AES/RE/14-18  In-pit co-disposal of tailings in an onshore dredge mine

05/09/14  M.C. Ensing
Summary

This research aims to provide an understanding of the behaviour of co-disposed coarse (sand) and fine (clay and silt) tailings when discharged into an onshore dredge pond, and the dependence of that behaviour on the sediment composition. This understanding contributes to the ultimate goal of finding a method by which fine tailings can be discharged back into the original dredge pond, while preventing these fines from recirculating through the production process. This would eliminate any need for external tailings dams, thus avoiding the risks traditionally associated with these dams. This study is part of an effort by Royal IHC to make dredge mining a competitive method for mining a wide variety of mineral deposits.

To describe the behaviour of co-disposed coarse and fine tailings, an analogue is found in debris flows. From this analogue, four key behavioural aspects of mud in water are defined: erosion of sediment or entrainment of water, hydroplaning, segregation of coarse and fines particles in a slurry, and the strength increase of the material after deposition. Literature on debris flows suggested that the cohesion of the material, in the description of debris flows usually captured in a yield stress, is instrumental in determining if any of the first three behaviours are to be expected. The fourth characteristic, strength increase, depends mainly on material specific relations between void ratio on one hand, and permeability and effective stress supporting capacity of the material on the other.

The concept of yield stress shows many similarities with that of the undrained shear strength. Therefore it is investigated if the yield stress can be calculated as a function of a material’s Atterberg limits – as is possible with the undrained shear strength – and thus provide a method of predicting the slurry flow behaviour. This removes the need for relationships between the less easily captured quantities as particle size distribution and clay mineralogy, and the yield stress.

Experiments are performed on modelling clay and three mixtures of this modelling clay with a silica flour to determine the relationships of both undrained shear strength and yield stress with the liquidity index. Testing is done on soils with different Atterberg limits to confirm dependence on liquidity index only. The undrained shear strength is measured using a fall cone test. The yield stress is measured using a viscometer. To determine the yield stress, a full rheogram is measured for each sample. A Bingham type rheological model, of which the yield stress is a parameter, is then fitted to this rheogram.

The yield stress and the undrained shear strength are plotted together as a function of liquidity index and proved to fit to the same curve, which thus describes the strength of a soil from a liquidity index of 0 up to 4.

The relationship between the yield stress and clay content of the tested samples is also investigated. It is found that the concentration of the clay-sized fraction in water correlated well to both the yield stress and undrained shear strength. Hence it is concluded that yield stress is primarily determined by the clay content, and that any other size fraction has negligible effect on the yield stress of slurries.

This changes when sand is added to diluted soil, as is suggested with the co-disposal method. At low sand concentrations, the effect of sand on the yield stress is still minor or even negligible. At higher concentration however, the sand noticeably increases the yield stress of the overall fluid. At a certain sand concentration, the sand grains form a network throughout the material, at which point
it can no longer be considered a fluid. Silt is found to have a similar influence as sand on the slurry yield stress.

The yield stress of the modelling clay in the experiments is plotted as a function of both the fines concentration in water and the sand concentration in the overall mixture. An optimum mixture range is identified based on earlier identified limits: As a minimum, the yield stress should be 35 Pa for a slurry to remain coherent at low slope angles. As a maximum, the pumpability with a centrifugal pump is used. This can be guaranteed up to a value of 200 Pa. However, for different pumps different values apply.

It is concluded that the clay fraction of a sediment is the critical mixture component for providing yield stress in when the material is slurried. Sand and silt sized material only have noticeable effects when their concentrations reach such levels that they form the main structure of the mixture. However, as the relationship between the carrier fluid’s clay content and yield stress is heavily dependent on the clay mineralogy, a more convenient method of estimating a slurry’s yield stress, and thus suitability for deposition, uses the liquidity index of the fines fraction.
Acknowledgement

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<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
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<tbody>
<tr>
<td>$A$</td>
<td>Clay activity [-]</td>
</tr>
<tr>
<td>$A_k$</td>
<td>Material constant for permeability-void ratio relationship [-]</td>
</tr>
<tr>
<td>$A_m$</td>
<td>Material constant for permeability-void ratio relationship [-]</td>
</tr>
<tr>
<td>$A_p$</td>
<td>Material constant for effective stress-void ratio relationship [-]</td>
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<td>$B_s$</td>
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<td>$c$</td>
<td>Cohesion [kPa]</td>
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<tr>
<td>$c_L$</td>
<td>Undrained shear strength at liquid limit [kPa]</td>
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<tr>
<td>$c_u$</td>
<td>Undrained shear strength [kPa]</td>
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<td>$d_f$</td>
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<td>Gravitational acceleration [m/s$^2$]</td>
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<td>$h$</td>
<td>Penetration depth [mm]</td>
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<tr>
<td>$H$</td>
<td>Height (of vane or rotor) [mm]</td>
</tr>
<tr>
<td>$H_{fl}$</td>
<td>Thickness of slurry [m]</td>
</tr>
<tr>
<td>$H_{max}$</td>
<td>Yield thickness of slurry [m]</td>
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<tr>
<td>$I_l$</td>
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<td>$k$</td>
<td>Permeability (specifically hydraulic conductivity) [m/s]</td>
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<td>Bingham viscosity [Pa·s]</td>
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<td>Mass [kg]</td>
</tr>
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<td>$m_{cl}$</td>
<td>Mass of clay [kg]</td>
</tr>
<tr>
<td>$M_{sl}$</td>
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<td>Mass of water [kg]</td>
</tr>
<tr>
<td>$M$</td>
<td>Torque [Nm]</td>
</tr>
<tr>
<td>$M_{peak}$</td>
<td>Peak torque readout of vane device [Nm]</td>
</tr>
<tr>
<td>$Q$</td>
<td>Weight of cone [mN]</td>
</tr>
<tr>
<td>$R^2$</td>
<td>Coefficient of determination [-]</td>
</tr>
</tbody>
</table>
\[ R_o \] Inner diameter of stator [mm]
\[ R_i \] Outer diameter of rotor [mm]
\[ Re_f \] Floc Reynolds number [-]
\[ s.g. \] Specific gravity [-]
\[ t \] Time [s]
\[ U_h \] Flow velocity of slurry [m/s]
\[ w \] Moisture content [-]
\[ w_l \] Liquid limit [-]
\[ w_p \] Plastic limit [-]
\[ w_s \] Settling velocity [m/s]
\[ W1 \] Mass of empty container [kg]
\[ W2 \] Mass of container + wet sample [kg]
\[ W3 \] Mass of container + dry sample [kg]
\[ Y_{\text{max}} \] Particle stability criterion [-]
\[ z \] Vertical Eulerian coordinate [m]

**Greek symbols**

\[ \alpha \] A sphericity factor [-]
\[ \beta \] Slope angle (in one case a sphericity factor) \(^{[\circ]}\) (or [-])
\[ \dot{\gamma} \] Shear rate \([1/s]\)
\[ \delta \] Deflection angle \([^\circ]\)
\[ \Delta \rho \] Difference between slurry density and ambient fluid density \([\text{kg/m}^3]\)
\[ \zeta \] Vertical material (Langrangian) coordinate
\[ \mu \] Dynamic viscosity \([\text{Pa} \cdot \text{s}]\)
\[ \mu_w \] Dynamic viscosity of water \([\text{Pa} \cdot \text{s}]\)
\[ \mu_0 \] Apparent viscosity \([\text{Pa} \cdot \text{s}]\)
\[ \xi_{cl} \] Clay mass fraction of solids [-]
\[ \xi_{cl0} \] Clay mass fraction at \(\lambda_p = 0\) in activity plot [-]
\[ \xi_{cli} \] Clay mass fraction of clay sized particles in suspending fluid [-]
\[ \xi_{cl-min} \] Mass fraction of clay mineral particles in suspending fluid [-]
\[ \xi_{fi} \] Mass fraction of fines in suspending fluid [-]
\[ \xi_{sa} \] Mass fraction of sand in slurry [-]
\[ \rho_b \] Bulk density of slurry \([\text{kg/m}^3]\)
\[ \rho_s \] Density of solids \([\text{kg/m}^3]\)
\[ \rho_w \] Density of water \([\text{kg/m}^3]\)
\[ \sigma_{t}^f \] Effective normal stress \([\text{kPa}]\)
\[ \sigma_{v}^f \] Vertical effective stress \([\text{kPa}]\)
\[ \tau \] Applied shear stress \([\text{Pa}]\)
\[ \tau_f \] Frictional shear stress \([\text{Pa}]\)
\[ \tau_y \] Yield stress (of slurry) \([\text{Pa}]\)
\[ \tau_{y}^* \] Yield stress of suspending fluid \([\text{Pa}]\)
\[ \phi \] Internal friction angle \([^\circ]\)
\[ \phi_{cl} \] Volumetric fraction occupied by clay [-]
\[ \phi_{co} \] Volumetric fraction occupied by coarse-sized solids fraction [-]
\[ \phi_{S} \] Volumetric fraction occupied by solids [-]
\[ \phi_{\text{max}} \] Maximum volumetric fraction of coarse-sized solids fraction [-]
1. Introduction

Waste disposal has become a critical part of any mining operation in the past decades. Failure to store mine waste, especially the very fine tailings, in a secure manner can result in the loss of one's license to operate. Traditionally wet fine tailings are stored in large basins, which are surrounded by artificial dikes, and are called tailings dams. These dams have a number of drawbacks and risks associated with them: depending on the size of the mining operation, these dams can span areas of multiple square kilometres. This effectively prohibits mining near populated areas. Additionally the surrounding dikes are susceptible to failure, which can lead to large flows as the tailings flow out of the dam. As fine tailings do not consolidate rapidly, the risk of dam failure and subsequent tailings flow remains for many years afterwards.

Dredge mining is the act of mining a mineral deposit using dredging equipment. Royal IHC (IHC) is a pioneer in this method, having assisted in designing and having built equipment for a number of operational dredge mines. The main advantages of this method are a high production rate and low operating costs when the material processing is performed in a wet process. A disadvantage is that dredge mining offers decreased flexibility as there are usually only a couple of dredgers per operation. There are three types of dredge mines: offshore, near-shore and onshore. In offshore dredge mining the minerals are dredged from the seafloor. An example of an offshore operation is the Peace in Africa, a ship that mines diamonds at 200m below the water surface off the coast of Namibia. This is not to be mistaken for deep-sea mining, which takes place at much greater depths. A broader insight into mining marine diamonds is offered by Foster (2014). Near-shore also includes alluvial mining. This means dredging the beds of rivers and lakes, as is done for instance in Surinam with gold. For onshore dredge mining an artificial pond is created for the dredge to float in while it dredges the pond bottom and the surrounding sediment. In South Africa, heavy minerals such as ilmenite, rutile and zircon are dredged onshore from the dunes north of Richards Bay (Figure 1). This study focuses on the onshore environment, where dredge mining has to compete with dry mining methods. Mol et al. (2012) provide an overview of Richards Bay Minerals and a comprehensive introduction into onshore dredge mining.

