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Foreword

This project-report serves as a thesis for completing the M.Sc. program in Resource Engineering at Department of Applied Earth Sciences, Delft University of Technology. The research was carried out within the context of a collaboration between IHC Holland (The Netherlands) and De Beers Marine (Pty) Ltd. (South Africa). Practically the project fell under the High Rate Mining Tool Development Program of De Beers Marine (Pty) Ltd. As such, the project, carried out from October 2002 to July 2003, was partly executed at testing facilities of De Beers Marine (Pty) Ltd. in Stellenbosch, South Africa.

A large number of people have contributed to the successful completion of this thesis project. To begin with, I thank the members of the M.Sc. graduation committee: Mrs. D.J.M. Ngan-Tillard, Mr. J.J. de Ruiter and Mr. G.L.M. van der Schrieck from TU Delft and Mr. H. van Muijen and Mr. P.M. Vercruijsse from IHC Holland for their guidance throughout the project. I sincerely thank Mr. M.L. Heyns, Mrs. M. Kruse and Mr. A.C. Labuschagne, and all others at De Beers Marine (Pty) Ltd. not called by name, for their devoted guidance and assistance during the practical experimentation in South Africa.

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Finally, I thank GOD for sending me a cousin R. Gangaram-Panday and his family and an aunt G. Ramdin without whose support my stay and studies in the Netherlands, and thus this thesis project, would not have been possible at all.
De Beers Marine (Pty) Ltd.
High Rate Mining Tool Development
Dealing with clay when mining diamonds offshore

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Summary

De Beers Marine is currently mining diamonds offshore near the Namibian coast (at water depths of 115 - 140 meters), using a vertical batch drill mining system ("Wirth drill"). Previously also continuous seabed crawlers have been used, but this method was discontinued because of unsatisfactory operational performance and recovery of the diamonds. New crawler-type mining tools (latest tool is the so-called "Gravel Wheel") are being developed due to their higher mining rate potential compared to the Wirth drill.

On 1/3 scale different versions of the Gravel Wheel had been thoroughly tested on imitated seabeds of gravel and initial tests had been done in clay. Since together with the gravel on average 5 to 10 cm (up to 30 cm) of underlying clay footwall may have to be loosened and removed more thorough tests have to be done in clay. The first reason to study the cutting into clay is the irregular gravel-footwall interface, which makes it nearly impossible to stay right at this interface all the time. Secondly, 100% of the diamonds have to be removed since the likelihood of diamonds occurring at the gravel-footwall interface is high. With the Gravel Wheel currently being tested, the focus of this study is to mine both gravel and clay with one tool in one step.

The main objective of this study is to define and improve the clay handling ability of the current Gravel Wheel model MKIII through scaled testing. The clay handling ability is specified as the ability to remain 100% clear from blockages and the ability to penetrate the clay footwall for 50 mm (full scale) whilst mining gravel. To reach to goals set, sub-objectives set are to establish a proper theoretical basis for the execution of scaled tests in clay, as well as derive basic design guidelines from experience in other related industries in order to evaluate and improve the Gravel Wheels' clay handling ability.

In the attempt to achieve similarity between model and full scale, three scenarios for scaled testing are identified. Although all three show scale effects, the most favourable scenario is selected to perform scaled tests. The main drawback of this scenario is that the clays' shear strength is not scaled back correctly in ratio to the force to mould the cut clay lumps (drag force). The consequence is that the test conditions on model scale are much tougher than on full scale. It is argued that if this scenario is used in scaled testing and if testing is successful, the tool will definitely work on full scale.

From experience in dredging several design guidelines have been derived to evaluate a tools' clay cutting ability and an initial evaluation of the Gravel Wheel has been made. The Gravel Wheel scores negative on the risk of bulldozing, openness of the tool and the converging shape of the suction duct and these features are expected to be most critical in the Gravel Wheels' design. Based on this analysis the scoop grizzly has been modified to reduce the risk of bulldozing.

The scaled tests (continued on 1/3 scale) prove that the maximum cut depth at which the tool remains free from blockage increases with increased rotational speed and modified scoop design. On model scale a 100 mm cut in clay has been achieved with a rotational speed of 20 RPM and a modified scoop design. Tests show that while mining gravel, a 20 mm and 50 mm cut in clay can be achieved at forward mining speeds of 2.5 and 1.7 m/min respectively. Further modifications to the scoop are made to reduce the obstructing side effect of the grizzly bars. The test results confirm that the risk of bulldozing and the converging shape of the suction duct are the most critical features in the Gravel Wheels design. The obstructing side effect of the grizzly bars has also been recognized as problematic.

It is recommended that more shear strength testing is done on offshore footwall clays to get more insight in the actual clay footwall shear strengths since currently only limited shear strength data of the clay footwall is available. Further more tests in clay and gravel should be performed to find the optimum operational parameters (e.g. forward mining speed) at different set cut depths: initial tests in clay and gravel have proven valuable to study the clay and gravel interaction and initial trends have been observed. The necessity to achieve a specific cut depth in clay depends all on the degree of undulation and the accuracy with which the clay-gravel interface can be followed. It is therefore finally recommended to further study these factors in order to better define the clay cut depth requirement of any mining tool.
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1. Introduction

1.1 Background

De Beers Marine is currently mining diamonds offshore near the Namibian coast (at water depths of 115 - 140 meters) using a vertical batch drill mining system ("Wirth drill") as shown in Figure 1.1. Previously also continuous seabed crawlers have been used, but this method was discontinued because of unsatisfactory operational performance and recovery of the diamonds. New crawler-type mining tools (latest tool is the so-called "Gravel Wheel") are being developed due to their higher mining rate potential compared to the Wirth drill. Figure 1.2 gives an overview of such a crawler type mining system.

The mining area near the Namibian coast is known as the "Atlantic 1 mining area" and its location is shown in Figure 1.3. A typical seabed in this area is as follows:
- 0.1 - 1 m overburden (usually smaller than 0.5 meters)
- 0.1 - 0.6 m gravel
- Expected 0.05 - 0.1 m footwall clay (up to 0.3 m) to be cut to recover all diamonds

On 1/3 model scale different tools have been thoroughly tested on imitated seabeds of gravel and initial tests have been done in clay. In the near future more tests have to be done incorporating clay, since together with the gravel on average 5 to 10 cm (up to 30 cm) of underlying footwall may have to be loosened and removed. The first reason for cutting clay is the undulation (irregular gravel-footwall interface), which makes it nearly impossible to stay right at the interface all the time. Secondly, 100% of the diamonds have to be removed since the likelihood of diamonds occurring at the gravel-footwall interface is high.

Theoretically 3 mining concepts are possible:
1. Mining in one single step (one tool)
2. Mining in two steps (two separate tools to separately mine clay and gravel)
3. Mining in one multi-phase step (same concept as in point two but the two tools combined in one tool)

If only gravel of 0.1 to 0.6 m was to be excavated, mining in one step is the solution on hand. Since gravel and clay are two extremes on the scale of material types and since their behaviour is also extremely different, mining both in one step may not be easy. It can be asked what type of clay, what thickness of clay or what ratio of gravel to clay thickness will start to give serious problems. Based on the currently expected 5 cm of clay to be cut and the necessity for a simple and cost effective system, mining in one step is preferred and will be studied first. With the Gravel Wheel currently being tested, the focus of this study is then to mine both gravel and clay in one single step. The other concepts will not be studied further.
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Figure 1.1 Vertical batch drill mining system: the Wirth drill.

Figure 1.2: Horizontal continuous crawler type mining system.
Figure 1.3: Location map of the Atlantic 1 mining area.
1.2 Concept of the Gravel Wheel

See Figure 1.4 for an overview of the the Gravel Wheel version MkIII. It is in simple terms a rotating drum with openings to hydraulically collect gravel and present it to the transport system. The Gravel Wheel has a rotating set of 5 ducts over a stationary inlet port. This serves to concentrate suction energy only over the quadrant in contact with the gravel to be mined. The suction interface of the tool has suction channels protected by non-blinding grizzlies. This is to ensure the suction interface is against the gravel bed to aid material collection. The tool rotates in the vertical plane and in an undercutting mode in order for the tool to penetrate and agitate the material to the required cut depth.

Appendices

Appendix 1 presents a more detailed description of the design intent and principle of operation of the Gravel Wheel.

Figure 1.4: Overview of the Gravel Wheel version MkIII.
1.3 Objectives

See Table 1.1 for the Gravel Wheel’s system specification as specified by De Beers Marine. The clay handling ability is specified as:

1. The ability to penetrate the clay footwall for 50 mm (full scale) whilst mining gravel and
2. The ability to remain 100% clear from blockages

<table>
<thead>
<tr>
<th>Requirement</th>
<th>Full Scale Specification</th>
<th>Third Scale Specification</th>
<th>Scale Factors</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Area Coverage Rate</td>
<td>5000 m³/day</td>
<td>962 m³/day</td>
<td>3.3β</td>
</tr>
<tr>
<td>2 Volumetric Mining Rate</td>
<td>216 m³/hr</td>
<td>13.9 m³/hr</td>
<td>3.3β</td>
</tr>
<tr>
<td>3 Recovery Efficiency</td>
<td>100%</td>
<td>100%</td>
<td>1</td>
</tr>
<tr>
<td>4 Penetration (Cut Depth)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4a Average gravel thickness</td>
<td>0.52 m</td>
<td>170 mm</td>
<td>3</td>
</tr>
<tr>
<td>4b Maximum gravel thickness</td>
<td>1.2 m</td>
<td>400 mm</td>
<td>3</td>
</tr>
<tr>
<td>5 Oversize Handling</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5a Remove cobbles &amp; boulders from mining lane</td>
<td>+180mm –560mm</td>
<td>+60mm –170mm</td>
<td>3</td>
</tr>
<tr>
<td>5b Remove slabs from mining lane</td>
<td>1m to 3m (100mm thick)</td>
<td>300mm to 1m (30mm thick)</td>
<td>3</td>
</tr>
<tr>
<td>6 Clay Handling</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6a Penetrate clay</td>
<td>50mm</td>
<td>16mm</td>
<td>3</td>
</tr>
<tr>
<td>6b Tool remain clear from blockages</td>
<td>100%</td>
<td>100%</td>
<td>1</td>
</tr>
<tr>
<td>7 Blockage prevention</td>
<td>100% free from jams &amp; blockages</td>
<td>100% free from jams &amp; blockages</td>
<td>1</td>
</tr>
<tr>
<td>8 Overlap</td>
<td>30% (minimise)</td>
<td>30%</td>
<td>1</td>
</tr>
<tr>
<td>9 Cycle Times</td>
<td>20-30% of time</td>
<td>for return &amp; re-position to start new mining lane</td>
<td></td>
</tr>
<tr>
<td>10 Availability</td>
<td>20 Hours per Day</td>
<td>83% mechanical availability including maintenance</td>
<td></td>
</tr>
</tbody>
</table>

Table 1.1: Gravel Wheel’s system specification.

The main objective of this study will be to define and possibly improve the clay handling ability of the current Gravel Wheel model MkIII through scaled testing. This includes the establishment of a proper theoretical basis for the execution of scaled tests in clay, as well as the understanding (analysis) of the clay footwall – Gravel Wheel interaction process. Further, basic design guidelines will be derived from experience in other related industries in order to evaluate and improve the Gravel Wheel’s design. Finally a translation of the model scale results to full scale conditions is required.
The objective falls apart in 3 main tasks:
1. Investigate the possibility to simulate clay (as found in the footwall) in scaled down tests. How are prevailing clay conditions as met by De Beers to be brought to test scale?
2. Define basic design guidelines for dealing with clay. How can operational parameters and tool design be adjusted to improve the clay handling ability?
3. Design test plan, perform tests and analyse test results to meet the objective set.

The tasks are performed in 3 phases and formulated as follows:

**Phase I:**
1. Investigate clay characteristics and properties under the specific offshore conditions (project constrains) as met by De Beers Marine:
   - Investigation of the local geology (especially the clay footwall)
   - Definition of major clay types and footwall conditions as met in the Atlantic 1 mining area
   - Investigation of the geotechnical characteristics of defined clay types
2. Investigate adhesion theory and clay cutting theory.
3. Evaluate possibilities of testing clay in scaled down tests to study the effect of clay on Gravel Wheel. What clay types have to be used in the lab to simulate a prototype? Give attention to scale laws (and if necessary, scale effects): How to scale up measured performance to full scale (production scale) again?

**Phase II:**
4. Define basic design guidelines for dealing with clay from experience in related industries listed below:
   1. Dredging
   2. Slurry shield tunnelling: Tunnel Boring Machines
   3. Offshore ploughing and pipe laying

**Phase III:**
5. Formulate experimental test program & perform tests:
   A. Soil lab tests
      Tests on clays to identify suitability for scaled tests
   B. Scaled laboratory tests with Gravel Wheel
6. Analysis of test results
7. Conclusions and recommendations:
   - Give guidelines on how to scale clay for testing the Gravel Wheel on 1/3 scale.
   - Give guidelines for equipment-design and constrains of process parameters (operational) when dealing with clay.
   - Give solutions for identified effects (e.g. recommendations on design adjustments of Gravel Wheel).
   - Give a translation of the model scale results to full scale conditions.

An overview of the tasks is given in Figure 1.5.
Figure 1.5: Schematic overview of goal and tasks.
This report is structured as follows. Chapter 2 to 6 provide the basis for the scale modelling analysis in Chapter 7. Chapter 2 and 3 deal with the geology and geotechnical properties of the footwall clay and test clays used. This in order to define the types of clay that can be encountered by the Gravel Wheel and also to define the test clay types as used in the scaled tests.

The adhesion theory and the cutting theory of clay are investigated in Chapter 5 and Chapter 6 respectively, since the adhesion and the failure mode of clay are recognized as important influencing factors in cutting of clay. Chapter 7 introduces first the scaling theory and then investigates the possibility of scaling clay and performing scaled tests in clay to set the stage for correct scaled testing. Chapter 8 covers phase II of the project resulting in the definition of design guidelines to deal with clay and the evaluation of the Gravel Wheel based on these design guidelines.

Chapter 9 presents and analyses the results of the planned scaled tests performed to study the effect the clay footwall has on the Gravel Wheel's prototype operation. Conclusions and recommendations are finally covered in Chapter 10.
2. Geology of the deposit in the Atlantic 1 area

2.1 Geological deposit characteristics

A typical sequence (stratigraphic horizons), from top to bottom, of the deposit sediments is as follows (Burger, 2002):

1. Overburden (clay/silt); usually less than 0.5 meters (range 0-5 meters)
2. Shell layer; up to 0.5 meters thick
3. Slabs (mainly sandstones) & boulders
4. Gravel (diamondiferous); between 0.1 and 0.6 meters thick (average 0.52 m);
5. Footwall (clay, sand or cemented); depending on undulation, 5 to 10 cm, sometimes even 30 cm, of the footwall may have to be excavated during current mining operation to ensure complete extraction of the diamonds (Appel, 2000). Much of the heavy mineral concentration is found on the footwall surface and so it is desirable to “clean up this interface, resulting in detachment and entrainment of pieces of footwall in the system” (Pether, 2002).

![Geological sequence diagram](image)

**Figure 2.1:** Typical geological sequence.

The interplay between the characteristics of the different layers in the succession, along with the morphology and structure, dictates the mineability. Based on this and other criteria mineralization zones have been identified which qualify for high-rate mining. This will not be further dealt with in this report.

2.2 The footwall

Both sand and clay footwall are present in the Atlantic 1 mining area.

Sand footwall is present as a range from sand to semi-consolidated sand right through to sandstone. For high rate mining, sandstone footwall will not be a requirement as it is considered difficult from a mineability perspective. All the unconsolidated footwall types will however be encountered by such a high rate mining system (Pether, 2002).

Two broad clay types are encountered based on age: Eocene (Bartonian) clays and Cretaceous (Coniacian) clays (Pether, 2002). See Appendix 2A for a map showing the different geological ages encountered in the Atlantic 1 mining area.

The available geotechnical data from the clay footwall stems from 2 datasets. The first dataset (see Appendix 3: Theory on clays and clay minerals Appendix 4 for this dataset called “dataset 1”) consisting of 70 samples includes data collected since 1996 and the spatial distribution of these samples is shown in Appendix 2A. The samples are largely limited to mining areas in regions J and N and sampled areas in regions P and G (Pether, 2002).
The B17 area (see Appendix 2A, region F) was more thoroughly sampled during the testbed preparations in 1999 (Bosma, 1999); in this report this dataset is distinguished as “dataset 2” (see Appendix 4; Gtech. sample number 103 and 117 to 121). Shear strength measurements were done on 61 samples and from only 5 representative samples other geotechnical data was derived. At that time it was concluded by Bosma (1999) that the data from this dataset was more representative that the data of dataset 1 which fell in the regions around the B17 area.

Since a revision of the first data set in 2002, the understanding of the different footwall clay types has grown considerably (Pether, 2002) with four main clay types being distinguished; these will be explained in more detail below. Pether (2002) added the 5 samples of dataset 2 to dataset 1 and he did identify these 5 samples as exceptional: different than the rest of the Cretaceous samples in Regions P and G. In this study the second dataset, although containing only 5 samples (but taken to be representing 61 samples), has been recognized as belonging to a separate footwall clay type. This also clears the wrong conclusions drawn in Bosma (1999).

### 2.2.1 Texture

See Appendix 2B for the classification of the different footwall clays by texture. The Eocene footwall samples come mainly from Region N. They have a high proportion of clay fraction. Their sand contents are variable, but seldom exceed 30%. The sand is mainly glauconite of authigenic marine origin. Samples of the Cretaceous footwall are texturally more diverse and range from clay-rich to silt-rich textures and to sandy muds and silts. The footwall from Region P tends to have a higher clay content. The footwall from Region J tends to be silt-rich, whilst the footwall nearby Region F is much siltier and sandier than those of region J (Pether, 2002 & Bosma, 1999). Table 2.1 summarizes the textural data of the footwall clays. The data of the two clay types that have been used in the scaled tests is also included for comparison. These clay types will be further dealt with in chapter 3.

<table>
<thead>
<tr>
<th></th>
<th>Clay (%)</th>
<th>Silt (%)</th>
<th>Sand (%)</th>
<th>Gravel (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eocene</td>
<td>Range</td>
<td>45-88</td>
<td>11-39</td>
<td>0-31</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>62,6</td>
<td>26,6</td>
<td>9,5</td>
</tr>
<tr>
<td>Cretaceous_1</td>
<td>Range</td>
<td>16-76</td>
<td>23-74</td>
<td>1-46</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>40,4</td>
<td>48,9</td>
<td>9,1</td>
</tr>
<tr>
<td>Cretaceous_2</td>
<td>Range</td>
<td>23-67</td>
<td>11-55</td>
<td>1-45</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>48,6</td>
<td>34,9</td>
<td>14,0</td>
</tr>
<tr>
<td>Cretaceous_3</td>
<td>Range</td>
<td>10-38</td>
<td>47-63</td>
<td>9-30</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>21,4</td>
<td>56,8</td>
<td>20,8</td>
</tr>
<tr>
<td>Test clay-soft</td>
<td>Range</td>
<td>46,0</td>
<td>29,0</td>
<td>22,0</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>45,0</td>
<td>28,0</td>
<td>26,0</td>
</tr>
</tbody>
</table>

Table 2.1: Summary of texture of different clay types.

### 2.2.2 Mineralogy

The clay mineralogy has been determined by infrared (IR) spectrometry using the Portable Infra-red Mineral Analyser (PIMA), a field tool designed primarily for ground truthing of airborne IR remote-sensing data. X-ray diffraction (XRD) clay analysis techniques were used to calibrate the PIMA technique and to develop IR spectral processing and interpretation methods that most reliably produce results reflecting the actual clay mineralogy (Pether, 2002).

All the footwall samples have a strong illite component of >50%. However, Eocene samples are mainly illite-smectite mixtures, whereas Cretaceous samples are mainly illitic-kaolin mixtures and form distinct clusters in the ternary diagram (see Appendix 2C). Based on smectite content the Cretaceous samples can be divided into ones with low and ones with intermediate smectite content. Notably, there is a facies change within the Cretaceous interval, from dominantly fluvial in the south (Regions J and K) to more open-marine conditions in the north (Region P). This is apparently accompanied by an increased number of beds with smectitic compositions. Based on texture the Cretaceous samples can be further divided into ones with relatively low (Region P) and ones with high silt content (Region F); last division is based on dataset 2.
2.2.3 Conclusions
It is clearly evident that footwall in the areas of interest form four categories corresponding with age and depositional environments (Pether, 2002 & Bosma, 1999):

1. Southern Cretaceous footwall beds are silt, mud and clay lithologies and the clay fractions in the essentially fluvial-paralic sequence are basically kaolinitic illites. Half of the samples have low smectitic contents of <3% and the other half have smectitic contents between 3 and 10%
   This category/type of clay will be further called "Cretaceous 1".
2. Northern Cretaceous footwall beds have higher clay contents and environments identified are marine and paralic. In the P-G samples kaolinite is often absent, but smectite is present and varies between 10 and 20%
   This category/type of clay will be further called "Cretaceous 2".
3. Eocene footwall samples are marine sandy muds and clays with low kaolinite content and smectite contents of 15-40%
4. Northern Cretaceous footwall samples of region F that are sandy silts with low clay fractions. Unfortunately no record of the clay mineralogy exists. This clay type will be further referred to as "Cretaceous 3". This is the fourth clay type that is recognized based on the data from dataset 2.

The geotechnical characteristics of the different clay types will be discussed in detail in chapter 3.

Pether (2002) acknowledges that the spatial distribution of the footwall samples remains limited to the primary areas of mining activity and that more spatial data is necessary to get a better understanding of the different clay types. In spite of this limited spatial distribution, already 4 different clay types are recognized. This indicates that in the Atlantic 1 mining area there is a high variability in clay types which can change over very short distances. Although this high variability, the four recognized clay types cover a wide range of clay categories and are thought to be representative enough for possible clay types to be encountered by the high rate mining system.
3. Geotechnical characterisation of footwall clay and test clay types

3.1 Definition of soil and soil classification

Different definitions of soils exist. Soil is an unconsolidated assemblage of solid particles which may or may not contain organic matter, the voids between the particles being occupied by air and/or water. The range of particle sizes encountered in soil is very wide: from above 200 mm to less than 0.0001 mm. In an engineering context soil means material that can be worked in, on or with, without drilling or blasting.

Classification of soils is based on certain fundamental properties and provides an ordered framework for their systematic description. Such a classification ideally should indicate the engineering performance of the soil type and should provide a quick means of identification.

The fundamental property upon which most engineering classifications of soils are based is the particle size distribution, since it is readily measurable and has an important influence on soil behaviour. Boulders, cobbles, gravels, sands, silts and clays are distinguished as individual groups and their size range differ depending on the used classification system (see paragraph 3.1.2). The major groups are divided further into subgroups on a basis of grading or plasticity. The classification based on plasticity is further explained in paragraph 3.1.3.

Because we are dealing with natural footwall clays, the terms "clay" and "soil" are used in this report without distinction, i.e. both have the same meaning in this report.

3.1.1 Basic properties of soil (Mitchell, 1993)

Soil consists of three phases – solids, water and air. The interrelationships of the weights and volumes of these three phases are important since they help define the character of the soil. One of the most fundamental properties of a soil is the void ratio \( e \), which is the ratio of the volume of the voids to that of the volume of the solids.

\[
e = \frac{V_v}{V_s}
\]  

(3.1)

The different footwall clays cover a wide range of void ratios as shown in Table 3.1 and the void ratio histogram in Appendix 5.

Water plays a fundamental part in determining the engineering behaviour of soil and the moisture content/water content is expressed as a percentage of the mass of the solid material in the soil.

\[
w\% = \frac{M_w}{M_s} \times 100%
\]  

(3.2)

The moisture content of the different clay types will be dealt with in paragraph 3.1.3.

Degree of saturation \( S_r \) expresses the relative volume percentage of water in the voids.

\[
S_r = \frac{V_w}{V_v}
\]  

(3.3)

The marine clays encountered by De Beers Marine have degrees of saturation around 100% as can be seen from Table 3.1.
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The bulk density $\rho$ of a soil is the ratio of its total mass to that of its total volume.

$$\gamma = \frac{M}{V} \quad (3.4)$$

The dry density $\rho_d$ is the mass of the solid particles divided by the total volume.

$$\gamma_d = \frac{M_s}{V} \quad (3.5)$$

In these equations, $M$ is the total mass ($M_s+M_w$) of the soil of volume $V=V_s+V_w$, $M_s$ is the mass of the solid particles of volume $V_s$, $M_w$ is the mass of the water of volume $V_w$ and $V$ is the volume of the pores (or voids).

Two clay types were available for testing, named "test clay – soft" and "test clay – stiff". A summary of the basic properties of the identified clay types together with the test clay types is presented in Table 3.1. See also Appendix 5 for a histogram of the dry density.

<table>
<thead>
<tr>
<th></th>
<th>Void ratio (-)</th>
<th>Degree of saturation (%)</th>
<th>Dry density (kg/m$^3$)</th>
<th>Bulk density (kg/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eocene</td>
<td>Range</td>
<td>0,92- 1,62</td>
<td>85-119</td>
<td>1060-1377</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>1,2</td>
<td>104,3</td>
<td>1193,0</td>
</tr>
<tr>
<td>Cretaceous_1</td>
<td>Range</td>
<td>0,371- 0,93</td>
<td>78,3-143</td>
<td>1328-1949</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>0,60</td>
<td>104,58</td>
<td>1688,0</td>
</tr>
<tr>
<td>Cretaceous_2</td>
<td>Range</td>
<td>0,56- 1,21</td>
<td>82-115,1</td>
<td>1232-1778</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>0,87</td>
<td>100,48</td>
<td>1477,0</td>
</tr>
<tr>
<td>Cretaceous_3</td>
<td>Range</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Test clay-soft</td>
<td>1 sample</td>
<td>0,79</td>
<td>88,9</td>
<td>1472</td>
</tr>
<tr>
<td>Test clay-stiff</td>
<td>1 sample</td>
<td>0,47</td>
<td>99,4</td>
<td>1773</td>
</tr>
</tbody>
</table>

Table 3.1: Summary of basic properties.

3.1.2 Particle size distribution

The particle size distribution expresses the size of particles in a soil in terms of percentages by weight, of boulders, cobbles, gravel, sand, silt and clay. The limits for these sizes based on the British Standards (BS 1377) are shown in Figure 3.1.

