two low-density polyethylenes sheet of 0.4mm thickness were used to prepare a slight barrel shaped cylinder. This design of the membrane was introduced in order to minimize confinement due to the membrane. Then, the
The first membrane was stretched around the bottom plate. To ensure an airtight seal, the side of the plate was greased and o-rings were stretched over the membranes which were kept in place by two grooves. The second membrane was pulled in on top of the first membrane and sealed using grease and o-rings the same way as the first (or inner) membrane.

2. Then, a steel mold was placed around the membrane, which split both horizontally and vertically (Figure 3.4a). The steel mold facilitates the sample building process and compaction of the specimen in multiple layers. To prevent fine materials from being sucked out of the specimen a PVC plate and a geotextile were placed between the bottom plate and the specimen, as well as on the top plate.

3. Compaction was carried out to have a certain density of the test materials for all the tests. In order to construct a homogenous sample for triaxial testing, the density must be controlled throughout the sample. The selected method for creating a homogenous sample was as follows: the specimen was compacted in 7-8 layers. For each layer the same amount of material was weighed. For each layer, the material was poured into the mould and compacted by hand tamping. The compacting tool used has deadweight of 14.645Kg with a circular base of 12 cm diameter (Figure 3.4b). With the experience gained from the first trial tests it was found that 25 blows were sufficient to achieve the targeted density before a given new layer was added, and compacted by repeating the procedure. The method was found to be satisfactory and all samples were prepared to an initial density of 1720 +/- 10 kg/m3.

4. Then, the top plate was sealed in the same way as the bottom one as discussed in step 2.

5. At last, the vacuum was applied through the top and bottom of the specimen before removing the steel mould. Due to the angularity of the material used, densification often results in damage or puncturing of the membrane surrounding the sample. In order to build up, a required confining pressure, the holes in the membrane were found using a “snoopy” and sealed with tape. Then, instrumentation was installed.
Figure 3.4: triaxial specimen preparation. Clockwise a) steel mold b) compaction by hand tamping c) applying vacuum d) instrumentation

3.2.3 Instrumentation and Axial and radial deformation measurements

Firstly, 3 small clamping blocks at 120° were glued and, 3 small glass plates were glued in between these blocks. The former was used to hold the rings in place while the latter were to be used as a platform for the radial LVDT-s. As shown in Figure 3.4c two rings were placed surrounding the samples circumference at 1/3H and 2/3H. On each ring, the 3 radial LVDT transducers were placed at 120°. The axial deformation was measured using three axial transducers fitted to the rings spaced at 120° to measure relative movement between the lower and upper rings spaced at 200mm and measure
displacements as much as 10 mm. Their accuracy of 0.01mm corresponds to 0.005 % strain. They could be reset to zero at any time during the test. Their readings were averaged to calculate axial the strain at the midsection of the sample. The system had been proved to be accurate in the measurement of volume change during previous studies (Niekerk, 2002). Moreover, the axial displacement was measured by recording the movement of the piston relative to the top of the cylinder.

3.2.4 Test procedure

Triaxial compression testing was performed. Axial load, axial strain, radial strain, and confining pressure were continually recorded during testing. All the tests were carried out on a specimen with an initial density of 1720 +/-10 kg/m³, at a constant vertical displacement rate of 2 mm/min. Confining pressures of 7.5kPa, 15kPa, 30kPa and 60kPa were used in the tests. This pressure range is adequate to simulate the confining pressure existing within the 10m high dike.

3.2.5 Data processing

3.2.5.1 Strain calculations

Figure 3.5 shows the measurements on the middle section and full length of the sample as a function of time. The measurement for the whole sample corresponds, of course to the rate of vertical deformation which was set at 2mm/min. The measured axial strain in the middle section of specimen was different from the whole sample. The increase in axial strain was larger in the whole sample than at the middle section. This difference could be due to closing of gap between the cap and sample at the commencement of the test or the punching of the aggregates at the top of the sample. The stiff loading plate constrains their rearrangement, the aggregates can only move downwards. For the tests conducted in this study, due to triaxial equipment limitation (instrumentation) all middle section measurements were taken up to <10 % axial strains.

The axial strain and radial strains are calculated by averaging the 3 vertical and, respectively 3 horizontal displacements measured at the central part of the sample divided by the initial height (20cm) and respectively, the initial radius (15cm). The volumetric strain is calculated by summing the vertical strain and twice the radial strain. The dilatancy is calculated by dividing volumetric strain by the axial strain. The dilatancy angle is the arctangent of this value.
3.2.5.2 Stress calculations

a. Area correction

A standard area correction was used, assuming that the sample deforms as a right circular cylinder.

The area correction is given by equation (4.1):

\[ A_c = \frac{A_o (1 - \varepsilon_{vol})}{1 - \varepsilon_a} \]  

Equation 4. 1

Where:

- \( A_c \) = corrected area of specimen
- \( A_o \) = initial area of specimen
- \( \varepsilon_{vol} \) = volumetric strain of specimen
- \( \varepsilon_a \) = axial strain of specimen

b. Correction for Membrane Strength

The need for membrane correction was considered because axial strain levels as high as 10 % were reached in some tests. However, taking into consideration the design of the membrane, the radial stress developed in the rubber membrane which reduces the confining pressure is expected to be low. Therefore, it was not evaluated. The membrane correction was not used in the interpretation of test results.
c. Correction for aggregate self weight

During sample preparation in the split mould, the only stress is due to its own weight and the weight of the top cap. For a sample height of 0.6 m, a diameter of 0.3 m and a density of 1720 kg/m$^3$, the vertical stress $\sigma_v$ due to the self weight of the sample equals to 10KPa at the bottom of the sample. The vertical stress is not uniform, it increases linearly from top to bottom of the sample. At the end of the sample preparation when the mold is restraining lateral movement, the horizontal stress equals to:

$$\sigma_h = K_o \gamma h$$  \hspace{1cm} \text{Equation 4.2}

where:

- $K_o$ is the coefficient of lateral earth pressure ($K_o = 1 - \sin \varphi$),
- $\gamma$ is unit weight, and
- $h$ is the height measured from the top sample.

This corresponds to a horizontal stress $\sigma_h$ of 2KPa, which is significant in comparison to the applied low confinements (7.5 & 15KPa). Once the split mould is removed, it is considered that this horizontal stress vanishes. The vertical stress gradient persists. Nevertheless it was not considered in the calculation.
3.3 Medium Shear Box Tests

3.3.1 Introduction

To compare the results of triaxial shear tests with those of the shear box tests, shear box tests were performed on previously used specimen in triaxial compression test. It was compacted directly in the shear box to a density of 1720 Kg/m$^3$ and tested under low normal stresses (1.8-33KPa). The Medium shear box apparatus, specimen preparation and testing procedure are explained below.

3.3.2 Medium Shear Box Test Apparatus

The Medium-scale direct shear test apparatus shown in Figure 3.6 was used for the test. The apparatus consists of a direct shear box that is 0.5m wide and long and 0.4m high, vertical and shearing loading units, a steel frame, force and displacement measuring devices, and a data acquisition system. The upper box is prevented from moving vertically or rotating; the load plate moves independently. According to the Shabuya 1997 classification (section 2.2) it is type C (without Teflon).

Figure 3. 6: The TU Delft medium size shear box (top); Sketch of the medium size shear box (bottom) (Modified from van der Linden, 2010).
Loading system

a. Vertical Loading System
The vertical load is applied on the specimen with a loading steel plate dead weight on top. As a result, it is low. At the maximum, 800 kg of steel plates are applied onto the top plate which corresponds to a normal stress of 32 kPa. The plates are prevented from toppling down by safety straps hanging loosely onto the portal crane. During shearing, the top plate is tilted when the aggregates are sheared and dilate. As a result, the dead weight applied has both a normal and shear load component. Tilt angles were recorded during the whole shearing test.

b. Shearing loading and load cells
The shearing load is applied by an electric motor with a worm wheel and reduction gear to the lower shear box while the upper shear box is onto the steel frame. The load cells placed between the steel frame and the upper box record the horizontal force applied by the content of the lower box being sheared against the material contained in the upper box. The capacity of each load cell is 50 kN so that the maximum shear strength that can be generated during testing is 500 kPa at 20% horizontal strain. The horizontal forces measured by both load cells can differ. In this report, they are summed. The friction between the upper and lower box is less than 85 N. It was measured by conducting shear tests using empty boxes.

Strain measuring system

a. Vertical Displacement Measurement
The range of the vertical displacement is only 2 cm, it was re-set during the test to extend the measuring range if necessary. The loading plate can tilt during the shearing test. Therefore, vertical displacement was corrected for tilt in the result analysis.

b. Horizontal displacement Measurement
The range of the horizontal displacement is 20 cm while the maximum allowable horizontal displacement is 10 cm, which corresponds to 20% horizontal strain. The dial gauges and the force transducers are connected to the mp3 data acquisition system, an in-house software developed at TU Delft.
During shearing, the aggregates tend to lift up the upper box when they dilate. To prevent rocks from getting trapped in between the edges of the boxes during shearing the vertical movement of the top shear box was restricted by inserting wooden blocks between the top shear box and the steel frame. Note also that the rotation of the upper box would impose a moment on the horizontal force transducers susceptible to damage the transducers. The effect of the wooden blocks on the shear test is unknown. During some of the tests force transducers were placed at the four corners of the top box to measure the normal forces that develop in the wooden blocks that restrain the top box from moving up (Figure 3.7). In one test, Teflon was used to minimize the friction between the normal force transducer’s and the box.

![Image of wooden blocks and normal force transducer](image)

**Figure 3.7: Transducers placed at the four corners**

### 3.3.3 Medium size shear box experiment procedure

**Sieving**

The same sieving method as for the triaxial test was applied (section 3.2.3). 175 kg of material was used to prepare a sample the material was reused. The medium scale direct shear test is 500*500*400 mm. In this study, the maximum particle size ($d_{\text{max}}$) used was 37.5mm. The $d_{\text{max}}/W$ is 1/13.3 and $d_{\text{max}}/H$ is 1/10.6, with $W$ the width of the box and $H$ its thickness. So both, the Japanese and ASTM standard have been fulfilled. Nevertheless, the dimensions of the medium size shear box do not respect the height to width ratio of the ASTM standard.
**Shear speed**
All tests were conducted at the same shearing speed (10 mm/min).