The bottom panel of Figure 1 shows a schematic long section of an onshore dredge mine. At one end of the mine (the ‘front’) production takes place, while at the other end (the ‘rear’) the pond is filled in with the waste material. In the dredge pond, a cutter suction dredge (CSD) lays anchored to two or more anchors laying on either side of the dredge, and a spud pole attached to the rear of the dredge. The dredge can be rotated by pulling on the anchor cables, using the spud pole as a pivot. The pole is attached to the dredge via an hydraulic ram, allowing the dredge to apply pressure on the cutter head. The cutter head is attached to the dredge via the ‘ladder’. The ladder contains the suction pipe and is suspended by cables coming from the front of the dredge, allowing for a variable depth of the cutter head. A cutting depth of 15m is typical for CSD’s used in mining. After cutting the material is pumped onwards to further processing, where the ore is separated from the waste. The waste material is pumped to the rear of the pond where it is deposited on the pond edge (Hogeweg, 2014, pers. comm.). The photograph in Figure 1 provides a sense of the scale of the tailings disposal problems, as the deposition area can be seen to stretch out far into the distance.

While dredge mining finds its main application in commodities typically associated with coarser sediments, some deposits also contain large amounts of fine material such as clay and silt in the
overburden, the mineralised zone, or both. Currently, deposits with large amounts of fine overburden and surrounding sediments are typically mined with dry mining methods. These methods allow the finer material to be separated from the mineralised area in the dry state, significantly reducing the problems associated with fine sediments: dry fines have less volume and have more strength than wet fines. Fines coming from the mineralised zone have passed through the processing plant and are thus in a wet state. These are currently deposited in tailing dams with all the associated risks and costs.

1.1 Motivation
Dredge mining offers a unique opportunity for the secure and cost-efficient disposal of wet fine sediments, as the need for an external dam can be severely decreased or even eliminated. In onshore dredge mining, an artificial pond is already created during the mining process. This pond can likely used to dispose of the wet fine tailings. With no tailings dam, the operation has a significantly smaller footprint. This also eliminates the risk of dam failure, as no dams need to be constructed. Filling the pond with tailings also removes the need to separately reclaim the pond area. The pond is usually reclaimed as a lake, meaning the area cannot be used for other purposes anymore. By filling the pond more land remains available for other purposes, such as agriculture.

The main risk associated with this disposal method is recirculation of fine sediment through the production process. To limit this recirculation, resuspension of the fines in the pond water, as well as flowing of the fines slurry across the entire pond should be prevented. Adding coarse sediments such as sand – which is also a waste material associated with dredge mining – to the slurry may increase the rate of consolidation and thus decrease the consolidation time. The investigation therefore focuses on the deposition of a mixture of fine and coarse sediments.
1.2 Aim and objectives
This research aims to provide an understanding of the behaviour of co-disposed coarse and fine tailings when discharged into an onshore dredge pond, and the dependence of that behaviour on the sediment composition with an emphasis on preventing fine sediment from recirculating through the production process.

To achieve this goal, the following objectives are defined:

1. Identify and describe the (flow) mechanisms involved in the depositional behaviour of the wet fine sediments and that have an impact on material resuspension, flow mobility and consolidation behaviour as a function of material properties.
2. Define limits to the described mechanisms beyond which submerged tailings deposition is considered not feasible.
3. Describe the effects of the addition of coarse sediments to the fine sediment on the flow mechanisms identified in objective 1.
4. Identify and attempt to plug knowledge gaps in the prediction of the depositional behaviour of the mixture of fine and coarse sediments.
5. Quantitatively link the described limits (objective 2) to the composition (sand/fines/water) of the deposited mixture.

1.3 Research questions
The overall research question is:

What is the effect of the different components of a mixture of sand, fines and water on the mixture’s ability to be deposited in a dredge pond such that no disposed material re-enters the production cycle?

To attain the objectives stated in the previous section, the following secondary research questions are posed:

Objective 1

1.1 By what mechanism is fine sediment suspended in the pond water and how is this related to the material properties?
1.2 How is flow mobility related to the material properties?
1.3 How is consolidation behaviour related to the material properties?

Objective 2

2.1 Which behaviour should the mixture exhibit in the depositional mechanisms identified in Objective 1 for the depositional method to be feasible?
2.2 What are the limits in material properties for the feasible adoption of the proposed deposition method?

Objective 3

3.1 How are the material properties identified as relevant in Objective 1 related to the mixture composition?
Objective 4

4.1 What extra knowledge is needed to successfully use the knowledge from objectives 1, 2 and 3 to be able to complete Objective 5?

Objective 5

5.1 What are the mixture compositions that correspond to the limits on the material parameters as defined in Objective 2?

1.4 Hypothesis
It is hypothesised that the method is feasible at low water contents and at high sand contents of the mixture and becomes increasingly feasible with decreasing fines content.

1.5 Report outline
The report is structured according to the objectives: Chapter 2, the literature review, deals with the determination of the behavioural characteristics of fine sediment (mud) flows in water (objective 1). The main flow mechanisms acting on subaqueous mud flows are presented, and the material properties controlling them are identified. Chapter 3 analyses the relationships between these flow mechanisms and the sediment properties. A literature study into the effects of coarse sediment addition is also presented in this chapter (objective 3). From these analyses, a knowledge gap in the prediction of a slurry’s yield strength prediction presents itself, as the role of silt is found to be ill-defined (objective 4). This role is subsequently investigated in Chapter 4. In Chapter 5 the results from the experiments and literature are combined to present a graphical representation of the material compositions for which the presented disposal method is mechanically feasible (objective 5). Included in Chapter 5 is a discussion on the validity of the results and the further investigation needed before the method can be successfully implemented. Chapter 6 presents an overview of the conclusions and recommendations.

1.6 Scope and limitations
The research only deals with the mechanics of the mixed waste material as it is deposited in the pond. The assumption is that the material is deposited on a submerged slope, after which it may or may not flow down, depending on the mechanics of the situation. Any chemical or biological activity taking place during the process is not considered. The material is assumed to be fully homogeneously mixed throughout the entire process. The boundary conditions for this assumption are defined in Chapter 2. An overview of included and excluded topics is presented in Table 1.
Table 1: The limitations to the scope of this study.

<table>
<thead>
<tr>
<th>Category</th>
<th>Included</th>
<th>Excluded</th>
</tr>
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<tbody>
<tr>
<td>Mining and deposition</td>
<td>Onshore dredge mining method</td>
<td>All other mining methods, particularly offshore and alluvial dredge mining</td>
</tr>
<tr>
<td></td>
<td>Deposition of waste</td>
<td>Transport of waste</td>
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<td></td>
<td>Subaqueous deposition</td>
<td>Subaerial Deposition</td>
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<td></td>
<td>Use of spigots</td>
<td>Use of diffuser</td>
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<tr>
<td>Depositional phenomena</td>
<td>Resuspension</td>
<td>Chemical alteration</td>
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<td></td>
<td>Flow mobility</td>
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2. Literature review – Behaviour of mud in water
In this chapter, the mechanisms affecting the behaviour of fine sediment slurries in water are identified and described. To acquire an understanding of the processes happening within a slurry, the behaviour of fine sediment in water and the effects of sand when mixed with the mud are described in Section 2.1. After this, the mechanical behaviour of fluids in general, and dense sediment-water slurries in particular is explained in Section 2.2. Finally, the more complex interactions of dense muddy slurries with their surroundings as they flow down slopes are explained in Section 2.3. Section 2.4 summarises and concludes this chapter.

2.1 Characteristics of cohesive sediment in water
This section aims to provide sufficient background information to give the reader an understanding of the complex internal structure of muddy slurries and the dependency of this structure on the slurry composition. The section is structured to follow the settling and consolidation process of mud particles in water; while the initial subsections deal with individual mud particles and low sediment concentrations, the later sections deal with space filling networks of sediment at sediment concentrations approaching those of actual soils. The final subsection explains the segregation process, a key phenomenon occurring along the entire sedimentation process.

2.1.1 General description
In the classification of soil materials, a division is made between clay, silt and sand, based on particle size. In this thesis, the classification is done on the basis of British Standard 1377:1 (1990b). This means that clay is defined as smaller than 2 µm, silt is larger than 2 µm, but smaller than 63 µm. Sand grains are again larger than 63 µm, up to 2 mm. Particles larger than 2 mm are referred to as gravel, but are outside the scope of this thesis. The clay and silt size groups are usually grouped together and called the fines. In sedimentology and other disciplines where these fines are found in water, they are mostly referred to as mud (Winterwerp and Van Kesteren, 2004). Note that the mentioned names refer to size, and have no reference to the mineral content.

Mud consists of mineral solids and organic content. For the purposes of this thesis, the organic fraction is left out of scope. The focus lies on the role of mineral solids instead. Silicates are usually the main component of the mineral solids fraction, with precipitated salts, oxides or hydroxides making up the rest of the mineral solids, depending on chemical conditions. The most common silicates are quartz, feldspar and clay minerals. As clay minerals are the softest of the silicates (quartz being the hardest), a relatively large portion of the clay mineral content in a soil falls in the clay particle size fraction – where they are dominant – and (almost) all of the clay minerals fall in the mud fraction. As quartz and feldspar minerals also comprise a large part of the silt fraction, mud does not consist solely of clay minerals, although they do play a major part in its behaviour. In the sand fraction, clay minerals are absent (Winterwerp and Van Kesteren, 2004).

In this thesis, sediment is always characterised by three parameters: the clay-sized fraction of all solids, the silt sized fraction of solids, and the median sand grain size. These parameters can be used to simplify the full particle size distribution, while remaining adequately detailed for most hydraulic engineering purposes (Merckelbach, 2000; van Ledden, 2003).
Clay minerals consist of silica tetrahedrons—just as quartz—but arranged in sheet form. These sheets are combined with sheets of alumina/magnesium-hydroxide octahedrons. The sequence of alternation between these sheets determines the clay mineral. This structure of clay particles, combined with their small size gives them a high specific surface area and an electrical charge distribution, where their basal surfaces becomes negatively charged (Winterwerp and Van Kesteren, 2004).

2.1.2 Flocculation
Clay particles in water aggregates to form larger structures. This aggregation process knows two stages, coagulation and flocculation. Coagulation is the process where individual clay particles combine to form primary mud particles, and is in literature also sometimes referred to as aggregation. Flocculation is the process where these primary mud particles combine to form flocculent structures (Van Olphen, 1977). Note that in this thesis, aggregation refers to the combined process of coagulation and flocculation.

The high specific surface area and electrical charged surfaces, combined with their small size means that clay particle-particle interactions are dominated by attractive van der Waals forces and repulsive electrostatic forces when the clay is mixed with water. If the repulsive forces are stronger than the attractive forces, clay particles repel each other and remain isolated and dispersed. Conversely, if the attractive forces are stronger than the repulsive forces, clay particles are forced towards each other, collide, and bond to form a stable structure (Litzenberger, 2003). This structure is then again subject to the same forces as those acting on its constituents, only on a different scale.