Figure 3.1: Particle size distribution based on BS 1377 (Whitlow).
Usually two methods are used to determine the particle size distribution: sieving and sedimentation. Sieving can be done either dry or wet and is used for the particle size analysis of sands and gravels (with size greater than 60 μm). Two types of sedimentation techniques are commonly used: pipette and the hydrometer test. These are used for size analysis of fine-grained soils (with size smaller than 60 μm).

The results of particle size analyses are given in the form of a series of fractions, by weight, of the different size grades. These fractions are expressed as a percentage of the whole sample and are generally summed to obtain a cumulative percentage. Cumulative curves are then plotted on a semi-logarithmic paper to give a graphical representation of the size distribution (grading curves). See for example Figure 3.2 (Whitlow, 2001).

![British Standard test sieves](image)

Figure 3.2: Typical particle size distribution curves (Whitlow).

A further quantitative analysis of grading curves may be carried out using geometric values known as grading characteristics. First of all, 3 points are located on the grading curve to give the following characteristic sizes:

- \(d_{10}\) = maximum size of the smallest 10% of the sample
- \(d_{30}\) = maximum size of the smallest 30% of the sample
- \(d_{60}\) = maximum size of the smallest 60% of the sample

From these characteristic sizes the following grading characteristics are defined:

Effective size = \(d_{10}\) mm

Uniformity coefficient: \[ C_u = \frac{d_{60}}{d_{10}} \] (3.6)

Coefficient of curvature: \[ C_g = \frac{(d_{30})^2}{d_{60} \times d_{10}} \] (3.7)

Both \(C_u\) and \(C_g\) will be unity for a single-sized soil, while \(C_u < 3\) indicated uniform grading and \(C_u > 3\) a well-graded soil. A well-graded soil has a \(C_g\) between 1 and 3.
Unfortunately the complete particle size distribution of the footwall clay types was not readily available for analysis. Only the fractions of clay, silt, sand and gravel are known as presented in chapter 2 (Appendix 2B). For the purpose of this project the complete particle size distribution of the test clay types has been determined by Geoscience Laboratories (Pty) Ltd. The size distribution for the fraction below 60 μm is determined by means of hydrometer analysis.

See also Appendix 2B for a comparison of the test clay types to the different footwall clays. The two test clay types classify as sandy muds and resemble the sandier Eocene and Cretaceous 2 footwall clay samples. The Cretaceous 1 and 3 clay types are much siltier and/or sandier.

### 3.1.3 Plasticity & consistency limits

The consistency of a soil is its physical state characteristic at a given water content. Four consistency states may be defined for cohesive soils as can be seen in Figure 3.3 (Whittow R., 2001).

![Figure 3.3: Consistency states of soil (Whittow).](image)

The transition from one state to the next is gradual and arbitrary limits, called the Atterberg limits, are defined:

- Liquid limit (w_L): the water content at which the soil ceases to be liquid and becomes plastic
- Plastic limit (w_p): the water content at which the soil ceases to be plastic and becomes a semi-plastic solid
- Shrinkage limit (w_s): the water content at which drying-shrinkage at constant stress ceases.

Casagrande (1932) deduced that the liquid limit of all soils corresponds approximately to a water content at which the soil has an undrained shear strength of about 2.5 kPa. Subsequent studies by different researchers derived similar results and also deduced that the liquid limit corresponds to a pore water suction of about 6 kPa (Mitchell, 1993). It also has been found that all soils at the plastic limit exhibit similar values of undrained shear strength reported by a number of researchers as being 100-200 kPa (Carter and Bentley, 1991). As such the consistency limit tests together with the actual water content of the soil in a sense give indirect measures of shear strength (see paragraph 3.2 on shear strength).

#### 3.1.3.1 Different indices

The two most important of the Atterberg limits are the liquid and plastic limits, which represent respectively the upper and lower bounds of the plastic states. Their difference is called the plasticity index (I_p):

\[ I_p = w_L - w_p \] (3.8)

The liquidity index of a soil is defined as its moisture content in excess of the plastic limit, expressed as a percentage of the plasticity index. It describes the moisture content of a soil with respect to its index limits and indicates in which part of its plastic range the soil lies, that is, its nearness to the liquid limit (Bell, 2000).
Liquidity index ($I_L$):

$$I_L = \frac{(w - w_p)}{I_p} \quad (3.9)$$

Where $w$ = natural water content

The significant values of $I_L$ are:

- $I_L < 0$: soil is in semi-plastic or solid state
- $0 < I_L < 1$: soil is in plastic state
- $I_L > 1$: soil is in liquid state

The consistency index ($I_C$) is the opposite of the liquidity index and defines the nearness of the moisture content to the plastic limit:

$$I_C = \frac{(w_L - w)}{I_p} = 1 - I_L \quad (3.10)$$

It is used to classify the different types of consistency of cohesive soils, as shown in Table 3.2 (Bell, 2000). The consistency determines what can be called the "mechanical mouldability" of the clay and is quite an important parameter in comparing the behaviour of different clay types under mechanical stresses.

<table>
<thead>
<tr>
<th>Class of clay</th>
<th>Consistency index, $I_C$</th>
<th>Undrained shear strength (kPa)</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very soft</td>
<td>&lt;0.5</td>
<td>&lt;20</td>
<td>Easily penetrated several centimetres by fist</td>
</tr>
<tr>
<td>Soft</td>
<td>0.5-0.75</td>
<td>20-40</td>
<td>Can be penetrated several cm by thumb, easily moulded</td>
</tr>
<tr>
<td>Firm</td>
<td>0.75-1</td>
<td>75-150</td>
<td>Readily indented by thumb, but penetrated only with difficulty. Cannot be moulded by fingers</td>
</tr>
<tr>
<td>Stiff</td>
<td>&gt;1</td>
<td>150-300</td>
<td>Indented with difficulty by thumbnail, brittle</td>
</tr>
<tr>
<td>Very stiff</td>
<td>&gt;3</td>
<td>&gt;300</td>
<td>Indented with difficulty by thumbnail, brittle</td>
</tr>
</tbody>
</table>

Table 3.2: Consistency of cohesive soils (Bell).

3.1.3.2 Activity

Plasticity is determined by the amount and the type of clay minerals present since clay minerals greatly influence the amount of attracted water held in the soil. Appendix 3 goes in-depth into the theory of clay to better understand their geotechnical properties. The degree of plasticity of a clay fraction itself is termed the activity of the soil:

$$Activity = \frac{I_p}{c} \quad (3.11)$$

Where:

- $c$ = the percentage by weight of clay fraction ($<2 \mu m$).

The activity gives an indication of clay mineral type is. Typical activity values are shown in Table 3.3 (Carter and Bentley, 1999). Classification of clays based on activity is presented in Table 3.4.
Mineral | Activity (Mitchell)
--- | ---
Montmorillonite | 1.0-7.0
Illite | 0.5-1.0
Kaolinite | <0.5

Table 3.3: Typical values of activity (Mitchell).

<table>
<thead>
<tr>
<th>Activity</th>
<th>Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;0.50</td>
<td>Inactive</td>
</tr>
<tr>
<td>0.50-0.75</td>
<td>Inactive</td>
</tr>
<tr>
<td>0.75-1.25</td>
<td>Normal</td>
</tr>
<tr>
<td>1.25-2.00</td>
<td>Active</td>
</tr>
<tr>
<td>&gt;2.00</td>
<td>Active</td>
</tr>
</tbody>
</table>

Table 3.4: Classification of activity of clay minerals (Grim).

3.1.3.3 Plasticity

Plasticity is the ability of a material to be moulded (irreversibly deformed) without fracturing. Plastic soils are often described as cohesive to distinguish them from non-plastic soils which are described as non-cohesive or granular. Since the plasticity of fine soils has an important effect on engineering properties like shear strength, plastic consistency is used as a basis for their classification.

Sils and clays that fall under the fine-grained soil group are generally subdivided according to their liquid limit into low, medium, high, very high and extremely high plasticity subgroups:

- Low plasticity: \( W_L < 35\% \)
- Intermediate: \( 35\% < W_L < 50\% \)
- High: \( 50\% < W_L < 70\% \)
- Very high: \( 70\% < W_L < 90\% \)
- Extremely high: \( W_L > 90\% \)

Each subgroup is also given a symbol. The ranges and symbols are shown in the plasticity chart which is a plot of the plasticity index vs. liquid limit, as introduced by Casagrande (see Figure 3.4). Silts tend to plot below, and clays above the A-line (Whitlow, 2001).

Figure 3.4: Plasticity chart for classification of fine grained soils.
3.1.3.4 Classifications of the footwall and test clay types
A summary of the different clay types under study in this report regarding the Atterberg limits is given in Table 3.5.

<table>
<thead>
<tr>
<th>Clay Type</th>
<th>Liquid limit (%)</th>
<th>Plastic limit (%)</th>
<th>Plasticity index (%)</th>
<th>Moisture content (%)</th>
<th>Consistency index (-)</th>
<th>Activity (-)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eocene</td>
<td>Range</td>
<td>64-144</td>
<td>26-50</td>
<td>38-94</td>
<td>33.6-66.09</td>
<td>0.51-1.17</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>109</td>
<td>39</td>
<td>70</td>
<td>48.1</td>
<td>0.85</td>
</tr>
<tr>
<td>Cretaceous_1</td>
<td>Range</td>
<td>30-95</td>
<td>14-38</td>
<td>13-65</td>
<td>13.1-37.2</td>
<td>0.38-1.8</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>53</td>
<td>24</td>
<td>28</td>
<td>23.66</td>
<td>1.07</td>
</tr>
<tr>
<td>Cretaceous_2</td>
<td>Range</td>
<td>44-122</td>
<td>18-37</td>
<td>26-87</td>
<td>20.9-46.7</td>
<td>0.48-1.1</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>85</td>
<td>30</td>
<td>56</td>
<td>31.79</td>
<td>0.93</td>
</tr>
<tr>
<td>Cretaceous_3</td>
<td>Range</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>46</td>
<td>22</td>
<td>24</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Test clay-soft</td>
<td>1 sample</td>
<td>38</td>
<td>19</td>
<td>19</td>
<td>26.7</td>
<td>0.59</td>
</tr>
<tr>
<td>Test clay-stiff</td>
<td>1 sample</td>
<td>39</td>
<td>18</td>
<td>21</td>
<td>17.9</td>
<td>1</td>
</tr>
</tbody>
</table>

Table 3.5: Summary of footwall clay and model clay data.

See Appendix 5 for histograms of the plasticity and consistency indices.

Based on this classification, the Cretaceous 1 clay has an intermediate plasticity, while the Cretaceous 2 has a very high to extremely high plasticity. Some of the Cretaceous 1 samples even plot under the A-line indicating that they are silts rather than clays. The Eocene clay is extremely plastic (see Figure 3.4). The consistency index of the Cretaceous clays is on average near one. This means that the natural water content is mostly around its plastic limit and that the clay can be classified as stiff to very stiff. The Eocene clay with an average consistency index of 0.86 can be classified as stiff. The activity of the footwall clays, which is on average around one, indicates (based on Table 3.3) that the clay types occurring in the clay are illite and montmorillonite and this is in accordance with the PIMA analysis of the mineral content of the clays (mainly illite and smectites).

The soft and stiff test clays with consistency indices of respectively around 0.6 and 1 classify as firm and stiff respectively. The mineralogy of the test clays was also determined by PIMA analysis. The soft test clay consists of 90% kaolinite and 10% illite, while the stiff test clay is 100% kaolinitic. The test clays have an intermediate plasticity. An activity below 0.5 for the test clays indicates the occurrence of mainly kaolinites what is also confirmed by the PIMA analysis.
3.2 Shear strength

3.2.1 Theory

The shear strength is defined as the maximum shear resistance which a soil can offer under defined conditions of effective pressure and drainage. When under undrained conditions, before drainage can take place, it is called the undrained shear strength or undrained cohesion or simple shear strength or cohesion; denoted by symbol $c_u$. The shear strength is not a fundamental property of a soil, but is related to the conditions prevailing in situ and can vary with time. The value measured in the laboratory is likewise dependent upon the conditions imposed during the test and in some instances upon duration of the test.

The shear strength of clays depends not only on the soil type and composition, as for sands, but on the following three significant factors:

- Water content
- Degree of saturation
- Undrained or drained condition

It is usually assumed that the shear strength of soils is governed by the Mohr-Coulomb failure criterion:

$$s = c + \sigma \tan \phi$$  \hspace{1cm} (3.12)

$s$ = shear stress at failure along any plane
$\sigma$ = normal stress on that plane
$c$ and $\phi$ = shear strength parameters; cohesion and angle of shearing resistance

This is shown graphically on the Mohr-diagram given in Figure 3.5 (Carter and Bentley, 1991).

![Figure 3.5: Mohr diagram representing the general Mohr-Coulomb failure criterion (Carter and Bentley).](image)

A complication arises because the normal stresses within a soil are carried partly by the soil skeleton itself and partly by the water within the soil voids. Considering only the stresses within the soil skeleton, equation (7) is modified to:

$$S = c' + (\sigma-u) \tan \phi'$$ or $$S = c' + \sigma' \tan \phi'$$  \hspace{1cm} (3.13)

$u$ = the pore water pressure
$\sigma'$ = $(\sigma-u)$, effective normal stress (on the soil skeleton)
$c'$ and $\phi'$ = shear strength parameters related to the effective stresses.

Thus when considering the shear strength of soils, there is a choice:

Either the total, combined response of the soil and pore water can be considered; or the specific response of the soil skeleton can be separated from the pore water pressure by considering effective stresses.

The effective stress approach gives a true measure of the response of the soil skeleton to the load applied to a saturated soil that is allowed to drain. If the rate of application of the load is sufficiently slow, the pore water can escape/drain away, pore pressures will not build up and the total stresses will equal the effective stresses. For drained conditions, or in terms of effective stresses, it is found that the

<table>
<thead>
<tr>
<th>Document Number</th>
<th>Rev</th>
<th>Additional Reference</th>
<th>Page Number</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>29</td>
</tr>
</tbody>
</table>
shear strength of soils is principally a frictional phenomenon, with $c'=0$, as illustrated in Figure 3.6 (Carter and Bentley, 1991).

This does not appear to be the case for over-consolidated clays which have a built-in pre-stress, or for partially saturated clays in which the particles are drawn together by surface tension effects, giving them some cohesion.

When soil is loaded, the increase in confining pressure within the soil skeleton squeezes the particles closer together, reducing the volume of voids. However, in saturated clay this cannot take place unless some of the pore water can drain from the voids. Thus, for saturated clay in conditions of no drainage (undrained condition), an increase in confining pressure cannot be carried by the soil skeleton but results instead in an equal increase in pore water pressure.

Since shear strength depends on the effective stresses, transmitted by inter-particle contacts, and these remain unchanged irrespective of the applied confining pressure, it follows that the undrained shear strength will also be independent of confining pressure. Because of this, samples of saturated clay tested in a quick undrained test give Mohr's circles of constant diameter and an apparent cohesion value as shown in Figure 3.7, even though, in effective stress terms, the material is basically frictional. Thus, in a sense, the phenomenon of cohesion is an illusion brought about by the response of pore water pressures to imposed loads. To underline this point, the term apparent cohesion is often used.

Partially saturated soils, tested in undrained conditions, will show a behaviour which is intermediate between that for drained conditions and for saturated undrained conditions, depending on the degree of saturation.

This means that the undrained shear strength of saturated clay is a fixed value and is equal to the apparent cohesion.

Apart from direct measurement of the apparent cohesion correlation with other index test values is possible and explored extensively by different researchers. The value of the undrained shear strength may be estimated by moulding a piece of clay between the fingers and applying the observation indicated in Table 3.2.

As discussed in the previous paragraph the liquid limit and plastic limit are moisture contents at which the soil has specific values of undrained shear strength. The strength is thus related to the natural
moisture content in relation to the liquid and plastic limit conveniently expressed by the liquidity index. Curves relating the remoulded undrained shear strength to the liquidity index have been established by Skempton and Northey (1952); see Figure 3.8 (Carter and Bentley, 1991).

The natural shear strength of the undisturbed clay may be different from the remoulded shear strength depending on the consolidation history of the clay as well as the fabric characteristics. This phenomenon is known as the sensitivity and is expressed as the ratio of the natural shear strength to the remoulded shear strength. The concept of sensitivity was developed to explain the loss of undrained shear strength when a undisturbed natural clay is remoulded. It is most apparent in soft, lightly consolidated clays which have an open structure and a high moisture content. Different researchers found a relation between sensitivity and liquidity index; that found by Skempton and Northey (1952) is shown in Figure 3.9.

Combining both the relation between remoulded shear strength and liquidity index and the relation between sensitivity and liquidity index, a relation can be found between the natural shear strength of undisturbed clays and the liquidity index. This is shown in Figure 3.10 which provides a useful predictive tool for assessing the shear strength of undisturbed soils.

Figure 3.8: Correlation between remoulded shear strength and liquidity index (Carter and Bentley).
Figure 3.9: Correlation between sensitivity and liquidity index (Carter and Bentley).

Figure 3.10: Relationship between natural shear strength and and liquidity index (Carter and Bentley).
3.2.2. Measuring the undrained shear strength

The undrained shear strength of saturated plastic clays is usually determined from compression tests, the shearbox test being less satisfactory for these soils. However their shear strength can be measured directly by the vane apparatus or a quick shearbox test. Using the quick shearbox test it is assumed that virtually no drainage takes place from a clay during the short period (usually up to 20 minutes) of the quick shear test (Head, 1988).

A series of quick tests with different normal pressures on saturated clay gives a failure envelope similar to Figure 3.11a in which the line is virtually horizontal. A quick test on an overconsolidated clay gives a slightly curved envelope as indicated in Figure 3.11b. Sandy or silty clay generally have some cohesion as well as internal friction, although the internal friction angle from the quick shear test is less than that for sands. See Figure 3.11c (Head, 1988).

![Figure 3.11: Failure envelope for saturated clay (a), overconsolidated clay (b) and sandy or silty clay (c).](image)

3.2.2.1 Test methods

Direct shear tests (Head, 1988):

1. Shear box test in which relative movement of two halves of a square block of soil takes place along a horizontal surface
2. Vane test, in which relative rotational movement takes place between a cylindrical volume of soil and the surrounding material.

The shearbox test is the simplest, the oldest and the most straightforward procedure for measuring the immediate or short term shear strength of soils in terms of total stress. In the test a soil is placed in a rigid metal box consisting of two halves. The lower half can slide relative to the upper when pushed by a motorized drive unit, while a normal pressure is applied.

In the vane test a four-blade cruciform is pushed into the soil and then rotated. The torque required to cause rotation of the cylinder of the soil is measured, which enables the undrained shear strength of the clay to be calculated (See Figure 3.12a and 3.12b for a laboratory and a field shear vane respectively).
Another device used to measure the undrained shear strength is the pocket shearmeter, or pocket vane (also called torvane), shown in Figure 3.12c. It operates on a similar principle to the laboratory vane apparatus, but is applied to a surface and rotates a relatively thin disk of soil. It can be used on site or in the laboratory as well. The instrument should be regarded as an aid to visual classification of soil in the zone inspected, and is not as a substitute for other methods measuring the shear strength for design purposes. The shear strength is measured by pushing the vanes into the soil and turning the knob until a maximum reading is achieved on the dial. This is calibrated to read directly in shear strength units.

![Figure 3.12: Different shear vanes.](image)

### 3.2.3. Undrained shear strength data of the different clay types

Based on data set 1 the following summary of shear strength, measured by means of shear box tests, is presented in Table 3.6.

<table>
<thead>
<tr>
<th>Clastic Type</th>
<th>Range (kPa)</th>
<th>Average (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eocene</td>
<td>9.7-63.5</td>
<td>38.2</td>
</tr>
<tr>
<td>Cretaceous_1</td>
<td>10.8-74</td>
<td>40.54</td>
</tr>
<tr>
<td>Cretaceous_2</td>
<td>30.5-97.1</td>
<td>53.37</td>
</tr>
</tbody>
</table>

Table 3.6: Summary of strength properties based on data set 1.

Based on correlations of natural shear strength and liquidity index of clays as presented in paragraph 3.2.1, and given the low liquidity indexes of the different clay types (almost zero), the undrained shear strength should range from 80 to 120 kPa (see Figure 3.10).

The second testing program in the B17 area (dataset 2) recorded the sampled clay footwall material (Cretaceous 3 clay) to be stiff and very stiff in places, even after disturbance. The footwall samples collected at sea were split open to expose undisturbed surfaces and tested using a hand held Geonor shear vane (Bosma, 1999). The average, median and standard deviation values are respectively 172 kPa, 155 kPa and 66 kPa. A histogram of the data is shown in Figure 3.13. This seems to be closer to reality as expected by above mentioned correlations. Unfortunately the moisture content of the Cretaceous 3 clay samples was not measured and thus the consistency cannot be calculated.
De Beers Marine (Pty) Ltd.
High Rate Mining Tool Development
Dealing with clay when mining diamonds offshore

Histogram: Shear strength of Cretaceous 3 clay

![Histogram of shear strength of Cretaceous 3 clay.](image)

**Figure 3.13: Histogram of shear strength of Cretaceous 3 clay.**

As can be seen the "shear box"-strength values of dataset 1 are much lower than the one of dataset 2. During the course of this project the contradiction was observed and it was decided to do more undrained shear strength measurements on the footwall clay using a hand held shear vane (torvane). In March 2003 tests were conducted in Region V but only 8 samples were taken due to failure of the torvane equipment. The observed values are shown in Table 3.7 (each value is the average of 3 measurements). This confirmed that the shear strength values are much higher than would be concluded from the values in dataset 1.

The author believes that the lower values of dataset 1 can be explained by the fact that shear box tests may not be the correct method to measure the undrained shear strength and the tests are possibly not completely undrained. Since the shear strength values of dataset 1 are not considered as being representative for the footwall clays, they will not be used in further analysis. This cuts down the available measured shear strength values to only the Cretaceous 3 clay type and the yet unidentified clay type of region V.

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Shear strength (kPa)</th>
<th>Sample Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>68</td>
<td>Hard grey blocky clay</td>
</tr>
<tr>
<td>2</td>
<td>192</td>
<td>Stiff grey clay</td>
</tr>
<tr>
<td>3</td>
<td>83</td>
<td>Hard grey blocky clay</td>
</tr>
<tr>
<td>4</td>
<td>134</td>
<td>Stiff grey clay</td>
</tr>
<tr>
<td>5</td>
<td>54</td>
<td>Hard grey blocky clay</td>
</tr>
<tr>
<td>6</td>
<td>136</td>
<td>Stiff green grey clay</td>
</tr>
<tr>
<td>7</td>
<td>143</td>
<td>Stiff green grey clay</td>
</tr>
<tr>
<td>8</td>
<td>60</td>
<td>Hard grey blocky clay</td>
</tr>
</tbody>
</table>

**Table 3.7: Shear strength data footwall clay in region V.**

Distinguishing between hard grey blocky clay and the stiff grey clay (as noted by the geological observer), their shear strengths are on average 66 and 151 kPa respectively. Experience shows that the sandier and siltier clays like the Cretaceous 1 and Cretaceous 3 clay exhibit higher shear strengths than clayey clay types (Eocene and Cretaceous 2 clay types), this probably to differing consistency indices. It can thus be expected that the Eocene and Cretaceous clay types have slightly lower shear strength than the Cretaceous 3 clay types.
The shear strength of the "test clay – stiff" was measured together with the water content to confirm the shear strength – liquidity index (or consistency index) relationship as predicted by Figure 3.10. Figure 3.14 shows the acquired trend which is similar to the predicted one and it can be seen that at a liquidity index of 0 (consistency index of 1) the shear strength of the clay is about 120 kPa.

Figure 3.14: Measured relationship between shear strength and liquidity index.
3.3 Adhesion

Although the adhesion is not considered to be a geotechnical characteristic of clays it is included here because of its importance in this study. A separate chapter is devoted to adhesion since it is a major factor in designing scaled tests. Here follows just the comparison of the adhesion of the different clay types based on correlations presented in chapter 5.

The adhesion is normally not measured in geotechnical tests and internationally standardized tests do not exist. Correlations in literature are used to get a qualitative and quantitative measure of adhesion.

Based on experience from slurry shield tunnelling Thewes (1999) poses that clay causes excessive adherence problems when the plasticity index is larger than 20% and the consistency index is between 0.75 and 1.25. See Figure 3.15 for the plot of the available data of the different clay types in the adherence potential graph of Thewes. Applying the experience from slurry shield tunnelling on the Gravel Wheel operation it can be concluded that the Cretaceous 1 footwall clay type has a moderate to high adherence potential and the Eocene and Cretaceous 2 clays exhibit high adherence potential. The stiff test clay plots at the border of the moderate to high adherence potential. And the soft test clay has according to this graph a low adherence potential, due to its low consistency index.

![Figure 3.15: Adherence potential for slurry shield tunnelling machines (Thewes, 1999).](image)

Thewes also presented quantitative trends for shear and tensile adhesion, see chapter 5. These trends are used to get a value of the tensile adhesion for the different clay types at a consistency index of 1, as presented in Table 3.8. The ranges of values are quite wide and should only be used for a general comparison of the clay types. Since the soft test clay has a consistency index of about 0.6 the average tensile adhesion will be much less than the listed 10 kPa.

<table>
<thead>
<tr>
<th>Adhesion (kPa)</th>
<th>Eocene</th>
<th>Cretaceous 1</th>
<th>Cretaceous 2</th>
<th>Test clay-soft</th>
<th>Test clay-stiff</th>
</tr>
</thead>
<tbody>
<tr>
<td>Range</td>
<td>10-30</td>
<td>10-45</td>
<td>15-35</td>
<td>0-20</td>
<td>0-25</td>
</tr>
<tr>
<td>Average</td>
<td>23</td>
<td>27</td>
<td>25</td>
<td>10</td>
<td>6</td>
</tr>
</tbody>
</table>

Table 3.8: Correlated tensile adhesion values of different clay types.
Trying to get a quantitative measure of the tensile adhesive strength of the clays, a simple "adhesometer" was designed and tested (see Figure 3.16 and Appendix 6) based on similar adhesion testing done by Thewes (1999). Adhesion measurements done on the stiff test clay showed values ranging from 6 and 9 kPa. This is fairly similar to the predicted adhesion value of 6 kPa. More testing has to be done on different clay types to evaluate the accuracy and reliability of the adhesometer.