**Shear strain**
A 20% shear strain was achieved for all the medium size shear box tests.

**Compaction**
All tests were performed in a dry condition at density of between 1710-1720 Kg/m$^3$.

**Test**
Four different vertical loads were used: 4kPa, 12kpa, 22kPa and 33kPa.

* In order to reduce any breakage at the interface between the specimen and the rigid base, a 2cm thick plastic was placed on the base of the lower shear box.

**3.2.4 Data processing**
The forces measured by the two force transducers are summed and the friction force between the two shear boxes (upper and lower) which was obtained by running a test without samples is deducted. The normal load at the commencement of the test is calculated by dividing the dead weight by the cross sectional area, which is equal to the size of the shear box (0.5m*0.5m). During shearing the area upon which the normal load is acting reduces. The width remains constant but the length reduces by the amount measured by the horizontal displacement transducer. The corrected normal stress is calculated dividing the dead weight by the new cross section area. The shear stress is calculated by dividing the measured shear force by the corrected cross sectional area. The dilatancy angle is arctangent of the ratio of vertical to horizontal displacement.

**3.3.5 Colored grains inside shear box**
Colored grains were used to understand the particle movement inside the shear box during shearing. The coloring was obtained by immersing the grains into a methylene blue solution. Colored grains (numbered 1 to 20) were placed at chosen locations and depths in the shear box. Figure 3.8 shows the initial layout of the colored grains in the
shear box. In addition, colored grains with the same grain size distribution gradation as the host material were placed at the interface between the top and bottom shear boxes (Figure 3.9). They were distributed throughout the plane uniformly during sample preparation. The total displacements of the colored particles were recorded after test.

Sieving was performed after each test to determine the grain size distribution of the colored sample. The weight of each colored grain was recorded before and after each test as well. A series of shear tests were conducted at normal stresses of 22 and 33.6KPa. Both tests were deformed to a 20% shear strain.

Figure 3.8: Layout on the shear box

Figure 3.9: colored Bremanger aggregates (plane view)
3.3.6 Tilt test

Theory on Sliding

Sliding of a block on a plane only occurs when the dip of a plane exceeds the angle of friction. Toppling can also occur when the weight line of the tilted block falls outside its base, i.e. when

\[ \frac{b}{h} = \tan \psi \]

where \( b \) is the width,
\( h \) is the height and
\( \psi \) is the inclination angle of the block

Hudson & Harrison (1997) represented the conditions for the sliding and toppling as shown as in Figure 3.10.

Figure 3. 10: Sliding and toppling instability of a block on an inclined plane (from Hoek and Bray, 1977)

The tilt box is made of two halves. Therefore, three conditions of toppling have to be considered:

i) toppling of the upper box by rotation around its bottom edge
ii) toppling of the whole box by rotation around its bottom edge
iii) toppling of the upper box by rotation around the bottom edge of the lower box
This corresponds to:

i) $\tan \psi > \frac{2b}{h}$

ii) $\tan \psi > \frac{b}{h}$

iii) $\tan \psi > \frac{2b}{3h}$

**Tilt Test**

Tilt test was carried out using the equipment shown in Figure 3.11. It has the same dimension as the medium shear box (500mm x 500mm x 400mm). The test material for the tilt test was prepared the same way as for medium shear box mentioned above (section 3.3.3). Toppling conditions i, ii and iii correspond to inclination angles of $68^0$, $51^0$ and $39^0$ respectively.

As shown in Figure 3.11, in the test, the box was gradually tilted by winding up a pulley an attached rope on base of the lower block at a low rate to minimize any impact. The static angle of repose, $\varphi_i$, was registered through the observations as described in (Yamaguchi et al, 2010) of the following angles:

(i) The angle at which some pieces of material dropped from the specimen;
(ii) The angle at which part of the specimen surface started to collapse;
(iii) The angle at which the entire surface of the specimen began to collapse, and;
(iv) The angle at which the specimen as a whole started to collapse.

Yamaguchi et al. (2010) considered the static angle of repose corresponds to the angle at which the entire surface of the specimen began to collapse.

Figure 3.11: Tilt test equipment
3.4 Small Shear Box Tests

3.4.1 Introduction

To compare the results of medium shear box tests and triaxial shear tests with those of the small shear box tests, small shear box tests were performed on scaled down specimens (0.3-3.35mm), compacted directly in the shear box, at high density (1720 kg/m³) under low normal stress σ (1.8-56kPa) comparable to that used in the medium shear box test and triaxial test. The small shear box test apparatus, specimen preparation and testing procedure are explained below.

3.4.2 Description of Small Shear Box Test Apparatus

The small scale direct Shear test apparatus comprises a direct shear box assembly for a 100mm x 100mm x 40mm thick specimen, a steel frame, a thyristor controlled drive unit, a loading ring, a weight hanger, a loading yoke and a data acquisition system. The vertical load is applied by the yoke which is placed on the loading cap and by putting weight on the weight hanger. For greater normal load, the slotted weights can put on the hanger from the level. In this case, the applied weight is multiplied by a factor of 11 because of the length of the beam (Figure 3.12).

The lower shear box is fixed to a carriage. The shearing load is applied to the carriage as well as the lower shear box while the upper shear box is fixed (Figure 3.12). The maximum load of the loading ring is around 4.5 KN. The dial gauges and the force transducers are connected to the data acquisition system WINCLISP program v4.51.

The shear apparatus is the combination of type A&C following Shibuya 1997 classification; the upper box is not completely prevented from moving vertically or rotating; the top platen moves independently and has freedom to move vertically and to rotate (Figure 3.13).
Figure 3. 12: Small Shear box test apparatus
3.4.3 Small Shear Box Test Procedure

The preparation of the rockfill specimen and testing procedure closely follow the conventional standard laboratory shear box test procedures for soils as described in (Mulder & Verwaal, 2006). Details of the procedures are presented below.

Crushing

First the integrity of the small aggregates was investigated using a micro-CT scanner. After the scanning the images were processed with Qwin to reconstitute the sample (Figure 3.13). From the images it is clear that the small grains contained many micro-cracks that can facilitate their breakage during shear box testing. The smaller grains might have originated from corner chopping of bigger aggregates that have been exposed to collision during transportation. They were discarded and it was decided to produce small size aggregates by controlled crushing.

Rock was crushed in three to four cycles. In the first cycle the crushed material was elongated and very angular. Each time the inner size of the crusher was reduced, the crusher broke the rock into smaller grain size. After screening to separate the different sized particles, the crushing process continues until the requirements proposed by (Fumigalli, Mosconi, & Rossi, 1970) are meet. i.e., equality of strength in all fractions and similarity of particle shape. By visual inspection, the small size aggregates were found to have the same sphericity and angularity as the aggregates tested in the large size equipments.
Sieving

The first stage in the preparation of the test specimens was the separation of the crushed Bremanger sandstone into different grain fractions by dry mechanical sieving. The grains were separated into five categories: $0.3\text{mm}<P<0.425\text{mm}$, $0.425\text{mm}<P<0.6\text{mm}$, $0.6\text{mm}<P<1.18\text{mm}$, $1.18\text{mm}<P<3.35\text{mm}$ and $3.35\text{mm}<P<6.3\text{mm}$. During this stage, the corresponding percentage for each grain fraction was estimated. The quantity of material required was determined from the volume of the test cell. The material weighed from each fraction was then mixed thoroughly. ASTM D 3080-90 requires a minimum specimen thickness of six times the maximum particle diameter and a minimum specimen width of 10 times the maximum particle diameter and the specimen used in this direct shear test is 100*100*40 mm. In this case, the whole categories are within the requirement. The $d_{\text{max}}/W$ is 1/30 and $d_{\text{max}}/H$ is 1/12. So the Japanese standard is also fulfilled. It should be noted that the small shear box also respect the thickness to width ratio criterion of the ASTM standard.

Figure 3. 14: Images obtained from CT-scan, the cracks are indicated with an arrow
Preparation of specimen and test

- The mass of the sample corresponding to the volume of the shear box and the required compaction density was quantitatively measured. For each specimen, the material is weighed before being placed inside the shear box it was calculated to yield the same density as that obtained in previous medium shear and triaxial shear tests.

- The shear box was cleaned, and the top half was placed and fastened together using the two shear box alignment screws. The alignment screw ensures that the prepared specimen is not disturbed during preparation and placement of the sample. The sample was then placed into the shear box and tampered in three layers. The shear box is then placed in the shear test device. Then the shear box restraining screws were tightened to secure the shear box. The transducers for measuring the vertical and horizontal deformations were adjusted so that the sensors can measure at least 10 mm of deformation. The two alignment screws were removed before the direct shear test was started.

- After placing the compacted specimen in the shear box, the transducers for measuring the vertical and shear deformations were adjusted. The additional dead weight to be placed on the hanger was calculated to give the similar normal stress $\sigma$ (kPa) as that obtained from previous shear tests, and placed on the hanger.

- The motor was switched on and readings were taken at 2 seconds interval until the peak strength was reached. Readings were continued until the ultimate strength was reached. The motor was stopped, then the vertical displacement transducer was raised and the dead weights were lifted off. The box was reversed manually until the initial shear displacement was obtained and the shear box was removed. For each test the same procedure was followed.

The effect of shear rate deformation was studied by using deformation rates of 1mm/min and 2 mm/min. The displacement rates were set by choosing the appropriate pair of gear wheels, and the right position of the gear lever. The effect of compaction was also studied by using specimens with different densities.
3.4.4 Data processing

The normal load before the commencement of the test is calculated by dividing the dead weight (corrected by the lever arm length if needed) by the cross sectional area, which is equal to the size of the shear box (0.1m*0.1m). During shearing the area upon which the normal load is acting reduces. The width remains constant but the length reduces by the amount measured by the horizontal displacement transducer. The normal stress is corrected by dividing the dead weight by the new cross section area. The shear stress is calculated by dividing the measured shear force by the new cross sectional area. The dilatancy angle is arctangent of the ratio of vertical to horizontal displacement.
4 LABORATORY TEST RESULTS AND DATA ANALYSIS

4.1 Triaxial compression Test Results

This section presents the results of tests performed in the large-scale triaxial compression apparatus following the test procedure presented in Section 3.2.4. The deviatoric stress-axial strain and the volumetric strain-axial strain curves are presented.