When small particles combine to form a dense larger structure that contains little water, the process is called coagulation. These coagulates are subject to the same forces as their constituents and can also combine to create even larger structures, but as their specific area decreases these forces are relatively weaker. At some point these forces become so weak that, instead of the van der Waals forces, attractive electrical forces dominate attraction. The coagulates can then still combine, but due to the distribution of electrical charge over their surface, they form a more open structure. This process is called flocculation. The formed flocs have a structure that resembles that of a house of cards, with most of the volume that is enclosed by the floc being filled with the surrounding fluid. Flocs have water contents ranging from 80%-98% water (Winterwerp and Van Kesteren, 2004). Note that flocculation generally has a detrimental effect on later self-weight consolidation, as the water is

![Figure 2: Schematic representation of a flocculated structure. Note the repetition in the agglomerate structure. (Winterwerp and Van Kesteren, 2004)](image-url)
trapped within the low-permeable flocs, rather than in the more permeable void space between the flocs (Fowler and Morkel, 2009).

Electrical forces are generally stronger than van der Waals forces. This is especially true at larger inter-particle distances, as van der Waals forces decrease with distance to a power of four, while electrical forces only decrease exponentially with distance. This does however mean that at close proximity between the particles, the van der Waals forces may be larger than the electrostatic force. Hence there is a distance at which the two forces are in equilibrium. At distances closer than this equilibrium point, the particles will coagulate. Beyond the equilibrium point the particles will disperse (Litzenberger, 2003). If the clay particles are mechanically brought into close proximity with each other (e.g. by turbulence), coagulation and flocculation progress faster. This was confirmed by tests on flocculation in turbulent and laminar flow environments (Tsai et al., 1987).

Besides agitating the suspension, another method of enhancing the agglomeration rate of clay particles is the addition of chemicals to the suspension. There are two types of chemical that have a distinctly different effect (Fowler and Morkel, 2009). The first type is a coagulant which, as the name implies, stimulates coagulation. It does this by reducing the repulsive electrostatic force between particles. When this force is weakened, the range around a particle in which the attractive force is dominant increases, leading to quicker coagulation. Chemicals that work as coagulants are generally salts, which consist of two or more ions –of which at least one is positive. The ions become free-moving when the salt is dissolved in water, and the positive ions are attracted towards the negatively charged surface of the clay particles. These cations then bind to the surface and reduce the net negative charge, reducing the electrostatic force between the particles (Litzenberger, 2003).

The second type of chemical is a flocculant. As the name implies, this increases the rate of flocculation. Flocculants are polymers, long organic strains consisting of only one molecule. These molecules bind to the surface of the suspended coagulates, similarly as would happen in coagulation, but because of their length, they bind multiple coagulates at the same time. In this way, they accrete to form a larger structure to which more coagulates can bind, to eventually form a large flocculated structure (Winterwerp and Van Kesteren, 2004).

Different clay types are more, or less sensitive to flocculation. A good measure for this is the so-called clay activity (see Section 2.1.5), which is indicative of charge density at the surfaces of the clay particles. More active clays such as montmorillonite do not coagulate much, but can form large flocs in which a lot of water is bound. Conversely low activity clays such as kaolinite coagulate more and form denser flocs.

Manning et al. (2013) performed flocculation experiments on different ideal mixtures of mud and sand and compared these to natural alluvial sediments. The tested mixtures are a pure mud, a 75:25, a 50:50, and a 25:75 mud : sand mixture. The sand in this case is limited to a size range of 100-200 µm, as they deemed that larger particles settle too quickly to flocculate. They found that with higher mud content, mainly macroflocs – agglomerations of several smaller flocs and coarse sediment – form, with lower mud content leading to the formation of only microflocs, consisting only of mud particles. To find out whether the flocs consisted only of coagulated mud, or also incorporated sand grains, they calculated the mass balance and concluded that in both instances sand grains had been incorporated in the mud, but that sand grains had a preference for microflocs. They also performed TEM analysis on the floc structure. In the formation of macroflocs, sand grains are captured between
microflocs, forming the macrofloc. At low mud content, macroflocs consist only of mud. Probably the sand grains settle before they can be incorporated in the macroflocs. In microflocculation, the mud particles are attached to the surface of the sand grain, instead of to other mud particles, resulting in mud-coated sand grains. At high mud content, microflocs consisted only of mud. In all cases, flocs incorporating sand grains had higher settling velocities than the other flocs. The authors also found that for increasing sediment concentrations (up to 5g/litre) the rate of flocculation increased and larger flocs were formed.

In the experiments of Manning et al. (2013), only the mud-rich mixtures are able to capture the sand grains in the floc structure before they settle out of suspension. Apparently, if the sand grains are to be successfully and uniformly incorporated in the flocculated structure of the mud, the mixture should consist of an abundance of mud, rather than of sand.

Understanding the flocculation processes is a key part of understanding the internal structure of muddy slurries. If allowed to settle, the open and house of cards-like structure of flocs eventually become space-filling, giving the entire fluid some form of strength. It explains why fluids which consist mainly of water, also have this strength, and why this strength becomes larger at lower water contents. The aggregation processes also explain the difference in water absorption capacity for different minerals, or in other words, why different clay types have different strengths at the same water content. And finally, this process allows us to understand why slurries which are mixed in different ways have different strengths.

2.1.3 Settling

Describing the settling behaviour of mud flocs is not as straightforward as describing the settling of sand. The main cause of this is the continued flocculation of particles during settling. While the settling velocity of individual sand particles can be accurately described by the Stokes Equation (1), for mud particles only a characteristic settling velocity for the entire suspension can be defined. The way this characteristic settling velocity is described depends on its application, and scaling issues are to be considered before application (Winterwerp and Van Kesteren, 2004).

\[ w_s = \frac{(\rho_s - \rho_w)gd^2}{18\mu} \]  

In Equation (1), \( w_s \) is the settling velocity for a single particle with diameter \( d \), and density \( \rho_s \) in water with density \( \rho_w \) and dynamic viscosity \( \mu \). Winterwerp and Van Kesteren (2004) showed that for a floc with diameter \( d_f \) and fractal dimension \( n_f \) the settling velocity can be calculated as:

\[ w_s = \frac{\alpha}{18\beta} \frac{(\rho_s - \rho_w)g}{\mu} d_p^{3-n_f} d_f^{n_f-1} \left( 1 + 0.15Re_f^{0.687} \right)^{n_f} \]  

Where \( \alpha \) and \( \beta \) are sphericity factors, \( d_p \) is the diameter of the primary mud particles and \( Re_f \) is the floc Reynolds number. The fractal dimension \( n_f \) is a measure of the floc’s internal void space. For the purposes of this thesis it suffices to know that a solid sphere has \( n_f=3 \), and settling mud floc typically has \( n_f = 2 \). Equation (2) is actually a general form of Equation (1): for a spherical \( (\alpha = \beta = 1) \) and Euclidean \( (n_f =3) \) particle in the Stokes’ regime \( (Re_f<<1) \) Equation (2) reduces to Equation (1). At a
certain point, the concentration of mud flocs reaches the point that the flocs start interacting with each other. For example, the updraft of water caused by the settling of one floc imposes a frictional force on a neighbouring floc. Other interactions that may occur include, but are not limited to, collisions and particle-particle attraction. Because the interaction between the flocs prevents free settling of flocs and decreases the settling velocity, this regime is called the hindered settling regime.

This hindered settling regime continues until the volumetric concentration of flocs reaches unity, as then the flocs fill the entire space, and thus cannot settle any further without consolidating (see Section 2.1.4). The concentration at which the flocs form this space filling network is called the gelling concentration \( c_{gel} \). For a mixture of sand and mud, the gelling concentration is the concentration at which the mud flocs form a space filling network between the sand particles. This means that when the sand particles form a network, the gelling concentration does not exist. This also implies that the gelled mud should be strong enough to prevent the sand particles from settling through the network. Assuming no network formation between the sand particles, Van Kesteren et al. (2007) calculated that for sand with a maximum grain size of 500 μm, a mud network yield strength of 0.5 Pa is required to support the sand (see Section 2.1.6).

As the gelling concentration is the lowest concentration at which the sediment network spans the entire volume of the slurry, it is also the point at which the slurry develops a shear strength. At lower concentrations the water ‘network’ is not able to support a shear stress, but deforms plastically almost immediately. This may also mean that from the gelling concentration on, forces acting on the slurry have to overcome the internal cohesion before any resuspension of sediment can take place.

### 2.1.4 Consolidation

When the volumetric fraction of mud flocs reaches unity, or reaches the skeleton void fraction in the case that granular material is present, and the mud forms a space filling network, settling of particles stops. Any further reduction in volume of the material is now governed by the self-weight consolidation process (Toorman, 1996; Winterwerp and Van Kesteren, 2004). In settling column experiments, the difference between the settling and self-weight consolidation regimes is marked by a discontinuity in the sediment concentration, see Figure 3 (Torfs et al., 1996). During self-weight consolidation, the remaining water within the network is expelled by the pressure of the overlying material and pressure from the overburden is transferred from the pore water to the mud network. As the permeability of cohesive sediment is relatively low, this process takes a long time.

![Figure 3: Schematic example of density profile in a settling column.](image-url)
For small consolidation strains, Terzaghi (1943) derived Equation (3) describing the change in effective vertical stresses ($\sigma'_v$) in the soil over the height of a soil column ($z$) and time ($t$). Because the strains are small, he assumed a constant void ratio and thus also a constant permeability, which he captured in the consolidation coefficient ($c_v$).

$$\frac{\partial \sigma'_v}{\partial t} - c_v \frac{\partial^2 \sigma'_v}{\partial z^2} = 0$$

(3)

In self-weight consolidation following settling, the initial void ratio is high in respect to the void ratio of normally consolidated clay, which would be the end-state of self-weight consolidation. Hence the total strain over the duration of the process is large. The low permeability means that initially most of the overburden pressure is carried by the water. With increasing consolidation, more bonds between the mud particles form, and the mud carries an increasing portion of the total stress (Berlamont et al., 2011). In other words, the pore pressures decrease and the effective stress increases. With a lower pore pressure to keep the pores open, the void ratio decreases. This then leads to decreasing permeability (Berilgen et al., 2006; Berlamont et al., 2011). When no unloading takes place in between consolidation phases, the total effective stress is always equal to the strength of the bonds between the mud particles. Note that this means that when particle bonds are stronger, for instance because of coagulant or flocculant addition, higher effective stresses are needed for the same amount of consolidation.