Figure 3.16: Adhesometer for measuring tensile adhesion.

3.4 Permeability

Since soils consist of discrete particles, the pore spaces between particles are all interconnected and water is free to flow within the soil mass. The capacity of a soil to allow water to pass through (flow through) is termed its permeability (or hydraulic conductivity). In saturated conditions, one-dimensional flow is governed by Darcy's law, which states that the flow velocity is proportional to the hydraulic gradient:

\[ v = ki = k \frac{\Delta h}{\Delta L} \]

Where \( v \) = flow velocity  
\( k = \) coefficient of permeability  
\( i = \) hydraulic gradient = \( \frac{\Delta h}{\Delta L} \)  
\( \Delta h = \) difference in total head/pressure over a flow path length of \( \Delta L \)

The coefficient of permeability \( k \) may be defined as the flow velocity produced by a hydraulic gradient of unity. The value of \( k \) is used as a measure of the resistance to flow offered by the soil, and is affected by several factors:

- The porosity of the soil
- The particle size distribution
- The shape and orientation of soil particles
- The degree of saturation
- The type of cat-ion and thickness of absorbed layers associated with clay minerals
- The viscosity of the soil water, which varies with temperature
- The presence of fissures and cracks

The range of values for \( k \) is extremely large as can be seen in Figure 3.17 (Whitlow, 2001).
### Figure 3.17: Range of values of k in m/s (Whitlow).

The value of k can be measured using field tests, or tests conducted in the laboratory. In the case of the footwall clay in the Atlantic 1 mining area it is impossible to perform field tests. A number of approximate empirical relations have been suggested between k and other soil properties. It is mentioned by Whitlow (2001) that comparative studies show that none of these empirical relationships is particularly reliable and that it is far more realistic to obtain estimates for k using field or laboratory tests.

The most frequently used approximation is one suggested by Hazen for filter sands and no approximations have been found for clays.

#### 3.6 Final remarks

It is recognized that current properties all pertain to small isotropic samples. The in-situ properties could be different due to e.g. bedding planes and joints and stress conditions. More comprehensive tests are necessary to get insight into the anisotropy of the clay footwall.
4. Influence of clay on diamond mining process

4.1 Sub-processes

To be able to define the sub-process(es) of the mining operation which are most effected by clay, a basic understanding of these processes is necessary. The following sub processes have been identified for the Gravel Wheel operation:

1. Loosening/cutting of the soil
2. Pick-up (slurrification) of loosened soil
3. Transport (vertical and horizontal) of the material
4. Processing on board of the vessel
5. Disposal
6. Fixation and positioning of the crawler on the sea bottom

In the next paragraphs a description of the above-mentioned processes is given.

4.1.1 Loosening of the soil

In the excavation process the material must first be loosened. The cohesion of the particles has to be broken and the material must be displaced. Two main methods/techniques are possible: passive and active excavation.

Passive excavation is achieved by erosion, where the soil is disintegrated by a flow of water, either just by suction (like a vacuum cleaner) or by adding high-pressure water jets. This type excavation is hydraulic in nature and is also known as hydraulic excavation technique.

Active (mechanical) excavation of the soil is achieved by mechanical excavation tools such as blades, teeth, pick/points, cutters, knives, buckets or a draghead.

From an energetic point of view active (mechanical) excavation techniques are preferable to passive (hydraulic) excavation techniques. In the case of hydraulic excavation techniques the excavation and slurrification process coincide and conditions for transport are difficult to control (Van Kesteren, 1992).

Soil characteristics-parameters

Soil characteristics play an important role in the process of loosening. Cohesive soils like clay can be very difficult to cut due to its cohesive and adhesive character. To loosen cohesive, ductile soils like clay or clay like material, it will in most cases be necessary to use a cutting tool. Undrained cohesion (Cu) or the undrained shear strength, adhesion and in-situ density are important parameters in this respect. The higher the undrained cohesion and adhesion, the more difficult it is to cut the clay. The sizes and forms of the lumps formed are governed by the design and the operational parameters of the excavation tool. The downstream processes are all affected by the size and form of the clay lumps and that's why a more detailed analysis of the cutting process and the failure type of the clay footwall is performed as presented in chapter 7.

4.1.2 Pick-up of loosened soil (Slurrification)

The loosened material must be prepared for transport. This can be either hydraulically (by pumps) or mechanically. For pumping, the soil particles must be intensively mixed with water to create a mixture that can be pumped. Large particles may clog the pipe/screen. Clay particles can form clay balls, which increase the hydraulic force necessary to pump the mixture up. The efficiency of the operation will depend on the method employed to loosen the material prior to suctioning. The most important soil parameters are particle size distribution and permeability to water. For cohesive soils yield stress, viscosity and the tendency to form clay balls are also of importance.

With modern high efficiency pumps and high concentration slurry pumping possibilities, the slurrification of the loosened soil is of extreme importance. This requires an integrated design between the actual cutting tool, suction inlet and onward pipe inlet of the hydraulic system.
It is expected that this process is one of the most critical ones. Experience in both dredging and
tunnelling show that careful design and operation is necessary for continuous and smooth operation.

4.1.3 Transport
The transport process consists of vertical hydraulic transport to the onboard processing plant. A pump is
usually installed as low as possible, preferably submerged on the seabed crawler. Airlift systems can
also be used.

Hydraulic transport is a marriage between the pump characteristics and the pipeline resistances on the
one side and the fluid characteristics, soil properties and the settling behaviour of the soil on the other.

In the case of cohesive soils attention must be paid to the formation of clay balls. In this respect the
following parameters are of significance:
- Plasticity limits (cohesion, clay ball formation)
- Specific gravity and water content of the soil in-situ (maximum concentration, compactness)
- Shearing stress (pipeline resistance, cohesion)
- Particle size distribution (pipe pile resistance, max. hydraulic diameter)
- Permeability (pumping process, pipeline resistance)
- Structure
- Lime content

In the case of the Gravel Wheel mostly gravel and a certain amount of clay lumps will have to be
transported. Experience in dredging is that usually there are no big problems occurring even with
transport of solely clay lumps. The combined transport of gravel and clay lumps is expected to be
easier.

4.1.4 Processing
From the mixture entering the processing plant the diamonds have to be extracted. The recovery should
be high and the remaining waste material should be disposed of. The processing installation can be
placed on the ship or be separated through a storage buffer or transport. During processing important
soil characteristics are again, cohesion, density, particle/lump size distribution etc. The first processing
steps are usually screening of the oversize. Having larger oversized clay lumps clogging of the screens
can occur. Of more importance is the possibility of diamonds embedded in the clay lumps. In order to
avoid this high pressure water jetting has been proposed to disintegrate the clay lumps (DEBTECH-

4.1.5 Disposal
The waste of the processing step will have to be deposited offshore if permitted or on a dump,
reclamation point, storage site or other location onshore.

4.1.6 Fixation and positioning of the crawler
A vital part of the mining process is the way of fixation and positioning of the seabed crawler. During
operation the crawler must be accurately moved around to enable proper production.

The most important thing in fixation of the crawler is the compensation of the excavation forces. This
can be achieved by the weight of the crawler (taking into account the buoyancy force) in combination
with the soil strength on which it is standing.

Control of the positioning is also of importance to reach high accuracy of the excavation.

This is also affected by the clay footwall on which the crawler will be operating.
Report MB 15 (IHC, 1997) studies the drawbar pull which can be expected, track sinkage, soil response
as well as slippage.

Also of interest is the adherence effect on the whole tool and the ability to cope with/work in highly
adhering clay footwall. Although this can be very interesting to investigate it is not the focus of this study
and will not be detailed further.
5. Adhesion processes

5.1 Introduction

Experience in dredging, tunnelling, drilling and the agricultural industries show that wherever clay is being excavated, problems with the adherence of clay are likely to occur. The problems associated with adhering clay are:

- Adherence of clay to equipment parts (cutting tool): reduction of equipment performance like loss of bit/tooth penetration and cutting force, thus hindering the progress and continuation of the operation
- Adherence of clay in fluid passages: fluid transport (and thus efficient removal of cuttings) is hampered by blocked fluid passages (screens/grids, pipelines)
- Increased torque required on the cutting tool
- Increased wear by reduced cooling effectiveness

This chapter is meant to give a theoretical and practical knowledge on adhesion and adherence processes.

5.2 Theory

Myers (1991) defines adhesion as follows:

"Adhesion is the state in which two bodies are held together by intimate interfacial contact in such a way that mechanical force or work can be applied across the interface without causing the bodies to separate."

Other terms used to describe adhesion are adherence and stickiness. In this report they will be used without distinction, i.e. both have the same meaning.

In general Myers distinguishes four individual mechanisms of adhesion:

a) Thermodynamic adhesion; related to the molecular interactions such as van der Waals and electrostatic forces.
b) Chemical adhesion; caused by the formation of chemical bonds such as crystallization of salts, use of adhesives.
c) Mechanical adhesion; caused by interlocking of the two phases at irregularities in the interface.
d) Physical adhesion; caused by capillary menisci between single particles (in partially saturated soil) or by capillary stresses due to differential pore pressures (in fully saturated soil).

A distinction is made between true adhesion and apparent adhesion. The adhesion caused by the first two is called true adhesion, and that caused by the last two is called apparent adhesion.

Two basic types of adhesion can be recognized, see Figure 5.1 for a graphical presentation of these types:

a) The tensile adhesion: the force per unit area needed to pull off the clay perpendicular to the contact surface
b) The shear adhesion: the force per unit area needed to initiate sliding parallel to the contact surface

![Figure 5.1: Measuring adhesion a. tensile adhesion and b. shear adhesion (Zimnik, 1999).](image-url)
The tensile and shear adhesion can be split up into the true tensile adhesion and true shear adhesion (so forces depending on bonding and cementation between the two different surfaces) and the apparent tensile adhesion and apparent shear adhesion (stresses generated by physical and mechanical forces). In Figure 5.2 a schematic overview of the terminology is given.

![Schematic overview of used terminology](image)

**Figure 5.2: Schematic overview of used terminology.**

The difference in shear adhesion and tensile adhesion is a result of the apparent shear or tensile adhesion, since the interlocking of microscopic asperities and the capillary stresses that develop during testing can depend on the testing method. When the created mechanical and capillary stresses are subtracted from the total shear or tensile adhesion the true shear or tensile adhesion is obtained. It is expected that the true tensile adhesion is equal to the true shear adhesion (Baalen, van 1999).

Research by Thewes indicates that for the specific situation of natural clays adhering to a steel surface only thermodynamic, mechanical and physical adhesion prevail. Based on further literature research Thewes argues that in the case water is present at the clay-steel interface, from these mechanisms the physical adhesion is the most dominant (due to the presence of water the van der Waals forces are minimal and due to the water the interlocking effects at irregularities are minimized).

From the physical adhesion mechanisms the capillary menisci only play a significant role at low degrees of saturation (lower than 30%). With higher degrees of saturation between 30 and 80% the number of capillary menisci decreases and so thus the adhesive force caused by them, while the capillary forces increase gradually. Beyond 80% only the adhesion caused by capillary forces remain (Thewes, 1999).

In the case of marine clays with degrees of saturation of around 100%, capillary forces will be the main cause of tensile adhesion. Thewes (1999) explains the tensile adhesive force as follows: at high degrees of saturation and with the occurrence of water at the clay-steel interface capillary forces develop. This capillary "underpressure" has to be overcome when applying tensile stresses on the clay.
5.3 Factors influencing adhesion

There are a lot of factors influencing the adhesion:

A. Factors regarding the clay properties
   - Texture
   - Clay mineral type
   - Water content
   - Void ratio

B. Factors regarding the test conditions
   - Tensile or shear loading (see paragraph 5.2)
   - Sliding speed
   - Type and roughness of surface contact
   - Contact pressure and time
   - Presence of water (saturation, pressures and geochemistry)

The most important ones are discussed in more detail below.

5.3.1 Texture

As previously mentioned in chapter 2 a fine-grained soil has different fractions of clay, silt and sand and gravel. Thewes (1999) refers to tests conducted by Wolkewitz that indicate that adhesion increases with the percentage of clay-sized particles (<2μm). See Figure 5.3.

![Figure 5.3: Correlation of clay fraction and adhesion (Thewes, 1999).](image)

5.3.2 Clay mineral type

Not only the fraction of clay-sized particles is of importance. If one would grind quartz particles down to sizes smaller than 2 μm it would not show plastic properties as that of clay nor much adhesion (see Figure 5.4).

It is observed that different type of clay minerals have different adhesive properties. Several investigations (Boisson (1981), Jancsezc (1991), Kalachov (1975)) on the tensile adhesion of different clays to steel surfaces have been performed. In Figure 5.4 an overview is given of the influence of the clay mineral type (Kalachov, 1975).
3.3 Water content

The properties of clays are affected by the total water content, the chemical composition of the water and by the bonding strength between the clay particles and the water molecules.

Thewes (1999) established to measure the tensile adhesion of several natural clays as a function of the water content (see Figure 5.5 with consistency index as relative measure of water content). Also a large investigation on this subject was performed in Russia. Kalachov did a range of measurements for different clay minerals as shown in Figure 5.4 and Figure 5.6. Results of measurements show that the curve of water content versus tensile adhesion is a parabola with a maximum near the plastic limit. These measurements show the same trend as the investigation done by Thewes.

Figure 5.6 shows that the adhesion has a maximum value at certain water content for each normal stress. The maximum value of adhesion is to be found at higher normal stresses and with the water content, which lies near the plastic limit.
Figure 5.5: Tensile adhesion as function of consistency index (Thewes, 1999).
Figure 5.6: Tensile adhesion as a function of the water content for different normal stresses for a. kaolinite and b. illite (Kalachov, 1975).
5.3.4 Type and roughness of contact surface

Boisson (1981) published a report in which he presented the results of sliding clay over different surfaces. He published a graph, in which the adherence of clayey material over surfaces of plexiglass, leather, smooth steel and rough steel was measured (Figure 5.7). From these results can be concluded that different contact materials give different adhesion values.

![Figure 5.7: Adhesion as a function of the cohesion for different types of surfaces (After Boisson, 1981).](image)

The contact surface between two materials is different for each combination of materials. Investigations have been done on the adhesive shear stress of a clay-steel contact with varying steel surface roughness (Tsubakihara, 1993). He concluded that there existed a critical steel roughness. He states that when the roughness is smaller than the critical roughness, sliding occurs at the clay-steel interface. When this critical roughness is exceeded shear failure occurs within the clay specimen.

5.3.5 Sliding speed

There appears to be a relationship between the sliding speed and the adhesion. In the dredging industry extensive research has been done as they are dealing with adherence problems when cutting clay (Miedema, 1992).

Miedema tried to describe both the cohesive and adhesive behaviour during the cutting of clay by means of adapting the strain rate process theory. This theory provides insight into the influences of certain variables (like strain rate (or deformation rate) which is the ratio of sliding speed and cutting depth) on soil behaviour.

In Figure 5.8 the graph is displayed which shows Miedema's empirical relation between strain rate and the cohesive shear strength fitted onto experimental data. According to Miedema a similar relationship also applies for the adhesion based on the same adapted strain rate process theory.
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Figure 5.8: Empirical curve for cohesive shear strength as a function of logarithmic strain rate (after Miedema, 1992).

The effect of sliding speed on the friction coefficient between clays and steel has not been found. Coulomb's law states that the friction coefficient $\mu$ is independent of sliding speed. However from observations of tests on other materials, it has become clear that $\mu$ decreases with increasing sliding speed, sometimes even exponentially (Adamson, 1982).
5.4 Correlations of adhesion with geotechnical properties of clay

The range of adhesion measured by different researchers (see paragraph 5.3) is very wide since the adhesion depends greatly on the conditions under which the tests are conducted. A slight difference in test conditions may change the nature and type of the adhesion measured thus making it difficult to interpret the results and to apply trends found in another research to that of the Gravel Wheel. It is believed by the author that the research conducted by Thewes is recent and the conditions that occur during Gravel Wheel operation resemble that of slurry shield tunnelling (in slurry shield tunnelling clayey soils are also actively cut and hydraulically transported away). This justifies the use of Thewes’ results for the purpose of this study.

Thewes (1999) has done an extensive research on shear and tensile adhesion of natural clays and he has derived experimental correlations of tensile adhesion to the internal factors mentioned in paragraph 5.3 and to the plasticity index. He first explored experience from the slurry shield tunnelling industry and then did a theoretical and practical research on the phenomenon of adhesion. From his first exploration of experimental data gathered from slurry shield operations, Thewes (1999) poses that clay will show excessive sticking behaviour (adherence) when the plasticity index is larger than 20% and the consistency index between 0.75 and 1.25 (see Figure 5.9).

![Adherence potential: Consistency index vs Plasticity index](image)

**Figure 5.9**: Adherence potential graph from slurry shield tunnelling (Thewes, 1999).

In the rest of his research he did detailed tests off which some of the results are presented below.

5.4.1 Correlation to internal factors

Between the tensile adhesion and a combination of the three internal factors: particle size distribution, texture and void ratio, an approximately linear relationship can be found (Thewes, 1999). In order to study the influence of water content on the tensile adhesion Thewes worked mostly with clays with a consistency indices ($I_c$) of 0.85 and 1. The measured tensile adhesion of clays with an $I_c$ of 0.85 lie all around 10 kPa (see Figure 5.5) and show little correlation with other geotechnical properties of the clay. The following correlations derived are only valid for clays at an $I_c$ of 1. Further Thewes used constant and pre-specified external parameters and changed the contact time ($t_A$). Thewes also wetted the clay surface prior to testing and varied the wetting time ($t_B$).

In plotting only the tensile adhesion against the clay fraction (for a contact time, $t_A$, of 10 minutes and wetting time, $t_B$, of 0.25 minutes), the plotted data show a lot of scatter without a specific trend. But if the normal adhesion is plotted against the sum of illite ($I$) and montmorillonite ($M$) fractions a fair trend can be observed (see Figure 5.10 and Figure 5.11).
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![Graph of Tensile Adhesion vs Clay Fraction](image)

**Figure 5.10:** Tensile adhesion vs clay fraction (Thewes, 1999).

![Graph of Tensile Adhesion vs Illite and Montmorillonite Fraction](image)

**Figure 5.11:** Tensile adhesion vs illite and montmorillonite fraction (Thewes, 1999).

Thewes defines a "normalized" tensile adhesion $a_{t,e}$ as follows:

$$a_{t,e} = a_t e$$  \hspace{1cm} (5.1)

In which:
- $a_{t,e}$ = the normalized tensile adhesion
- $a_t$ = measured tensile adhesion
- $e$ = void ratio

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<th>Rev</th>
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<td>51</td>
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</table>
When this normalized tensile adhesion is plotted against the I+M-fraction a strong correlation (regression coefficient ($R^2$) = 0.92) is found, see Figure 5.12. See Figure 5.13 for the correlations found for different contact times.

![Graph](image)

**Figure 5.12:** Normalized tensile adhesion vs illite and montmorillonite fraction for $t_A=10$ min (Thewes, 1999).

![Graph](image)

**Figure 5.13:** Normalized tensile adhesion vs illite and montmorillonite fraction for all $t_A$ (Thewes, 1999).
Thewes conducted his tests with clay types which all had an I+M-fraction above 10%. But in Figure 5.13 the trend is extrapolated to the I+M fractions below 10%. Since all contact times give similar results it can be reasonably concluded that if no illite and montmorillonite are present (I+M-fraction is zero) a normalized tensile adhesion of below 10 kPa can be expected.

5.4.2 Correlation to the plasticity index

In the analysis of the relationship between tensile adhesion and plasticity index (I_p) Thewes (1999) found a similar strong positive correlation with the normalized tensile adhesion plotted against the plasticity index; see Figure 5.14 and Figure 5.15. From the extrapolation of the trends to the plasticity index axis, he reasonably concluded that the normalized adhesion becomes zero when the plasticity index lies between 20% and 30%.

![Figure 5.14: Normalized tensile adhesion vs plasticity index for t_A=10 min (Thewes, 1999).](image)

![Figure 5.15: Normalized tensile adhesion vs plasticity index for all t_A (Thewes, 1999).](image)
The definition of high and intermediate and low adherence potential as used in Figure 5.9 is based on experience in slurry shield tunnelling. A high adherence potential is defined as being highly problematic in which continuous operation with a conventional slurry shield tool is not possible; daily delays with manual cleaning cannot be prevented. A moderate adherence potential is defined as being problematic, but problems can be minimized by small adjustments to operation and tool. Low adherence potential clay does not cause any abnormal problems.

Based on his experimental work Thewes has found that the distinction between moderate and high adherence potential at a plasticity index of 20% corresponds to his results that the normalized tensile adhesion reaches values below 5-10 kPa at plasticity indices between 20 and 30% (Figure 5.15). Based on this he concludes that clays with a high adherence potential are expected to have normalized tensile adhesions above about 5 kPa. Clays with intermediate adherence potential are expected to have normalized tensile adhesion values below about 5 kPa.

### 5.4.3 Correlation of tensile and shear adhesion

Thewes conducted shear and tensile tests on the same clays and made correlations between the shear and tensile adhesion, see Figure 5.16. He found that the shear adhesion is always lower than the tensile adhesion and that the tensile to shear adhesion ratio lies between 2.5 and 4.

![Figure 5.16: Correlation shear and tensile adhesion (Thewes, 1999).](image)

The correlation between shear and tensile adhesion is shown in Figure 5.16. The lines represent different ratios of tensile to shear adhesion, with a ratio of 2.5 and 4 being highlighted. The graph illustrates the relationship between the two types of adhesion for different clay samples.

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5.5 Dealing with stickiness

Methods to deal with stickiness of soil fall apart in basically 3 groups as shown in Figure 5.17 (Appel, 1999):

1. Mechanical methods
2. Electrochemical methods and
3. Additives

Both preventive and curable methods are included.

![Methods to reduce the sticking of soil in a TBM](image)

**Figure 5.17**: Methods to deal with stickiness in slurry shield tunnelling (Appel, 1999).

### 5.5.1 Mechanical methods

The initial design is thought to be the single most important preventive methods of dealing with stickiness. Attention has to be paid to the design of the gravel wheel as a means to prevent and reduce the stickiness of soil. For example, avoid dead corners and angles where less flow occurs, avoid converging parts, create enough turbulence; see also chapter 5 on this. Jetting may additionally help in avoiding stickiness. Much is dependent of the possibility and the way jetting can be/is applied to an existing design. The application of jetting has not been investigated in this study due to time constrains.

### 5.5.2 Electrochemical methods

Electro-osmosis is the process in which water migrates through the clay and is liberated as a thin film at a metal clay interface (Baalen van, 1999). The basic idea is that this water film prevents the sticking of clay to the metal. The migration of water is induced by a potential applied to the clay. The cat-ions in the clay are attracted towards the cathode, and as they migrate towards the cathode they drag water along with them. This leads to the appearance of a thin layer of water at the clay-cathode interface, resulting in a loss of contact and therefore reduced adhesion between the cathode and the clay. The application of electro-osmosis has been investigated by van Baalen (1999) and it can be recommended to further investigate possibilities of applying electro-osmosis on the Gravel Wheel. Since this is beyond the scope of this project it has not been dealt with further.

### 5.5.3 Additives

Research by Appel (1999) shows that chemical additives used by him were not effective in reducing clay ball formation. Since in Gravel Wheel mining operation mostly gravel with additionally clay will be excavated the fines in the gravel can act as natural mechanical "additives". This might help in reducing the stickiness of clay lumps formed by the cutting process.
5.6 Experience from tunnelling on stickiness

Main differences of tunnelling operations with future Gravel Wheel mining operations are that tunnelling only takes place in cohesive soils while in the Gravel Wheel operation the main focus will be on the gravel and where the clay footwall should be dealt with effectively. Caution in interpreting experience of tunnelling is a must and careful translation of (application to) gravel wheel operation should be made.

Thewes (1999) identifies 4 mechanisms that are fundamental causes for clogging potential (see Figure 5.18):

1. Adhesive sticking: this causes the clay to stick to part of the tool/equipment. The tensile and shear adhesive forces are larger than the fluid drag forces acting on that piece of clay.
2. Bridging: large clay lumps may block fluid passages: the transport forces acting on the lumps are not large enough to push the lumps through the passage.
3. Cohesive sticking: due to cohesive forces, two or more clay lump coalesces and a larger clay lump results. This may cause growth of lumps stuck to parts of the tool due to adhesive sticking or they may cause growth of bridging lumps.
4. Reduced disintegration (natural, softening): lumps that have the tendency to stick are usually so stiff that they will hardly disintegrate in the transport fluids. They will remain intact and not disperse/disintegrate easily.

![Figure 5.18: Mechanisms that are fundamental causes for clogging.](image)

According to Thewes, the clogging problems with adhering clay are primarily caused by adhesive sticking and bridging. Once these mechanisms have taken place, cohesive sticking and the reduced disintegration will cause escalation of the problems. The prevention/minimization of clogging is therefore thought to lie primarily in the design and operation (mechanical methods) of the Gravel Wheel and possibly in affecting the adhesive properties (electrochemical methods or additives) of the clay.

Affecting the cohesive sticking or increasing the disintegration of the clay will be less effective and may be useless, because initial sticking has already taken place. However when the lumps are hydraulically transported, cohesive sticking should still be prevented, because the pump efficiency will be enormously reduced when clay lumps coalesce.

Disintegration of clay lumps during the hydraulic transport process is difficult, if not impossible, to affect, if not naturally taking place.

Clay lumps can cause the following problems in Gravel Wheel operation:

- Discontinuation of the operation/production due to bridging: blockage of fluid passages
- Decreases pump efficiency and lower mixture concentration due to the clay lumps
- Complicate processing and decrease recovery of diamonds
5.7 Clay lumps behaviour: formation (aggregation) or disintegration

During excavation of cohesive soils, lumps (cuttings) of different sizes are produced, see Figure 5.19. The initial size and form of the lumps depends on both the soil type and the cutting process (dictating failure type). During transport of the lumps they either aggregate or disintegrate depending on material and transport characteristics.

5.7.1 Clay lump formation
The clay lumps (also called clay balls) encountered in the discharge can have two basic origins:

1. Produced lumps from the cutting process that have remained intact during transport or
2. Clay balls that have aggregated from smaller lumps.

Investigations by Huisman (1996) indicate that in the specific tunnelling operations considered the clay is such that the aggregation process can be considered of minor importance compared to the production of clay lumps by the cutting process. This means that together with reduced/minimal disintegration the initial size of clay lumps is important. If these can be limited due to operational parameters they will cause fewer problems in operation.