4.1.1 Stress-Strain Behaviour

Deviatoric stress-strain-volume change behaviors of the Bremanger sandstone rockfill subjected to triaxial testing are shown in Figure 4.1. In these figures, HD stands for the high density (1710 Kg/m³) and ND stands for normal density (1560 kg/m³), the last two digits of the test label represent the confining pressure value in kPa.

The linear stress-strain response of the Bremanger sandstone rockfill becomes stiffer as confining pressure increase. This approximately linear behavior continues for the confining pressure of 60 kPa from the start of loading up to an axial deformation of about 0.5%. As expected, the peak deviator stress increases with the increasing confining stress. At low confining pressure $\sigma_3$, all specimens, showed volumetric dilatancy ($\varepsilon_v > 0$), whereas at confining pressure $\sigma_3$ of 60kpa, the specimen first compresses then dilates. In these graphs, the dilation (volume increase) is considered to be positive, and compression to be negative.

The strains measured at peak deviatoric stress $(\sigma_1 - \sigma_3)_p$ for different values of confining pressures $\sigma_3$ are plotted in Figure 4.2. The volumetric strain ($\varepsilon_v$) and axial strain at peak deviator stress increases with the confining pressure (8.9 and 3.6% respectively at $\sigma_3' = 7.5$kPa to 13.7 and 5.8% respectively at $\sigma_3' = 60$kPa).
4.1.2 Shear Strength

In Figure 4.3, the variation of the peak principal stress ratio with confining pressure is shown. At low confining pressure the specimen shows a significantly high \((\sigma_1 / \sigma_3)p\), which decreases as the confining pressure increase. The high values of principal stress ratio at peak exhibited by the Bremanger sandstone rockfill at low confining pressure can be attributed to the greater frictional interlock of predominantly sub-angular particles.
Figure 4.3: Influence of confining pressure on principal stress ratio at peak deviatoric stress.

Figure 4.4 presents a comparison of the variation of shear strength with the increase in normal stress. The results were fitted with the Mohr-Coulomb’s failure criterion and the power curved law. The shear strength envelope is non-linear; it is associated with the dilatant behaviour of Bremanger sandstone rockfill at low normal stress and is better described by a power relationship than the Mohr-Coulomb. The Mohr-Coulomb’s fit gives a friction angle of $49.9^\circ$ and cohesion of 14.6 KPa, its $R^2$ is 0.998 and less than the $R^2$ of the power curve which is 0.999.

Figure 4.4: Failure envelope of the Bremanger sandstone rockfill tested in triaxial conditions
4.1.3 Influence of Confining Pressure on internal Friction Angle and Dilatancy angle

Figure 4.5 shows the variation of the internal friction angle $\varphi_p$ and dilatancy angle $\Psi_p$ with increase in confining pressure. When the confining pressure was increased from 7.5 kPa to 60 kPa, the internal friction angle dropped from $62.9^\circ$ to $52.8^\circ$ and dilatancy angle dropped from $30.9^\circ$ to $16^\circ$. The high values internal friction angle at low confining pressure are believed to be related to the inter particle contact forces that are well below the crushing strength of the parent rock, and the ability of the interlocking particles to dilate under lower stress levels.

Figure 4.5: Influence of confining pressure on the internal friction angle and dilatancy angle (lower angles)
4.2 Medium size direct shear box Test Result and Discussion

4.2.1 Introduction

This section presents the results of tests performed in the Medium size direct shear box following the tests procedure presented in section 3.4.

4.2.2 Medium size direct shear box experiment results

General trends

Figure 4.6 shows the shear stress-horizontal displacement and vertical-horizontal displacement behaviors of the Bremanger sandstone rockfill subject to medium direct shear box tests. The general shear characteristics of Bremanger sandstone rockfill are summarized as follows:

❖ The stress-strain behavior of the Bremanger sandstone rockfill is nonlinear, stress dependent.

❖ An increase in normal stress is associated with an increase in shear strength.

❖ The stress-strain curves show a marked curvature and a distinct drop after failure.

❖ The strain at failure varies from 3.0 to 6.0%. The strain at failure does not increase significantly with the normal stress, which might be due to the low normal stress as well as the short normal stress range.

4.2.3 Secant friction angle and dilatancy angle

The secant friction angle at failure decreases with an increase in normal stress. The relationship between friction angle and normal stress is non-linear. The same as the secant friction angle, the dilatancy angle decreases as the normal stress increases (Figure 4.7). The secant friction angle varies from $75^\circ$ to $82^\circ$ and the dilatancy angle varies from $20^\circ$ to $29^\circ$. 
4.2.4 Stress-dilatancy

The stress ratio and dilatancy relations are shown in Figure 4.8. This type of graphical representation can be used to analyze the shearing processes (compression, expansion, failure, and softening/hardening) (Lee et al., 2009; Wood, 1990; Taylor, 1948). Starting from the commencement of shearing test both the stress ratio and dilatancy increase up to the peak strength. After the peak the stress ratio decreases significantly while the dilatancy reduces but remains high.
In all the tests both the stress ratio at failure and dilatancy at failure follow a similar trend; decrease with increasing normal stress. This can be explained by the fact that shearing under low normal load is resisted by interlocking of particles rather than crushing of particles and causes dilatancy. This leads to an increase in the friction angle.

![Dilatancy vs. Stress Ratio](image)

**Figure 4.8: Dilatancy and stress ratio**

**4.2.5 Effect of tilting of normal load on shear strength**

During medium shear box testing, measurements of the inclinations/tilting angle of the normal load were taken throughout the test. At failure a maximum tilt angle of 1.5° was recorded at a normal stress of 33KPa. The difference between the corrected and uncorrected normal stress is insignificant (<0.5%). Furthermore, the extra shear stress due to tilting is very low always less than 2.5% of the total shear stress (Appendix B).

**4.2.6 Particle breakage**

Generally, particle breakage did not occur as expected at such low normal stress. However, significant amount of dust were produced after each test. There was however no visual difference in the aggregates appearance.
4.2.7 Shear box alignment and restrain

In order to determine the effect of the wooden blocks (that were used to restrict the movement of the upper shear box) on the shear strength of the rockfill, vertical force transducers were introduced in between each wooden block and the steel frame. Two tests were performed with and without Teflon; Teflon was used to reduce the transmission of horizontal force between the upper box and steel frame via the wooden blocks. Figure 4.9 shows the measurements from the four transducers along with the measurements of the shear forces. During the test, the wooden blocks at the back were less strained than those at the front. Similar vertical forces were measured at the same location, both at the back and front positions. Similar horizontal forces were also measured. However, this was not observed during the test using Teflon. This could be due to imperfection in the sample preparation or the shear box alignment. The effect of the wooden blocks is not obvious, but it can be said the stress distribution in the shear box is not uniform.

![Vertical force vs horizontal displacement](image)

Figure 4.9: Normal forces at the four shear box corners (v1front, v2front, v3back and v4back) and the two horizontal forces (KN-1 and KN-2): a) without Teflon; b) with Teflon.

4.2.8 Repeatability of Medium Scale shear box tests

Repeatability of results using low normal stresses in medium direct shear box is low. Four tests were conducted on high density rock fill under a normal stress of 33KPa. All tests exhibited strain-softening behavior, with failure defined at peak strength. The stress strain curves of samples tested (Figure 4.10) vary specially at peak shear stress, within [+] or [-] 10.0 KPa. The secant friction angle φ' of the sample varies from 74.3° to
77.4°, with an average of 75.5° and a standard deviation of 1.2°. For the analysis the averaged friction angle is used. In case of horizontal-vertical displacement the difference is insignificant particularly prior to failure.

![Graphs showing shear box test results](image)

Figure 4. 10: Repeatability of medium shear box test results conducted under 33KPa. Top: stress-horizontal displacement, bottom: vertical-horizontal displacement

### 4.2.9 Rockfill shear strength model

The relationship between shear strength and stress level can be described by Mohr-Coulomb, power curve failure or other advanced models.

**The Mohr-Coulomb strength model**

The results of the medium-scale shear box tests are shown in Figure 4.11. The strength constants, \(c\) and \(\varphi\), derived from the approximate line in accordance with the Mohr-Coulomb failure criterion are indicated with and without cohesion.

The Mohr-Coulomb failure envelope does not adequately describe relationship between shear strength and normal stress. However, it can be used as a rough estimate for shear strength and friction angle for this rockfill.
Figure 4.11: Relation between normal stress and shear stress at failure based on the results of the medium-scale box shear tests.

**The power curve model**

In Figure 4.12, the power curve is fitted to the medium scale shear box test results. The $R^2$ increases from 0.9882 ($c \neq 0$) and 0.9349 ($c=0$) for the MC with and without cohesion respectively to 0.9969 for the power curve.

Figure 4.12: Relation between normal and shear stress using power curve strength model

The regression coefficient of the parabolic curve is higher than the one corresponding to the linear curve. It can be concluded the failure envelop is described better by the power curve at low stress range.


4.2.10 Tilt test results

The results of the tilt test are summarized in table 4.1. The angle at which the entire surfaces of the specimen began to collapse and the whole specimen started to collapse were the same.

Table 4. 1: Results of tilt tests

<table>
<thead>
<tr>
<th>Test-Number</th>
<th>Density (Mg/m3)</th>
<th>Normal load (KPa)</th>
<th>Static Angle of Repose $\varphi$ i $(^\circ)$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Situation(i)</td>
<td>Situation(ii)</td>
</tr>
<tr>
<td>1</td>
<td>1.71</td>
<td>3.36</td>
<td>36</td>
</tr>
</tbody>
</table>

The results were far lower than expected. It was checked whether overturning failure (direct toppling) in place of sliding had taken place. The angle of situation i ($36^\circ$) seems to correspond to toppling of the upper box by rotation around the bottom edge of the lower box. The angles of situations iii and iv ($51^\circ$) correspond to the angle at which toppling of the whole box by rotation around its bottom edge occurs.

Barton (2008), used a tilt apparatus with a base to height ratio of 2.5 (base: 5m and 2 x 1/2 box height= 2 m) for measuring a tilting angle as high as $83^\circ$. It should be noted that this angle is higher than the three toppling angles defined in section 3.3.5. This is not understood why. If the base to height ratio used by Barton is applied to a 0.4m thick tilt apparatus, the tilt apparatus should be at least 1m long.

4.2.11 Particle movement

In Figure 4.13, the relative and total particle displacements sustained by the coloured particles are plotted. The relative displacement represents the displacement of the colored particles with respect to the shear box walls.