The non-constant permeability means that the consolidation equation of Terzaghi (1943) is not valid for self-weight consolidation. To account for the larger variability of void ratio and permeability, Gibson et al. (1967) derived Equation (4a).

$$\frac{\partial e}{\partial \tau} + \left(\frac{\rho_s - \rho_w}{\rho_w}\right) \frac{d}{de} \left[ k \frac{\partial e}{\partial \zeta} \right] + \frac{\partial}{\partial \zeta} \left[ \frac{k}{g \rho_w (1 + e)} \frac{d \sigma'_v \partial e}{de \partial \zeta} \right] = 0$$

(4a)

$$\frac{\partial \phi_s}{\partial \tau} - \left(\frac{\rho_s - \rho_w}{\rho_w}\right) \frac{d}{dz} \left[ k \phi_s^2 \right] + \frac{1}{g \rho_w} \frac{d}{dz} \left[ k \phi_s \frac{d \sigma'_v}{de} \right] = 0$$

(4b)

Note that Equation (4a) is in so called material coordinates, where:

$$\zeta = \int \phi_s \, dz$$

(5)

To facilitate solving the self-weight consolidation equation, Toorman (1996) rewrote the Gibson equation into an Eulerian form and using the total volumetric sediment concentration ($\phi$), which is shown in Equation (4b). For an accurate solution to Equation (4), the $k$-$e$ and the $\sigma'_v$-$e$ relationships of the material in question need to be accurately known. These relationships are known as the material functions. They have a general empirical form, which is shown in Equation (6) and Equation (7) both as a function of void ratio and of volumetric sediment concentration. Merckelbach (2000) proved these empirical equations from theory by considering mud flocs as fractal structures.
In these equations all capitals denote material-specific constants, which can be determined from settling column tests. Merckelbach (2000) also showed that $B_k = -B_k$, similarly Winterwerp and Van Kesteren (2004) showed that $B_p = 1/B_m$. Recently, these parameters have also been correlated to the index properties (see Subsection 2.1.5) (Berilgen et al., 2006; Paul, 2011).

### 2.1.5 Geotechnical properties

Typical of soils consisting mostly of mud is that their strength relies mainly on cohesion (see Section 2.1.2). A typical property of cohesive soils is that their cohesive strength depends strongly on their water content. To capture this behaviour, soils are generally classified on the basis of the so-called **Atterberg limits**. There are two Atterberg limits, which are the **Liquid limit** ($w_L$) and **Plastic Limit** ($w_P$). Both limits represent water content as a percentage of the solids by weight, at which the soil passes from a brittle to a plastic state ($w_P$) and from the plastic to the liquid state ($w_L$). These transition points are comparable to the water content at an undrained shear strength of 1.7 kPa ($w_L$) and 170 kPa ($w_P$) (Sharma and Bora, 2003; Wood, 1985). The difference between the Liquid and Plastic limits is called the **Plasticity Index** ($I_P$).

When the clay fraction of sediment controls its mechanical properties, there is a linear relationship between the clay content of the soil and the Plasticity Index. The slope of this relation is called the **activity** ($A$), and is indicative of the main clay mineral involved. Activity is calculated using:

$$A = \frac{I_P}{\xi_{cl} - \xi_{cl}^0} \quad (8)$$

Where $\xi_{cl}$ is the clay mineral content as weight percentage of solids, and $\xi_{cl}^0$ is the intercept with $I_P=0$. Note that $\xi_{cl}^0$ is also the clay mineral content at which the soil starts displaying cohesion.

For instance kaolinite, which is the least active clay mineral, has an activity of 0.4, while montmorillonite has an activity of up to 7, which is the highest of all clays (Winterwerp and Van Kesteren, 2004).

The strength parameters of a soil with a given water content ($w$) can be correlated using the **liquidity index** ($I_L$), which is defined as:

$$I_L = w - w_p - \frac{w}{I_p} \quad (9)$$

The undrained shear strength ($c_u$) can then be determined using an established relation between $c_u$ and $I_L$ for a given mineral activity. Over the years, different authors found similar, but slightly different relationships. Vardanega and Haigh (2014) analysed different relationships published in the past 40 years. They found that by slightly modifying the equation found by Wroth and Wood (1978) they were able to fit most of their collected data. This modified equation is given in Equation (10). In this equation they make use of $c_{ul}$, which is the undrained shear strength taken at the liquid limit.
This circumvents the problem found by Koumoto and Houlsby (2001), that use of different standards across the world leads to different shear strengths at the liquid limit. Vardanega and Haigh (2014) used $c_L = 1.7$ kPa. Note that their relation also implies that the strength at the plastic limit is 35 times larger than at the liquid limit. The authors found that the largest deviation to the rule was at liquidity indices near the plastic limit, and chose to exclude that liquidity range from the analysis.

$$c_u = c_L35^{(1-l_L)}, \quad 0.2 < l_L < 1.1$$

This relationship is also valid for values of $l_L$ slightly above 1 ($w_L$), but quickly becomes inaccurate for high values. The method itself is already not of high accuracy, but allows for a quick initial estimate. The same method can be used for an assessment of permeability ($k$) (Winterwerp and Van Kesteren, 2004).

The relationship between shear strength and liquidity index (and consequently water content) clearly shows that with increasing consolidation state, thus decreasing water content, the shear strength of a mud-sand mixture increases. Merckelbach (2000) investigated the fundamentals of this relationship between consolidation state and shear strength using the same geometrical theory he applied in deriving the material functions. He also noted that yield stress and bed shear stress are not necessarily the same, as the plane of failure is actually made up of several smaller failures at an angle to the overall failure plane. The yield stress operates along the plane of failure, but the critical shear stress—the bed strength—operates along these smaller failure planes. The equation for the critical shear strength he arrived at has a similar form to the Mohr-Coulomb failure criterion. He thus proposed to write the formulation in Mohr-Coulomb terms, with an added component to model the effects of creep:

$$\tau_c = c\phi_{cl} + (ab + a'_p)\tan \phi'$$

In this, $c$ is the cohesive strength and is defined by the number of active inter-aggregate bonds per aggregate, $ab$ denotes the number of inactive bonds as a result of creep and, and lastly $\phi'$ is the internal friction angle and is defined by the ratio of the number of active inter-particle bonds to the total number of active bonds in the mud. Notable about this definition is that the material cohesion, which intuitively finds its origin in the connection between mud particles, is mostly related to mud aggregates. Conversely the friction angle of the material, which one would intuitively associate mostly with inter-aggregate strength, is related to inter-particle strength. To complicate matters further: Merckelbach (2000) also relates the friction angle to the sand content. He concludes that with increasing sand content, the friction angle actually decreases. This is again contrary to intuition, which says that with increasing sand content, the relative occurrence of friction between sand grains increases. For this reason, the method of Merckelbach (2000) for calculation of mud strength is not adopted in this thesis.

### 2.1.6 Segregation

Sediment in a mixture often exhibit segregation, meaning that particles of different sizes settle at different rates. Coarser particles settle more quickly than fine particles, because their ratio of mass (downwards force) to hydraulic surface (resisting force) is larger, see also the Stokes settling velocity in Equation (1). This leads to a fining upwards deposition of the sediment in the slurry. At a certain solids concentration of the slurry, the hindrance in the settling is so large that particles of different
sizes settle at the same velocity. Slurries above this solids concentration are referred as non-segregating. For muddy slurries, this concentration is typically equal to the gel concentration.

Equation (12) shows the boundary condition for a single grain to settle in a slurry. In this equation, $D$ is the grain size, $Y_{\text{max}}$ is a shape dependent measure (0.065 for spheres) and $\tau_y$ is the yield strength, which is explained in the next section; suffice it to say it represents the amount of hindrance on the settling particle. As long as Equation (12) is true, the particle settles through the slurry (Van Kesteren et al., 2007).

$$\frac{\tau_y}{(\rho_s - \rho_w)gd} < Y_{\text{max}}$$

(12)

If the slurry yield stress is large enough to support the largest particle sizes of the constituting or of admixed sediment as per Equation (12), the slurry is non-segregating. For a maximum particle size of 1 mm this corresponds to a yield stress of 1 Pa.

An example of a segregating environment is a river delta, where coarse material is deposited near the shore while clay and silt-sized particles are transported out to sea. On the other hand, a non-segregating slurry does not exhibit differentiated settling behaviour, and upon drying it leaves behind a homogeneous mass of sediment (Fitton, 2007). This means that in a non-segregating slurry, sand is well spread throughout all of the sediment and not just deposited near the initial deposition point. As this is a condition for the enhanced consolidation of the disposed material, the rest of the study focuses on non-segregating slurries.
2.2 Slurry flow

This section is a short introduction into rheology in general and the rheology of sediment slurries in particular. Rheology is the science of deformation and flow of matter, especially fluids and plastic solids. In this section the focus is on the flow of so-called non-Newtonian fluids, as opposed to Newtonian fluids, which are briefly explained in Paragraph 2.2.1.

2.2.1 Newtonian flow

Slurries can exhibit a wide variety of behavioural aspects. The flow behaviour of a fluid is referred to as its rheology. A first distinction is usually made between Newtonian flow behaviour and non-Newtonian flow behaviour. Newtonian behaviour is described by a linear relationship between the shear stress acting on a fluid and the shear rate of that fluid as in Equation (13).

\[ \tau = \mu \dot{\gamma} \]  

In Equation (13), \( \tau \) is the shear stress, \( \dot{\gamma} \) is the shear rate and the constant relating the two is a material property called the dynamic viscosity (\( \mu \)). An example of a Newtonian fluid is water, which has a viscosity of 0.001 Pa·s at room temperature.

2.2.2 Non-Newtonian flow

Non-Newtonian fluids do not exhibit this linear relationship. Two common properties of non-Newtonian fluids are the yield stress and shear thinning or thickening. When a fluid has a yield stress, it means that a certain amount shear stress is required for the fluid to flow at all. At shear stresses below the yield stress, the fluid behaves like an elastic solid. This is similar to yield strengths in soil mechanics, which denotes the boundary between elastic and viscoelastic behaviour. An example of a rheological model that incorporates the yield stress concept is that of a Bingham fluid. The Bingham rheological model is defined by Equation (14) in which \( \tau_y \) is the yield stress and \( K \) is the Bingham viscosity.

\[ \tau = K \dot{\gamma} + \tau_y \]  

One effect of this yield stress is that so-called plug flow can occur. Plug flow occurs when a portion of the fluid is flowing, but is not sheared. The flow velocity over the cross-section of this plug is constant. The shear stress in the plug is smaller than the yield stress, thus no shear occurs, and the flow velocity does not change, see also Figure 4.

Figure 4: Schematic representation of the shear stress and flow velocity profiles of a Bingham fluid during flow in a pipe. Near the centre of the pipe the shear stress is smaller than the yield stress, leading to a constant velocity (plug flow) near the pipe centre.
When a fluid is shear thinning, it means that the derivative of the shear stress over shear rate diagram becomes smaller, i.e. what in a Newtonian fluid is viscosity decreases with increasing shear rate. A shear thinning fluid is easier to accelerate when it is already being rapidly sheared than when it does not flow yet. Conversely a shear thickening fluid would become harder to shear with increasing shear rate. Purely shear thinning or thickening rheological models are usually called \textit{power-law fluids} and are defined by Equation (15). A fluid’s shear thinning or shear thickening property is usually called thixotropy or rheopecty respectively.

\[ \tau = K\dot{\gamma}^n \]  

(15)

For a value of the flow behaviour index \((n)\) below unity, the fluid is shear thinning. For a value of \(n\) above unity, the fluid is shear thickening. Note that if \(n\) equals unity the fluid behaves in a Newtonian fashion. Schematic rheograms of all four mentioned rheological models are given in Figure 5.