Certain criteria have to be met in order for clay balls to grow (Leschensky, 1992), see Table 5.1. If these criteria are not met the clay balls will degrade, rather than grow.

<table>
<thead>
<tr>
<th>Liquid limit (%)</th>
<th>35-50 to 80-120</th>
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<tr>
<td>Plastic limit (%)</td>
<td>&gt;20-30</td>
</tr>
<tr>
<td>Density (kN/m³)</td>
<td>&gt;15-17</td>
</tr>
<tr>
<td>Shear strength (kPa)</td>
<td>&gt;25-50</td>
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Table 5.1: Criteria for growth of clay balls (after Leschensky, 1992).

It is not clear what is the basis and what are the limits for these figures' validity. It is assumed that they are based on experience.
5.7.2 Disintegration

Basically two mechanisms that cause degradation of clay balls exist, Figure 5.20 (Peter Appel, 2000):

- Softening/ natural disintegration (erosion by the slurry)
- Mechanical impact (clay breaks into smaller chunks)

Softening is the process in which water penetrates a clay lump, resulting in weakening of the surface layer of that lump (Appel, 2000). This weakening is caused by the swelling of the clay in the surface layer: the clay particles increase in distance from each other and the attractive forces decrease. Due to the water influx, the surface layer strength is reduced and collision and hydraulic erosion take place more rapidly and have more effect. The softening process itself is based on the pressure and relaxation situation and the physical-chemical interaction between clay particles and the surrounding water.

When the water content of the clay increases, the clay becomes more dispersible. Also, clay that is not compact will abrade faster because it has a lot of open spaces where the water can flow through and the surface area of clay is higher. The erosion process caused by softening can be described with the following formula (this formula is only valid when $\frac{dr}{r} < 0.1$):

$$\frac{dr}{dt} = \frac{-2 \cdot C_v \cdot (\frac{1}{1 - n_0} \cdot D_{50} \cdot (\frac{1}{1 - n_0})^{1/3} \cdot 1.8 \cdot C_u - T_{cr})}{10 \cdot D_{50} \cdot \tau_{cb} - \tau_{cr}}$$

(5.2)

In which:
- $\frac{dr}{dt}$ = change of radius of the clay ball as a function of time
- $C_v$ = consolidation effect
- $D_{50}$ = grain size underpassed by 50wt% of the grains
- $n_0$ = in-situ porosity of the clay ball
- $\tau_{cb}$ = shear stress exerted on the clay ball by the transport fluid
- $\tau_{cr}$ = shear stress below which no erosion takes place
- $C_u$ = undrained shear strength

The composition and properties (e.g. salinity) of the transport fluid are of importance to the softening process. If the salinity of the clay lump and the surrounding fluid is very different, then osmotic water flow can occur. The flowing water will promote abrasion while it presses itself through the pores. When
the salinity of the clay lump and the surrounding water is similar, osmotic processes are of minor importance (this is assumed to be the case for the gravel wheel operation).

There are more factors that influence the softening behaviour of clay. According to Arulanandan (as mentioned by Huisman, 1996), the softening properties of clay depend on the following:

- Type of clay
- Orientation of the clay particles
- Water content
- Cohesion
- Ratio of exchangeable Ca$^{2+}$/Na$^{2+}$
- Salt concentration
- pH
- Temperature

5.7.2.2 Mechanical impact

The mechanical impact can be further divided in mutual collisions and in collisions with the transport system. Collisions occur locally, whilst the softening happens on a long time-scale. Mechanical impact will result in sudden stress situations and cause breaking of clay balls, instead of a slow but steady abrasion process like softening (Appel, 2000).

A clay ball can break by shearing or by tearing. Factors that influence the breakdown of clay are the flow velocity, turbulence, strength and concentration of the clay balls. Fissuring, layering, organic content and other disturbances that influence the strength of the clay will also affect the breakdown process.
6. Loosening process – Cutting

6.1 Introduction

In general clay is more difficult to cut/dredge/handle than sand. This is due to the cohesive properties of clay (van der Schrieck, 1998).

1. It has to be cut; the productivity depends on the cutting performance.
2. The slurrification is the second important aspect with dredging. Depending on the characteristics, the cut parts can form stable lumps or not. This can lead to ball formation.

For describing the cutting of cohesive soils like clay it is important to distinguish undrained and drained failure of soil. During drained failure, pore water is allowed to flow/migrate from high water pressure to low water pressure areas (caused by dilatation). Undrained failure is characterized by the lack of such pore water migration due to high cutting velocities under normal soil cutting processes (Delft Hydraulics, 1996). See also paragraph 3.2.1 for the fundamental difference of drained and undrained failure. Cutting processes in clay as in dredging, tunnelling and gravel mining with the Gravel Wheel are thus always undrained.

In soil cutting analysis, both the 2-dimensional and 3-dimensional approach is possible. The 2-dimensional approach is valid for soil cutting tools with wide blades relative to their depths of operation, see Figure 6.1a. Mc Keys mentions that researchers (Payne, 1956) noted that when a vertical soil-cutting tool is not very wide (3-dimensional case), a large proportion of the cut soil moves sideways as shown in Figure 6.1b (McKeys, 1985).

![Figure 6.1: 2-dimensional and 3-dimensional failure (Mc Keys).](image)

Many studies have been done using the 2-dimensional approach since it is easier to do analytical calculations and perform tests. Figure 6.2 shows a side view sketch of the 2-dimensional case with the relevant parameters:

- $h =$ cutting depth
- $\alpha =$ cutting angle with horizontal
- $\beta =$ angle of shear (rupture) surface with horizontal
- $l =$ length of the blade
- $w =$ width of the blade (perpendicular to plane of the sketch; not shown)

![Figure 6.2: Side view of cutting with relevant parameters (van Vliet).](image)
6.2 Failure types

Depending on the cutting process (cutting tool design and operational parameters) and the properties of the clay (mineralogical content, strength) there are three basic failure types distinguished by Hatamura and Chijiiwa (1975), see Figure 6.3:

1. Flow type or ductile failure
2. Shear type or localization of deformation in a shear plane
3. Tear type or localization of deformation in cracks

In addition, 2 other special types are distinguished (Schrieck van der, 1998, Van Kesteren et al, 1992):

1. Brittle failure type 1
2. Brittle failure type 2

According to van Vliet (1996) only the 3 basic failure types exist and the brittle failure types are a combination of the basic failure types in combination with layering and irregularities in the soil like structures (cracks and fissures).

![Flow failure](image1)

![Brittle failure type 1](image2)

![Shear failure](image3)

![Brittle failure type 2](image4)

![Tear failure](image5)

Figure 6.3: Failure types (van der Schrieck).

**Flow type of failure**
Flow type of failure characterises itself by a continuous deformation of the soil. The production is characterized by a continuous chip (van Kesteren et al., 1992).

**Shear type of failure**
The deformation is localized in shear planes and these shear planes appear periodically from the cutting edge of the cutting tool. The production is characterized by a continuous lump/chip weakened by discrete shear planes (van Kesteren et al., 1992).

**Tear type of failure**
This failure mode is characterized by crack propagation initiated by a stress concentration at the border of the plastic zone around the cutting edge (van Kesteren et al., 1992). Different from the shear and flow type of failure, the different chips are not connected to each other.

The shear and flow type of failure are similar. The flow type can be seen as having a series of shear planes at infinitely small distance. The forces during shear type are similar to that of flow type of failure. Further the influence of clay-steel adhesion on the cutting forces is similar: at low clay-steel adhesion the cut slice curls away from the blade, while at high adhesion the clay stays on the blade. Miedema (1987) even distinguished a separate failure type: curling failure (see Figure 6.4) when the clay-steel adhesion is low.

<table>
<thead>
<tr>
<th>Document Number</th>
<th>Rev</th>
<th>Additional Reference</th>
<th>Page Number</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>61</td>
</tr>
</tbody>
</table>
6.2.1 Occurrence of failure types

Hatamura and Chijwiia (1975) did a lot of cutting tests on different types of material. The conditions under which these tests were done are:
- Cutting velocities 0.1 - 14 cm/s
- Cutting depths 2.5 - 15 cm
- Cutting angles 30° - 90°
- Cutting blade lengths 5 - 20 cm and widths 4.7 - 33 cm

They found that normally clay deforms in flow or shear type and that compacted clay deforms in tear type. More specific, quantitative conclusions were drawn but since the material types were not specified they have no meaning and are not presented. Only the qualitative conclusions are included. The cutting velocities are also far lower than the cutting velocities (1-3 m/s) dealt with in Gravel Wheel operation. This makes the results not directly usable for predictions in our case.

They performed UCS tests and tensile strength tests to clarify the correspondence of the failure types with failure conditions. In Figure 6.5 the relation between failure types and failure conditions as proposed by them is given. The figure shows that the tear type of failure occurs when the tensile strength ($\sigma_t$) of the soil is smaller than the undrained shear strength. The flow and shear type failure patterns occur when the tensile strength is larger than the cohesion of the soil. The problem is that the tensile test is normally not done in conventional soil mechanics. A standard test is not available.

![Failure conditions of soils](image)

**Figure 6.5: Relation between failure type and failure conditions (Hatamura and Chijwiia).**
Testing with different operational parameters on a certain plastic silty clay material they found that the tear type (or mixed type of tear and flow) appears when the cutting angle is smaller than 45°. And that pure flow type appears when \( \alpha \) is greater than 60°. They also found that (for a given tensile strength) the tensile stresses are large at low cutting angle. The tensile stresses decrease as the cutting angle increases, and becomes zero for the first time at \( \alpha = 90° \). From this it can be concluded that the tear type of failure is induced by this tensile stress and flow type appears when this tensile stress is small.

Van der Schrieck (1998) states that flow type of failure occurs at high deformation rates and low shear strength \((c_u<100\text{kPa})\). Deformation rate is the ratio of cutting velocity and cut depth. Shear failure type occurs at lower deformation rates and low shear strength \((c_u<100\text{kPa})\). Tear failure can occur with stronger clay \((c_u>100\text{kPa})\) and low knife angles. They occur due to the fact that the tensile strength of the material is reached. Essential is that the cutting velocity is lower than the crack propagation velocity.

Experimental research by Delft Hydraulics (van Kesteren, 1992) states that theoretically all 3 failure modes can occur in any soil and therefore the soil failure behaviour cannot be uniquely referred to as either ductile (flow failure) or brittle (tear failure). The occurrence of each failure mode and their mutual transition is not only determined by soil parameters (like strength) but also by operational parameters like cutting velocity \( (v_c) \) cutting depth \( (h) \) and cutting angle \( (\alpha) \) as shown in Figure 6.6.

![Diagram of Failure Type as Function of Deformation Rate and Cutting Angle](image)

**Figure 6.6: Failure type as function of deformation rate and cutting angle (van Kesteren).**
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---

**Deformation rate**

---

**High strength**

---

**Low strength**

---

**Tear type – flow type transition**

---

**Flow**

---

**Tear**

---

**Cutting angle**

---

**Flow type**

---

**High rate**

---

**Low rate**

---

**Figure 6.7: Influence of strength on boundary of tear to flow type of failure.**

With increase of the blade angle failure type goes from tear to flow type (or shear type) of failure (see Figure 6.6). Increase of deformation rate (at constant cutting angle) also causes a shift from tear to flow type of failure. Shear type of failure only occurs at higher cutting angles. Higher strengths of the clay move the boundary between tear and flow type of failure towards higher deformation rates (see Figure 6.7). The deformation rate at which boundary of flow to tear type occurs is approximated by a model of Delft Hydraulics and is presented in sub-paragraph 6.1.2.2 below.

Delft Hydraulics found that tear type of failure occurs when the cutting velocity does not exceed the crack propagation velocity. The propagation velocity depends on drainage of micro-cracks and fissures and that opening of such cracks can only take place if water gets the time to flow into these fissures. Otherwise the cracks do not open and flow type or shear type of failure will occur.

### 6.2.2 Tear to flow type of failure model

For the transition of flow type to shear type failure no model has been developed, since these have similar cutting forces. The transition from flow type or shear type to tear type has been modelled by Delft Hydraulics (van Vliet & van Kesteren, 1996). The model only gives an indication of the flow to tear type transition and should be used with caution. All inputs should be understood since the sensitivity of the model to some of the inputs is high.

The conditions for the occurrence of tear failure are (van Vliet & van Kesteren, 1996):

1. The deformation rate should be low
2. The cutting velocity should be smaller than the maximum crack propagation velocity
3. The cutting depth should be greater than the critical cutting depth

Next follows the way these limiting conditions can be quantified.

**Ad.1 the deformation rate should be low**

The critical deformation rate \( \frac{v}{h} \) above which tear failure will not occur can be calculated by:

\[
\frac{v}{h} = \frac{m_k k E^2}{\rho g a^2}
\]  

(6.1)
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In which:

\( v \) = cutting velocity (m/s)
\( h \) = cutting depth (m)
\( m_v \) = compressibility coefficient
\( \nu \) = poisson ratio
\( k \) = permeability coefficient (m/s)
\( E \) = Young's modulus (≈ \( 50 \times c_u \) kPa, if not measured)
\( \rho \) = density of water (kg/m\(^3\))
\( g \) = gravitation (9.81 m/s\(^2\))
\( a \) = length micro crack (m)

The compressibility coefficient is approximated by:

\[
m_v = \frac{(1-2\nu)(1+\nu)}{E(1-\nu)}
\]  
(Note: E in Pa)  

(6.2)

The micro crack length can be approximated from the void size distribution; the maximum void size is the criteria (norm) used. But the void size distribution is not known and it is assumed that the maximum void size is one tenth of the maximum particle diameter.

This approximation is also applied to clay:

\[
a \approx \frac{1}{10} d_{\text{max}}
\]  

(6.3)

Ad.2 the cutting velocity should be smaller than the maximum crack propagation velocity

If the cutting velocity is higher than the maximum crack propagation velocity, the cutting tool can outrun the crack propagation. The maximum crack propagation velocity is limited by the maximum velocity with which the plastic strain, in the zone of the crack tip, can take place. This maximum velocity can be approximated by:

\[
v_{\text{max}} = \int_0^{\varepsilon_{\text{max}}} \frac{1}{\rho} \frac{d\sigma(\varepsilon)}{d\varepsilon} d\varepsilon
\]  

(6.4)

In which:

\( \sigma(\varepsilon) \) = stress strain relation (from tri-axial tests)
\( \rho \) = density clay
\( \varepsilon_{\text{max}} \) = strain value at failure

Ad.3 the cutting depth should be greater than the critical cutting depth

At last tear failure can occur if the stress at the crack tip is large enough for unstable crack propagation to take place. For this the cutting depth should be large enough. If the cutting depth is smaller than a certain critical cutting depth no unstable crack propagation, and thus no tear failure, is possible.

The critical cutting depth can be calculated by:

\[
h_{cr} = \frac{K^2}{(b.c_u)^2}
\]  

(6.5)

In which:

\( h_{cr} \) = critical cutting depth
\( K \) = critical stress concentration factor
\( b \) = constant (5 to 6)
K can be approximated by following formula and this is valid for undrained shear strengths between 100 and 300 kPa:

\[ K = 0.13c_u \]  

(6.6)

It can be seen that when this approximation is used \( h_{cr} \) is reduced to:

\[ h_{cr} = \frac{0.13^2}{b^2} = 5.59 \times 10^{-4} \]

This means that, in determining the failure type when cutting prototype clay with cutting depths above 2.5 cm, this limiting condition is always satisfied (for tear type of failure to occur) and does not have to be explicitly mentioned. Only the first 2 conditions should be calculated.

### 6.3 Predictions of failure type

Based on theory from Hatamura and Chijwiia and given the soil characteristics (see chapter 3), both flow type and tear type of failure can be expected. The clay as found in situ is stiff to very stiff (with undrained shear strengths of 80-200 kPa) with water content near the plastic limit. Rough estimates of the tensile strength as being 10% of the UCS (with \( UCS = 2c_u \)), give possible tensile strengths of 16-40 kPa. This range falls below the range of the undrained shear strength (80-200 kPa), and thus according to Hatamura and Chijwiia tear type of failure will occur.

According to van der Schrieck, with the undrained shear strength greater than 100 kPa, tear type of failure can be expected at low blade angles, as is the case with the scoops of the Gravel Wheel.

According to van Kesteren, shear type of failure usually does not occur at low blade angles (Figure 6.6) and flow or tear type of failure can be expected in Gravel Wheel operation. An attempt was made to determine the failure type based on the model of Delft Hydraulics (presented in previous paragraph) although most the inputs required (Young’s Modulus, poisson ratio and permeability) were not available. With rough estimations of the inputs, the model predicted tear type of failure for both prototype and test clays. This estimate is considered to be unreliable since in the scaled tests only flow type of failure was observed.

It is concluded that the Eocene and Cretaceous 2 clay types (with shear strengths between 80 and 120 kPa) are more likely to show flow type of failure just like the test clays during scaled testing since they resemble the test clay types (see paragraph 3.1.2). The sandier and siltier clay types (Cretaceous 1 and 3) with expected higher shear strengths (150 – 200 kPa) have a higher chance of failing in tear mode.

In order to use the model of Delft Hydraulics in the future for failure type prediction the input parameters have to be available. It is recommended that if footwall samples are taken tri-axial compression and permeability tests are also performed.

### 6.4 Cutting forces

Cutting forces of clay depend on:

1. Ground parameters: undrained shear strength (cohesion), adhesion, layering, existence of cracks and fissures
2. Geometry of cutting tool: blade angle (\( \alpha \)), blade width (b), blade length (l)
3. Operational parameters: cutting velocity (v), cutting depth (h), (deformation rate = \( v/h \)), wear rate

\[ F_h = f(c,u,a,v,b,h,\sin\alpha,l) \]  

(6.7)

The cutting forces depend also on the type of failure that occurs. The cutting forces decrease from flow, shear to tear type of failure. In dredging it is than usual to use models for plastic type of failure being the worst case, and thus expecting an upper limit for cutting forces.
In general with higher cohesion (c) and larger cut area (b x h) the cutting forces are higher. Further the higher the deformation rate the higher the cutting force.

6.4.1 Basic cutting force prediction (Schrieck, 1998)
The most simplistic way to induce failure in clay is by means of a uni-axial compression test (see Figure 6.8).

![Simplistic failure model diagram](image)

Figure 6.8: Simplistic failure model.

For a given cohesion $c_u$ it follows that the shear strength $\sigma_1$ at failure equals:

$$\sigma_1 = 2c_u \tag{6.8}$$

Given that the shear strength is applied on the surface with dimensions $b \times h$ it follows that the force to induce failure equals:

$$F = 2c_u bh \tag{6.9}$$

Defining specific energy for cutting as $F/(b \times h)$ this specific energy equals $2c_u$ for the uni-axial compression test.

Since inducing failure in soil by means of cutting with a blade is assumed to be more difficult than the uni-axial compression test the specific energy can be expected to be higher than $2c_u$.

Generalizing above mentioned reasoning the horizontal cutting force $F_h$ can be approximated by the following formula:

$$F_h = const \times c_u bh \tag{6.10a}$$

or

$$\frac{F_h}{c_u bh} = const \tag{6.10b}$$

From tests with sharp blades in clay with plastic type of failure, empirical values as in Table 6.1 have been found for the constant as function of the deformation rate (Schrieck, 1998): In practice the effect of the sharpness of the knife and adhesion leads to a force that is 2 to 3 times higher as can be derived from Table 6.1.

<table>
<thead>
<tr>
<th>Deformation rate (v/h)</th>
<th>Constant</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>3</td>
</tr>
<tr>
<td>50</td>
<td>6</td>
</tr>
<tr>
<td>250</td>
<td>9</td>
</tr>
</tbody>
</table>

Table 6.1: Empirical values of the constant as function of deformation rate (Schrieck, 1998).
6.4.2 Simplistic cutting force model for plastic failure

In general with cutting of soil the following resisting forces have been identified (Mc Keys, 1985):

1. Gravitational forces
2. Frictional forces
3. Cohesion
4. Adhesion
5. Surcharge pressure (if present)

During cutting of clay, the major resisting forces acting are the cohesive and adhesive forces. Since the cohesion (and adhesion) forces are much larger than the gravitational force, the latter may be neglected. In addition, many clay soils show little internal friction so that the internal frictional forces may be disregarded too.

Based on the previous simplification a simplistic model has been developed to predict cutting forces for plastic clays that fail in plastic or shear mode. This model has been developed based on the following assumptions (see Figure 6.9):

1. Failure occurs along a straight shear plane in which the shear resistance equals the cohesion
2. Movement occurs along the cutting blade with the adhesion as the shear resistance.
3. Apart from the two above-mentioned shear forces only normal forces work on the shear plane and the cutting blade.

The horizontal cutting force relation can be found (see Figure 6.2 for some of parameters):

\[ F_{cut, \text{horizontal}} = c w h \left( \frac{h^* \sin \alpha}{h \sin \beta \sin(\alpha + \beta)} + \frac{a h^* \sin \beta}{h \sin \alpha \sin(\alpha + \beta)} \right) \]  \hspace{1cm} (6.11)

In which:
- \( F \) = cutting force
- \( c \) = undrained shear strength
- \( w \) = cutting width
- \( h \) = cutting depth
- \( h^*/h \) = empirical correction due to deformation of clay in front of knife (see Figure 6.10)
- \( a \) = adhesion

![Figure 6.9: Simplistic cutting force prediction model.](image)
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\[ h_b = \text{cutting blade height} \]
\[ \alpha = \text{cutting angle} \]
\[ \beta = \text{angle of shear surface with horizontal} \]

\[ v = \frac{v_s}{h} \]
\[ t = \tau_y + \tau_0 \ln(1 + \frac{v_s}{h}) \]
\[ \varepsilon_0 \]

**Figure 6.10: Empirical correction \( h' / h \).**

Miedema (1989) modified his fundamental physical model on cutting of sand for application in clay and derives the same equation above. In addition to that he gives a deformation rate dependent cohesion and adhesion (equation 6.12):

\[
\tau = \tau_y + \tau_0 \ln(1 + \frac{v_s}{h})
\]

In which:

\[ \frac{v_s}{h} = \text{deformation rate} \]
\[ v_s = \text{cutting velocity} \]
\[ \tau = \text{deformation rate dependent cohesion or adhesion} \]
\[ \tau_y = \text{deformation rate independent cohesion or adhesion} \]
\[ \tau_0 = \text{dynamic factor} \]
\[ \varepsilon_0 = \text{frequency} \]

Equation 6.12 gives the dependence of the deformation rate on the shear strength; \( \tau \) can be either the cohesion (c, undrained shear strength) or adhesion (a).

**6.5 Conclusions**

It is concluded that the Eocene and Cretaceous 2 clay types are more likely to show flow type of failure just like the test clays during scaled testing since they resemble the test clay types. The sandier and siltier clay types (Cretaceous 1 and 3) with expected higher shear strengths have a higher chance of failing in tear mode.

Attempts to use Delft Hydraulics' model to predict failure type was unsuccessful due to lack of known input parameters. In order to use the model of Delft Hydraulics in the future for failure type prediction the input parameters have to be available. It is recommended that if footwall samples are taken tri-axial compression and permeability tests are also performed.
7. Scale modelling of clay-cutting tool interaction process

7.1 Introduction

Scale models are experimental models structured to mirror the true physical behaviour of an original phenomenon, or a prototype that for some reason (e.g. too large, too expensive and unmanageable) cannot be explored on the prototype level (Schuring, 1977).

Scaling applies not only to linear dimensions but also to all other important dimensions like time, force, and density, in scaling called 'quantities'. If the prototype is scaled correctly, forces, velocities, deformations, and all other relevant quantities measured on the scale model permit predictions of the corresponding quantities of the prototype.

The same physical laws governing the prototype must prevail in the model. Unfortunately it is not always possible to satisfy this demand, and this deficiency is the main disadvantage of scale modelling. Still valuable information can be derived from such models by making simplifications. For example a process can result in a dysfunction of a tool; by exaggerating this process in the model, it will be hampered more severely than the prototype. If the model still performs under these circumstances conclusion can be drawn that the prototype will certainly do so.

Scale modelling can be advantageous for 4 reasons:
1. The problem is too complex or too little explored to be amenable to an analytical solution.
2. Scale models permit transformation to manageable proportions of systems that may be too large or expensive for direct experimentation
3. Scale modelling shortens experimentation
4. Scale modelling promotes a deeper understanding of the phenomenon under investigation

Scale model testing cannot be performed without insight into the given phenomenon, an insight that will grow into an understanding of the basic structure of the system as the model tests progress. Hence, scale model experiments are qualified to help establish both the design and the fundamental nature of new engineering hardware.

7.2 Principles of scaling

7.2.1 Scale factors
A scale factor of a physical dimension or quantity (e.g. length) can be defined as its prototype value divided by its model value:

\[
\text{scalefactor} = \frac{\text{quantity - prototype}}{\text{quantity - model}} \tag{7.1}
\]

For example the length scale factor is expressed as:

\[
n_L = \frac{L_{\text{prototype}}}{L_{\text{model}}} = \frac{L_P}{L_M} \tag{7.2}
\]

In which
\[
n_L = \text{length scale factor} \\
L_{\text{prototype}} = \text{length of prototype} \\
L_{\text{model}} = \text{length of model}
\]

As can be seen the scale factor will be unity when a physical dimension or quantity is of equal magnitude for prototype and model.

The five 'primary' quantities as defined by The International Systems of Units are length, time, force, temperature and electric current.
In scale modelling, we are concerned only with ratio of quantities defined as products of five ‘primary’ quantities to the appropriate power. Therefore, only the primary scale factors of these five quantities need to be accounted for; all other ‘secondary’ scale factors are easily derived from them. This is a consequence of physics in combination with our standard system of measurement; all other quantities can be expressed in these five primary quantities.

See Appendix 7 for a table of some secondary quantities in terms of primary quantities (Schuring, 1977). This table is also valid for their scale factors as will be explained in paragraph 7.2.2.

Some of the major physical fields amenable to scale modelling are listed in Table 7.1, arranged according to the number of participating primary scale factors. The phenomena we have to deal with in modelling the mining processes of the Gravel Wheel, belong to the fields of geometry, kinematics and dynamics and thus only the primary scale factors of length, time and force have to be considered.