The displacement of particles was higher at the middle of the shear box than in the upper box. The particles of the bottom shear box had little horizontal and downward displacements. Traces of particle movements were observed on the surfaces of the top box. No particle movements or scratches were observed on the face onto which the shear force is applied (face 1). Straight particle movement traces are found on the face of the top shear box (face 2) on the force transducer side; the face opposite to face 1. On the faces parallel to the shear displacement (face 3 and 4), curved traces are found (Figure 4.14).
Figure 4. 13: Particle positions after shearing (left), and corrected for box movement (right)

Figure 4. 14: Traces of particle movement found inside the upper shear box
Particle movements within the Medium size shear box are shown schematically in Figure 4.15. Simulations of direct shear tests on sand by Liu (2006) with a discrete element model have shown particle movements similar to those illustrated in Figure 4.15. Downward particle movement at the back and upward particle movements at the front of the box were observed, resulting in inclination of the normal load.

The thickness of the zone where particle movement is observed (Figure 4.13) is 25cm. which corresponds to less than ten times \( D_{\text{max}} \) (37.5mm) which is often cited as the shear band thickness in literature.

![Diagram of particle movements during shearing in Medium-scale direct shear](image)

**Figure 4. 15:** Schematic of particle movements during shearing in Medium-scale direct shear
4.3 Small size direct shear box Test Results and discussion

4.3.1 Introduction

This section presents the results of tests performed in the small size direct shear box following the test procedure presented in section 3.3. The results show the stress strain and shear-normal displacement of specimens of Bremanger sandstone rockfills when subjected to shear. The changes in the behavior of Bremanger sandstone rockfill specimens caused by the degree of compaction and shearing rate are analyzed. To ensure the repeatability of the results, some tests were repeated three times.

4.3.2 Small size direct shear box experiment results

General trends

Figure 4.16 presents the shear stress-horizontal displacement and vertical displacement-horizontal displacement behavior of the Bremanger sandstone rockfill subject to small direct shear box tests. The general shear characteristics of Bremanger sandstone rockfill are summarized as follows:

- The stress–strain behavior of the Bremanger sandstone rockfill is nonlinear and stress dependent.

- The stress-strain curves show a marked curvature and a distinct drop after failure in dense samples (HD1&HD2). Whereas, loose samples (ND1) show less curvature.

- All tests show post-peak softening behavior.

- An increase in normal stress is associated with an increase in shear strength.

- At low normal stresses (1.8kPa & 12kPa), the sample dilates without any measurable contraction at the start of the tests. At higher normal stresses (22kPa & 33kPa) first the sample contracts then dilates.
Figure 4.16: Shear stress-horizontal displacement and vertical displacement-horizontal displacement behavior of Bremanger sandstone rockfill tested using small size shear box.
4.3.3 Secant friction angle and dilatancy angle

The secant friction angle at failure decreases as the normal stress increases. The relationship between friction angle and normal stress is non-linear. In same as the friction angle, the dilatancy angle decreases as the normal stress increases (Figure 4.17). Dense samples and samples shear at higher rates have higher secant friction angle as well as dilatancy angle, while normal density samples and samples sheared at lower rates have the lowest friction angle and dilatancy angle.

Figure 4. 17: Corrected Normal stress vs. Secant friction angle (left) and Corrected normal stress vs. dilatancy angle at failure (right)

4.3.4 Strain at failure

The strain at failure varies from 2.0 to 6.0 %. In all the tests, axial strain at failure increases with normal stress. The shear strain at failure is higher for normal density samples than dense samples.

4.3.5 Stress–dilatancy

Figure 4.18 shows the stress ratio ($\tau/\sigma$) and dilatancy (dH/dD) relations.

- Volume expansion occurs in all samples. The degree of volume expansion and the stress ratio decreases as the normal stress increases.

- Normal density samples have lower dilatancy at failure than dense samples.

- Dense samples have higher magnitude of stress ratio and dilatancy than loose samples.
In theory, if the deformation of the aggregates would be homogeneous in the shear box, extrapolating the stress ratio vs. dilatancy curve to a zero dilatancy would give the stress ratio at the critical state. In practice, strain is localized during shearing and the critical state is reached in the shear band only. The total sample dilatancy corresponds to both the dilatancy of shear band and the dilatancy of material outside the shear band.

### 4.3.6 Factors affecting shear behavior

#### 4.3.6.1 Normal stress

The internal friction angle $\phi$ decreases as normal stress increases, which generally agrees with some of the trends discussed in (section 4.2.3). The internal friction angle is determined from Mohr-Coulomb linear failure envelopes constructed as best-fit lines. The high values of the friction angle at low normal load are believed to be related to the inter-particle contact forces that are well below the crushing strength of the Bremanger sandstone, and the ability of the interlocking particles to dilate under lower stress levels.
Table 4. 2: Results of small shear box test (c=0)

<table>
<thead>
<tr>
<th>Sample</th>
<th>Test No.</th>
<th>Density (Mg/m³)</th>
<th>Peak strength</th>
<th>Φ(deg)</th>
<th>𝑅²</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>(KPa)</td>
<td>τ(KPa)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>HD (1)</td>
<td>1</td>
<td>1.74</td>
<td>1.80</td>
<td>4.80</td>
<td>51.8</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1.73</td>
<td>12.0</td>
<td>19.14</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>1.73</td>
<td>22.0</td>
<td>31.53</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>1.73</td>
<td>33.0</td>
<td>47.11</td>
<td></td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>1.73</td>
<td>55.0</td>
<td>79.36</td>
<td></td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>1.73</td>
<td>114</td>
<td>147.90</td>
<td></td>
</tr>
<tr>
<td>ND (1)</td>
<td>1</td>
<td>1.64</td>
<td>1.8</td>
<td>2.969</td>
<td>44.8</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1.64</td>
<td>12.0</td>
<td>13.73</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>1.64</td>
<td>22.0</td>
<td>24.73</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>1.64</td>
<td>33.0</td>
<td>34.33</td>
<td></td>
</tr>
<tr>
<td>HD(2)</td>
<td>1</td>
<td>1.74</td>
<td>1.8</td>
<td>6.24</td>
<td>52.8</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>1.73</td>
<td>12</td>
<td>20.25</td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>1.73</td>
<td>22.0</td>
<td>34.17</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>1.73</td>
<td>33.0</td>
<td>42.10</td>
<td></td>
</tr>
</tbody>
</table>

4.3.6.2 Density

The compacted samples have a higher friction angle than normal density samples. The internal friction angle of the dense sample (1.73Mg/m³) is about 7° higher than that of the normal density sample (1640Kg/m³). The stress-strain curves of the high density show a marked curvature and a distinct drop after failure while that of the normal density show minimal curvature and less drop in the friction angle.

Furthermore, the denser specimens of Bremanger sandstone rockfills displayed larger dilation when compared with specimens prepared at a lower density. This can be explained by the dense material requiring more dilation to fail as particles have less room to move during shearing which cause a distinct drop in the friction angle after failure.
4.3.6.3 Strain rate

In order to determine the effect of shearing rates on shear strength of the Bremanger sandstone rockfill, two series of tests were done. First series of tests at a shearing rate of 2 mm/min were performed; and second series of tests at a shearing rate of 1 mm/min. The result shows an increase in internal friction angle of the rockfill is about 1° higher at 2 mm/min (Table 4.2). It appears that the shear strength of the Bremanger sandstone rockfill material is not noticeably affected by the deformation rate. A likely justification for the slight difference is that at higher shearing rates, less time is allowed for rearrangement of particles inside the shear box, which results in a greater friction angle of the material (Al-Mhaidib, 2005).

4.3.7 Repeatability of small direct shear tests

Repeatability of the testing method was evaluated by conducting three replicate tests. All tests exhibited a strain-softening behavior, with a clear peak strength. The stress-strain curves of samples tested are nearly identical (Figure 4.19), with the shear strength at a given normal stress (116 KPa) varying within (+/-) 3.0 kPa. The $\phi$ for the sample HD1 (High density 1) varies from 51.1° to 52.4°, with an average of 51.9° and a standard deviation of 0.71°. The dilatancy angle for the sample ranges from 7° to 10.2° with an average of 8.9° and a standard deviation of 1.6°. The friction angle obtained using small scale shear box testing is highly repeatable, but dilatancy shows a higher variability.

![Graph showing shear stress vs. horizontal displacement and vertical vs. horizontal displacement](image)

Figure 4.19: The stress-strain of three specimens under the same test condition (116 KPa).
**4.3.8 Particle breakage**

Figure 4.20 shows grading curves after a series of high load tests along with the initial grading curve. The percentage of particles retained in each sieve is determined by sieving the sample using a set of sieves (0.3 to 3.35 mm) before and after testing at initial and final stages of the test. Due to breakage or abrasion of particles, the percentage of the particles retained in large size sieves decreased and the percentage of particles retained in small size sieves increased. However, the difference between was very small. A likely explanation for low particle breakage is that the tests were done at normal loads. Which are not high enough to cause crushing for a strong material like Bremanger sandstone.

![Grain Size distribution curve](image)

Figure 4. 20: Sieve curves before and after test for small shear box.

**4.3.9 Rockfill shear strength**

**Mohr-Coulomb Model**

Mohr-Coulomb failure envelopes for the Bremanger sandstone rockfill tested using small scale shear box are shown in Figure 4.21. A Mohr-Coulomb failure envelope for each test was defined by linear least-squares regression for c≠0 and c=0. The regression coefficient $R^2$ of c≠0 is higher than c=0. The apparent cohesion in rockfill is not very important, therefore here only the latter is considered. Failure envelopes for the samples tested at low shear rate (1mm/min) at high and low density exhibited a high degree of linearity ($R^2 =0.994$ for both) whereas the test performed at high shearing rate has a lower regression coefficient ($R^2 =0.941$). Generally, the non-linearity of the failure at low normal stress is not captured well by this model.
The power curve model

In Figure 4.22, the power curve is fitted to the small scale shear box test results. The regression coefficients of these parabolic curves are higher than the ones corresponding to the linear curves. The non-linearity at low normal stress is better represented by this power curve envelope than Mohr coulomb envelope. The fitting result parameters are listed in Table 4.3.

Table 4.3: Shear strength parameters for Bremanger sandstone rockfill where $\tau_f = A(\sigma)^b$.