Sometimes the strength of a non-Newtonian fluid is characterised by the apparent viscosity \((\mu_0)\). The apparent viscosity is what the viscosity of the fluid at a given shear rate would be if the fluid were Newtonian instead. Put differently, the apparent viscosity is the shear stress divided by the shear rate, but is only valid for that shear rate: it is different for other shear rates as the fluid isn’t actually Newtonian. An example of this is also provided in Figure 5C.

Some fluids follow a different path in the stress-strain rate domain depending on whether strain rate increases, or decreases. An example is given in Figure 5D. In the depicted rheogram the shear rate is first increased, and then decreased again. The rheogram shows a clear shear thinning effect as the shear rate is increased. When the shear rate is then lowered, the rheogram remains linear, Bingham-like, for most of the shear rate range. At low shear rates the rheogram curves and approaches the origin again. This type of behaviour is for instance observed in mayonnaise. Effects like this, where the material strength depends on the shear path or flow history are grouped under the term \textit{hysteresis}.

\[ \begin{array}{c|c|c|c|}
\text{Shear stress} & \text{Shear rate} & \text{Shear rate} & \text{Shear rate} \\
\hline
A & K & 1 & \mu_0 \\
B & 1 & \gamma_0 & \mu \\
C & \text{dashed} & \text{full line} & \text{long dash} \\
D & \text{red lines} & \text{concept of apparent viscosity} \\
\end{array} \]

\text{Figure 5: Schematic rheograms of a Newtonian fluid (A), a Bingham plastic (B) and two power-law fluids (C). In C the dashed rheogram represents a shear thinning fluid, the full line a shear thickening fluid (long dash). In C the red lines depict the concept of apparent viscosity: the apparent viscosity of the shear thinning fluid is } \mu_0 \text{ at shear rate } \gamma_0.\

\text{2.2.3 Sediment slurry rheology}\

When sediment is suspended in water at low concentrations, the slurry behaves as a Newtonian fluid, with the added phenomenon of sediment settling. The sediment settles to the bottom, leaving clear water at the top and increasing the slurry concentration at the bottom. If the slurry is flowing, turbulent flow conditions can prevent this settlement and keep the slurry fully mixed. A Newtonian
slurry deposited on a slope flows to the bottom and forms a horizontal surface there, like water in a lake. When the flow stops, the sediment settles out over time and forms a horizontal deposit.

When the sediment concentrations in the slurry are increased, the slurry gains a yield strength. This yield strength prevents some of the suspended particles from settling, as long as their weight is less than the required shear stress for the slurry to become fluid and thus allow their settlement. When external shear is applied to the slurry, the fabric of the slurry is broken up, thus lowering the viscosity and strength, giving the slurry a shear thinning behaviour. Parsons et al. (2001) did note that while mud-sand mixed slurries exhibit shear thinning behaviour (0.4 < $n < 0.7$), the clay content had a large influence on the amount of shear thinning occurring. For slightly increased clay contents (+2-3%) they found that the rheology tends to approach Bingham conditions ($n$ approximately unity). Because the difference between the Herschel-Bulkley (yield stress & power law) and Bingham models in the observed shear rate range is limited, they recommend that the muddy slurries be modelled as Bingham plastics. Furthermore, Locat and Demers (1988) found that clay slurries that behave as Bingham fluids typically exhibit strong hysteresis effects, whereby previously sheared soft clays had significantly lower resistance to flow than undisturbed clays. Their yield stress did remain the same under both circumstances.

The Bingham flow model is also advocated by a number of other authors (Hampton, 1972; Mulder and Alexander, 2001) as a simplification of the Coulomb-viscous model for soils. This model is similar to the Mohr-Coulomb failure envelope, but with an added viscosity element as depicted in Equation (16). The Coulomb-viscous model reduces to the Bingham model when the yield stress is taken equal to the combined cohesive and frictional strengths. This means that the yield strength of a slurry is equal to its Mohr-Coulomb shear strength.

$$\tau = c + \sigma_n' \tan \phi' + \mu \dot{\gamma} \quad (16)$$

For the purposes of this investigation yield stress is an interesting material property, as a yield stress fluid can – contrary to Newtonian fluids – remain stationary on a slope and thus prevent the tailings from flowing out to the production area. The maximum thickness ($H_{\text{max}}$) of the yield stress fluid on the slope depends on the material density ($\rho_b$), slope angle ($\beta$) and yield stress. This maximum thickness may also be called the yield thickness. A force balance reveals the following relationship between the mentioned parameters:

$$H_{\text{max}} = \frac{\tau_0}{g \rho_b \sin \beta} \quad (17)$$

In summary, fine sediment slurries behave as Bingham plastics: they possess a yield stress. If the shear stress on the material exceeds the yield stress, for example because sedimentation leads to exceeding of the yield thickness on a slope, the material starts to flow. During flow, the shear rate increases linearly with increase in the applied shear stress. Assuming that nothing else happens to the slurry during flow, it flows out until it is reduced to the yield thickness. The next section looks into the mechanisms acting on a flowing slurry of fine sediment that affect this stable equilibrium.
2.3 Slurry flowing down a slope

When slurry flows down a slope, its basic behaviour is defined by the rheological model (viscosity, yield strength) as presented in the previous section. However, a number of other parameters also affect the behaviour of the slurry. In this section the effects of the slope material, segregation, and the ambient fluid on the slurry flow is analysed. For this, a number of analogies are considered. For subaerial flow a central (tailings) discharge facility is seen as analogues to disposal of a dense fines slurry on a slope. The effect of flow (shear) on slurry segregation is described by Sisson et al. (2012) in their model on fines capturing in thickened tailings disposal. Finally, an analogy for subaqueous flow of dense slurries is found in experiments used to model the behaviour of submarine debris flows.

2.3.1 Subaerial tailings disposal

For modelling the subaerial disposal of thickened tailings, there are two methods that can be considered as industry standards. These are the Fitton method (Fitton, 2007), based on a slope angle where sedimentation and erosion along the slope are in equilibrium, and the McPhail method (McPhail, 2008), which is based on the assumption that the slurry must dissipate all of its energy during flow down the slope.

Fitton method

Fitton (2007) researched the subaerial deposition of non-segregating slurries in a central discharge facility. His field observations led him to conclude that in large scale deposition, the final geometry of the deposit is governed by channel flow of the non-segregating slurry. In this, the thickened tailings flow down the path of least resistance, which effectively consists of a channel formed by previous flow paths. He also noted that, if the flow rate increases, the underlying tailings are eroded. He then assumed, based on the works of Winterwerp et al. (1990) that there is a slope at which all solids remain suspended, but the bed is not eroded, calling this the equilibrium slope. His approach to finding the ultimate tailings slope consisted of finding the equilibrium slope, both experimentally and theoretically. From experiments it appeared that the slope is dependent mainly on the flow rate and (though less strongly) on slurry solids concentration. He further noted that segregating slurries have a significantly flatter slope than non-segregating slurries. When evaluating the rheology, he found that the Herschel-Bulkley model fitted the tailings behaviour better than the Bingham model. He did not succeed in determining the segregation threshold: existing tests are only able to determine the segregation threshold under static conditions, which differed from the dynamic segregation threshold as estimated from his experimental results. Pirouz (2006) already showed that when the slurries Fitton used where sheared (dynamic segregation), they exhibited a higher segregation threshold value than under static conditions. Fitton reasoned that an accurate measurement of the segregation threshold in small scale laboratory experiments is unlikely, because it would also depend on the amount of shear in the flow channel.

An important conclusion drawn by Fitton (2007) concerns the suitability of labscale experiments in the prediction of the slope profile. He made a comparison between a large amount of previous studies and his own and found that all laboratory scale studies failed in accurately representing the channel flow mechanism. Since this mechanism is a defining aspect of the problem of slope prediction, lab scale experiments cannot be relied upon for the estimation of a full scale slope profile.
**McPhail method**

Another method of determining the slope profile is determining at what slope the energy of the flowing energy is fully dissipated by the time it reaches the toe of the slope. This was investigated by McPhail (2008) for subaerially deposited thickened tailings slurries. His is a numerical model, and assumes a known initial slope and a limited length of the slope, which would be the case in a tailings storage facility. In the case investigated in this thesis the length of the slope follows from the slope angle, so this analysis would not directly work. However the idea of energy dissipation along a slope does look promising, as the method is able to predict beach profiles quite accurately.

### 2.3.2 Segregation in subaerial and subaqueous flow

Sisson et al. (2012) investigated the fines capture ratio in oil sands tailings for different depositional basins and different slurry properties. For this, they also determined the sedimentation rate of the coarse particles in the slurries. They noted that non-segregating slurries do exhibit some segregation in shear. For their calculations, they considered a non-segregating slurry flowing down a slope. While plug flow is observed in most of the slurry, the bottom part near the slope is sheared, and thus the sand grains in the slurry start to settle out of the slurry, forming a sandy bed. The carrier fluid is then partly trapped in the pore space of this sandy bed. As the sand settles out of the slurry, the sand to fines ratio (SFR) also drops. If the SFR drops below the static segregation threshold, the slurry segregates when the flow stops, and the fines are lost from the slurry to the surrounding water. Note that the SFR can only drop below this segregation threshold if the plug flow is only an apparent

![Figure 6: The three scenarios for deposition as predicted by Sisson et al. (2012). In (a) the SFR does not drop below the static segregation threshold, and the non-segregating tailings (NST) reaches the bottom of the pond intact. In (b) the SFR does fall below the segregation threshold, leading to the formation of a sand bed and expelling of the fluid fine tailings (FFT). In (c) the tailings stream encounters turbulent condition or strong waves in the pond, leading to entrainment of water and the slurry falling below the segregation threshold, leading again to sand deposition and expelling of the FFT. Mature fine tailings (MFT) is the name for FFT that have consolidated somewhat. In (a) the MFT forms in the NST, but is expelled from it as the porosity between the grains decreases during consolidation.](image-url)
plug flow, and actually does exhibit shear. They identified three different scenarios for the deposition of a non-segregating slurry, which are displayed in Figure 6. In Figure 6a the tailings remain non-segregating, and form a bed at the bottom of the basin where they consolidate. During consolidation, the pore space between the grains decrease and some of the pore fluid are expelled. The fines slurry in the pores consolidates as well and obtains a higher solids content than the original carrier fluid. This denser fluid in water is referred to as mature fine tailings (MFT). In Figure 6b the tailings become segregating under static conditions. This means that after deposition at the bottom, the tailings segregate immediately. The sand grains fall to the bottom, leaving the carrier fluid behind. The carrier fluid has then not yet undergone consolidation and as it carries no more coarse solids it is called the fluid fine tailings (FFT). In Figure 6c the water in the basin is not still, as in cases a and b. It is either flowing turbulent instead, or there is significant wave action. This leads to shearing of the NST at the water interface and causes entrainment – or mixing – of the water in the slurry. With this higher water content, the slurry becomes non-segregating. As in case b, the sands settle and the FFT are left behind. In both cases b and c the FFT slowly flocculate and consolidate to form the denser MFT.