<table>
<thead>
<tr>
<th>Unit</th>
<th>Primary scale factors</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>length</td>
</tr>
<tr>
<td>Geometry</td>
<td>x</td>
</tr>
<tr>
<td>Kinematics</td>
<td>x</td>
</tr>
<tr>
<td>Statics</td>
<td>x</td>
</tr>
<tr>
<td>Dynamics</td>
<td>x</td>
</tr>
<tr>
<td>Thermodynamics</td>
<td>x</td>
</tr>
<tr>
<td>Heat and mass transfer</td>
<td>x</td>
</tr>
<tr>
<td>Electrostatics</td>
<td>x</td>
</tr>
<tr>
<td>Magneto-hydrodynamics</td>
<td>x</td>
</tr>
</tbody>
</table>

Table 7.1: Participating primary scale factors for several major physical fields.

### 7.2.2. Dimensionless products or pi-numbers and scaling laws

Scale modelling requires that the same physical laws or equations govern prototype and model. In practice this is achieved by generating dimensionless ratios of quantities, which are equal for both the prototype and the model (Schuring, 1977). The concept of dimensionless ratio of quantities will be explained by the example below.

Take for instance Newton’s law of inertia; force is mass times acceleration:

\[ F = ma \]  

(7.3)

With \( m \propto \rho V \propto \rho t^3 \), \( a \propto \frac{v}{t} \), \( v \propto \frac{l}{t} \) and \( t \propto \frac{1}{v} \) this can be rewritten as:

\[ F \propto \rho l^2 v^2 \]  

(7.4)

In which: \( v \) = velocity, \( l \) = length, \( t \) = time, \( m \) = mass, \( a \) = acceleration, \( F \) = force, \( \rho \) = density.

If this law governs both prototype and model then:

\[ F_p \propto \rho_p l_p^2 v_p^2 \quad \text{and} \quad F_M \propto \rho_M l_M^2 v_M^2 \]  

(7.5)

Taking the ratio of forces in prototype and model the ratio becomes:

\[ \frac{F_p}{F_M} = \frac{\rho_p l_p^2 v_p^2}{\rho_M l_M^2 v_M^2} \]  

(7.6)

With all quantities of prototype on one side and that of model on the other of the equation, this relation converts into:
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\[
\frac{F_p}{\rho_p l_p^2 v_p^2} = \frac{F_M}{\rho_M l_M^2 v_M^2} = \pi
\]  \hspace{1cm} (7.7)

The dimensionless products are called pi-numbers, denoted by the Greek letter \( \pi \), and since they are equal for prototype and model the subscripts can be left out. This \( \pi \) number could have been directly derived from Newton’s law, equation 7.4.

\[
\pi = \frac{F}{\rho l^2 v^2}
\]  \hspace{1cm} (7.8)

There are common and principle \( \pi \) numbers. The relation above is an example of a principle \( \pi \) number, the Newton number. Other well-known examples of other principle \( \pi \) numbers are the Froude- and Reynolds-number.

These \( \pi \) numbers can also be written in terms of scale factors, called scale laws:

\[
n_F = n_p n_l^2 n_v^2
\]  \hspace{1cm} (7.9)

In conclusion, if a law or equation governing a process is identified, a scale law can be derived directly as follows:

\[
F \propto \rho l^2 v^2 \Rightarrow \pi = \frac{F}{\rho l^2 v^2} \Rightarrow n_F = n_p n_l^2 n_v^2
\]  \hspace{1cm} (7.10)

How scale modelling is applied to a real problem will be demonstrated by an example. Take for example a simple block (prototype) with dimensions \( l \times l \times l \) at rest on a table (Figure 7.1). We want to know the force \( F_p \) that is necessary to make the block start moving. Although an analytical solution is available, for the sake of this example it is assumed there is not. Further assumption made is that the force will have to be calculated by measurement of the force \( F_M \) on a scale model. The question to be answered is: what is the force scale factor? The force scale factor will be derived with scale modelling principles.

First we have to look at which physical laws are acting on the block:

1. Newton’s law of gravity \( F_g = m \, g \)
2. Law of frictional resistance \( F_R = \mu \, F_g \)

![Figure 7.1: Example of simple block at rest on surface.](image)
These laws are rearranged in terms of primary quantities as shown in Table 7.2. From these relations the governing scale laws can be derived.

<table>
<thead>
<tr>
<th>Physical laws</th>
<th>Representative laws in terms of primary quantities</th>
<th>Scale laws</th>
<th>Elimination of common scale factor(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( F_g = mg = \rho gV_g )</td>
<td>( F_g = \rho g l^3 )</td>
<td>( n_{F_g} = n_\rho n_g n_l^3 )</td>
<td>( n_{F_g} = n_\rho n_g n_l^3 )</td>
</tr>
<tr>
<td>( F_p = F_R = \mu F_g )</td>
<td>( F_R = \mu F_g )</td>
<td>( n_{F_R} = n_\rho n_F )</td>
<td>( n_{F_R} = n_\rho n_F )</td>
</tr>
</tbody>
</table>

Table 7.2: Scale laws of simple block example.

Considering the gravitational acceleration to be equal for prototype and model, \( n_g \) equals unity, which results in the following force scale law:

\[
\frac{n_{F_R}}{n_{F_g}} = n_\rho n_\mu n_l^3 \tag{7.11}
\]

Assume for example that \( n_\mu \) and \( n_\rho \) are unity.

If we take a geometrical model with a length scale of 3 (\( n_l = 3 \)) the force required to move the prototype will be 27 times the force measured in the model.

In scale modelling dimensionless products play a key role. As mentioned above, in modelling the mining processes of the Gravel Wheel belong to the fields of geometry, kinematics and dynamics. The dimensionless products have thus to be equal for model and prototype to:

A. assure geometrical similarity between prototype and model

It follows that all dimensions are equally scaled down by the same factor resulting in the following condition:

\[
n_s = n_p = n_\rho = n_l
\]

B. assure kinematic and dynamic similarity between prototype and model

Kinematic and dynamic similarity is required to scale all processes correctly. To correctly reproduce for example the flow path of particles in hydraulic transport, all forces acting on that transported particle like the drag force and the gravity force should be correctly scaled in equal proportions. This is achieved by making use of scale laws with force ratio scale factors to match ratio of forces on model scale to those on full scale. See Figure 7.2 for an illustration of such force ratios.

![Figure 7.2: Illustration of force ratio's.](image-url)
For kinematic and dynamic similarity between prototype and model the force ratio scale factors (for example $\frac{n_{F_{\text{ad}}}}{n_{F_{\text{drag}}}}$ and $\frac{n_{F_{\text{g}}}}{n_{F_{\text{drag}}}}$) should be 1. A scale factor below 1 means that the force in the nominator of the ratio is overrepresented or not fully scaled back in the model (or force in denominator is underrepresented). A scale factor above 1 means that the force is underrepresented in the model.

### 7.2.3 Scale effects

If more physical laws, and thus more than one scale factor, are involved in a scaling problem, correct scaling may be difficult due to conflicts between the different interrelations among the primary scale factors.

Take again for example the hydraulic transport of particles in a flow.

![Diagram showing forces and flow direction](image)

The physical laws/forces that can be identified are:

1. Inertial forces, $F_i$
2. Gravitational forces, $F_g$
3. Fluid drag forces, $F_{\text{drag}}$

In Table 7.3 the scale laws and scaling requirements are derived from the identified physical laws/forces.

<table>
<thead>
<tr>
<th>Physical laws</th>
<th>Representative laws in terms of primary quantities</th>
<th>Scale laws</th>
<th>Force ratio scale laws</th>
<th>Scaling requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 $F_i = ma$</td>
<td>$F_i \propto \rho l^2 v^2$</td>
<td>$n_{F_i} = n_{\rho} n_i^2 n_v^2$</td>
<td>$n_{F_i} = \frac{n_{\rho}}{n_g} n_i^2 n_v^2$</td>
<td>$n_v = \sqrt{n_i}$</td>
</tr>
<tr>
<td>2 $F_g = mg = \rho Vg$</td>
<td>$F_g \propto \rho gl^3$</td>
<td>$n_{F_g} = n_{\rho} n_g n_i^3$</td>
<td>$n_{F_i} = \frac{n_{\rho}}{n_g} n_i$</td>
<td>$n_{F_i} = \frac{n_{\rho}}{n_g} n_i^2$</td>
</tr>
<tr>
<td>3 $F_{\text{drag}} = C_D \frac{1}{2} \rho v^2$</td>
<td>$F_{\text{drag}} \propto C_D \rho l^2 v^2$</td>
<td>$n_{F_{\text{drag}}} = n_{C_D} n_{\rho} n_i^2 n_v^2$</td>
<td>$n_{F_{\text{drag}}} = \frac{1}{n_{C_D}}$</td>
<td>$n_{C_D} = 1$</td>
</tr>
</tbody>
</table>

Table 7.3: Derivation of scale laws and scaling requirement for illustration of scale effects.

The first scaling requirement, $n_v = \sqrt{n_i}$, is also known as the Froude scaling condition, with the Froude number being the ratio of the inertia and gravity force:

$$Froude = \frac{\rho l^2 v^2}{\rho g l^3} = \frac{v}{\sqrt{gl}} \quad (7.12)$$
Since the drag coefficient $C_D$ is a function of the Reynolds number which is equal to $\text{Re} = \frac{\rho v d}{\eta}$ the second requirement that $n_{C_D} = 1$ can only be satisfied if $n_{Re} = 1$. That's leads to the following requirement with $n_{\rho} = 1$ and $n_\eta = 1$, known as the Reynolds scale condition:

$$n_{Re} = \frac{n_{\rho} n_{\eta} n_i}{n_\eta} = 1$$  \hspace{1cm} (7.13)$$

$$n_v = \frac{1}{n_i}$$  \hspace{1cm} (7.14)$$

The two derived scale conditions $n_v = \sqrt{n_i}$ and $n_v = \frac{1}{n_i}$ cannot be satisfied simultaneously. This is known as a scale effect.

### 7.2.4 Relaxation of model design requirements

The above-mentioned conflicts or scale effects could be dealt with by what is called relaxation. Relaxations might mean identifying the essential physical laws/forces and less important/weak physical laws/forces, and than neglecting/disregarding the weak ones. Or the phenomenon can be divided into smaller, manageable parts that are studied separately. Carrying on with the above mentioned example relaxation will be demonstrated. The drag coefficient is dependent of the flow regime (laminar or turbulent) which can be predicted by the Reynolds number as shown in Figure 7.3.

![Figure 7.3: Drag coefficient as function of the Reynolds number.](image)

If the flow is turbulent with Reynold numbers above $4 \times 10^5$ the drag coefficient is independent of the Reynolds number and is constant. This means that if the flow in both model and prototype is turbulent the requirement that $n_{C_D} = 1$ is satisfied. From this follows that the Reynolds scale condition $n_v = \frac{1}{n_i}$ can be disregarded and the scale conflict recognized before has been dealt with.
7.2.4.1 Application to Gravel Wheel's operating condition

To calculate the Reynolds number for full scale conditions the following input values have been selected based on data supplied by De Beers Marine:

\[ d_p = 22.5 \text{mm} \]
\[ v_p = 6 \text{m/s} \]  

Calculating the Reynolds number results in:

\[
Re_p = \frac{\rho \cdot d_p \cdot v_p}{\eta_p} \quad \text{or with } (v_p = \frac{\eta_p}{\rho}) \Rightarrow Re_p = \frac{d_p \cdot v_p}{\nu_p} \approx \frac{0.0225 \times 3}{1.48 \cdot 10^{-6}} \approx 4.6 \times 10^4 (-) \tag{7.16}
\]

In which:

\[ \nu_p = \frac{40 \cdot 10^{-6}}{20 + T} \text{ (m}^2/\text{s}) \text{ and } T \text{ in degrees Celcius} \]
\[ d_p = 0.0225 \text{m} \]
\[ v_p \approx 6 \text{m/s} \]
\[ T_p = 7^\circ C \]

When Froude is used to scale full scale conditions to an arbitrary chosen 1/3 and 1/4 scale, the calculations that apply to identify the hydraulic model conditions and the resulting Reynolds numbers for the 2 model scales are shown in Table 7.4.

<table>
<thead>
<tr>
<th>Scale conditions</th>
<th>Full scale</th>
<th>Model 1/3 scale</th>
<th>Model 1/4 scale</th>
</tr>
</thead>
<tbody>
<tr>
<td>n_i = 3</td>
<td>n_i = \sqrt{n_i}</td>
<td>n_i = \sqrt{n_i}</td>
<td>n_i = \sqrt{n_i}</td>
</tr>
<tr>
<td>d (m)</td>
<td>d_p = 0.0225</td>
<td>d_M = \frac{d_p}{3} = 0.008</td>
<td>d_M = \frac{d_p}{4} = 0.006</td>
</tr>
<tr>
<td>v (m/s)</td>
<td>v_p = 6</td>
<td>v_M = \frac{v_p}{\sqrt{3}} \approx 3.46</td>
<td>v_M = \frac{v_p}{\sqrt{4}} \approx 3.00</td>
</tr>
<tr>
<td>T (°C)</td>
<td>T_p = 7</td>
<td>T_M = 10</td>
<td>T_M = 10</td>
</tr>
<tr>
<td>Re (-)</td>
<td>Re_p \approx 4.6 \times 10^4</td>
<td>Re_M \approx \frac{0.008 \times 3.46}{1.33 \cdot 10^{-6}} \approx 9.1 \times 10^4</td>
<td>Re_M \approx \frac{0.006 \times 3}{1.33 \cdot 10^{-6}} \approx 1.3 \times 10^4</td>
</tr>
</tbody>
</table>

Table 7.4: Reynolds number for 1/3 and 1/4 model scale.

From the above it can be concluded that the numerical calculations validate the C_D coefficient to be independent of the Reynolds number for both model and full scale (\(Re_M \land Re_p \geq 4.10^3\)) conditions when a 1/3 or 1/4 scale is applied.
7.3 Modelling of clay – Gravel Wheel interaction process

Given the complexity of the problem of scaling the whole mining process of the Gravel Wheel, it is obvious that many physical laws will have to be satisfied. This means that a number of scale factors will have to be satisfied causing conflicts that have to be minimized by relaxation. This has been studied by Theo van der Werken (2001). In his study van der Werken only dealt with gravel and did not include clay; he identified 6 physical processes: sedimentation, erosion, cutting of sediment, breaching, density currents, pump suction and jet stowing for testing in gravel. The purpose of this study is the sealing of clay in order to study the effects that clay has on the Gravel Wheel operation.

Three main situations in which clay occurs are identified. For each, the governing physical processes/forces (see Figure 7.4) can be identified in and around the Gravel Wheel:

I. Cutting of clay
   a. Required cutting force
   b. Available force/drive power on cutting edge

II. Slurrification
   a. Failure of clay determining the primary shape of the clay lumps
   b. Force to break clay in lumps
   c. Strain at point failure
   d. Hydraulic transport

III. Hydraulic transport
   a. Hydraulic transport
   b. Gravitation
   c. Adherence
   d. Force to mould clay lumps

Figure 7.4: Forces acting in and around the Gravel Wheel.

The following steps have to be followed in designing model experiments:
1. Derive physical laws/equations that govern/describe the physical processes
2. Derive scaling laws
3. Derive primary scale factor relations
4. Design scaled tests
7.4 Derivation of scaling laws for identified physical processes

7.4.1 Cutting of clay
The simplistic cutting force model as introduced in paragraph 6.4.2 (equation 6.11) will be used to describe the cutting process during Gravel Wheel operation. This model predicts cutting forces when clays fail in plastic or shear mode. Cutting forces in tear mode of failure are lower than in plastic and shear mode of failure (this while cutting clay with the same shear strength), thus the model presents an upper limit to the cutting force (see paragraph 6.4). This simplification is seen as adequate since the cutting force is usually not a limiting factor in testing.

The ratio \( \frac{h'}{h} \) in equation 6.11 is assumed to be one and the relative adhesion (adhesion/cohesion ratio) to be constant. This simplifies equation 6.11 to the following simple representative form:

\[
F_{cut} \propto cl^2 \Rightarrow n_{Fcut} = n_c n_i^2
\]  
(7.17)

The force \( F_{drive} \) that is delivered to the cutting edge and which has to be equal to the cutting force \( F_{cut} \) can be derived by the following formula:

\[
P \propto F_{drive} \cdot v_i = F_{cut} \cdot v_i \Rightarrow n_p = n_{Fcut} n_v \Rightarrow n_p = n_c n_i^2 n_v
\]  
(7.18)

\( P \) is the power supplied to the cutting edge.

Inertial forces have always to be accounted for and the following scale law is derived:

\[
F_i = ma \Rightarrow F_i \propto pl^3 v^2 \Rightarrow n_{F_i} = n_p n_i^2 n_v^2
\]  
(7.19)

7.4.2 Slurriification

7.4.2.1 Failure type determining the primary shape of the clay lumps
The failure type is a function of the shear strength (cohesion), deformation rate, \( \frac{v_i}{h} \) and cutting angle (see paragraph 6.2). Their effects on failure type cannot be ignored and is accounted for by checking in both prototype and model that the failure type of the clay is similar based on the theory presented in paragraph 6.2. It was mentioned in paragraph 6.2 that using Delft Hydraulics' model to predict the failure type in both prototype and model failed due to lack of input data.

Since the type of failure is a function of the cohesion, deformation rate and cutting angle a reasonable attempt to match the failure type in the scaled and prototype operation would be to:

1. Use a test clay with a similar cohesion as the footwall clay types.
2. Match deformation rate.
3. Cut with the same cutting angle.

This can be quantified by the following scaling laws:

Similar failure type \( \Rightarrow n_c = 1, n_v = 1 \) and \( n_a = 1 \)

\[
(7.20)
\]

To better take the effect of the shear strength on the failure type into account the following scaling law, which makes common sense, can be derived. For clay to behave mechanically similar under different conditions (thus also during cutting) it should be of similar consistency \( \left( \frac{w}{w_L} \right) \). This makes common sense since clay with different consistencies differ in what has been called the "mechanical mouldability" (see paragraph 3.1.3.1). This translates to the following scale law, substituting the shear strength scaling law \( n_c = 1 \):

\[
\]
Similar consistency $\Rightarrow n_{lc} = 1$  \hspace{1cm} (7.21)

7.4.2.2 Force to break the clay in lumps

The formation of clay lumps is governed by the force necessary to break the clay in lumps (by hydraulic force). This force, $F_b$, is considered of similar form as the cutting force from equation 7.17.

$$F_b = cl^2 \Rightarrow n_{fb} = n_c n_i$$  \hspace{1cm} (7.22)

7.4.2.3 Strain at point of failure

When clay becomes less consistent it behaves more plastic and less brittle. This results in a higher strain before failure takes place and this effect has been called the strain at point of failure. Assume the strain situation as shown in Figure 7.5. The cut clay lump cannot extend into the infinite and due to e.g. gravitational and hydraulic forces it will break off at certain points into separate lumps. The strain at point of failure is a function of the plasticity of the clay; a more plastic (and less consistent) clay will show a higher strain to failure.

![Figure 7.5: Stretch to point of failure.](image)

Suppose a curvature radius $R$. From theory of mechanics it is known that the maximum strain $\varepsilon_{max}$ at $R+\frac{1}{2}h$ (and its resulting representative form) is:

$$\varepsilon_{max} = \frac{1}{2} \frac{h}{R} \Rightarrow n_{e_{max}} = n_{\varepsilon_{max}} = \frac{n_h}{n_R}$$  \hspace{1cm} (7.23)

The displacement $\Delta L$ of the lump with length $L$ is approximately equal to $d\varphi \times L$.

For geometrical similarity the following scale laws apply:

$$n_{e_{max}} = n_{\varepsilon_{max}} = 1 \text{ and } n_{\varphi} = n_{\Delta L} = 1$$  \hspace{1cm} (7.24)
7.4.2.4 Fluid drag force

\[ F_{\text{drag}} = C_D A \frac{1}{2} \rho v^2 \Rightarrow F \propto C_D \rho l^2 v^2 \Rightarrow n_{F_{\text{drag}}} = n_{C_D} n_\rho n_l^2 n_v^2 \]  

Fluid pressures and pressure differentials are important in hydraulic transport and as such the pressure is written in terms of scale factors as follows:

\[ \Delta P = \frac{F_{\text{drag}}}{A} \Rightarrow n_{\Delta P} = \frac{n_{C_D} n_\rho n_l^2 n_v^2}{n_l^2} = n_{C_D} n_\rho n_v^2 \]  

(7.25)

(7.26)

7.4.3 Hydraulic transport

Different from the cutting process are the processes taking place when the soil already has been cut and formed into lumps. The following physical laws/equations/forces govern the remaining processes during hydraulic transport; they are already rewritten in terms of primary quantities:

- Fluid drag force
  See paragraph 7.4.2.4.

- Gravity (soil weight)
  \[ F_g = mg = \rho Vg \Rightarrow F_g \propto \rho gl^3 \Rightarrow n_{F_g} = n_\rho n_g n_l^3 \]  

(7.27)

- Adhesion
  The process of adherence of clay to a steel surface is presented here in the simplest way. The adhesive shear strength \( \tau_a \) (adhesion) of a piece of clay to a surface can be simply written as:
  \[ \tau_a = a + \sigma_a \tan \phi \]  
  Friction angles \( \phi \) in clay are low and can be neglected.
  Thus, the force required to overcome the adhesion to a surface area \( A \) can be written as:
  \[ F_{\text{ad}} = aA \Rightarrow F_{\text{ad}} \propto a l^2 \Rightarrow n_{F_{\text{ad}}} = n_a n_l^2 \]  

(7.28)

(7.29)

- The force to mould the clay lumps, \( F_m \), can be written as:
  \[ F_m = cl^2 \Rightarrow n_{F_m} = n_c n_l^2 \]  

(7.30)
Summarizing the above derived scale laws they are listed in Table 7.5.

<table>
<thead>
<tr>
<th>Process/force</th>
<th>Scaling laws</th>
<th>Elimination of constant and common quantities (see notes below)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cutting of clay</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Inertial forces</td>
<td>( n_{Fi} = n_p n_l^2 n_v^2 )</td>
<td>( n_{Fi} = n_l^2 n_v^2 )</td>
</tr>
<tr>
<td>Required cutting force</td>
<td>( n_{Fc} = n_p n_l^2 )</td>
<td>( n_{Fc} = n_p n_l^2 )</td>
</tr>
<tr>
<td>Delivered power on cutting</td>
<td>( n_p = n_c n_l^2 n_v )</td>
<td>( n_p = n_c n_l^2 n_v )</td>
</tr>
<tr>
<td>edge</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slurrification</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fluid drag force</td>
<td>( n_{Fdrag} = n_{CD} n_p n_l^2 n_v^2 )</td>
<td>( n_{Fdrag} = n_l^2 n_v^2 )</td>
</tr>
<tr>
<td>Failure type</td>
<td>( n_v = 1, n_a = 1 )</td>
<td>( n_v = 1, n_a = 1 )</td>
</tr>
<tr>
<td>Consistency</td>
<td>( n_{l_c} = 1 )</td>
<td>( n_{l_c} = 1 )</td>
</tr>
<tr>
<td>Force to break clay in lumps</td>
<td>( n_{Fb} = n_c n_l^2 )</td>
<td>( n_{Fb} = n_c n_l^2 )</td>
</tr>
<tr>
<td>Strain at point of failure</td>
<td>( n_{h/R} = 1, n_{SL/L} = 1 )</td>
<td>( n_{h/R} = 1, n_{SL/L} = 1 )</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hydraulic transport</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fluid drag force</td>
<td>( n_{Fdrag} = n_{CD} n_p n_l^2 n_v^2 )</td>
<td>( n_{Fdrag} = n_l^2 n_v^2 )</td>
</tr>
<tr>
<td>Gravity</td>
<td>( n_{Fg} = n_p n_g n_l^3 )</td>
<td>( n_{Fg} = n_l^3 )</td>
</tr>
<tr>
<td>Adhesion</td>
<td>( n_{Fad} = n_a n_l^2 )</td>
<td>( n_{Fad} = n_a n_l^2 )</td>
</tr>
<tr>
<td>Force to mould clay lumps</td>
<td>( n_{Fm} = n_c n_l^2 )</td>
<td>( n_{Fm} = n_c n_l^2 )</td>
</tr>
</tbody>
</table>

Table 7.5: Derived scaling laws.

Notes:

1) It is assumed that both the density scale factor \( n_p \) and gravitational acceleration scale factor \( n_g \) are equal to unity.

2) In paragraph 7.2.4.1 it was shown that the drag coefficient \( C_D \) is independent of the Reynolds number \( Re \) (\( Re_{M} < Re_{P} \geq 4 \times 10^3 \)) when testing on 1/3 and 1/4 scale, thus \( C_D \) is the same in both prototype and model: \( n_{C_D} = 1 \).

3) \( n_l \) is here initially the scale factor of the clay lump sizes, \( l \) is a dimension of the size of the clay lumps. Since the clay lump size can be expected to decrease as with the tool size, \( n_{l-clay\, lump} \) equals \( n_{l-tool} \).
7.5 Derivation of primary scale factor relations

Making use of force ratio scale factors Table 7.6 has been constructed from Table 7.5 in order to scale the processes in the different situations correctly.

<table>
<thead>
<tr>
<th>Situation</th>
<th>Forces</th>
<th>Scale laws</th>
<th>Force ratio scale factors</th>
<th>Necessary requirements for scaled testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cutting of clay</td>
<td>Inertial force</td>
<td>$n_F = n_i^2 n_v^2$</td>
<td>$n_{Fcut} = n_c n_i^2$</td>
<td>$n_c = n_v^2$</td>
</tr>
<tr>
<td></td>
<td>Required cutting force</td>
<td></td>
<td>$n_{Fcut} = \frac{n_c}{n_I} n_v^2$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Delivered power on cutting edge</td>
<td></td>
<td>$n_p = n_c n_i^2 n_v$</td>
<td>$n_p = n_c n_i^2 n_v$</td>
</tr>
<tr>
<td>Slurrification</td>
<td>Fluid drag force</td>
<td>$n_{Fdrag} = n_i^2 n_v^2$</td>
<td>$n_{Fdrag} = n_c n_i^2$</td>
<td>$n_c = n_v^2$</td>
</tr>
<tr>
<td></td>
<td>Force to break clay in lumps</td>
<td></td>
<td>$n_{Fb} = n_c n_i^2$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Failure type</td>
<td></td>
<td>$n_{Fb} = \frac{n_c}{n_i} n_v^2$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Consistency</td>
<td></td>
<td>$n_{lc} = 1$</td>
<td>$n_{lc} = 1$</td>
</tr>
<tr>
<td></td>
<td>Strain at failure</td>
<td></td>
<td>$n_h = 1, n_{\frac{ML}{L}} = 1$</td>
<td></td>
</tr>
<tr>
<td>Hydraulic transport</td>
<td>Fluid drag force</td>
<td>$n_{Fdrag} = n_i^2 n_v^2$</td>
<td>$n_{Fdrag} = n_c n_i^2$</td>
<td>$n_c = n_v^2$</td>
</tr>
<tr>
<td></td>
<td>Gravity</td>
<td></td>
<td>$n_{Fg} = n_i^3$</td>
<td>$n_{Fg} = \frac{n_i}{n_v^2}$</td>
</tr>
<tr>
<td></td>
<td>Adhesion</td>
<td></td>
<td>$n_{Fad} = n_a n_i^2$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Force to mould clay lumps</td>
<td></td>
<td>$n_{Fm} = n_c n_i^2$</td>
<td></td>
</tr>
</tbody>
</table>

Table 7.6: Derivation of force ratio scale factors and necessary testing requirements.