<table>
<thead>
<tr>
<th>Category Full Name</th>
<th>Category Symbol</th>
<th>A</th>
<th>b</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>High density (0.3mm&lt;P&lt;3.35mm)</td>
<td>HD-1</td>
<td>2.923</td>
<td>0.768</td>
<td>0.996</td>
</tr>
<tr>
<td>High density (0.3mm&lt;P&lt;3.35mm)</td>
<td>HD-2</td>
<td>4.148</td>
<td>0.650</td>
<td>0.998</td>
</tr>
<tr>
<td>Normal density (0.3mm&lt;P&lt;3.35mm)</td>
<td>ND-1</td>
<td>1.732</td>
<td>0.833</td>
<td>0.999</td>
</tr>
</tbody>
</table>
Summary of test conditions used in triaxial compression, medium scale shear box and small scale shear box tests on Bremanger sandstone rock fill are given in Table 4.3.
Table 4.3: Summary of testing

<table>
<thead>
<tr>
<th></th>
<th>Small scale shear box</th>
<th>Medium scale shear box</th>
<th>Triaxial Compression</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear box size (w<em>l</em>h) [mm]</td>
<td>100<em>100</em>40</td>
<td>500<em>500</em>400</td>
<td>600*300</td>
</tr>
<tr>
<td>Grain size [mm]</td>
<td>0.3&lt;P&lt;3.35</td>
<td>6.3&lt;P&lt;37.5</td>
<td>6.3&lt;P&lt;37.5</td>
</tr>
<tr>
<td>$D_{\text{max}}$ [mm]</td>
<td>3.35</td>
<td>37.5</td>
<td>37.5</td>
</tr>
<tr>
<td>$D_{50}$ [mm]</td>
<td>1.4</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>$C_u$ [-]</td>
<td>3.6</td>
<td>2.2</td>
<td>2.2</td>
</tr>
<tr>
<td>$H_{\text{spec}}/D_{\text{max}}$ [-]</td>
<td>12</td>
<td>10</td>
<td>8</td>
</tr>
<tr>
<td>$W/D_{\text{max}}$ [-]</td>
<td>30</td>
<td>13</td>
<td>8</td>
</tr>
<tr>
<td>Range of load [kPa]</td>
<td>1.8-33</td>
<td>3.8-33</td>
<td>7.5-60 ($\sigma_3$)</td>
</tr>
<tr>
<td>Strain rate [mm$^{-1}$]</td>
<td>0.01</td>
<td>0.02</td>
<td>0.003</td>
</tr>
<tr>
<td>Boundry condition</td>
<td>Combination of Type A&amp;C</td>
<td>Type C (but without Teflon inside the wall to reduce friction)</td>
<td>-</td>
</tr>
<tr>
<td>Density [g/cm$^3$]</td>
<td>1.72-1.73</td>
<td>1.72-1.73</td>
<td>1.72-1.73</td>
</tr>
<tr>
<td>Friction angle (c≠0) [$^\circ$]</td>
<td>50.5</td>
<td>74.9</td>
<td>49.9</td>
</tr>
<tr>
<td>Dilatancy angle</td>
<td>Stress [kPa]</td>
<td>Stress [kPa]</td>
<td>Confining stress [kPa] &amp; after converting to Normal stress [kPa]</td>
</tr>
<tr>
<td></td>
<td>Dilatancy [$^\circ$]</td>
<td>Dilatancy [$^\circ$]</td>
<td>Dilatancy [$^\circ$]</td>
</tr>
<tr>
<td>1.8</td>
<td>17.2</td>
<td>3.8</td>
<td>29.2</td>
</tr>
<tr>
<td>12</td>
<td>13.5</td>
<td>12</td>
<td>28.2</td>
</tr>
<tr>
<td>22</td>
<td>11.8</td>
<td>21.5</td>
<td>26</td>
</tr>
<tr>
<td>33</td>
<td>11</td>
<td>33.6</td>
<td>23</td>
</tr>
</tbody>
</table>
5 DISCUSSIONS

The results of medium and small scale shear tests are compared with the results of the triaxial tests done on the same material. Moreover, these results are compared with the direct shear tests results conducted on the same material but with different aggregate size, density and gradation by Sun (2010).

Table 5.1: Summary of used abbreviations

<table>
<thead>
<tr>
<th>Test</th>
<th>Grain size (mm)</th>
<th>Density (Mg/m³)</th>
<th>Category Symbol</th>
</tr>
</thead>
<tbody>
<tr>
<td>Triaxial compression</td>
<td>6.3&lt;P&lt;37.50</td>
<td>High density (1.72)</td>
<td>Triaxial</td>
</tr>
<tr>
<td>Medium size shear box</td>
<td>6.3&lt;P&lt;37.50</td>
<td>High density (1.72)</td>
<td>MHD</td>
</tr>
<tr>
<td>Small Size shear box</td>
<td>0.3&lt;P&lt;3.35</td>
<td>High density (1.72)</td>
<td>HD-1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>High density (1.72)</td>
<td>HD-2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Normal density</td>
<td>HD-2</td>
</tr>
</tbody>
</table>
<pre><code>                           |                 | (1.64)           |                 |
</code></pre>
<p>| small size shear box          | 1.18&lt;P&lt;3.35     | Normal density   | NS             |
|                 | (1.63)           |                 |
|                 | High density     | HS             |
|                 | (1.71)           |                 |
|                 | Mixture          | MS             |
|                 | (1.65)           |                 |
|                               | 3.35&lt;P&lt;6.30     | Normal density   | NB             |
|                 | (1.65)           |                 |
|                 | High density     | HB             |
|                 | (1.71)           |                 |
|                 | Mixture          | MB             |
|                 | (1.65)           |                 |
| Medium size shear box         | 31.5&lt;P&lt;80       | Normal density   | M3             |
|                 | (1.398)          |                 |
|                               | 30&lt;P&lt;50         | Normal density   | M1             |
|                 | (1.398)          |                 |
|                 | High density     | M2             |
|                 | (1.56)           |                 |</p>
5.1 Comparison of Triaxial Compression, Medium-Scale direct shear and Small-scale direct shear tests

5.1.1 General Trend

The stress-strain behavior of all tests is similar, which is nonlinear and stress dependent.

5.1.2 Secant friction angle and dilatancy angle at failure

The secant friction angles and dilatancy angles obtained from the triaxial compression tests are shown in Figure 5.1 along with the secant friction angles and dilatancy angles from the small scale shear box and medium scale shear box tests on the Bremanger sandstone rockfill. For the triaxial compression tests, the normal stress and shear stress on the failure plane (assumed to be at 45+Φ/2) were determined from Mohr’s circle so that the results from direct shear and triaxial compression could be compared.

In Figure 5.1, the secant friction angle which is obtained from the medium scale shear box tests is much higher than the angle obtained from triaxial compression and small scale shear box tests. The secant friction angle measured in triaxial compression test under the lowest confining pressures 7.5KPa & 15KPa (about 17kPa & 30KPa after converting into normal stress respectively) is greater than the angle derived from the small scale shear box tests. We recognize that at higher stress (>100KPa) both are equivalent or very close. The dilatancy angle measured in the medium size shear box is higher than that measured in the small size shear box, but lower than triaxial compression test.

This finding suggests that the conclusions made by Ghanbari 2008 and others, which indicate that Φ measured in triaxial compression and direct shear differs by less than ±4° do not apply in this study on Bremanger sandstone. The likely justification for high difference in secant friction angle between medium scale shear box and small scale shear box is the design of the apparatus. In the former the upper box is prevented from moving vertically or rotating, this was done by inserting wooden blocks between the upper box and the shear box frame. According to Shabuya classification (1997), it is of type C but without Teflon on the sides to reduce friction, where as the small shear box has little freedom to tilt and move vertically. The small shear box is combination of type A&C for detail refer section 2.3.
5.1.3 Internal friction angle

A comparison of internal friction angles obtained from the triaxial compression (TC), medium size shear box (MS-SB) tests and those obtained from the small size shear box (SS-SB) tests are shown in Figure 5.2. The friction angle measured in TC is higher than SS-SB by 2.2°, whereas the friction angle of MS-SB is 25° higher than SS-SB.
5.2 Comparison with Previous tests

5.2.1 Comparison Medium size box test results

The secant friction angles obtained in medium size shear box are compared with previous test results obtained using the same apparatus on coarser sample (31.5mm<P<80mm & 30mm<P<50mm) at lower density (1.4-1.56g/cm$^3$) (Figure 5.3). On one hand the secant friction angle measured in this study are expected to be higher as the tests were done on denser (1.72 g/cm$^3$) and better graded ($C_u$=2.2) samples. On the other hand the tests performed by the author were made on smaller particles ($D_{max}=37.5$mm) and one cannot conclude on the particle size effect from the literature study. The secant friction angles measured in this study are similar to those obtained in previous studies. The same trend is also observed in the dilatancy angle. Scale and boundary effects are thought to dominate the test results.

Figure 5.3: Corrected normal stress vs. the secant friction angle at failure (left) and Corrected normal stress vs. the dilatancy angle at failure (right)

5.2.2 Comparison Small Scale shear box tests

Both the secant friction angle and dilatancy angle obtained using small size shear box is on average around $10^0$ lower than in previous studies done using the same apparatus but on coarser aggregates having a lower particle strength and being less well graded. Note that unlike previous studies materials used in this study were crushed from coarser materials to get stronger particles. The difference in secant friction angle is not as expected. For instance if we compare HD1 and category HS, both have the same density (1.71g/cm$^3$), but different $D_{max}$ (HD1-3.35mm & category HS -6.3mm), different material
strength (category HS has a lower particle strength than HD1) and different gradation (HD1- 0.3-3.35mm & category HS- 3.35-6.3mm). The category HS has a higher secant friction angle and dilatancy angle than HD1 (Figure 5.4). The increase in friction angles as a function of grain size, low particle strength and narrow gradation is an unusual trend and can be explained by several reasons. Firstly, it might be because of the scale/boundary effects. Secondly, it could be due to the differences in relative densities.