The above processes make limited use of traditional methods for calculating the sedimentation or erosion rate of a system. This is because these last methods consider the sediment as individual particles and rely on the particle size and density to predict its behaviour in different sedimentary systems. They are therefore unable to handle the non-segregating characteristic – where particles move together – of the slurries that are considered here.

When slurries are segregating sedimentation analysis can be used – as is done by Sisson et al. (2012) – to analyse the segregation behaviour in the sheared zone of the flow. For this they make use of the regular free settling and hindered settling equations from Section 2.1. For instance using the Stokes equation (Equation (1)), only changing the \( \rho_w \) to the density of the carrier fluid and the \( \mu_w \) to the apparent viscosity (Section 2.2) of the sheared slurry:

\[
w_s = \frac{(\rho_s - \rho_ft)gd^2}{18\mu_0}
\]  

(18)

**2.3.3 Submarine debris flow**

Submarine debris flows are high concentration, high density sediment gravity flows consisting of a muddy matrix in which coarse material is suspended (Mulder and Alexander, 2001). Typical sediment concentrations of debris flows range from 30% to 80% by volume, of which at least 50% is non-cohesive. Cohesive flows with less than 50% non-cohesive particle content are typically referred to as mud flows, which behave similarly but result in different types of deposits (Mulder and Alexander, 2001). Debris flows typically form on continental slopes, showing that they can occur at low slope angles (1°-5°). They can incorporate volumes of multiple cubic kilometres and can flow out over hundreds of kilometres (Ilstad et al., 2004c). While natural debris flows are of a scale much larger than a tailings disposal scenario, there has been a lot of small scale experimental work done on flow of dense slurries down a submerged slope. The results of these experiments show that two important characteristics of debris flows are their coherence and the occurrence hydroplaning. These characteristics are discussed in this section.
Coherence

Marr et al. (2001) performed experiments with slurries consisting of kaolin clay, fine sand (110μ), and water flowing down a submerged slope of 4.6°, which served as analogues to submarine debris flows. A schematic section of their experimental setup is presented in Figure 7. To characterize the behaviour of these submarine debris flows, they used the coherence of a flow as a qualitative measure of the ability of the slurry to resist breaking apart under the high dynamic stresses encountered in subaqueous debris flows. This breaking apart of the slurry occurs chiefly at the head of a debris flow, where stresses are the highest, and leads to the formation of a turbidity current above the flow (Hampton, 1972). Marr et al. (2001) divided the flows they observed in experiments into three categories (weakly, moderately, strongly coherent debris flows) to illustrate the range of behaviour that they observed in their experiments. Because coherence is a function of dynamic stresses, a single flow can fall in different categories as shear stresses and pressures change. The three different types as characterised by Marr et al. (2001) are displayed in Figure 8 and described below:

Strongly coherent flows show strong cohesive behaviour, leading to little or no breakup of the flow head. Although they usually do show a small turbidity plume behind the head, it is highly dilute. The interface between the debris flow and the overlying turbidity current is sharp. Slurry flow in the head and body is laminar. In many cases, dynamic pressures at the front of the head allow a small layer of water to slip underneath the head, leading to so-called hydroplaning where the front of the head flows over this layer of water rather than over the bed (see also Hydroplaning).

Moderately coherent flows do show erosion and breakup of the flow head, forming a substantial turbidity plume around and behind the head. The boundary between the plume and the gravity flow is still distinctly visible. Flow in the head and body is laminar. Granular material in the body is fixed in place in the matrix.

Weakly coherent flows have little internal strength, and no flow head is observed as it is broken up into a turbulent suspension. A significant and concentrated turbidity plume forms. The slurry flow in the body is still (nearly) laminar. The boundary between the flow and plume is indistinguishable. In the body, grains are suspended in the fluidized matrix, rather than being fixed.

Hampton (1972) investigated the formation of a turbidity plume from a debris flow. He noted that at the head of the flow a shear region exists that is sheared in the opposite direction to that of the flow. He found that the sheared material is moved towards a low pressure zone directly behind the head of the flow. There, with not enough pressure acting on the material to keep it from eroding, it is thrown into suspension in the turbidity plume. He then reasoned that the dynamic shear stresses at
the head must be greater than the yield stress for shear to occur. As this shear is necessary to feed material into the low pressure zone, he reasoned that if the dynamic shear stresses do not exceed the shear strength, no mixing takes place. Hence, if a slurry has a higher yield strength, it is more coherent.

Mohrig and Marr (2003) analysed the theory of Hampton (1972). For this they used yield stress and flow velocity data from Mohrig et al. (1998) and Marr et al. (2001), and defined the dynamic shear stress at the head \((\tau_f)\) according to Equation (19) where \(U_h\) is the flow velocity of the head. They then calculated the ratio of \(\tau_y/\tau_f\), and compared this to observed coherence of the flow. There they found that all but one of the strongly coherent flows have a \(\tau_y/\tau_f > 0.2\), and all less coherent flows have a \(\tau_y/\tau_f < 0.2\), see also Figure 9. They note that this supports the theory of Hampton (1972) that the character of the flow, specifically that at the front of the flow, changes when the dynamic pressures exceed the slurry yield strength. They did not find such a specific boundary value for the transition between moderately and weakly coherent flows.

\[
\tau_f = \frac{1}{2} \rho_w U_h^2
\]

Ilstad et al. (2004b) continued the work of analysing the behaviour of debris flows. Where previous research focused on the head of the flow, they used particle tracking to better describe the flow of sand grains within the body flow. In strongly coherent flows they observed that the body directly behind the flow is thinner than further behind, probably caused by the same velocity difference.
between head and body that causes block forming. In less coherent flows this effect is not observed. In both moderately and weakly coherent flows, they observed settling of sand grains in the body, already within seconds after the head had passed. Besides this, they confirmed that in strongly coherent flows there is no slip at the bottom of the flow, and that above a small shear zone at the bottom, plug flow is observed.

The relation between yield strength and coherence is further illustrated by the results of Sawyer et al. (2012). They placed different mixtures of silt, clay, and water in a small tank on a 10° slope inside a larger water-filled tank. The downslope wall of the small tank was subsequently opened, allowing the slurry to flow out. The slurries were characterised by their yield strength.

To categorise the resulting debris flows, the authors introduced the flow factor ($F_f$), which they define as the ratio of the driving shear stress to the yield stress ($F_f = \tau / \tau_y$). Their results show that for increasing flow factor, the amount of turbidity generated increases. In their experiments, the value of the flow factor ranges from 0.95 to 6.23. At the highest of the tested flow factors, flow is rapid (16 cm/s) and substantial turbidity is produced. Around a flow factor of 3, flow is still rapid (>10 cm/s), but only minor turbidity is produced. For flow factors in the range from 1 to 2, flow rate decreases to below 1 cm/s, and no turbidity is formed. The deposit formed by the flow shows block forming at the front, with tension cracks forming behind.

Comparing the categorisation by flow factor to the principle of coherence, there is reason to believe the two are related: As flow factor decreases, yield strength increases (assuming equal driving forces), there is less breakup of the slurry in flow, hence the slurry is more coherent. Another similarity is that Marr et al. (2001) put the boundary between strongly coherent and less coherent at a yield stress of 30 Pa. Strongly coherent flow was observed by Sawyer et al. (2012) for slurries with a yield stress of 32 Pa and higher, see also Figure 14 & Figure 18. Less coherent flow was observed for yield stresses of 25 Pa or lower. This suggests a critical yield strength for flow down flat slopes (5°-10°) of around 30 Pa.

Figure 9: The ratio of yield strength over frictional stresses plotted for a number of experimental debris flows (‘runs’). (Mohrig and Marr, 2003)
Hydroplaning

Hydroplaning is the phenomenon where a debris flow propagates over a thin layer of water that separates the debris flow from the underlying sediments. This layer of water is the result of the dynamic pressures in front of the debris flow becoming large enough to support the buoyant weight of the debris flow (Mohrig et al., 1998). Hydroplaning only occurs in subaqueous flow and is the cause of the differences in behaviour between subaerial and subaqueous debris flows, namely higher mobility, block formation and lack of reactivation in subaqueous flows (Mohrig et al., 1999).

When debris flows under water, the head of the flow is subject to dynamic pressures due to the displacement of water in front of the flow. These pressures cause the head to deform. As a result, the head becomes slightly thicker than the body of the flow. This leads to a low pressure zone right behind the head (Hampton, 1972; Ilstad et al., 2004a). At the same time a small layer of water forms below the bulbous part of the head (see also Figure 10). When the pressure at the front of the flow and the low pressure zone behind the head become strong enough to support the buoyant weight of the debris flow, the layer of water intrudes further along the interface of the flow and the slope, and the flow starts hydroplaning (Ilstad et al., 2004a). The point at which the flow traverses from normal frictional flow to hydroplaning can be estimated using the densimetric Froude number ($Fr$), given in Equation (20). When the Froude number exceeds a value of approximately 0.3, hydroplaning sets in. At a Froude number of approximately 0.4, the flow is said to be fully hydroplaning (Mohrig et al., 1998).

$$Fr = \frac{U_h}{\sqrt{\frac{\Delta \rho}{\rho_w}} g H_{max} \cos \beta}$$

The water between the flow and bed acts as a lubricating layer, reducing the drag on the flow. With less drag, the flow can run out over a longer distance than it would without hydroplaning. It is then said that the mobility of the flow increases. The velocity of the flow does not increase as a result of hydroplaning; instead an equilibrium velocity is attained within seconds after the release of the flow. Subaqueous flows differ from subaerial ones in this behaviour, as subaerial flows decelerated continuously in all experiments of Mohrig et al. (1999)

Near the head the velocity does increase with hydroplaning. As a result of the difference in velocity between the head and the body, the head gradually separates from the rest of the flow. This was observed by Ilstad et al. (2004a) in experiments and in the field, where sometimes the head of the flow can be found at distances many times greater than the run-out distance of the flow body. Along the way, the head erodes slowly, and smaller blocks can be seen between the body and the head.

Erosion of the bed

Mohrig et al. (1999) investigated the reactivation (or erosion) of former debris flows as a result of another debris flow flowing over it. Their research covers both the subaerial and subaqueous environments. They found that in a subaerial environment the overlying flow would erode the previous deposit, something that Fitton (2007) also observed. As a result of the added mass, the combined reactivated flow has a higher velocity than when the debris flowed over a hard surface. The subaqueous flow did not reactivate the existing deposit, depositing a continuous layer over the existing deposit instead. They assumed that the cause lies in the lubricating effect of the layer of
However, they also noted that with just static loading of the second layer on the original deposit, the maximum thickness as defined by the yield strength would not have been achieved, and the underlying deposit would remain static. Instead they reasoned that the hydroplaning effect merely prevented the large dynamic stresses associated with a flowing slurry from acting on the underlying deposit.

As the bed in all subaqueous experiments is deposited below yield thickness, it remains questionable whether the same reactivation of sediment as observed in the subaerial cases can occur. It is clear that in the case of a hydroplaning flow, the yield thickness at least needs to be surpassed by the combination of bed and flow thicknesses, before there is a possibility of reactivation. It is likely that this reactivation then occurs from a static situation, when the hydroplaning flow has come to rest. Which may start at the tail of the flow and move forward, as hydroplaning occurs only near the head of the flow, while the tail rests on the bed. This situation differs from the subaerial case, as there reactivation starts at the head of the flow.