Setting the force scale ratios to 1 gives the requirements necessary for scaled testing in the last column. For correct scaling the following requirements have thus to be satisfied:

Requirement 1: $n_i = 1, n_a = 1$

Requirement 2: $n_{lc} = 1$

Requirement 3: $n_v = \sqrt{n_i}$

Requirement 4: $n_a = n_v^2$

Requirement 5: $n_c = n_v^2$

Requirement 6: $n_h = 1, n_{\frac{ML}{L}} = 1$
Satisfying requirement 3 (for correctly scaling the drag and gravity forces) and requirements 4 and 5 (for the cohesion and the adhesion) at the same time gives an additional requirement: \( n_a = n_c = n_f \). This practically means that the adhesion and cohesion of the clay should be scaled down by the length scale factor. By scaling the cohesion down it will be impossible to satisfy requirement 2 of similar consistency, since the consistency is strongly correlated with the cohesion (paragraph 3.2.1). Also the clay will be more plastic with a higher strain at failure (requirement 6). This conflict between different scaling requirements will result in scale effects when performing scale tests.

7.6 Evaluation of scale effects

To evaluate the scale effects that can occur, different scale scenarios are designed to reduce the scale effects as much as possible. These scale scenarios and the magnitudes of the subsequent scale effects are summarized in Table 7.7. In the first row the chosen test conditions for each scenario are listed and then the resulting magnitude of the testing requirements follows. The different scenarios and their effects will be discussed in following subchapters.

<table>
<thead>
<tr>
<th>Scenario 1</th>
<th>Scenario 2</th>
<th>Scenario 3a</th>
<th>Scenario 3b</th>
</tr>
</thead>
<tbody>
<tr>
<td>- Scaled &quot;Froude&quot; velocities</td>
<td>- Prototype clay velocities</td>
<td>- Scaled &quot;Froude&quot; velocities</td>
<td>- Scaled &quot;Froude&quot; velocities</td>
</tr>
<tr>
<td>- Scaled clay; cohesion and adhesion scaled</td>
<td>- Prototype clay</td>
<td>- Prototype clay</td>
<td>- Scaled clay; only adhesion scaled</td>
</tr>
<tr>
<td>( n_L = 3 )</td>
<td>( n_L = 3 )</td>
<td>( n_L = 3 )</td>
<td>( n_L = 3 )</td>
</tr>
<tr>
<td>( n_v = \sqrt{3} )</td>
<td>( n_v = 1 )</td>
<td>( n_v = \sqrt{3} )</td>
<td>( n_v = \sqrt{3} )</td>
</tr>
<tr>
<td>( n_{SP} = n_v^2 = 3 )</td>
<td>( n_{SP} = 1 )</td>
<td>( n_{SP} = 3 )</td>
<td>( n_{SP} = 3 )</td>
</tr>
<tr>
<td>( n_c = 3 )</td>
<td>( n_c = 1 )</td>
<td>( n_c = 1 )</td>
<td>( n_c = 1 )</td>
</tr>
<tr>
<td>( n_a = 3 )</td>
<td>( n_a = 1 )</td>
<td>( n_a = 1 )</td>
<td>( n_a = 1 )</td>
</tr>
</tbody>
</table>

1. Failure type

\( n_v = \frac{1}{h} \sqrt{3} \)
\( n_a = 1 \)
\( n_v = \frac{1}{h} \sqrt{3} \)
\( n_a = 1 \)
\( n_v = \frac{1}{h} \sqrt{3} \)
\( n_a = 1 \)
\( n_v = \frac{1}{h} \sqrt{3} \)
\( n_a = 1 \)

2. Consistency

\( n_{IC} \leq 1 \)
\( n_{IC} \approx 1 \)
\( n_{IC} \approx 1 \)
\( n_{IC} \approx 1 \)

3. Strain at failure

\( n_h = 1 \)
\( n_h \leq 1 \)
\( n_h \leq 1 \)
\( n_h \leq 1 \)
\( n_h \leq 1 \)
\( n_h \leq 1 \)
\( n_h \leq 1 \)
\( n_h \leq 1 \)

4. Gravity force

\( n_{FG} \leq 1 \)
\( n_{FG} = 3 \)
\( n_{FG} = 1 \)
\( n_{FG} = 1 \)

5. Adhesion

\( n_{Fad} \leq 1 \)
\( n_{Fad} \leq \frac{1}{3} \)
\( n_{Fad} \leq 3 \)
\( n_{Fad} \leq 3 \)

6. Force to mould clay lump

\( n_{Fm} \leq 1 \)
\( n_{Fm} \leq 3 \)
\( n_{Fm} \leq 3 \)
\( n_{Fm} \leq 3 \)

Table 7.7: Scale scenarios and scale effects.
7.6.1 Scenario 1
This scenario assumes Froude velocities with $n_v = \sqrt{n_f}$ and as such the clay cohesion and adhesion should be scaled back by the length scale factor, $n_a = n_c = n_f$. This results in the adhesion, gravity force and force to mould clay lumps to be scaled correctly (requirements 3, 4 and 5 satisfied). The consequences are that deformation rate is a factor $\sqrt{3}$ too big and the consistency is too low. The clay will be too plastic/soft to correctly represent the failure type. The clay will fail more plastic with a higher strain at failure. Due to the higher strain at failure, failure of the clay into lumps is delayed resulting in relatively larger lumps. Testing on model scale is thus considered more difficult than full scale. It is argued that if this scenario is used in scaled testing and if testing is successful, the tool will definitely work on full scale.

7.6.2 Scenario 2
One way to satisfy requirement 2, 4, 5 and 6 at the same time is to use prototype velocities and prototype clay. The deformation rate is now a factor 3 too big and the failure type is thus strongly skewed towards plastic mode. But the consistency and stretch to failure are right and this is thus a more favourable condition compared to scenario 1. On the other hand requirement 3 is not satisfied. The gravity force is underrepresented on model scale by a factor of 3. If the pick-up, slurrification and hydraulic transport processes are mostly dominated by the fluid drag force and to a lesser extend by the gravitational force than it could be stated that this under-representation is acceptable. But if this is not the case the model scale underestimates the prototype operation and this is considered an unfavourable condition for testing on model scale: on model scale the conditions are easier than on full scale.

7.6.3 Scenario 3
Due to practical reasons testing has been done under scenario 3 (in two variations called scenario 3a and scenario 3b). In scenario 3a use is made of prototype clay but with Froude velocities. In this scenario the adhesion and cohesion are now a factor 3 too high. The force to mould the clay lumps and the force to break the clay in lumps are a factor 3 too low and the clay is much tougher than required. When testing on model scale, the system is thus presented to a much tougher condition than can be expected on full scale. Scenario 3b makes use of a clay that is of same consistency and shear strength as the prototype clay but has an adhesion that is scaled back by the length scale factor (how this is achieved in our case will be explained in paragraph 7.7.2). Having a clay with lower adhesion may influence the failure type, but the effect of adhesion on failure type is expected to be small and is ignored. Requirement 4 is now satisfied and with requirement 5 remaining unsatisfied the test condition on model scale is much tougher than on full scale. It is again argued that if scenario 3b is used in scaled testing and if testing is successful, the tool will definitely work on full scale.

7.6.4 Experimental comparison of scenarios
An actual laboratory comparison of the different testing scenarios was considered but not performed. The main reason for this being the fact that actual full scale velocities ($n_f = 1$) could not be reached at the existing De Beers Marine testing facility.
7.7 Selection of scale scenario for testing at De Beers Marine

7.7.1 Shear strength & consistency index

The average (median) shear strength of the footwall clay types has been found to be between 115 and 165 kPa (see paragraph 3.2.3). They are classified as stiff with an average consistency index around 1. A “1/3 scale” clay would need a shear strength between 38 and 55 kPa.

Two clay types were available for testing, named “test clay - soft” and “test clay - stiff” (see chapter 2 and 3 for details on their properties).

The “test clay - soft” was available from a site close to the testing facility. De Beers Marine had done initial tests close to scenario 1 (shear strength smaller than 40 kPa, low consistency). The clay was supplied dry and by adding water a stiff mixture was made. The claybed was constructed by manual compacting the stiff mixture in the testtank. The tests were always successful even under extreme bulldozing conditions. The fact that tooi did not block although bulldozing conditions were present was at first thought to be caused by the very plastic behaviour (low consistency) of the test clay. Back analysis concluded that the clay being used had shear strengths between 20 and 40 kPa (or even lower) which classifies as soft to almost very soft. Together with this the claybed was very inhomogeneous and the combined effect of both the inhomogeneity and the softness of the clay lead to much easier conditions than intended. Testing with clay around and below 40 kPa under submerged conditions makes the consistency and shear strength difficult to control since the clay will deteriorate fast by easily absorbing more water. As such it has been concluded the “test clay-soft” did not satisfy the strength/consistency-requirement as set by scenario 1 and this is the reason that scenario 1 has not been followed in the scaled tests.

The pick-up and slurrification processes in the suction mouth of the Gravel Wheel are dominated by both the fluid drag force and the gravitational force. Since the gravitational force in the suction mouth area cannot be neglected the under-representation of the gravitational force in scenario 2 will make 1/3 scale testing the conditions easier than required and this is unacceptable. This could not be proven since actual full scale velocities ($n_r = 1$) could not be reached at the existing De Beers Marine testing facility.

Testing at 1/3 scale was thus continued with scenario 3b (with Froude velocities). Although the force to break the clay in lumps is not correctly scaled (see Table 7.7), in actual Gravel Wheel operation the clay lumps are cut by the blade and do not have to be produced by the force of the fluid flow (see Figure 9.9). As such this incorrect scaling (of the force to break the clay lumps) is not relevant for the Gravel Wheel operation. Only the force to mould the clay lumps is too low in scaled testing. If it is possible to make the tool work under these difficult scaled conditions it is expected to work under full scale conditions. As such it has been tried to use a clay which has a consistency similar to the prototype clay as found offshore and with an adhesion scaled back by a factor of 3.

The second clay type available for testing, “test clay-stiff”, was delivered by a local fired-clay brick factory. Making use of high pressure extrusion equipment during the brick fabrication process, homogeneous clay bricks with high shear strengths and consistency indices of around 1 could be delivered. The shear strength of the “test clay - stiff” at a consistency index of 1 lies around 120 kPa (see paragraph 3.2.3). During the tests conducted the clay had shear strengths between 80 and 120 kPa. As such the shear strength and consistency of this test clay are close to the properties as required for scaled testing under scenario 3b.

7.7.2 Adhesion

The footwall clay types encountered in the Atlantic 1 mining area are expected to have on average tensile adhesion values of 25 kPa with maximum values of 45 kPa (see Table 3.8). This means that for correct scaling the adhesion of clay to be used in testing should be on average 9 kPa to maximum 15 kPa. Unfortunately the clay types that are available for testing have either low illite and smectite content or are purely kaolonitic and the adhesion of these clays can be expected to be low.

Since this stiff test clay is 100 % kaolonitic, estimations of adhesion values are low with an average of around 6 kPa (at an $I_c$ of around 1). See Table 7.8 for a summary of required and actual adhesion values. Having no clay with higher adhesion values, tests were carried out with the stiff test clay being...
the best option available. This has as a consequence that in the scaled tests the adhesion is somewhat underestimated compared to the full scale operation.

<table>
<thead>
<tr>
<th>Adhesion</th>
<th>Full scale</th>
<th>Ideal 1/3 scale</th>
<th>Actual 1/3 scale</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>25-45 kPa</td>
<td>9-15 kPa (n_a = 3)</td>
<td>6 kPa (n_a &gt; 3)</td>
</tr>
</tbody>
</table>

Table 7.8: Adhesion for full scale and 1/3 scale condition.

In order to have a clay with a higher adhesion it is being considered to get actual footwall clay from the current offshore Wirth drill operations. Using footwall clay in the scaled tests would over-represent the adhesion since the adhesion in the scaled tests would be the same as in full scale operation. If the tool still performed well using actual footwall clay in scaled testing it could be stated that it would also be successful in full scale operation.

7.7.3 Failure type and deformation rate

In paragraph 7.4.2.1 it was mentioned that Delft Hydraulics' model to predict the failure type was unsuccessful and failure type similarity was attempted in a simple but reasonable way. Following is to only exclude possible errors made using this simplification.

During scaled testing only flow type of failure (ductile failure) was observed. Considering that the flow type of failure is unfavourable compared to tear type of failure, following can be said for the failure type similarity between full scale and model scale:

"Since during scaled testing only flow type of failure was observed it can be stated that if on full scale the clay fails in tear failure mode the scaled tests are performed under more difficult conditions. In the case that in prototype operation the failure type is also flow type of failure the requirement of similar failure type is met".

If actual footwall clay is to be used in the scaled testing, the failure type in the scaled tests will be at least as unfavourable as in the prototype operation owing to the fact that the deformation rates in the scaled tests are always higher, see Table 7.9.

<table>
<thead>
<tr>
<th>Model</th>
<th>Cutting velocity (m/s)</th>
<th>Deformation rate (1/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10 RPM</td>
<td>0.52</td>
<td>10.5</td>
</tr>
<tr>
<td>20 RPM</td>
<td>1.05</td>
<td>41.9</td>
</tr>
<tr>
<td>Prototype</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.8 RPM</td>
<td>0.91</td>
<td>6</td>
</tr>
<tr>
<td>11.6 RPM</td>
<td>1.81</td>
<td>24.2</td>
</tr>
</tbody>
</table>

Table 7.9: Deformation rates for model scale and full scale.
7.8 Summary

Scenario 1 with scaled clay and froude velocities is theoretically the best scenario for scaled testing. But the clay available for testing at De Beers Marine testing facility did not meet the strength/consistency-requirement set.

Testing with clay around and below 40 kPa under submerged conditions makes the consistency and shear strength difficult to control since the clay will deteriorate fast by easily absorbing more water. Because the strength/consistency-requirement as set by scenario 1 are difficult to be met it makes this scenario unfeasible for scaled testing under conditions prevailing at De Beers Marine testing facility.

Not being able to test under scenario 1 testing has been conducted under scenario 3b. Although testing under this scenario presents the tool to a tougher condition than required, it is argued that if the scaled tests are successful the tool will definitely work on full scale. Consistency indices and shear strengths of test clay type used are close to actual footwall clay types, satisfying the consistency and shear strength requirements. The adhesion of the test clay is lower than required and in scaled testing the adhesion is underrepresented.

The failure mode is difficult to predict due to lack of sufficient input data into the failure type (Delft Hydraulic) prediction model. Efforts to keep failure type similar is kept simple by keeping shear strength, deformation rate and cutting angle similar on both model and prototype scale. Since flow type of failure is observed in scaled testing, full scale operation can only be more favourable.
8. Experience from other industries

Since offshore diamond mining with ROV's is relatively new, there is not much experience in this field and if there is it is not available in public sources. In order to be able to predict possible effects of ROV operation experience of similar processes in other fields can be useful; fundamental knowledge of the different processes (e.g. excavation) obtained from dredging research can be applied for improvement and development of offshore diamond mining techniques.

At first other fields like slurry shield tunnelling, offshore pipe laying and agriculture were also to be considered, but since it added not more information to that already available from dredging, it was not further explored in detail.

The aim for this exercise is to come up with a list of design guidelines by which the Gravel Wheel and any other similar tool can be evaluated and compared.

8.1 Offshore diamond mining system design philosophy

In the design of mining equipment to mine diamonds offshore from a 150 m deep diamond bearing gravel layer on top of a unconsolidated footwall the following concept/philosophy is followed.

Geological deposit characteristics and (superimposed) mining requirements (e.g. mining rate), together named mining conditions (that are translated/expressed into mining system specifications), set the requirements for the whole mining system to be designed. The optimum mining system design is called the mining solution. To derive a mining solution based on the set mining conditions experience from existing solutions to similar conditions can be used. Since the dredging industry has already existing dredging solutions to known dredging conditions this knowledge base has been used in finding a mining solution for the offshore diamond mining industry.

In finding a solution for the complete mining system the approach taken is to split the mining system into smaller subsystems and find solutions for these sub-systems. As such the mining tool, being the system directly interacting with the orebody, is identified as a sub-system. The offshore diamond mining conditions for the mining tool have been already summarized in paragraph 1.3.

Using abovementioned philosophy De Beers Marine's current "High rate mining tool development" program has come to several mining tool concepts that have been tested and are being tested on model scale. From these concepts the Gravel Wheel has been thoroughly tested in gravel trying to satisfy the gravel mining requirements first. In these trials several modifications led to the Gravel Wheel version MkIII.

In dredging a similar design philosophy is followed. The offshore diamond mining conditions differ from the dredging conditions and this may result in a different optimal solution (see Figure 8.1: the difference between conditions and solutions in dredging and offshore diamond mining is symbolized by Δ).
This study focuses only on the development of the Gravel Wheel version MkIII and specifically on the study of the clay handling requirements set. The approach followed is to derive design guidelines from (clay cutting) experience in the dredging industry and to evaluate the Gravel Wheel’s clay handling ability with these guidelines with necessary caution (see Figure 8.1).
8.2 General

In the book on general dredging principles by Bray (1979) the following short description can be found: Clay is a material which can normally only be excavated by scooping, scraping or cutting. Its dredgeability is governed by:

1. Its resistance to this excavation and
2. Subsequently by the ease with which it can be handled

The resistance to excavation is directly related to the force required to induce failure of the material and thus dependent on the undrained cohesion (undrained shear strength, c) of the clay. An attempt to assess the dredgeability has been made by Bray (1979): see Table 8.3 for the dredgeability of clays.

<table>
<thead>
<tr>
<th>Consistency index (-)</th>
<th>Undrained shear strength, c (kPa)</th>
<th>Dredging rate (m³/h)</th>
<th>N-value (SPT)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft clay</td>
<td>0.4</td>
<td>45</td>
<td>200</td>
</tr>
<tr>
<td>Medium clay</td>
<td>0.6/0.87</td>
<td>50/200</td>
<td>50</td>
</tr>
<tr>
<td>Hard clay</td>
<td>1.3</td>
<td>&gt;250</td>
<td>25</td>
</tr>
</tbody>
</table>

Table 8.1: dredgeability of clays (Bray, 1979).

Two distinct problems in ease of handling, if clay is cohesive/plastic, are:

1. The likelihood of clogging and/or balling of the clay in cutter and pipelines and
2. The adhesion of the clay to various steel surfaces of the cutting tool prior to entering the suction line.

The first depends on shear strength of the clay in combination with the Atterberg limits (plasticity index). Especially the consistency index is important. The closer the natural water content to the liquid limit the more the clay tends to go in suspension. Clay in suspension causes no problems during transportation and discharging. Clay balls results in severe increase of the required pump power to transport the mixture.

The second problem mentioned depends on the adherence properties (shear and tensile adhesion) of the clay, which are also related to the Atterberg limits and consistency index.

8.3 Important soil properties in dredging

A detailed description of the most important soil properties in dredging is given by F.A. Verhoeven et al (1990). The dredging process is divided into 4 sub-processes and the behaviour of soil under these processes is described. The following extract (focussing on clay) has been derived from this paper. Weak point of article is that it only gives the important soil properties but no quantitative method to use the qualitative approach.
Basic division into dredging processes is as follows:

1. Loosening process
   The failure behaviour of clay during cutting is to a large extent determined by the stiffness. Softer clays generally show plastic failure behaviour while with stiffer clays failure often starts from a tensile crack. The specific energy (energy per cubic meter of soil cut) to cut stiff clay will be relatively less than for the softer clay types. The cutting forces in softer clay are determined by the undrained shear strength and the adhesion in contact with the steel blade. Assuming the adhesion to undrained shear strength ratio to be constant, the shear strength can be considered as the only relevant parameter.

2. Mixing process
   The mixing process (especially for cutter dredgers) plays an important role because a poor mixing process leads to a considerable spill. In clay the mixing process is different than in sand. Just after cutting, clay consists mainly of lumps and a certain amount of dissolved material, which limit the concentration in the pipeline due to difficulty to suction them. So, for clay the mixing process is hardly sensitive to the soil properties, but more to the size and weight of the lumps.

3. Transport (hydraulic)
   The friction losses of clay-water mixtures will depend on:
   - The rheological properties of the slurry
   - The density of the slurry
   - The percentage of clay lumps
   The clay slurrifies during transport and the degree of slurrification is a function of the clay type (undrained shear strength and plasticity index). At the beginning a higher percentage of clay lumps is present in the mixture than at the ends.

4. Disposal
   In regular dredging the dredged soil is disposed off as e.g. a landfill. In our case the transported mix of gravel and clay will have to pass a processing step but experience from dredging regarding important clay properties can still be of use. The behaviour of disposed clay is predominantly determined by the percentage of lumps in relation to the percentage of slurry. The degradation of clay lumps is governed by the consistency of the clay (or the undrained shear strength) and the Atterberg limits (plasticity index).

A summary of the important properties of clay in the different steps is given in Table 8.2:

<table>
<thead>
<tr>
<th>Process</th>
<th>Relevant properties of clay</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loosening</td>
<td>- undrained shear strength</td>
</tr>
<tr>
<td></td>
<td>- (adhesion)</td>
</tr>
<tr>
<td>Mixing/slurrification</td>
<td>- lump size distribution</td>
</tr>
<tr>
<td></td>
<td>- density of lump</td>
</tr>
<tr>
<td>Transport</td>
<td>- rheological properties</td>
</tr>
<tr>
<td></td>
<td>- undrained shear strength</td>
</tr>
<tr>
<td></td>
<td>- plasticity index</td>
</tr>
<tr>
<td></td>
<td>- wet density</td>
</tr>
<tr>
<td>Disposal</td>
<td>- undrained shear strength</td>
</tr>
<tr>
<td></td>
<td>- plasticity index</td>
</tr>
</tbody>
</table>

Note: The soil parameters between parentheses are relevant for the processes mentioned as well, but generally the other parameters can be regarded as sufficient

Table 8.2: Important soil properties of clay (Verhoeven, 1990).
### 8.4 Practical design rules from dredging

#### 8.4.1 General

Table 8.3 gives an overview of types of dredgers applied in clay and their general characteristics.

<table>
<thead>
<tr>
<th>Type of dredge</th>
<th>Name</th>
<th>Plain suction dredge</th>
<th>Cutter suction dredge</th>
<th>Wheel suction dredge</th>
<th>Trailing suction hopper dredge</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Excavation tool</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clay</td>
<td></td>
<td>-</td>
<td>+</td>
<td>+</td>
<td>0</td>
</tr>
<tr>
<td>Silt</td>
<td></td>
<td>0</td>
<td>+</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>Gravel and Sand</td>
<td></td>
<td>+</td>
<td>+</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>Rock</td>
<td></td>
<td>-</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Excavation/loosening</td>
<td></td>
<td>Passive – erosion</td>
<td>Active – mechanical</td>
<td>Active – mechanical</td>
<td>Active – jet/mechanical</td>
</tr>
<tr>
<td>Pick-up/ slurrification</td>
<td></td>
<td>Suction mouth/ waterflow</td>
<td>Suction mouth/ waterflow</td>
<td>Suction mouth/ waterflow</td>
<td>Suction mouth/ waterflow</td>
</tr>
<tr>
<td>Vertical transport</td>
<td></td>
<td>Pump/pipe</td>
<td>Pump/pipe</td>
<td>Pump/pipe</td>
<td>Pump/pipe</td>
</tr>
<tr>
<td>Connection tool with ship/pontoon</td>
<td></td>
<td>Flexible</td>
<td>Rigid</td>
<td>Rigid</td>
<td>Flexible or rigid</td>
</tr>
</tbody>
</table>

Note: + = GOOD  
0 = RESTRICTED  
- = NO APPLICATION

Table 8.3: Features of dredging equipment.
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This table only gives an introduction to the features of the different dredging equipment and should not be used for selection purposes. Initial differences can be seen but no conclusive selection criteria should be derived. These 4 types of dredgers are chosen since they are the ones that use hydraulic transport as means of material transport.

In this study only the cutter suction dredge and the wheel suction dredge will be dealt with in more detail, since they are the main type of dredging equipment used to cut clay as can be derived from Table 8.3. The restricted operation of the trailer suction hopper dredge in clay is not investigated, but currently new systems are being developed to deal with clay, with application of high pressure water jets in the draghead of a conventional hopper dredger (Vandycke, 2002).

8.4.2 Cutters suction dredgers
8.4.2.1 Introduction
The cutter suction dredger (CSD), see Figure 8.2, is the most common type of dredger. One of the main components of the cutter suction dredger is the cutterhead which is situated at the entrance of the suction pipe and has two primary functions:

1. To loosen and disintegrate the soil (agitate softer materials and cut harder materials) into particle sizes compatible with the pumping system
2. To ensure that the disintegrated material can be removed hydraulically by placing it in the high velocity stream at the suction intake in the necessary quantity (Turner, 1996 & Verhoef, 1997).

Figure 8.2: Sketch of the cutter suction dredge.

The cutterhead is commonly of a basket type (straight edge is another, rarely used, type), with spiral arms which are integral with the front hub and back wearing ring (see Figure 8.3a). Different variations on design of the basket type exist, see Figure 8.3.