![Corrected Normal Stress vs. the Secant Friction Angle at Failure](image)

Figure 5. 4: Corrected normal stress vs. the secant friction angle at failure
**5.3 Factors affecting shear behavior**

**5.3.1 Maximum particle size**

The internal friction angles of category HS, HB, HD-1, MHD, M2 and triaxial were compared because they have similar density (HS: 1.71 g/cm³; HB: 1.71 g/cm³; HD-1: 1.72 g/cm³; MHD: 1.71 g/cm³ M2: 1.56 g/cm³). Figure 5.6 shows that the internal friction angle increases with the maximum particle size in direct shear box testing. However, if the result obtained using triaxial compression testing is used instead of those obtained with medium size shear box testing and tests conducted under the same particle strength and having parallel grading are only considered, then the internal friction angle decreases with $D_{\text{max}}$. Taking into account the design of the medium size shear box the latter trend can be taken as representative for Bremanger sandstone rockfill.
5.3.2 Density

In all the tests the same trend was found: the higher the density, the higher the shear strength and dilatancy. The stress-strain curves of all the dense samples showed a marked curvature and a distinct drop after failure. This is not the case for the normal density samples.

Figure 5.6: Internal friction angles vs. Max. Size (the internal friction angles are obtained from the best linear fit of the test results under similar normal stresses)

5.4 Summary

Summary of triaxial compression, medium size shear box and small size shear box performed on Bremanger sandstone on dense and loose samples at low normal stress $\sigma$ range are given in Table 5.2.

Table 5.2: Summary of shear tests on Bremanger sandstone

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Without cohesion (C=0)</th>
<th>With cohesion (C≠0)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Friction Angle($^\circ$)</td>
<td>$R^2$</td>
</tr>
<tr>
<td>Triaxial</td>
<td>54</td>
<td>0.958</td>
</tr>
<tr>
<td>HD1</td>
<td>51.8</td>
<td>0.994</td>
</tr>
<tr>
<td>HD2</td>
<td>52.8</td>
<td>0.941</td>
</tr>
<tr>
<td>ND1</td>
<td>44.8</td>
<td>0.994</td>
</tr>
<tr>
<td>MHD</td>
<td>77.3</td>
<td>0.935</td>
</tr>
<tr>
<td></td>
<td>Without cohesion (C=0)</td>
<td></td>
</tr>
<tr>
<td>---</td>
<td>------------------------</td>
<td>---</td>
</tr>
<tr>
<td>NS</td>
<td>53.51</td>
<td>0.989</td>
</tr>
<tr>
<td>NB</td>
<td>60.93</td>
<td>0.955</td>
</tr>
<tr>
<td>HS</td>
<td>57.83</td>
<td>0.991</td>
</tr>
<tr>
<td>HB</td>
<td>62.54</td>
<td>0.953</td>
</tr>
<tr>
<td>MS</td>
<td>57.51</td>
<td>0.985</td>
</tr>
<tr>
<td>MB</td>
<td>58.36</td>
<td>0.885</td>
</tr>
<tr>
<td>M1</td>
<td>74.02</td>
<td>0.658</td>
</tr>
<tr>
<td>M2</td>
<td>79.1</td>
<td>0.766</td>
</tr>
<tr>
<td>M3</td>
<td>77.58</td>
<td>0.949</td>
</tr>
</tbody>
</table>
6 BASIC Friction ANGLE OF BREMANGER SADNSTONE

6.1 Introduction

Rock joint shear strength is one of the key properties used in the stability analysis and design of engineering structures in rock mass, e.g. slopes, tunnels and foundations (Hoek and Brown, 1980). Determinations of friction angles often expressed as basic, residual or peak friction angle are fundamental to the understanding of the shear strength of discontinuities in rock masses. Both angles of basic (μ_b) and residual (μ_r) friction angle represent minimum shear resistance. μ_b refers to smooth, planar surface in fresh rock and can be considered as a material constant. μ_r refers to the residual condition, which is attained after large shear displacement. If the natural joint surface is un-weathered, μ_r can be taken equal to μ_b. Methods for μ_b characterization include direct shear test or tilt tests on saw cut surface. Basic friction angle can be used as a conservative estimate of the friction angle along a discontinuity in a stability analysis except when the rock had been softened and altered or when the discontinuity had been polished by displacement. In the latter, residual friction angle is preferred.

In this study, the basic friction angle (Φ_b) is the main focus of the analysis. Φ_b is one of the basic parameter in Barton model in calculating the shear strength, (section 6.2.1). The basic friction angle is a measurement of the shear strength along an artificially planar diamond saw cut surface and is characteristic of the rock mineralogy and grain size. However, Bootsma (2010) finds out in his study that it was impossible to create two matching smooth surfaces by saw cutting due to oscillation of the saw as the Bremanger sandstone is a very strong rock. Barton (1971) recommended shearing tests on sandblasted samples. Small scale roughness due to sandblasting and sum of the friction angle of polished mineral surfaces is the basic friction angle (Bruce, 1978).

In this chapter, laboratory Golder shear box tests on sandblasted Bremanger rock blocks to determine basic friction angle are discussed.
6.2 Shear Strength of rocks

6.2.1 Shear Strength Criteria

This section reviews some available models that use the basic friction angle as a parameter in determining the shear strength. These models are discussed here in short.

Shear strength of planar surfaces

a- Mohr-Coulomb Criteria

In rock slope design the shear strength of sliding interfaces is often based on Coulomb’s model in which shear strength ($\tau$) is expressed as a function of cohesion ($c$), normal load ($\sigma$) and the friction angle ($\varphi$). It is assumed that the discontinuity surface is cemented and is absolutely planer. Coulomb criterion represents the relationship between the peak shear strength and normal stress by:

$$\tau = c + \sigma n \tan \varphi$$

Equation 6.1

The peak and residual shear strengths measured under different normal stresses results in two straight lines (Figure 6.1). The peak strength line has a slope of $\varphi_p$ and an intercept of $c$ on the shear strength axis. The residual strength line has a slope of $\varphi_r$.

Figure 6.1: Mohr coulomb criteria on rock strength
Shear strength of rough surfaces

b- Patton’s criteria
Patton showed on that shearing resistance consists of two components, a frictional resistance between two sawn surfaces of rock represented by $\Phi_b$, the basic friction angle, and a topographic component represented by $i$, the roughness angle along discontinuities.

$$\tau = \sigma_n \tan (\Phi_b + i)$$  
Equation 6.2

Where $\tau$ is joint shear strength, $\sigma_n$ is normal stress, $\Phi_b$ is basic friction angle, and $i$ is regular teeth inclination.

However, the equation (6.2) is only valid at low normal stress. At higher normal stress the teeth are sheared off, resulting in a shear strength behavior which is more closely related to the intact material strength than to the frictional characteristics of the surfaces (Figure 6.2).

![Figure 6.2: Patton’s experiment on the shear strength of saw-tooth specimens](image)

c- JRC model
After recognizing the inadequacy of the linear Coulomb and Patton models, Barton (1973) proposed an empirical criterion for the shear strength of rock joints.

$$\tau = \sigma_n \tan \{\Phi_b + \text{JRC \ Log 10 (JCS /} \sigma_n)\}$$  
Equation 6.3

Where $\tau$ is joint shear strength, $\Phi_b$ is basic friction angle, $\sigma_n$ is normal stress, JRC is the joint roughness coefficient, and JCS is the joint wall compressive strength. One main
factor of influence on shear strength of rock joints is the basic friction angle. Barton’s
criterion is only valid where joint walls are in rock-to-rock contact.

A list of basic friction angle values ($\Phi_b$) was compiled by Barton and Choubey (1977) is
shown (Table 6.1). Source publications are difficult to find, particularly those providing
the detailed rock descriptions (mineralogy and grain size). They belong to proceedings
of regional symposium published in the 1970’ies or are PhD theses not always in
English.

Table 6. 1: Basic friction angles of various unweathered rocks (Barton and Choubey,
1973)

<table>
<thead>
<tr>
<th>ROCK TYPE</th>
<th>BASIC FRICTION ANGLE ($\Phi_b$) (Degrees)</th>
<th>REFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Sedimentary Rocks</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sandstone</td>
<td>26 – 35</td>
<td>Patton, 1966</td>
</tr>
<tr>
<td>Sandstone</td>
<td>31 – 33</td>
<td>Krsmanovic, 1967</td>
</tr>
<tr>
<td>Sandstone</td>
<td>31 – 34</td>
<td>Coulson, 1972</td>
</tr>
<tr>
<td>Siltstone</td>
<td>31 – 33</td>
<td>Coulson, 1972</td>
</tr>
<tr>
<td>B. Igneous Rocks</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Basalt</td>
<td>35 – 38</td>
<td>Coulson, 1972</td>
</tr>
<tr>
<td>Granite (Fine)</td>
<td>31 – 35</td>
<td>Coulson, 1972</td>
</tr>
<tr>
<td>Granite (Coarse)</td>
<td>31 – 35</td>
<td>Coulson, 1972</td>
</tr>
<tr>
<td>Porphyry</td>
<td>31</td>
<td>Barton, 1971</td>
</tr>
<tr>
<td>Dolerite</td>
<td>36</td>
<td>Richards, 1975</td>
</tr>
<tr>
<td>C. Metamorphic Rocks</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gneiss</td>
<td>26 – 29</td>
<td>Coulson, 1972</td>
</tr>
<tr>
<td>Slate</td>
<td>25 – 30</td>
<td>Barton, 1971</td>
</tr>
</tbody>
</table>

6.3 Apparatus, Material used and Testing procedure

6.3.1 The Golder shear box apparatus

A Golder shear box apparatus was used for determining the basic friction angles using
sand blasted surfaces. The apparatus has an upper and a lower shear box, and the
sample is sheared by pushing the lower shear box horizontally. A normal load is applied
by means of a dead load system and therefore remains constant throughout the test.
Both applied shear force and shear displacement are monitored, as well as the vertical
displacement. The vertical displacement is measured at a single point (point 9) on the
level arm allowing a magnification of up to five times providing a relatively high degree of sensitivity. The setup is illustrated in Figure 6.3.

The Golder shear box tests have some limitations; the normal and shear load capabilities are limited and the shear displacements that can be accommodated are inadequate for residual strength determinations.