Figure 10: Schematic representation of the front of a debris flow, flowing to the left, in 3 different conditions. V.E. means Vertical Exaggeration, which is 5x in all instances. A: The flow is not hydroplaning ($Fr < 0.3$). This shape of the front is also observed in subaerial flows. B: The front is at the verge of hydroplaning ($Fr ≈ 0.35$). The head is already deforming, becoming thicker than the flow body. C: The flow is now fully hydroplaning ($Fr > 0.4$). The head of the flow is fully deformed and travelling faster than the body of the flow. This causes a thinning of the flow just behind the head. (Mohrig et al., 1998)
2.4 Conclusions
From analysing the literature, the following initial conclusions can be drawn, each with a small summary of the observations leading to them:

Fine sediments, especially clay minerals, display cohesion when mixed with water. This cohesion leads the sediment particles to form aggregates which subsequently settle to the bottom of the water basin. Near the bottom these sediments form a space filling sediment network, which resembles a soil, only has a several times higher water content. As the load on this sediment bed increases, for instance due to further sedimentation, the mud develops a strength to support this load. This process is known as consolidation.

To prevent resuspension of sediment, the slurry should have a cohesive strength strong enough to bind the fine sediment together to form a coherent slurry. This strength comes from the same inter-particle forces that cause coagulation and flocculation in suspended fine sediment, so enhancing slurry strength can probably be done using the same methods to enhance flocculation and coagulation.

To ensure that sand remains homogeneously mixed in the mud slurry, the slurry should be strong enough to overcome the gravitational force on the sand grains. In other words the slurry has to be non-segregating. The admixed sand gives the slurry higher density, leading to an increased consolidation rate.

The thixotropic and hysteretic behaviour of muddy slurries shows that when the applied shear stress exceeds the strength of the internal structure, this structure is broken down and much of the strength is lost. When the slurry is at rest, the internal structure reorganises itself and the strength is regained.

![Figure 11: Schematic overview of the behavioural phenomena identified in this chapter and their influence on the resuspension of sediment and the flow mobility. These last two behavioural aspects both lead to recirculation of sediment, thus should be limited or prevented.](image-url)
In the analysis of the flow of a submerged slurry, three key behavioural characteristics are identified. These are water entrainment, hydroplaning, and erosion (or reactivation) of the bed. These phenomena all have a direct impact on the mobility (also through resuspension of sediment) of the deposited material in the pond. Figure 11 schematically presents the qualitative relations between the flow phenomena and their effect on the sediment deposition.

The flow velocity is an important parameter that has an impact on both the amount of fines being resuspended from the slurry as on the erosion of the underlying sediment. If the slurry flows very fast, the slurry faces erosion and water entrainment. On the other hand, if the slurry does not flow fast enough to hydroplane, the flow bed is eroded.

From experiments on submerged mudflows it was observed that increasing the clay fraction of a sediment slurry increases yield stress. At the same time less material was resuspended from these slurries as they flowed down a slope. It therefore seems that the yield stress and clay fraction of the slurry are good indicators of a slurry’s coherence.

In conclusion, the main material properties that affect the behaviour of slurry flowing down a submerged slope are the slurry yield strength and the flow velocity.
3. Analysis of literature - Effect of material properties on behaviour

In this chapter the relationship between the slurry composition and its behaviour during submerged flow is investigated. Five key behavioural aspects of mud in water are defined in the previous chapter: entrainment of water, hydroplaning, segregation of coarse and fines particles in a slurry, erosion of the bed, and the strength increase of the material after deposition. In the next sections, the relationships between these five phenomena and the properties of the slurry and its constituting materials are investigated. Section 3.1 focuses on the role of the slurry bulk properties in predicting the flow behaviour, and draws mainly from the literature review. Section 3.2 aims at defining the relationships between the slurry constituents – water and sediment – on these slurry bulk properties. At the end of the chapter questions for further research are posed.

3.1 Role of slurry bulk properties

In this section, an overview of the slurry properties that influence the flow behaviour is presented. One of these is then selected for further investigation. To this end, the four key behavioural aspects as defined in the previous chapter are investigated for their governing parameters, focussing on the slurry properties. Most of the information in this section is drawn from the literature mentioned in Section 2.3. For the purposes of the parameter selection, the slurry is assumed a single homogeneous mass, and only properties applying to this bulk are considered. Where possible, the investigation of the role of any individual constituents is deferred to Section 3.2.

3.1.1 Entrainment of water

As is shown by Mohrig and Marr (2003) in Figure 9, the erosion of sediment from the flow is governed by the ratio of the yield stress to the dynamic stress $\tau_y/\tau_f$, where $\tau_f$ is defined according to Equation (19). If this ratio exceeds 0.2, erosion is likely to occur.

This means that the occurrence of erosion is governed by the yield stress and flow velocity (water density is assumed a universal constant). With a higher yield strength, the flow is more resistant to erosion, while a higher flow velocity increases friction, thus increases the likelihood the flow being eroded. The slurry velocity is however not an intrinsic material property, but also depends on external factors such as slope angle and length. Slurry properties that influence velocity are density and the rheological properties, in a Bingham model these are the yield stress and Bingham viscosity.

Until now, entrainment of water has been treated in this thesis as part of the erosion process. There is however a distinct difference between the two processes. While erosion is caused by large shear stresses acting on the surface of the flow head, entrainment is caused by the flow properties of the slurry itself. When the slurry has a relatively large internal permeability, the dynamic flow pressures can allow the ambient fluid to flow into the slurry, slowly increasing the water content of the slurry, thus decreasing its coherence (as observed by (Marr et al., 2001)) and hence increasing the erosion rate, until eventually the original slurry fully transfers into a turbidity current. The same dilution process applies to slurries where particles are only loosely bound together, and which flow under turbulent conditions. It is therefore likely that the onset of turbulence in the flow also impacts the coherence of the material. Because the onset of turbulence is controlled by flow velocity and rheological properties, i.e. viscosity and yield stress, this effect is assumed to be captured in the limit of $\tau_y/\tau_f$ as established by Mohrig and Marr (2003) and is not separately investigated any further.
3.1.2 Hydroplaning

Hydroplaning, or the uplifting of the debris flow by a thin layer of water, is observed only for relatively high velocity debris flows in subaqueous environments. Mohrig et al. (1998) showed that the densimetric Froude number – the dynamic pressure of the ambient fluid as defined in Equation (19) divided by the submerged weight of the overlying slurry – is an easy and a consistent indicator for hydroplaning. This confirms that flow velocity, but also slurry density, essentially governs the occurrence of hydroplaning. With a higher flow velocity, or lower slurry density the occurrence of hydroplaning becomes more likely. However, as mentioned in the previous sub-section on erosion, the slurry velocity is not an intrinsic material property, but is governed by both material and external properties. Slurry properties that influence the velocity are density and the rheological properties: yield stress and Bingham viscosity.

Hydroplaning also has a self-reinforcing effect: when hydroplaning sets in, friction is reduced and thus velocity increases. This means that the slurry velocity, which governs the onset of hydroplaning, is also itself influenced by the onset of hydroplaning. This is important, as the flow velocity influences not only hydroplaning but erosion as well.

There is however reason to assume that hydroplaning only occurs in strongly coherent flows, as experiments with less-than-strongly coherent flows did not exhibit any hydroplaning, while in some cases the Froude number would lead one to believe that they should hydroplane. In these instance a breakup of the flow head, instead of a lifting of the flow head is observed. Apparently these slurries were not strong enough to allow uplifting of their heads. This means that the ratio \( \frac{\tau_y}{\tau_f} \) and thus also the yield stress, is a factor to consider in evaluating the hydroplaning behaviour of a slurry. A higher yield strength increases the capacity of a flow to hydroplane. While an increase in yield strength and/or viscosity leads to a lower equilibrium velocity for the slurry – maybe even lower than the hydroplaning velocity – this is not likely to have an impact on the onset of hydroplaning, as the slurry velocity at the basin entry is in most cases much larger than the hydroplaning velocity.

3.1.3 Segregation

Sisson et al. (2012) stated that segregation does occur in the shear zone of a flowing slurry. Dynamic segregation should therefore be considered for all subaerial flows and for submerged flows that do not hydroplane. This is confirmed by the observations of e.g. Marr et al. (2001), who noted that in the non-hydroplaning flows (also the less coherent) segregation does take place. They also noted that in stronger flows, that do hydroplane, no segregation takes place. This means that the hydroplaning flows exhibit no shear during flow, because their thicknesses are below the yield thickness (as observed by Mohrig et al. (1999)). With no shear, the slurry continues to support all the suspended material. Because the supporting water layer dissipates all friction between the slurry and the bed, the slurry can still flow without being sheared in a similar way to plug flow.

This means that segregation in hydroplaning flows is governed by the static conditions, meaning that Equation (12) applies. Segregation is thus governed by yield stress and the size of the coarse particles.
3.1.4 Bed erosion
In the experiments of Mohrig et al. (1999), erosion of the bed was only observed in cases where the debris flow did not hydroplane. This is explained by the lack of dynamic stresses acting on the bed. The onset of erosion during flow is thus covered by the limits for hydroplaning. From the observations made by these same authors on dry flows, it is however inferred that when the yield height of the deposited sediment is reached, the sediment remobilizes. It is therefore expected that the onset of erosion after flow is governed by the density and yield stress profiles of the slurry and the bed. If the bed yield stress is large enough to support the additional load exerted by the non-hydroplaning slurry, the bed will not erode.

3.1.5 Strength increase after deposition
After deposition of the slurry, the shear strength of the slurry increases as a result of consolidation. This is interesting in the context of bed reactivation: if the bed is allowed to consolidate between two successive flows, the strength of the bed increases, and with that the yield thickness for reactivation of the bed also increases. The yield stress is therefore a function of the consolidation state. It would be interesting to determine what the yield stress is at a certain consolidation state, as this would allow for the production of mud at a certain yield stress.

The process of self-weight consolidation is actually a bit more complex than that, as the consolidation state depends mostly on the interaction between strength of the material, applied self-weight, and hydraulic conductivity of the material. In Section 2.1 it is noted that the strength and permeability of the material at a certain consolidation state depend very much on the material equations. As these equations are material specific discussing them is deferred to the next section. The material load is dependent on the slurry bulk, as it is a function of the submerged weight of the slurry, hence also of the slurry density.

3.1.6 Summary
Whether or not water is entrained in the debris flow is controlled by the material strength, or yield stress, and the flow governing properties.

The onset of hydroplaning is controlled by the slurry density and the flow governing properties. The increase in flow velocity associated with hydroplaning allows this phenomenon to sustain itself.

The occurrence of segregation is primarily associated with the occurrence of shear in the flow body. If the flow body is sheared, dynamic segregation occurs. If the flow body is not sheared, for example during hydroplaning, the occurrence of segregation is controlled by the yield stress and the grain size of the suspended particles.