<table>
<thead>
<tr>
<th>Document Number</th>
<th>Rev</th>
<th>Additional Reference</th>
<th>Page Number</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>93</td>
</tr>
</tbody>
</table>
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The first generation cutterheads had arms with cutting blades, either plain or serrated (see Figure 8.3b and Figure 8.3c). The second-generation cutterheads are designed with teeth (see Figure 8.4) instead of a plain or serrated cutting edge. The type of teeth used depends on the soil type to be cut. The design of the cutterhead (e.g. position of cutting edge or teeth) highly influences the efficiency of the cutting and suction operation. Toothed cutterheads have better positioned arms and teeth and make replacement of the wear parts faster and easier.

Figure 8.3: Various designs of the basket type cutterhead.

Figure 8.4: Several type of teeth used on toothed cutterheads.

Rough general guidelines for selecting cutterheads in various soil or rock types with respect to envelope, type and number of blades or teeth, are as follows (Vlasblom, 2001, adjusted):

1. For hard rock (figure Figure 8.5a):
   a. Envelope of the arms: use small contours
   b. Use toothed cutter with pick points to concentrate cutting forces
   c. Heavy and robust (wide arms) to be able to endure high cutting forces/stresses on only a few teeth

<table>
<thead>
<tr>
<th>Document Number</th>
<th>Rev</th>
<th>Additional Reference</th>
<th>Page Number</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>94</td>
</tr>
</tbody>
</table>
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2. For cohesionless soils like loose sands or very soft clays (Figure 8.5b):
   a. Envelope of the arms: conical contours
   b. Preferably plain or serrated edges; when the soil/sand is cemented use toothed cutter with narrow or wide chisels
   c. Generally 6 blades

3. For cohesive soils like stiff clay (Figure 8.5c):
   a. Envelope of the arms: large and spherical contours to prevent clogging of clay between the adjacent arms
   b. Toothed cutter with flared chisels
   c. Usually 5 blades

Figure 8.5: Envelope of arms for different soil types.

8.4.2.2 Design aspects (features) related to clay
Bladed or toothed cutterhead
The positioning of the arms of a blades cutterhead is such that the distance/gap between the arms decreases towards the hub of the cutter (converging shape). When cutting cohesive soils with high adherence properties this may cause clogging, obstructing the gap between the arms at this location. See Figure 8.6 for a clogged cutter. The toothed cutter has more flexibility in the shape and positioning of the arms at the end of the hub with more space between the arms being created and the converging effect somewhat compensated. That's one of the reasons why in stiff clay nowadays a toothed cutter is preferred above a bladed cutter.
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Figure 8.6: Cutter clogged by clay.

Shape of the cutter head
As mentioned before the cutterhead is designed with spherical contours for more open design to prevent clogging. The width of the arms is also made smaller to reduce adherence area.

The top of the cutter (in the vicinity of the hub) has a higher potential for clogging and is farthest away from the sphere of influence of the suction inlet. That makes the transport of the cut soil from the top most difficult. To reduce this problem, the cutterhead is made shorter to decrease the distance to the suction mouth (for more inflow of water to carry off the excavated soil and easier disposal of the cut soil). The height to diameter ratio is around 0.7-0.8.

Position of the teeth
The cutting angle is chosen in order to reduce cutting resistances. Normal cutting angles are chosen as small as possible.

The dynamic clearance angle $\beta$ which is a function of rotation speed should be such that unnecessary friction on the backside of the teeth/blade/ or arm will not occur during excavation and thus decreasing the cutting forces.

The teeth on the adjacent arms are placed in a staggered position. In the first cut, grooves are made by the first teeth. The series of teeth on the next arm cuts the material between the grooves made by the first one.

Shape of the blade or teeth/arm
When the clay is firm to stiff slices of soil are cut which can break off into clay lumps. The smaller the clay lumps the better this is for both the slurrification and hydraulic transport process. The arms/blades or teeth can be designed with a breaking edge (see Figure 8.4, flared tooth) to force the cut slice up and to let it break in order to minimise the size of the clay lumps formed. The second advantage of this breaking edge is that the area over which the clay can adhere is made smaller. The disadvantage is that the breaking edge does wear faster due to more intense contact with the soil.

Angular displacement
A greater angular displacement (Figure 8.7) makes for a smoother operation and greater penetrating force. The teeth come one by one in the soil thus making it possible that the forces increase gradually instead of an abrupt jump in cutting force (causing a shock effect and higher vibrations) when more teeth cut at once. The result is that the cutterhead has a better energy efficiency. A large displacement angle is particularly important when 5 blades instead of 6 blades are used (Turner, 1996).
Figure 8.7: Angular displacement for smoother operation.

The strength of the teeth and the arms has to be designed such that it can take bumps and other impacts during operation.

8.4.2.3 Operational aspects related to clay
The cutterhead is mounted on a supporting arm, called the "ladder", see Figure 8.8. The ladder can be vertically hoisted and lowered and is fixed to the vessel (cutter suction dredger). During operation the vessel pivots around a spud pile and two side winches (wires), attached to anchors, make the swinging motion possible (see Figure 8.9).

Figure 8.8: Cutter suction dredge.
The blades or teeth on the cutterhead perform the cutting operation by means of the rotation of the cutterhead and the swinging movement of the vessel. The simultaneous rotation and the lateral displacement of the cutter make the cutting edge of the blade or teeth moves along a so-called cycloid (see Figure 8.10).

Figure 8.10: Movement of teeth along so called cycloid.

As shown in Figure 8.10, the thickness of the slice, cut by one blade or teeth, increases going upward. The thickness of the slice is a function of the rotational speed of the cutterhead (usually around 26 to 30 rpm) and the swing speed (haul velocity, usually 5 – 20 m/min).

The thickness of the slice is limited by the number of blades of the pump propeller and cannot be increased above a value equal to 0.3 times the diameter of the suction pipe (with a dredge pump having 4 blades). Further limitation is also the spacing of the blades on the cutter.
Conclusions
Nowadays the cutterhead designed for stiff clay:
1. Is toothed for better position of the arms at the hub (more open and diverging space, or reducing convergence).
2. Is more spherical (more open design).
3. Is shorter; the height to diameter ratio is around 0.7-0.8.
5. Has narrower arms to reduce adherence area.
6. Rotational speed and swing speed should be chosen such that the thickness of the slice is optimised.

8.4.3 Wheel suction dredgers

8.4.3.1 Introduction
The wheel dredger has been recognized to be an efficient tool for both conventional dredging and mining. Its main component is the dredging wheel, as shown in Figure 8.11, consisting of hub and a ring connected by bottomless buckets which excavate the soil.

Figure 8.11: Dredging wheel on wheel suction dredger.

The wheel has the same primary functions as the cutterhead has on the cutter suction dredger. But due to its design it assures high production and efficiency by its more uniform cutting process, constant dredging output in both directions of swing, optimum mixture density, low spillage, almost absence of blockage and low sensitivity to debris such as rocks and tree stumps. The wheel dredger is utilized in more specific applications than the cutter suction dredge. The wheel dredger is better in clay and has high transport concentration possibilities but is more expensive. Selection of a wheel dredger above a cutter dredger depends on the area of application. Different contractors make more use of the cutter dredger because of its more general application. Wheel dredger used in more specific applications where higher costs are possible and high transport concentration can be dealt.

The design of the wheel has undergone quite some change since the first models. The first generation wheels were shaped like the wheel excavators used in opencast mining (see Figure 8.12) but poorly performed in cohesive and sticky soils; in the transition from the cutting process to the suction process, the closed buckets are very prone to clogging.
The new designs cope with these problems with the following features:

1. The buckets are made bottomless (see Figure 8.13), eliminating the risk of accumulation of soil in, and clogging of, the bucket.
2. By mounting of a large number of buckets close together (see Figure 8.13), the area enclosed by them become an integral part of the suction mouth and the soil right after being cut, moves within suction range, giving very low spillage. The small gap between two consecutive buckets also prevents big pieces to enter the system, resulting in fewer blockages of the suction mouth and dredge pump.
3. The suction mouth is extended into the bucket by means of a lip (see Figure 8.13), which actively guides the cut soil towards the suction mouth and completely prevents clogging of the buckets. And with suction applied close to the cutting area high mixture densities (of 70%) can be achieved.

Two types of dredging wheel operation can be recognized: undercutting or over-cutting. Figure 8.11 is an example of undercutting and Figure 8.13 of over-cutting. Differences in design and operation will be dealt with in next sections.

8.4.3.2 Design (features) aspects related to clay

Shape and position of buckets

The cutting edge of the buckets should be such that the cutting angle and clearance angle are optimal. See Figure 8.16 for the cutting action and feed of the cut slices to the bucket. To cut harder soil types like rock the bucket can be fitted with teeth as shown in Figure 8.14.

In the curved, closed buckets of the older dredging wheels, cohesive and sticky soil could easily stick and constantly accumulate in the bucket until the bucket was completely clogged. Straight, bottomless buckets eliminate the risk of accumulation of cohesive and sticky soil in the bucket.
Position of the suction mouth and lip is best closest to the location where the buckets end their cutting cycle. The application of this concept is shown in Figure 8.15 with a clear difference of suction mouth and lip position in upward and downward cutting.

Suction force is effectively applied; highest suction force where needed most. The mixture concentration entering the suction mouth and pipe is improved and as a direct consequence also the diameter of the pipeline can be reduced.

Direction of suction
In either direction (in under- or over-cutting, see Figure 8.15), the wheel performs better if the direction of rotation is the same as the direction of suction (wheel also acts as a pump the pumping direction of the wheel should be the same as the suction direction). Therefore, with under-cutting the suction pipe should be positioned at the top of the wheel, and with over-cutting, at the bottom, see Figure 8.15.

Figure 8.15: Left shows under-cutting, and right over-cutting configuration.
8.4.3.3 Operational aspects related to clay

The operation of the wheel suction dredger is similar to that of the cutter suction dredger, which is explained in the previous paragraph. Difference is the rotational direction of the cutting edges/blades or teeth with respect to the swing direction.

The dredge wheel performs the cutting action by means of the rotation of wheel and the swinging movement of the vessel. Due to the simultaneous rotary and swing movement of the bucket wheel, every point on the cutting edge of the buckets describes a helical path, just as the treads of a screw. The thickness of the cut slice is also here a function of both the rotational and swing speed.

In dredging wheel operation "bull dozing" should be avoided: bull dozing refers to the situation where the cut slice is so thick that the formed chip (deformed and thickened) touches the bottom of the preceding bucket; this is illustrated in Figure 8.16.

![Normal bucket feed and bucket feed at the start of bull dozing](image)

**Figure 8.16: Bull dozing.**

One of the advantages, hydraulically spoken, of the construction of the dredge wheel and its build in scraping lip, is that due to the rotation of the wheel, the flow will be directed through the lip towards the suction mouth. This creates a pumping action which reduces the entrance losses of the material while entering the suction mouth. Even a slight over pressure is created which benefits the dredging performance.

**Conclusion**

Dredge wheel design concepts:
1. Open bottomless buckets to reduce risk of accumulation and clogging potential
2. Closed design (large number buckets close to each other) reduces spillage
3. Narrow opening between buckets prevent large piece to enter suction mouth and pipe
4. Active guidance of cut soil towards the suction mouth by means of a lip
5. Positioning lip near the location where the buckets end their cutting cycle
6. Applying suction force (positioning of suction mouth) at location of lip
7. Direction of suction in accordance to direction of rotation
8.4.4 Features in dredging translated to design guidelines

In the previous paragraphs the cutter suction dredge and the wheel suction dredge have been analysed with respect to design and operational features related to cutting of clay. These are now translated to preliminary design guidelines for other equipment that will have to cut clay. One main difference in the Gravel Wheel is that its main function is not to cut clay for clay production but that the clay has to be dealt with it effectively during its actual function of mining a diamond bearing gravel layer.

Design guidelines:
1. Cut clay actively (mechanically, not hydraulically)
2. Open design of tool to reduce clogging potential
3. Avoid converging areas
4. Reduce contact area of cutting tool/edge with clay (adhesive area); shape of cutting edge/teeth
5. If possible introduce active guidance of cut clay
6. Optimise cutting angle and clearance angle
7. Reduce distance from suction mouth to area where cutting action takes place
8. Create symmetry in slurrification area/suction mouth
9. Reduce size of lumps compared to suction mouth opening; this by optimal operational parameters
10. Avoid bulldozing; find optimal/minimal cutting depth (thickness of slice) to opening of cutting tool ratio
11. Avoid sharp bends and dead corners in slurrification area and suction mouth
12. Create high turbulences in slurrification area/suction mouth for better slurrification
13. Let direction of suction be in accordance with cutting direction

In an initial evaluation a comparison is made of the cutter and wheel suction dredge and the Gravel Wheel based on derived design guidelines (see Table 8.4). In the following subchapter a more detailed explanation of the scores will be given per design guideline.

<table>
<thead>
<tr>
<th>Design guidelines</th>
<th>Cutter suction dredger</th>
<th>Wheel suction dredger</th>
<th>Gravel Wheel</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Active cutting</td>
<td>++</td>
<td>++</td>
<td>++</td>
</tr>
<tr>
<td>2 Openness of tool</td>
<td>++</td>
<td>++</td>
<td>--</td>
</tr>
<tr>
<td>3 Converging parts</td>
<td>+/-</td>
<td>++</td>
<td>--</td>
</tr>
<tr>
<td>4 Contact area cutting tool clay</td>
<td>++</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>5 Active guidance of cut material</td>
<td>-</td>
<td>+</td>
<td>NA</td>
</tr>
<tr>
<td>6 Optimum cutting angle</td>
<td>+</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>7 Distance suction mouth to location of cutting action</td>
<td>+/-</td>
<td>+/-</td>
<td>++</td>
</tr>
<tr>
<td>8 Symmetry</td>
<td>-</td>
<td>++</td>
<td>-</td>
</tr>
<tr>
<td>9 Size of lumps vs grid size</td>
<td>+</td>
<td>+</td>
<td>NA</td>
</tr>
<tr>
<td>10 Risk of bull dozing</td>
<td>+</td>
<td>+</td>
<td>--</td>
</tr>
<tr>
<td>11 Sharp bends/dead corners</td>
<td>++</td>
<td>++</td>
<td>-</td>
</tr>
<tr>
<td>12 High turbulences</td>
<td>+</td>
<td>+</td>
<td>+</td>
</tr>
<tr>
<td>13 Direction of suction vs direction of cutting</td>
<td>+/-</td>
<td>+</td>
<td>+/-</td>
</tr>
</tbody>
</table>

Note: + = positive score - = negative score

Table 8.4: Evaluation based on derived design guidelines.

The list of design guidelines should be used with caution and is also by no means extensive and complete. All guidelines do not have to be perfect for every design. Some guidelines overlap and one guideline can overrule the other (one guideline is dominant/more pronounced). For example the lip in a wheel suction dredger, actively guiding clay, makes the "contact area" and "distance suction mouth to location of cutting action" design guidelines less important.


8.4.5 Evaluation of Gravel Wheel

Active cutting
The Gravel Wheel’s scoop is actively cutting the clay and scores positive on this guideline.

Risk of bulldozing and openness of tool
Both guidelines are related in that the more open a tool, the lower the risk of bulldozing at constant operational parameters. Comparing the openness of the Gravel Wheel to the dredging tools the Gravel Wheel has definitely a closed design. The relatively narrow scoop openings are the only open areas on the whole drum. This is not without reason. The thought behind this design is that by concentrating the suction force on smaller areas right at the tool gravel interface the pickup of gravel and diamonds will be optimised and spillage reduced. Another feature is that the scoop with its grizzly bars has a primary screening function and as such has a set screen size.

The risk of bulldozing should be evaluated by comparing the thickness of the cut slice, which is a function of operational parameters, to a specified “tool opening size”. The risk of bulldozing is acceptable if the ratio of cut slice thickness to tool opening size is below a certain limiting value \( R_{\text{max}} \).

\[
\frac{\text{cut} \cdot \text{slice} \cdot \text{thickness}}{\text{tool} \cdot \text{opening} \cdot \text{size}} \leq R_{\text{max}} \tag{8.1}
\]

An initial calculation of thickness of the cut slice at defined optimum operational parameters (for gravel mining) was made and compared to the opening size at the scoop grizzly. From this exercise it was concluded that the Gravel Wheel’s risk of bulldozing was high in comparison to cuttersuction and wheel suction dredger.

Converging parts
The suction duct is converging from the scoop grizzly to the size suction port. Two suction ducts also converge via their respective side suction ports into one single suction pipe (see Appendix 1). This gives the Gravel Wheel a negative score on this design guideline.

Contact area cutting tool clay
The Gravel Wheel has a large surface area in the suction port that is in contact with the flow and the carried gravel and clay. Based on this observation it is stated that compared to the dredging equipment the amount of contact area of the Gravel Wheel is high. The Gravel Wheel is given a negative score for this design guideline.

It should be noted that the suction port of the Gravel Wheel is actually already part of the suction pipe (extension of the suction pipe) and that the suction is thus a factor higher in the suction port than at the cutting blade-clay interface of dredging equipment. But being conservative this factor is not considered in the negative score.

Active guidance of cut material
Since cutting takes place at the extension of the suction mouth there is no talk of active guidance of material as in the case of the wheel suction dredger.

Distance of suction mouth to location of cutting action & active guidance of material
The cutting action takes place at the scoop edge which is right in front of the suction duct. As noted before the suction port is an extension of the suction pipe and as such the distance of the suction mouth to the location of cutting is taken a value of zero. The advantage of this is that it is expected to reduce spillage and as such also reduce the diamond mining recovery.

Optimum cutting angle
The cutting angle of 30 degrees falls in the range of 30-45 degrees which is generally considered to be optimal for soil cutting.
Symmetry
The suction duct of the Gravel Wheel version MkIII is not fully symmetrical since its suction port is positioned at one side of the drum (see Appendix 1). How this a-symmetry affects the flow and the pick-up of particularly clay lumps in the suction duct negatively should be studied further. Previous tests with gravel have not identified any problems. Being on the conservative side, the score given to this design guideline is negative (-).

Size of lumps vs. grid size
The clay lumps are not screened since they are formed right at the screen. As such this design guideline does not apply to the Gravel Wheel version MkIII.

Sharp bends/dead corners
The flow has to change direction going from the suction duct into the suction pipe. This is an extra resistance to flow at a critical point. That is why the Gravel Wheel is given a negative score for this design guideline.

High turbulences
When cutting with the scoop at the suction duct inlet, there is not a real slurrification step as in dredging. The cut clay directly enters the suction duct and slurrification is supposedly taking place in the suction duct before entering the suction pipe. But the high velocities cause high enough turbulences in the suction mouth.

Direction of cutting vs. direction of suction
The cutting takes place in the tangential direction while the suction is radial. The cut clay slice is forced to change direction at the scoop-suction duct transition. How this effects the lump formation and pick-up of the clay lumps by the flow is not known. But compared to dredging equipment this is unusual and can negatively effect lump formation and pick-up. With dredging equipment the cutting blade is standing free in space and the clay lump can freely leave the blade at the backside of the blade.
9. Test work and results

9.1 Objectives

A series of scaled tests were conducted to study the clay on handling ability of the Gravel Wheel.

The objectives of the scaled tests were defined as follows:
1. Derive under which parameters the tool still remains free from blockage
2. Derive under which parameters the tool still performs its intended gravel mining function
3. Investigate the phenomena and their underlying mechanisms taking place at tool-soil interaction
4. Evaluate the tools design based on the design guidelines derived in Chapter 8
5. Recommend changes to the tool for increased clay handling ability

9.2 Method

The testing facilities of De Beers Marine were built for testing on 1/3 scale because this was considered just small enough to be able to practically and financially handle and just big enough not to introduce to much scale effects. In this study testing on the same 1/3 scale in continued

An initial evaluation of the Gravel Wheel based on the design guidelines as in Chapter 8 showed that it scores negatively on a few design guidelines off which the risk of bulldozing is thought to be the most critical one. If the tool starts bulldozing the flow will stop completely and if the blockage cannot be removed the tool will be totally blocked. It was also acknowledged that planning and implementing a new scoop design to influence (reduce) the risk of bulldozing could be achieved within the time planned for this project. The implementation of any other major change to the design would have required more time than available and the change would not have been directly justified.

Based on the evaluation of the Gravel Wheel in Chapter 8 a new scoop was designed to reduce the risk of bulldozing. The new scoop (the "5 cm scoop") is made as open as possible open (more "aggressive") with the limiting factor being the screen size of 60 mm on 1/3 model scale. Details on both scoop designs can be found in Appendix 8.

It was decided that the first series of tests would be focused on finding the operational parameters under which the tool remains free from blockages (objective 1) and finding the maximum cut depth that could be achieved with both the existing (further called the "3 cm scoop") and new scoop design.

"3 cm scoop"  
560 mm  
50x75 mm  
30 mm

"5 cm scoop"  
50 mm  
63x63 mm

Figure 9.1: Comparison of scoops.
The following variations were made in the scaled tests to derive under which parameters the tool still works fine without blocking:

1. Specific "bite size" (see paragraph 9.5.2) by changing following operational parameters:
   a. Cut depth
   b. Tool rotational speed and
   c. Forward mining speed

2. Design
   a. Scoop design

In the second series of tests a bed gravel was laid on top of clay to find the operational parameters under which the tool still performed its diamond bearing-gravel mining function (objective 2).

9.3 Experimental test set-up

See Figure 9.2 for the experimental test set-up.
The Gavel Wheel is attached to a boogie that can be moved along the tank at specified speeds (forward mining speed, accuracy 0.1 m/min) by means of 2 drives. The maximum speed that can be achieved is 20 m/min. The Gravel Wheels' attachment to the boogie is flexible and both the height of the tool and horizontal position of the tool can be controlled. The height can be set at mm accuracy.

The Gravel wheel itself can rotate by means of a hydraulic motor and the tool rotational speed can be set at required values with an accuracy of 0.1 RPM. The hydraulic system consists of the pump (which is also attached on the boogie) and connecting pipes as shown in Figure 9.3. The pump speed is also variable and can be set at specified values.
Implementing a clay testbed is easier said than done. The stiff clay as introduced in chapters 3 and 7 is used in the tests. The shear strength varied between 80 and 120 kPa. Apart from finding the right clay the actual implementation of a clay testbed was the second concern. Clay bricks, approximately 10x20x40cm in size, were manufactured by a local fired-clay brick factory and these clay bricks were delivered on pallets and wrapped in plastic to prevent it from drying out. It was assumed that the clay bricks were homogeneous per batch delivered. The strength of the bricks per delivered batch still varied due to drying out of bricks at the bottom of the pallets. The bricks were laid in the best possible orientation as can be seen in Figure 9.4. The bed was constrained on the sides and the front by sand bags to prevent the bricks from slipping over the underground during the tests. In some tests slippage of the bricks still occurred. Another feature of the clay testbed is that due to the size regular "shear planes" are introduced in the bed and this can be expected to have an influence on the tests and subsequent results. This effect cannot be quantified and is not considered further.

In the clay and gravel tests, gravel of all passing size (d_{100} = 60 mm) was laid over the clay testbed. No compaction was applied and the gravel just laid loose over the clay testbed.

Figure 9.3: Hydraulic system with pump and connecting pipes.

Figure 9.4: Stiff clay brick layout/orientation.
The following parameters can be electronically logged by the data logging system already in place and are used in subsequent analyses:

1. Pump vacuum pressure
2. Tool vacuum pressure; the tool vacuum gage gave up twice during the course of the tests and since there was no other gage available on such a short notice in the rest of the tests, the tool vacuum pressure was not used in the data analysis.
3. Hydraulic pressure of the hydraulically driven Gravel Wheel, which is a function of the effort to rotate the tool at set rotational speed and the cutting force to be applied at the scoop.

9.4 Test plan

The Gravel Wheel MkIII's optimum operational parameters mining in gravel were defined by previous tests (Smith, 2003) and were found to be as follows:

1. Tool rotational speed: 10 RPM
2. Forward mining speed: 2.5 m/min
3. Pump speed: 1000 RPM

These settings were taken as the base case setting and changes were made on these to study the clay handling ability of the tool.

The list with a summary of the tests carried out is presented in Appendix 9.

9.5 Test results and data analysis

9.5.1 Results 3 cm scoop

The scaled tests performed with the 3 cm scoop are graphically summarized in Figure 9.5 below. It shows successful and near successful tests under different operational parameters. The successfullness of a test is defined by the vacuum pressures that occur during the test. When the vacuum pressures go above the normal range of vacuum pressures it is considered near successful. When the tool is completely blocked the test is unsuccessful. The first series of tests with a forward mining rate of 2.5 m/min show that increasing rotational speed increased the cut depth achieved. A 2.5 m/min limit line is drawn below which successful operation is possible.

Increase in cut depth can also be achieved by reducing the forward mining speed. In the second series of tests the forward mining speed was reduced for a given cut depth in order to find the forward mining speed where successful operation was still possible. In the figure another two limit lines are drawn for 1.0 and 1.3 m/min respectively. This verifies that by reducing the forward mining speed a higher cut depth can be achieved.

![Figure 9.5: Test results 3 cm scoop.](image-url)
9.5.2 Results 5 cm scoop

A similar graphical summary of the test results using the 5 cm scoop is shown in Figure 9.6. Comparing these with the 3 cm scoop results it is observed that with the 5 cm scoop a small, but significant increase in cut depth of about 15% is achieved; this improvement is shown in Figure 9.7.

![Graph showing test results 5 cm scoop](image)

Figure 9.6: Test results 5 cm scoop.

![Graph showing comparison of achieved cut depth for different scoop types](image)

Figure 9.7: Comparison of achieved cut depth for different scoop types.

9.6 Bite size and length of cut

One of the governing dimensions in clay cutting is the “bite size”. A distinction is made between the “in-situ” bite size and the bite size “as cut” as shown in Figure 9.8. The in-situ bite size is the thickness of the slice given to the tool as occurring in-situ (see Figure 9.9) and which is a function of rotational speed, forward mining speed and cut depth. The bite size “as cut” is the thickness of the slice after it is cut: this is a function of the in-situ bite size, the cutting angle, the cutting velocity, the cohesion and the adhesion. Based on theory presented in Chapter 6 a simplified estimation of the bite size “as cut” is made.
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Dealing with clay when mining diamonds offshore

Figure 9.8: Illustration of "in-situ" bite size and bite size "as cut".

See Appendix 10 in which the bite sizes for several rotational speeds, forward mining speeds and cut depths are tabulated.