Figure 6. 3: A sketch of a laboratory Golder shear box (Bootsma, 2010)

6.3.2 Tested material and Sample preparation

Rock samples used in this study were prepared from S3 and S7, two samples tested by Bootsma. Their matching surfaces were first polished and, then textured by sandblasting. Sandblasting was operated manually on sandblast machine under the following conditions: time: ~1 min; distance between spray head and samples was roughly: 200mm; and spray angle: ~80-90°.
6.3.3 Test procedure

The test procedure follows as much as practical the ASTM D5607 standard practice. The tested surface area was 6.6cm. The shearing rate was about 1 mm/minute. In order to obtain the shear strength characteristics of the rock several direct shear tests (usually more than 2 tests) under different normal loads should be performed. Due to the difficulties in obtaining a sufficient number of identical samples, multiple tests were performed on the same sample. The specimen was sheared under five different normal loads (76 kPa, 153 kPa, 364 kPa, 677kPa and 1371 kPa) starting from low normal load.

6.3.4 Texture investigation using 3D pictures

In order to investigate the surface texture of the blocks 3D Leica pictures were taken and examined at its initial state (prior to sandblasting), after sandblasting and after shearing, at the center of each sample in an area of 2cm x 2cm. Both surfaces of S3 (S3-1 and S3-2) were analyzed. In addition, one image of original material with a texture supposed to be natural was taken and used for comparison, herein referred as original. The comparisons are made based on roughness parameters obtained by processing the image. The definition of the roughness parameters is given in Appendix C. All results were normalized with respect to the parameter of the “original” sample analyzed and plotted on histograms to identify any variation in the parameters value between prior to sandblasting and after sandblasting and after shearing. All the 3D
images parameters are plotted in a chart (Figure 6.5). From the charts it can be observed the following: the saw-cut surfaces (“Before”) parameters have lower values than the original sample, after sandblasting most values are higher than the polished and some parameters are higher than the original sample. After shearing most of the parameters have values lower than before shearing.

Furthermore, to make a simple comparison the most commonly used parameter in rock surface profile studies are used; the Pa, Pq, Pz and Pp (Joost, 2010) (Figure 6.6). These selected parameters follow the same trend as explained above. It is clear that the two faces (S3-1 and S3-2) have a different texture, S3-1 is rougher than S3-2. If more detailed statistical analysis is required then it is recommended to study the works by Reeves (1985).

Figure 6.5: Histograms showing the parameters prior to sandblasting and after sandblasting and after shearing. Results have been normalized with respect to the parameters of the “original” sample.
In Figure 6.7 both surfaces of S3 are compared at each stage (before sandblasting, after sandblasting and after shearing). Before sandblasting both surfaces (S3-1 and S3-2) have similar texture, after sandblasting the texture has improved (becomes rougher) in both surfaces, S1 have close similar texture as the original. After shearing S3-2 remains unaffected by the shearing, while the other surface S3-1 becomes smoother. This can be during shearing the area where measurement was taken may be was not in contact.
Figure 6. 7: Comparison of S3-1 and S3-2 after each stage, from left to right: before sandblasting, after sandblasting, after shearing.
6.3.5 Microscopic Study

Mineral analyses were done in previous studies. The mineral composition of the grains is as follows (van Tooren, 2010):

- Quartz – dominant mineral (70%)
- Feldspar – plagioclase, microcline, orthoclase
- Calcite
- Mica – muscovite, biotite (chlorite), chlorite
- Opaque minerals
- Lithic fragments – chert, quartzite, mylonite, volcanic rock

The damage caused due to the sandblasting is studied under microscope on thin sections. Four thin sections were prepared from each surface of the samples. Microstructures resulting from the sandblasting processes are deduced by comparing them with the structures found elsewhere in the specimen, in zones unaffected by the sandblast impact. Examination of cross sections on the sand blasted surfaces indicated that some minerals were eroded and damaged (Figure 6.8). It was observed that the eroding mechanism appeared to affect certain minerals probably calcite (Figure 6.8d). In certain sections of the surfaces very small new cracks were found to have been induced parallel to the surface perpendicular to the sandblasting. Conchoidal fracturing is the main feature observed in the quartz minerals (Figure 6.8a, b & c). The observed cracks under the microscope are generally within 0.1mm of the sandblasted surface. Larger part of the thin sections are displayed in Appendix D.

6.4 Results and Discussion

Shear stress against shear displacement and vertical displacement against shear displacement graphs are shown in Figure 6.9. The peak and residual shear strength (i.e. corrected for dilatancy) are estimated by plotting the best linear fit through points (pairs of normal stress-peak shear stress) (Figure 6.10) the cohesion and friction angle are determined.
Figure 6. 8: Microphotographs of sand-blasted surfaces (width of view 0.2mm) thin sections were prepared perpendicular to the sand blasted surface. The white arrows highlight damage (micro-cracking or erosion).

Figure 6. 9: Shear stress against shear displacement and vertical displacement against shear displacement.
Barton (1971) gave criteria to reject erroneous basic friction angle values. A basic friction angle value is not valid if:

- The shear stress-horizontal displacement curve has a clear peak related to surface interlocking,
- It is affected by oscillations caused by stick and slip shearing along too smooth surfaces, or
- It shows strain hardening resulting from the roughening of smooth discontinuity walls as shear displacement increases.

Based on these criteria, the curves obtained for normal stress up to 677 kPa are valid to determine the basic friction angle.

The shear box test results in a peak friction angle of $27.4^\circ$ and basic friction angle of $24.7^\circ$ based on residual strength values for sample S3. These values have been determined using normal stresses and shear that were corrected from dilatancy (Hencher et al, 1989). As the dilatancy measured during testing was minimum (less than $2.7^\circ$); the correction is not, anyway, very significant. Barton and Choubey (1977) reported that the basic friction angle for most smooth unweathered rock surfaces lies between $25^\circ$ and $35^\circ$. The low angle could be probably due to any of the following factors: i) failure to produce the natural texture of the Bremanger sandstone; firstly, selecting a surface representing the natural texture of the rock is subjective and
secondly, producing the desired texture is not easy with the current method; ii) damage caused by sandblasting on the grains might have weakened the surface or iii) the direct shear testing was undertaken using a five stage on a single sample. Any small-scale surface irregularities may have been sheared off by the initial stages of the tests, resulting in smoother surfaces and a lower basic friction angle.

6.5 Conclusions

The estimates produced by direct shear testing on sandblasted Bremanger sandstone resulted in friction angle that is higher than that obtained on polished saw cut samples and is lower than that obtained on fresh tensile cracks sheared up to their residual strength.
7 CONCLUSIONS AND RECOMMENDATIONS

In this study, different tests have been conducted on Bremanger Sandstone rockfill materials taken from the MV2 rock fill stock piles. The important tasks accomplished in the course of this research are briefly summarized below. In addition, conclusions together with future recommendations are presented.

7.1 Summary of Work Done

The strength parameter of the Bremanger rock fill governs the design of the runway for the large crane of Maasvlakte 2, the resistance to wave action of the cobble beach and the leveling of the sea water breaker foundation layers. In this thesis, tests have been done not to address the design requirements of Maasvlakte 2 but to have general understanding of the shear behavior of rockfills made of very strong rocks.

Experiments were conducted on the Bremanger sandstone rockfill to evaluate its shear strength as a function of different factors: normal stress, grain size, density and particle strength. All experiments were conducted on angular to sub angular rockfills and blocks. Several equipments were used to determine the shear strength, i.e. triaxial compression apparatus, medium-scale and small-scale shear boxes. The Golder shear box was used for determining the basic friction angle of the Bremanger sandstone on block specimens.

Large scale triaxial compression (600*300mm) tests were performed using vacuum pressure to generate confinement $\sigma_3$, at four confining pressure (7.5, 15, 30 and 60KPa) on a well graded 10-37.5mm size Bremanger sandstone rockfills at high density. Shear strength tests were also performed on the same material (size, gradation & density) at normal stresses of 3.7, 12, 22 & 33KPa using a medium scale shear box (500*500*400mm). In addition, to understand the movements of particles during shearing colored grains were systematically placed inside the medium scale shear box. Finally, the results of medium scale shear box were compared with the results of triaxial compression test having been carried out on the same material, as well as with the result from small scale shear box.
In order to determine the effect of grain size, scale and boundary effect, shear strength tests were performed on a parallel scaled down material sized 0.3-3.35mm using small scale shear box (100*100*40mm). The small aggregates were found to have lower crushing strength and are believed to come from the edges of bigger blocks, which have been damaged due to collision between blocks during transportation. A study done using CT-scanned images attests this fact. There were cracks through grains. Therefore, to have consistency in the material used, in the medium scale tests and the small scale shear box tests bigger aggregates were crushed till the right material is obtained in shape, strength and roughness, etc.

Finally, Golder shear box was used to determine the basic friction angle on sandblasted Bremanger sandstone. In the literature, rockfill strength is explained by combined effect of basic friction angle and dilatancy angle. The surface texture before and after sandblasting as well as after shearing were studied on images taken using a 3D-Leica microscope. Also, the damage due to the sandblasting was studied using microscope on thin sections prepared perpendicular to the sandblasted surface.

7.2 Observations and Conclusions

In this section, the major conclusions that can be drawn from this work are listed:

- The Bremanger sandstone rockfill showed mixed trends (dilation and contraction) in their volume change behavior, depending on the normal stress. At low normal stress dilates and at higher normal stress (>60kPa) first compresses and then dilates.

- The increase of the normal stress reduce the peak friction angle (in decreasing rates) and the dilation angle (Fig. 4.5 & Fig. 4.7 )

- In each of the three testing apparatus, the friction angle increases as density increases. The high density samples have peak values from which the shear stress reduces with further displacement. For the normal density samples this is not observed.

- The results of the medium shear box, small shear box and triaxial compression tests all conducted under low normal stress were compared. The medium shear
box yielded higher shear strength and peak friction angle, and small shear box on small aggregates yielded nearly the same peak friction angle as the triaxial compression tests on large aggregates at higher stress (>60KPa).

- Based on the results of small scale shear test series of the shear test, conclusions about effects of shearing rate and compaction are made. The angle of internal friction of well compacted specimen is about 5 to 7° less than normal density specimen whereas, the shearing rate does not have significant effect on the shear strength.

- The failure envelop is described better by the power curve at low stress range for all tests performed using different apparatus.

- Grain movements were studied using colored grains systematically placed inside medium scale shear box and the scratches on the sides of the medium scale shear box. The displacement of particles was higher at the middle of the shear box than in the upper box. The particles at the bottom shear box had very little horizontal and downward displacements. The variation in displacement can be due to the boundary condition imposed. Where there is a non fixed loading plate on top of the upper shear box and aggregates can move up in the upper box. In the lower box, the aggregates, initially compacted to their maximum density seem to be locked and cannot move.

- The stress distribution in the medium shear box is not uniform, high at the front and low at the back of the shear box.

- Results showed that the tests were less repeatable in medium scale than small scale shear test.

- During medium size shear box testing the effect of inclination of the dead weight on the shear strength is negligible at peak strength. The difference between the corrected and the uncorrected normal stress was less than 0.5 % at failure.

- Basic friction angle obtained using sandblasted surfaces of Bremanger sandstone corresponds to the lower bound value recommended by Barton (1973) for sandstones. This low value could be due to the damage caused on the grains
during sandblasting. Tests on fresh tensile cracks sheared several times far beyond their peak strengths lead to a much higher friction angle. It is not clear whether the Barton’s model for rock fill can be used for the Bremanger sandstone using such a high basic friction angle. Barton estimated the contribution of aggregate roughness and crushing using a trial and error approach assuming a much lower basic friction angle.

- During triaxial sample preparation in the split mould, the only stress is due to its own weight and the weight of the top cap. For a sample height of 0.6m and diameter of 0.3m, the vertical stress $\sigma_v'$ at the bottom equals to 10KPa. The vertical stress is not uniform; it increases linearly from top to bottom of the sample. There is a horizontal stress $\sigma_h'$ of 2KPa, which is significant in comparison to the used low confinements (7.5 & 15KPa). Once the split mould is removed, it is considered that this horizontal stress vanishes. The vertical stress still acts on the bottom of the sample which is not negligible in comparison to the applied confinement.
7.3 Recommendations for Future Work

Although this study has provided valuable information on varied aspects of Bremanger sandstone rockfill shear strength behavior under low stress, it is felt that the aspects discussed in the following section require further investigation.

- In this study powder was produced after each shear and triaxial tests however, aggregate roughness was not visually different before and after testing. In future studies it is recommended to investigate if re-testing of the sample has an effect on the shear strength of Bremanger sandstone rockfill.

- The boundary conditions applied in the medium scale shear box are not ideal, further investigation is needed to modify the apparatus. This can be lining the inside wall box with Teflon or grease in order to reduce the friction between the material and the wall and rollers between the horizontal force transducer and the upper shear box. Furthermore, the effect of wooden blocks on the friction angle measured in medium size shear box is unknown rollers can be placed between the frame and the vertical transducers in order to reduce the effect.

- In future studies, the relative density of the packing should be used instead of density to make comparison. This would require knowing the maximum and minimum porosity.

- For conservative design the right friction angle has to be selected. For stability and bearing capacity problems then the residual friction angles can be used provided they are reached for an admissible displacement. For excavatability/earth movement the peak friction angle has to be considered. It should be noted this angle is very high for the Bremanger rockfill (above 55°) at low normal stress (less than 20kPa).

- It is suggested that further laboratory tilt tests are performed to characterize the roughness of Bremanger sandstone rockfill, using different tilt box dimensions so that sliding occurs before toppling.

- Fluctuations of shear stress are routinely observed during medium size shear box tests. Li and Aydin (2010) have showed that, valuable information can be
extracted from fluctuations of the shear stress. It is therefore suggested to make
use of this information in the future.
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Newsletter_Maasvlakte_2 (may, 2010)


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APPENDIX

Appendix A: Triaxial Compression test results
Figure A-1: $\sigma_1$, $\sigma_3$ and $\sigma_1 + \sigma_3$ against time

Figure A-2: Internal friction angle and cohesion
Figure A-3: Dilatancy angle

Figure A-4: stress path during drained tests.
Axial and Radial measurements [60KPa]

<table>
<thead>
<tr>
<th>Time [sec]</th>
<th>Displacement [mm]</th>
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</thead>
<tbody>
<tr>
<td>500</td>
<td>0</td>
</tr>
<tr>
<td>1000</td>
<td>10</td>
</tr>
</tbody>
</table>

Displacement measurements for test conducted under 60kPa.

Figure A-5: Displacement measurements for test conducted under 60kPa.
APPENDIX B: The Effect of Inclination / Tilt of The Normal Load on The Medium Size shear Box Test Results

To examine the effect of the inclination, during shearing of the steel on which the dead weight is applied, on the test results, tilt of the steel plate was measured using field compass (Figure B-1). It was seen that the maximum inclination of normal load was less than $2^0$ at maximum shear stress.

![Figure B-1. Tilt measurements using field compass during medium box shear test. The upper photo has been taken at the end of the test, after 10cm horizontal displacement.](image-url)
Forces acting at some angle from the coordinate axes consisting of the vertical (y) and horizontal (x) axes can be resolved into mutually perpendicular forces called **components** \( F_x \) and \( F_y \):

\[
F_x = F \sin \theta_y \quad F_y = F \cos \theta_y \quad F = \sqrt{F_x^2 + F_y^2}
\]

\[
\tan \theta = \frac{F_x}{F_y}
\]

![Figure B-2: Forces components](image)

At failure a maximum tilt angle of \( 1.5^\circ \) was recorded during medium shear box test at a normal stress of 33KPa. The difference between the corrected and uncorrected normal stress is < 0.5% and the extra shear stress due to tilt is 2.6%.

\[
\tau_{\text{Corrected}} = \tau \times \sin \theta
\]

\[
\sigma_{N(\text{corrected})} = \sigma_N \times \cos \theta
\]

|--------------|--------------|----------|--------------------|-----------------------------|----------------------|--------------------------------------|
APPENDIX C: Rock surface profile parameters

Several parameters are used to quantify a rock surface profile (Figure C-1). The most widely used parameters to characterize the roughness of a profile are Pa, Pp, Pz, Pq and Pv.

<table>
<thead>
<tr>
<th>Sign</th>
<th>Value</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>RS</td>
<td>1.0493</td>
<td>Ratio of true area to projected area</td>
</tr>
<tr>
<td>TS</td>
<td>105.5mm²</td>
<td>True area</td>
</tr>
<tr>
<td>PS</td>
<td>100.6mm²</td>
<td>Projected area</td>
</tr>
<tr>
<td>XRL</td>
<td>1.03752</td>
<td>Medium ratio of true length to projected length for horizontal profiles</td>
</tr>
<tr>
<td>YRL</td>
<td>1.0121</td>
<td>Medium ratio of true length to projected length for vertical profiles</td>
</tr>
<tr>
<td>DS</td>
<td>2.00515</td>
<td>Fractal dimension of selected area</td>
</tr>
<tr>
<td>Sa</td>
<td>0.521 mm</td>
<td>Average height of selected area</td>
</tr>
<tr>
<td>Sq</td>
<td>0.604 mm</td>
<td>Root-Mean-Square height of selected area</td>
</tr>
<tr>
<td>Sp</td>
<td>1.207 mm</td>
<td>Maximum peak height of selected area</td>
</tr>
<tr>
<td>Sv</td>
<td>1.065 mm</td>
<td>Maximum valley depth of selected area</td>
</tr>
<tr>
<td>St</td>
<td>2.272 mm</td>
<td>Maximum height of selected area (SP + SV)</td>
</tr>
<tr>
<td>Ssk</td>
<td>0.158613</td>
<td>Skewness of selected area</td>
</tr>
<tr>
<td>Sku</td>
<td>1.83836</td>
<td>Kurtosis of selected area</td>
</tr>
<tr>
<td>Sz</td>
<td>2.228 mm</td>
<td>Ten point height of selected area</td>
</tr>
<tr>
<td>Pa</td>
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<td>Mean average height of primary profiles</td>
</tr>
<tr>
<td>Pq</td>
<td>0.046 mm</td>
<td>Mean Root-Mean-Square height of primary profiles</td>
</tr>
<tr>
<td>Pz</td>
<td>0.206 mm</td>
<td>Mean Maximum height of primary profiles</td>
</tr>
<tr>
<td>Pp</td>
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</tr>
<tr>
<td>Pv</td>
<td>0.095 mm</td>
<td>Mean Maximum valley depth of primary profiles</td>
</tr>
<tr>
<td>Pc</td>
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<td>Average mean height of profile irregularities of primary profiles</td>
</tr>
<tr>
<td>PSm</td>
<td>2.704 mm</td>
<td>Average mean spacing of profile irregularities of primary profiles</td>
</tr>
<tr>
<td>Ps</td>
<td>0.140954</td>
<td>Mean Skewness of primary profiles</td>
</tr>
<tr>
<td>Pku</td>
<td>2.5999</td>
<td>Mean Kurtosis of primary profiles</td>
</tr>
<tr>
<td>Pdq</td>
<td>0.180523</td>
<td>Mean Root-Mean-Square slope of primary profiles</td>
</tr>
<tr>
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<td>Average height of mean primary profile</td>
</tr>
<tr>
<td>Pq_m</td>
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<td>Root-Mean-Square height of mean primary profile</td>
</tr>
<tr>
<td>Pz_m</td>
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<tr>
<td>PC_m</td>
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</tr>
<tr>
<td>PSm_m</td>
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<td>Mean spacing of profile irregularities of mean primary profile</td>
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<td>Ps_m</td>
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<td>Kurtosis of mean primary profile</td>
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<tr>
<td>Pku_m</td>
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<tr>
<td>Pdq_m</td>
<td>0.0503149</td>
<td>Root-Mean-Square slope of mean primary profile</td>
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</table>

Figure C-1. List of profile roughness parameters determined using the image processing software of the LEICA 3D microscope.
APPENDIX D: Examination of Microphotographs on sand blasted surfaces

Four thin sections were prepared from each sand blasted surface in the direction perpendicular to the sand blasted surfaces. Microstructures resulting from the sandblasting processes are deduced by comparing with the structures elsewhere in the specimen, in zones unaffected by the sandblast impact. The full images are shown below. Then width of view is 0.3mm and the sandblasted surface is on the right side of the images. To highlight microcracks zooms of these images are presented in the section 6.3.5.

Figure D-1. S3.1, The white arrow highlights a crack parallel to the sandblasted surface in a quartz grain
Figure D-2. S3-2. The white arrows highlight curved cracks in a quartz grain.

Figure D-3. S7-1. The white arrow highlights cracks parallel and perpendicular to the sandblasted surface in a quartz grain.
Figure D-4. S7-2. The white arrows highlight possible erosion of a calcite grain.

**Scale (each unit is equal to 0.01mm)**

Figure D-5. Scale of figures D1-D4.