Tests are usually conducted in two ways:
1. Keeping the forward mining speed constant and increasing the cut depth
2. Keeping the cut depth constant and increasing the forward mining speed

In both the bite size is increased until the tool clogs. The overall maximum bite sizes "as cut" achieved with the 3 cm scoop and the 5 cm scoop are 2.4 cm and 2.9 cm respectively.

Another factor that comes into play is the length of the cut slices. The length of the cut slices increases with cut depth and may also influence clogging. See more on this in the paragraph 9.9 on the observed causes for clogging.

Figure 9.9: "In-situ" bite size at the Gravel Wheel.
9.7 Pump vacuum

The pump vacuum is a good indicator for clogging. During the tests only visual observation of the flow is used as an indicator for blockage. After the tests the logged data is analysed and "objective" evidence of blockage can be shown. The logged data even shows that partial blockage occurred which was not always recognized at the time of the test. It can be difficult to visually observe blockage during the test, partially due to lack of experience.

A plot of the maximum vacuum pressures of the different tests in the stiff test clays is shown in Figure 9.10. On the x-axis the in-situ cut depth is plotted; no distinction is made in the length of cut which also influences the test result. The figure intends to show how the pump vacuum behaves under different test conditions. The maximum pump vacuum pressures under easy conditions varies between -40 and -50 kPa. In the case of complete blockage vacuum pressures vary between -70 and -80 kPa.

A good reference is the plotted vacuum pressures of previously conducted tests in soft clay with the 3 cm scoop (with undrained shear strengths well below 40 kPa). These show that with soft clay the vacuum pressures stay below -40 kPa and blockage never occurred with the soft clay, not even under the so called bulldozing conditions.

Area1 and area 2 in the figure show successful tests with the 3 cm scoop and 5 cm scoop respectively; a small increase of bite size is achieved. Area 3 shows points of exceptionally high bite sizes which were successfully cut, but these concern shallow cuts and thus clay lumps with short lengths (see paragraph 9.11).

![Diagram showing the relationship between pump vacuum and in-situ bite size](image)

Figure 9.10: Pump vacuum as function of the in-situ bite size.
Figure 9.11 shows the vacuum readings of a normal test. The pump vacuum pressures during the test remain around -40 kPa. Figure 9.12 shows a vacuum reading of a test during which not long after start of the test the vacuum pressures increase to -80 kPa as evidence of a complete blockage. An example of the vacuum pressure in the case of bulldozing is shown in Figure 9.13. In this case the vacuum pressures increased to -75 kPa from the start of the test.

Figure 9.12: Vacuum readings with complete blockage not long after start.

Appendix 11 deals in more detail with vacuum measurements at De Beers Marine (Pty) Ltd. testing facility. The aim is to develop an understanding of the vacuum measurements at the suction side of the pump system and explain vacuum pressures measured during tests with different tools (under which the Gravel Wheel).
9.8 Hydraulic pressure

In Figure 9.14 the hydraulic pressure is plotted against the bite size. The maximum hydraulic pressure that can be delivered by the hydraulic pump is 200 bars and as such all tests fall under this tool limit. There is, as can be expected from theory, a strong and positive relationship (although the high scatter) between bite size and cutting force: see paragraph 6.1.3.

The hydraulic pressure as measured is the sum of both the pressure to turn the wheel and the pressure to cut the clay. Since the hydraulic pressure to turn the wheel is significantly different for the different rotational speeds this is accounted for in what is called the "corrected hydraulic pressure" which is the difference between the total hydraulic pressure and the hydraulic pressure needed to turn the wheel.

In Figure 9.14 the hydraulic pressures measured during tests with the soft clay is also plotted. As can be seen these are significantly lower (almost nonexistent) than tests conducted in stiff clay.

The high scatter is attributed to the large amount of influencing factors and the low accuracy with which they are known (for example: the hydraulic pressure is roughly read from hydraulic-time graph, the shear strength is not exactly know at the moment the test is conducted).
9.9 Causes for clogging

Clogging occurs:
1. Due to bulldozing
2. Or is a result of the clay lump size (bite size and length) and shear strength on the one hand and the converging inlet design on the other.

From observations the following interpretation of the two clogging mechanisms is given.

Theoretically bulldozing can be expected to start at a bite size as cut of 3.0 cm using the 3 cm scoops. It was observed that clogging started to occur from 2.7 cm. The mechanism that underlies bulldozing in this case is that when the flow is obstructed, the fluid flow is prevented to transport the cut clay lumps away. Then the tool starts building up clay chunks until the whole suction mouth is full; see Figure 9.15 and Figure 9.16.

The second cause for clogging is that at a certain clay lump size (combination of bite size and length), the lumps can get stuck in the converging part of the inlet towards the suction pipe when the clay is stiff enough to resist the drag force. That is why there is a certain limiting bite size above which the cut lumps are strong enough to resist the flow and get stuck in the inlet. Once the process of clogging starts the following lumps build up till the whole inlet in completely clogged as shown in the pictures. Another possibility is that at a certain clay lump size the amount of clay to be transported away becomes more than the 150 mm suction pipe can handle at one time.
9.10 Influence of shear strength

As already noted above the shear strength is an important factor in the clogging mechanism. One set of identical tests with only different shear strengths was conducted in which clogging occurred in the one with the stronger clay (shear strength between 70 and 80 kPa) and no occurrence of clogging with the softer clay (shear strength of 40-50 kPa). More tests are necessary to verify this.

9.11 Design guidelines

From the list of design guidelines presented in Chapter 8 the following are critical in the design of the Gravel Wheel:
1. Risk of bulldozing
2. Converging shape of suction duct
A new design guideline is identified: Potential obstructions in design: in the case of the Gravel Wheel the grizzly bars act as an obstruction to the cutting and slurryfication process. At the locations of the grizzly bars no cutting takes place along the cutting edge of the scoop. The clay builds up in front of the grizzly bar and forms a kind of bridge between the adjacent cut clay lumps. This now acts as an additional resistance to the flow. The clay lumps are held at the grizzly bar and the suction (drag force) on the lumps is not enough to release them.

Another reason for the clay lumps remaining in the scoop is that the clay is not cut completely to the clay surface, resulting in the clay bed being lifted in the last part of the cut. At a certain distance of the scoop edge to the surface tearing takes place towards the surface since there is nothing holding the clay down in its position. The overlying gravel does not have any confining effect at the clay-gravel interface since the gravel is lying loose on the clay. The clay remaining in front of the scoop again acts as a bridge between clay lumps. The clay lumps are only released (pushed free) by mechanical force of the next "bite".

One critical advantage of the grizzly bars is that it produces clay lumps to be produces and if missing, one single large sheet of clay would have been cut, probably being more problematic.

**Risk of bulldozing**

Equation 8.1 introduced a measure of the risk of bulldozing giving a limiting value below which bulldozing is not likely to occur.

This has been quantified with tool opening size set to the scoop opening size along the grizzly bars; the opening size of the 3 and 5 cm scoop are 5 and 6.3 cm respectively (see Appendix 8). The bite sizes that could be handled by the 3 and 5 cm scoop were found to be respectively 2.4 and 2.9 cm, giving a ratio of bite size to tool opening size of 0.48 and 0.46. These bite sizes also take into account the converging suction duct which additionally limits the bite size that can be handled by the system and are only valid to a cut depth of 100 mm. This design guideline is valid for shear strengths above 50 kPa.

This leads to the following quantified design guideline:

\[
\frac{\text{bite \cdot size} \text{"as \cdot cut"}}{\text{tool \cdot opening \cdot size}} \leq 0.45 - 0.50
\]

(9.1)

**Figure 9.17: Bite size "as cut" vs. tool opening size.**

**Converging shape of suction duct**

The maximum bite size the tool can theoretically take is the tool opening size itself. There seems to be a trend that with increasing cut depth, the bite size that can be handled by the system decreases. This is thought to be due to the increasing length of the cut clay lumps with increasing cut depth. There seems to be a cut depth of between 65 and 75 cm for the 5 cm scoop where there is a sudden drop in bite size handled as shown in Figure 9.18. The author believes that the length of the slice has reached a length that the converging suction duct directly influences the cut clay lumps. The convergence in the suction duct causes an additional resistance to the flow. This is more apparent for higher shear strength as noted in paragraph 9.10.
The distance of the tip of the scoop to the start of the converging section is measured to be 17 cm for the 5 cm scoop. The theoretical total length of clay lump that is cut at cut depths of 65 to 75 cm is about 25-26 cm, as can be read from Figure 9.19.

That the theoretical length is larger than 17 cm is considered to be due to the small thickness of the first part of the cut slice, making the effective length smaller. See Figure 9.20 for a sketch of the situation of the clay lumps in the suction mouth.

The following general design guideline is derived:

\[
\text{Effective length of clay lump} \leq \text{Length of parallel section suction duct} \tag{9.2}
\]

Figure 9.18: Sudden decrease in bite size “as cut” at specific cut depth.

Figure 9.19: Theoretical total length of clay lump cut.
Shear strength
Initial tests were conducted by de Beers Marine in soft clay with strength lower than 40 kPa. These tests never showed any sign of clogging although the bite size was much larger than the tool opening size. Based on this experience and the results of the tests in this study the following design guideline is derived:

"No design restrictions with respect to clay clogging if shear strength ≤ 40 kPa"  \( (9.3) \)

9.12 Test results clay and gravel mining tests

The second series of tests were performed to derive under which parameters the tool still was able to cut clay whilst mining gravel. In the tests mostly undersize gravel \(< 60 \text{ mm}) is used. There was some variation in the gravel used with variable amounts of near size and sometimes significant amounts of oversize. Only qualitative observations are used to evaluate the tools performance.

In first tests cutting into a bed of gravel and clay in one single pass an undesirable phenomenon is observed that further is named "carry-over" (see also paragraph 9.10.1). A lot of undersize gravel that should have been hydraulically collected by the tool was left on the mined out lane. The only way that that could have happened is when it was somehow carried over the back of the drum by the scoops acting as a spade. Significant to serious carry-over already occurred at cut depths in clay of 20 mm at a forward mining speed of 2.5 m/min and 10 RPM (see Figure 9.21). Significantly less carry-over was observed at a rotational speed 20 RPM (with same forward mining speed and rotational speed) compared to 10 RPM.

In order to avoid the undesirable carry-over to occur the scoops were re-designed as will be discussed in paragraph (9.10.2). The modification to the scoop resulted in a considerable reduction of carry-over. In order to evaluate the modified scoops not all 5 scoops were modified at once. One scoop was not changed and from the 4 other scoops there were two kinds (one set with one modification and another with two modifications).

With the modified scoops carry-over at 20 mm cut depth was insignificant and the few carried over undersized gravel parts is attributed to the one single old scoop still on the drum (see Figure 9.22). Reduced but still significant carry-over occurred at 50 mm cut depth. In order to be able to cut 50 mm of clay the forward mining speed was reduced to 1.7 m/min giving the system the same bite size (of 1.7 cm) as at the 20 mm cut. This indicated again the bite size as governing factor and proved that 50 mm of clay could be cut at a reduced forward mining speed of 1.7 m/min (See Table 9.1 and Table 9.2 for summary of the test results).
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Figure 9.21: Serious carry-over as observed on 13 May 2003.

Figure 9.22: Very little carry-over as observed on 17 June 2003.
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<table>
<thead>
<tr>
<th>Date</th>
<th>Forward mining speed and Rotational speed</th>
<th>Bite size (cm)</th>
<th>Cut depth</th>
<th>Scoop</th>
<th>Notes</th>
<th>Carry-over</th>
</tr>
</thead>
<tbody>
<tr>
<td>13 May</td>
<td>2.5 m/min, 10 RPM</td>
<td>2.5</td>
<td>20 mm</td>
<td>5 cm scoops</td>
<td>All passing gravel, no/little near/over size</td>
<td>Serious carry-over</td>
</tr>
<tr>
<td>13 May</td>
<td>2.5 m/min, 10 RPM</td>
<td>3.3</td>
<td>50 mm</td>
<td>5 cm scoops</td>
<td>All passing gravel, no/little near/over size</td>
<td>Serious carry-over</td>
</tr>
<tr>
<td>16 May</td>
<td>2.5 m/min, 20 RPM</td>
<td>1.7</td>
<td>20 mm</td>
<td>5 cm scoops</td>
<td>All passing gravel, no/little near/over size</td>
<td>Significant carry-over, but significantly less than 13 May</td>
</tr>
<tr>
<td>16 May</td>
<td>2.5 m/min, 20 RPM</td>
<td>2.2</td>
<td>50 mm</td>
<td>5 cm scoops</td>
<td>All passing gravel, no/little near/over size</td>
<td>Significant carry-over, but significantly less than 13 May</td>
</tr>
<tr>
<td>19 May</td>
<td>2.5 m/min, 20 RPM</td>
<td>1.7</td>
<td>20 mm</td>
<td>3 cm scoops</td>
<td>All passing gravel, no/little near/over size</td>
<td>Carry-over</td>
</tr>
<tr>
<td>19 May</td>
<td>2.5 m/min, 20 RPM</td>
<td>2.2</td>
<td>50 mm</td>
<td>3 cm scoops</td>
<td>All passing gravel, no/little near/over size</td>
<td>Carry-over</td>
</tr>
<tr>
<td>17&amp;25 June</td>
<td>2.5 m/min, 20 RPM</td>
<td>1.7</td>
<td>20 mm</td>
<td>Adjusted 5 cm scoops</td>
<td>All passing gravel, some near/over size</td>
<td>Little carry-over</td>
</tr>
<tr>
<td>17&amp;25 June</td>
<td>2.5 m/min, 20 RPM</td>
<td>2.2</td>
<td>50 mm</td>
<td>Adjusted 5 cm scoops</td>
<td>All passing gravel, some near/over size</td>
<td>Carry-over, but significantly less than 16 May</td>
</tr>
<tr>
<td>26 June</td>
<td>2.5 m/min, 20 RPM</td>
<td>2.2</td>
<td>50 mm</td>
<td>Adjusted 5 cm scoops</td>
<td>All passing gravel, significant near/over size</td>
<td>Serious carry-over</td>
</tr>
<tr>
<td>26 June</td>
<td>1.7 m/min, 20 RPM</td>
<td>1.7</td>
<td>50 mm</td>
<td>Adjusted 5 cm scoops</td>
<td>All passing gravel, significant near/over size</td>
<td>Little carry-over</td>
</tr>
<tr>
<td>3&amp; 8 July</td>
<td>1.7 m/min, 20 RPM</td>
<td>1.7</td>
<td>50 mm</td>
<td>Adjusted 5 cm scoops</td>
<td>All passing gravel, significant near/over size</td>
<td>Very little carry-over</td>
</tr>
<tr>
<td>3&amp; 8 July</td>
<td>1.9 m/min, 20 RPM</td>
<td>1.7</td>
<td>50 mm</td>
<td>Adjusted 5 cm scoops</td>
<td>All passing gravel, significant near/over size</td>
<td>Very little carry-over</td>
</tr>
</tbody>
</table>

Note: A mix of adjusted and original 5 cm scoops was used in the tests of 17 to 26 of June.

Table 9.2: List of tests carried out in gravel and clay.
9.12.1 Causes for “carry-over”
It has been observed that a lot of gravel is being carried over to the back of the drum ("carry-over") during the tests. This is thought to be caused by the clay lumps remaining in the scoop openings as shown in Figure 9.23 (see also paragraph 9.11) reducing the scoop openings considerably. Only the size fraction that can pass through the remaining opening is taken away. For the rest of the gravel, the grizzly acts as a scoop, carrying it over and depositing it behind the drum. Mining wise this is unacceptable/unwanted.

Figure 9.23: Clay lumps in the scoop opening.

9.12.2 Scoop modifications
The original 5 cm scoops are modified as follows to try to prevent the clay lumps to remain in the scoop inlet (see Appendix 8C for the design of the modified scoop):

1. The 12 mm round grizzly bars are replaced by 4 mm flat blades. This change is expected to reduce the obstructing behaviour of the round grizzly bars (see also paragraph 9.9) and prevent clay bridges to be formed.
2. A deflection plate was welded inside the scoop in order reduce contact area with the tool and to present the clay lumps much better to the flow. Holes in the back of the plate were added with the intention to avoid a low-pressure zone to be formed at the back of the deflection plate. These did not get clogged in any of the tests.
3. The 4 mm blades were designed with a 5 mm protrusion above the blade to try to prevent the clay bed being lifted in the last part of the cut (see paragraph 9.9).

Unfortunately it is observed that the problem of the clay being lifted in the last part of the cut has not been solved and this may result in an under-estimation of the actual prototype operation. More efforts have to be made to prevent this problem from occurring in the scaled test to achieve similarity between prototype and model.

9.12.3 Conclusions on clay and gravel tests
A limited number of tests have been carried out in a gravel and clay bed to get insight in the performance of the Gravel Wheel. Initial efforts have been made to counteract the observed carry-over phenomenon and steps have been set in the right direction. Unfortunately the number of tests carried out is not enough to define exact trends and it is recommended that more tests be carried out.

Considering that the requirement of the Gravel Wheel’s ability to penetrate the footwall is set at 50 mm on full scale (and thus 17 mm on 3rd scale) and considering the achieved 60 mm penetration (20 mm on 3rd scale) in clay, it could be stated that the requirement has been met.
But given the uncertainty of the undulation of the actual clay footwall, the unknown accuracy with which
the tool can follow the clay-gravel interface and the fact that the scaled model cannot cope with 50 mm
of clay (full scale 150 mm), this result is according to the author not yet satisfying. As such the Gravel
Wheel's design to improve the ability to penetrate clay still requires further thought and refinement.

9.13 Fourth Scale considerations

Re-considering the existing model Gravel Wheel as a 1/4 scale model instead of the intended 1/3 scale
shows a potentially remarkable increase in mining rate without actual scaled tool improvement. With the
1/3 scale model already achieving required mining rates, changing to 1/4 scale would mean achieving
mining rates far above initial requirements and expectations (see Table 9.3).

This change of thought has to be evaluated and the implications quantified. Below follows an initial re­
evaluation of the results achieved in this study. It is by no means complete and a more thorough
evaluation is recommended.

The implications on volumetric mining rate and area coverage rate have been quantified in Table 9.3. The
table gives a complete comparison of considering the tool at 1/3 or 1/4 scale with achieved forward
mining speeds of 2.5 and 1.7 m/min for 20 mm and 50 mm clay cuts (scaled values) respectively. The
forward mining speeds do not yet include any consideration of overlap of subsequent lanes (see Smith,
2003).

It can be concluded that although the tool can achieve 8000 m²/day at 2.5 m/min when considered as
1/4 scale, it only achieves a 50 mm cut (200 mm cut on full scale) at 1.7 m/min. Fortunately this still
gives a area coverage rate above full scale specification.

<table>
<thead>
<tr>
<th>Area Coverage Rate (full scale)</th>
<th>Volumetric Mining Rate (full scale)</th>
<th>Cut depth in clay (full scale)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Full scale specification</td>
<td>5000</td>
<td>216</td>
</tr>
<tr>
<td>Achieved at 1/3 scale and 2.5 m/min</td>
<td>5200</td>
<td>227</td>
</tr>
<tr>
<td>Achieved at 1/3 scale and 1.7 m/min</td>
<td>3500</td>
<td>154</td>
</tr>
<tr>
<td>Potential at 1/4 scale and 2.5 m/min</td>
<td>8000</td>
<td>349</td>
</tr>
<tr>
<td>Achieved at 1/4 scale and 1.7 m/min</td>
<td>5400</td>
<td>238</td>
</tr>
</tbody>
</table>

Note: The forward mining speeds are still scaled values to easily compare to test results.

Table 9.3: Comparison of 3rd and 4th scale considerations.

Further:
The actual screen size changes from 180 mm to 240 mm. The implication is that from the in-situ gravel
less will be screened and more will be taken up by the hydraulic system. This will have consequences
for the downstream gravel treatment and diamond recovery plant and this has to be further assessed in
detail.

Another, positive, implication is that because less material will have to be screened the occurrence of
blockage is greatly reduced since it is the oversize that is the primary cause of blockage at the screen
interface. A negative consequence is that the energy required by the system will increase, together with
the actual dimensions of the tool and mining system and feasibility studies will need to prove if such a
system can be practically and economically achieved.
9.14 Conclusions

The following conclusions can be drawn for clay handling:

1. By increasing the tool rotational speed an increase in cut depth can be achieved
2. The 5 cm scoop design is a small (15%) improvement over the 3 cm scoop design
3. Two mechanisms for clogging are recognized:
   a. Due to bulldozing
   b. As a result of the clay lump size and shear strength on the one hand and the converging inlet design on the other

Regarding gravel and clay mining following conclusions are drawn:

3. Unwanted carry-over of undersize gravel is observed when mining clay and gravel together which is caused by
   a. Grizzly bar design of Gravel Wheel
   b. Limitation of the test set-up
4. Improved scoop design reduces the observed carry-over considerably
5. A 20 and 50 mm cut depth has been achieved at forward mining speeds of 2.5 and 1.7 m/min respectively.

Minimal spillage has been observed during both the tests in clay only and the tests in clay and gravel. This confirms the expected advantage of the Gravel Wheel to minimize possible loss of diamonds through spillage.
10. Conclusions and recommendations

10.1 Conclusions

Investigation of the possibility to simulate clay in scaled down tests

- Apart from the 3 already defined clay types in the Atlantic 1 mining area, a fourth clay type (sandy-silty clay) has been recognized based on textural properties.

- In spite of the limited spatial distribution of footwall samples taken, already 4 different clay types have been identified. This indicates that in the Atlantic 1 mining area there is a high variability in clay types that can change over very short distances. Despite this high variability, these four clay types cover a wide range of clay categories and are thought to be representative enough for possible clay types to be encountered by the high rate mining system.

- The measured shear strength values of the dataset (with average of 40 kPa) which were previously used by De Beers Marine for scaled testing purposes, are not representative for the footwall clays. The actual shear strength is much higher than initially expected.

- Three testing scenarios were designed in the attempt to achieve similarity between model and prototype scale and all three scenarios showed scale effects. Testing with scaled clay and Froude velocities is theoretically the best scenario for scaled testing. But clay available for testing at De Beers Marine testing facility did not meet the strength/consistency-requirement set.

- Not being able to test with scaled clay, tests have been conducted with clay of full scale strength and Froude velocities. Although testing under this scenario presents the tool to a tougher condition than required, it is argued that if the scaled tests are successful, the tool will definitely work on full scale. Consistency indices and shear strengths of test clay type used are close to actual footwall clay types, satisfying the consistency and shear strength requirements. The adhesion of the test clay is lower than required and in scaled testing the adhesion is underestimated.

- Testing with clay around and below 40 kPa (probable range of scaled clay shear strengths) under submerged conditions makes the consistency and shear strength difficult to control since the clay will deteriorate fast by easily absorbing more water. Because the strength/consistency-requirement as set by scenario 1 are difficult to be met, it makes this scenario unfeasible for scaled testing under conditions prevailing at De Beers Marine testing facility.

- The adhesion of the stiff test clay is lower than the required adhesion on 1/3 scale making the model tests underestimate the full scale adhesion.

- The ductile failure type observed in the performed model tests is at least as unfavourable as on full scale.

- The stiff test clay type models the behaviour of the Eocene and Cretaceous 2 footwall clay types best.
Definition of basic design guidelines for dealing with clay

• From experience in dredging several design guidelines have been derived to evaluate a tool's clay cutting ability and an initial evaluation of the Gravel Wheel has been made.

• The Gravel Wheel scores negative on the risk of bulldozing, openness of the tool and the converging shape of the suction duct and these features are expected to be most critical in the Gravel Wheel's design. This evaluation resulted in a modified scoop design for testing in clay.

Definition of the clay handling ability of the Gravel Wheel

• The maximum cut depth at which the tool remains free from blockage has been experimentally defined with increased cut depth achieved with increased rotational speed and modified scoop design. On model scale a 100 mm cut has been achieved with a rotational speed of 20 RPM and a modified scoop design.

• On model scale and with modified scoop design a 20 mm and 50 mm cut in clay has been achieved at forward mining speeds of 2.5 and 1.7 m/min respectively.

• Three design guidelines, specific to the Gravel Wheel, has been derived:
  • The ratio of the bite size "as cut" to the tool opening size has to be smaller than a specific value of 0.45 to 0.50.
  • The effective length of the clay lump must be smaller than the length of the parallel section of the suction duct.
  • Clay with shear strengths below 40 kPa poses no clogging problems to the Gravel Wheel.

The test results, leading to these design guidelines, confirm that the risk of bulldozing and the converging shape of the suction duct are the most critical features in the Gravel Wheel's design.

• Translating these scaled test results to full scale conditions it can be concluded that the tool will definitely work on full scale cutting 60 mm and 200 mm of clay at forward mining speeds of 4.3 and 2.9 m/min respectively. The derived design guidelines are on the conservative side and if followed, testing under full scale conditions will be successful.

• The maximum cut depth achieved is a function of the bite size, length and strength of the cut clay lumps.

• Monitoring of pump vacuum and hydraulic pressure have a potential of giving a measure of the cut depth in clay and could be further developed as predictive indicators in future full scale operation.

• An undesirable phenomenon called carry-over has been observed in clay and gravel mining tests, limiting the cut depth that can be achieved considerably.

• The major cause for carry-over is the obstructing side effect of the grizzly bars in combination with limitations in the test set-up. Further modifications to the grizzly bars are made to reduce the obstructing side effect.

• Considering the scaled test at 1/4 scale instead of the initially intended 1/3 scale increases the maximum cut depth that can be achieved from 60 mm to 200 mm on full scale, whilst still satisfying mining rate requirements.

• The expected advantage of the Gravel Wheel to minimize possible loss of diamonds through spillage has been confirmed by the tests.
10.2 Recommendations

- Currently only limited shear strength data of the clay footwall is available and it is recommended that more shear strength testing is done (preferably with a Pilcon shear vane) to get more insight in the actual clay footwall shear strengths.

- More tests should be performed with the simple adhesometer of different, more adhesive clay types to assess the accuracy and reliability of the device.

- Initial tests in clay and gravel have proven valuable to study the clay and gravel interaction during the mining process and initial trends have been observed. More tests in clay and gravel should be performed to find the optimum operational parameters (e.g. forward mining speed) at different set cut depths.

- Each new concept of a mining tool should be evaluated with the derived design guidelines.

- The necessity to achieve a specific cut depth in clay depends all on the degree of undulation and the accuracy with which the clay-gravel interface can be followed. It is therefore recommended to further study these factors in order to better define the clay cut depth requirement of any mining tool.
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