The onset of erosion is chiefly controlled by the onset of hydroplaning. If the flow hydroplanes, no erosion is expected. If the flow does not hydroplane, erosion is governed by the vertical profiles of density and yield stress of the slurry and the bed.

The consolidation process is mainly governed by the material equations for effective stress and hydraulic conductivity, which depend mostly on the constituting sediments. The slurry density does influence the rate of consolidation.
The flow velocity and thickness are not intrinsic properties of the slurry, but are partially governed by the slurry density, viscosity and yield stress. These are also referred to as the flow governing properties.

This means that the entire flow behaviour is controlled by the following parameters:

- Yield strength
- Bingham viscosity
- Slurry density

A schematic overview of the relationships mentioned above is presented in Figure 12. Only consolidation is not presented, as it is controlled directly by the constituting materials. There we see that the viscosity only influences the behaviour indirectly, through the flow velocity. The slurry density has a certain influence on most aspects, except for segregation. Only the yield stress governs every aspect of the flow behaviour. It is therefore concluded that the yield stress is the most important parameter to describe mud flow behaviour.

![Figure 12: Schematic overview of the relationships between technical parameter (i.e. parameters that are dependent on intrinsic material properties) and the occurrence of the described phenomena. Relationship between flow thickness and bed erosion is dotted as it is inferred, but is not observed in the literature references. Sand grain size is a material property, but as it has direct impact on the occurrence of segregation it is included here.](image-url)
3.2 Role of sediment properties

In the previous paragraphs the technical parameters (e.g. the yield strength) that govern the critical phenomena were identified. An overview of this is presented in Figure 12. These properties are not uniquely related to a set of material properties. Actually the reverse is true: the material properties define the value of these parameters. We therefore look at the intrinsic material properties that control these technical parameters. Figure 13 displays a schematic overview of these relationships.

For clarity, the material properties that are investigated are as follows:

- Clay fraction of solids
- Silt fraction of solids
- Sand grain size
- Water content
- Clay mineralogy

In this section, the relationship between these material properties and the slurry yield stress is investigated. This is done in two steps. In Subsection 3.2.1, the particle size data from the literature presented in Section 2.3 is analysed for any relationships with slurry yield stress. In Subsection 3.2.2 other data is presented as well, and the effects of clay mineralogy are included in the analysis.

![Figure 13: Schematic representation of relationships between material properties (e.g. clay fraction) and technical parameters (e.g. yield strength). The connection between clay/silt ratio has a question mark, because it is unknown if and how the silt content affects yield strength. The consolidation process is included here, as the direct relationships between it and the material parameters are well understood.](image-url)
3.2.1 Yield stress as a function of particle size distribution

As already mentioned in Section 2.2, concentrated sediment slurries have a yield strength, and may exhibit a shear thinning behaviour, or a Bingham-like behaviour when the fraction of fine sediment is high. In this sub-section, the relationship between the different size fractions of sediment and the yield stress of the bulk fluid is analysed.

Marr et al. (2001) associated both an increasing clay fraction of solids and a decreasing overall slurry water content with increasing yield stress, as a higher clay content leads to higher cohesion and a lower water content means the matrix is less fluid. These relations were experimentally verified: slurries containing only sand and water, and no clay, displayed no coherence, hence had no yield stress. Figure 14 shows the observed flow behaviour in relation to yield stress and the clay and water contents of the mixture. Their approach would mean that not the fines content in itself, but the ratio of clay to other solids governs the yield strength.

Conversely, Sisson et al. (2012) investigated the rheology of slurries consisting of a mixture of sand, clay and water, and determined that the clay content of the carrier fluid determines the yield strength. For their analysis, they distinguished between the grains (sand) and the carrier fluid (a water and clay mixture). Furthermore they found that sand content only has an influence on the viscosity, and that the sand to fines ratio (SFR) determined whether or not a slurry could be considered segregating. For different values of the fines content in the carrier fluid, they noted different values of the SFR at which the slurry became non-segregating. From this observation it is concluded that the SFR also impacts the yield stress, as it is the yield stress that governs segregation behaviour, not the viscosity.

![Figure 14: A plot of water mass fraction of the slurry versus clay (kaolinite) mass fraction of solids. Numbers next to points denote the yield stress in Pa of the kaolinite-sand-water mixtures. Contours of the yield strength are also displayed. The open data point indicates that the experimentally determined yield strength is uncertain, as segregation of sand particles occurred. Comparable to Figure 18, showing test results with bentonite. (Marr et al., 2001)](image-url)
There are clearly two approaches to analysing the slurry composition. In the first approach the slurry is assumed to be a more homogeneous mass, and the yield stress is assumed to be a function of water content of total solids, and of the clay-sized fraction of the solids. This approach is taken by Marr et al. (2001), Illstad et al. (2004b) and Sawyer et al. (2012). In the second approach the slurry is mentally divided into a fluid part containing the water and fine sediment, and a solid part containing the coarser sediment. The yield stress is then assumed to come entirely from the fines concentration in water. This method is used by Sisson et al. (2012), who add that admixing coarse material increases the slurry viscosity and reduces segregation.

To compare these two approaches, Figure 15 & Figure 16 present the data of Marr et al. (2001), Illstad et al. (2004b) and Sawyer et al. (2012) (no yield stress data is available from Sisson et al. (2012) in a set of axes in accordance with the first and second approach respectively.

Figure 15 presents the combined results as a function of clay fraction of solids, and water content as a mass percentage of solids, in accordance with the first approach. The experiments of Marr et al. (2001) and Illstad et al. (2004b) were performed in the same flume, with comparable slopes and flow rates. The slurries they used consisted of kaolin clay, water and added sand: Marr et al. (2001) used 300μ sand, and Illstad et al. (2004b) used 500μ sand. Sawyer et al. (2012) did not add sand to the tested slurries, but instead experimented with different clay to silt ratios, data from their experiments is denoted with an added ‘+’ sign. From these results, a linear trend can be distinguished. Here the trend is clearly towards a higher yield stress for a larger clay fraction and a lower water content.

In Figure 16, where the data are plotted using the water content divided by the clay content and the fraction of total slurry volume occupied by the coarse fraction. While there aren’t data points, some trends can be observed in the figure. Firstly, it becomes obvious that for a higher water content, the yield stress is lower. It seems that the coarse fraction has no or limited influence on the yield stress at low concentrations, as the initial trend is more or less horizontal. When the coarse fraction dominates the slurry, and starts forming a skeleton, the yield stress does seem to increase. This relation is apparent at volumetric sand fractions higher than 0.5. However, as the data is sparse, no definite conclusions should be drawn.

The effect of granular particles being suspended in a yield stress fluid is also investigated in the field of chemical engineering. Chateau et al. (2008) compared a theoretical analysis to experiments done on suspensions of glass beads in a yield stress fluid. They found that the ratio between the yield stress of the mixture and that of the suspending fluid depends only on the volume fraction occupied by granular particles. Their results are presented in Figure 17. They noted that the effect of adding coarse particles increased as the concentration of coarse particles approached the point where a grain skeleton is formed. This point they defined as the maximum concentration of coarse material. At concentrations near and beyond this maximum, the material no longer flows as a fluid, but becomes increasingly frictional. After this maximum the material is no longer a viscous fluid, but a grain flow (Iverson, 1997). The relationship between the yield stress and the solid volume fraction they found is presented in Equation (21).

\[
\frac{\tau_y}{\tau_m^0} = \left(1 - \frac{\phi_{co}}{\phi_{co,m}}\right)
\left(1 - \frac{\phi_{co}}{\phi_{co,m}}\right)^{-1.25}\phi_m
\]  

(21)
Figure 16: Measured yields strengths plotted as a function of water content of fines (<2μm) and sand volumetric fraction of the bulk fluid. Increasing the fines water content leads to a lower yield strength, although the nature of this relation remains unclear. Sand volume appears to have limited effect at low values. At high values this effect seems to increase (dotted trendline). Data from Marr et al. (2001), Ilstad et al. (2004b) and Sawyer et al. (2012). Experiments from Sawyer et al. were performed with silt instead of sand, these data points are denoted with an extra '+' sign.

Figure 15: Measured yields strengths plotted as a function of water content and clay fraction, both of solids. Decreasing water content and increasing clay content both lead to a higher yield strength, apparently in a linear relationship (dotted trendlines). Data from Marr et al. (2001), Ilstad et al. (2004b) and Sawyer et al. (2012). Experiments from Sawyer et al. were done with silt instead of sand, these data points are denoted with an extra '+' sign.
In this equation $\tau_y / \tau_y^*$ denotes the ratio between the mixture and suspending fluid yield stresses and is called the dimensionless yield stress, $\phi_{co}$ denotes the ‘coarse’ volume fraction, and $\phi_{max}$ denotes the maximum coarse particle volume fraction. Alberts (2005) found that for monodisperse spherical particles typically $\phi_{max} = 0.6$.

As the role played by the sand grains is mainly a frictional one, generating resistance to flow, the question of whether the slurry sand content also reduces the erodibility of the fine slurry material. As sand does not provide the slurry with more cohesion, the inclusion of sand may have no, or a detrimental effect on the coherence of a slurry. On the other hand, as Sisson et al. (2012) pointed out, segregation of sediment may be a direct consequence of a fluidisation of the sediment (flow). Increased resistance to flow would then lead to decreased erodibility of the fine sediment. As this subject is too fundamental for the purposes of this thesis, it is decided to follow this last line of thought and investigate what the influence of sand could be. If later research points out that sand does not play a role, this influence can always be ignored.

While the relationship found by Chateau et al. (2008) may provide a solution to the influence of the sand structure in the slurry, the role of silt in the generation of yield stress is more obscure. In both Figure 15 & Figure 16, the data points where silt content has been counted as sand content are denoted by an extra + sign. While these point do not differ much from the overall trends, it could be argued that in the silt-rich slurries the yield stresses are actually higher than can be expected from the trend. This would mean that silt does contribute to the yield strength, but probably not as much as clay (at high silt content, yield stresses are only slightly higher than at low silt content). This strength effect could be due to the incorporation of silt in mud flocs. On the other hand, it could be similar to the influence of the sand, but that the silt has a lower maximum volumetric fraction – see Equation (21) – than the sand used in the other tests. The question here is if the silt fraction can be considered colloidal, because it settles and consolidates together with the clay, or not.
3.2.2 Influence of clay mineralogy

In a real world situation the clay mineralogy is never as well defined, nor has as pure a consistency as is the case in the experiments mentioned in the previous subsection. It is therefore important to get an understanding of how the clay type influences the relationship between yield stress and the sediment size distribution. That the clay mineralogy has an effect on this relationship is illustrated by the experiments of Marr et al. (2001), who not only experimented with kaolin clay slurries, but also with bentonite slurries (Figure 18). Comparing the bentonite results with the results from kaolinite (Figure 14) we see that to achieve the same yield stress, much less bentonite is required than is kaolinite.

The same problem of having to account for different mineralogies occurs in classical soil mechanics as well, when for instance the undrained shear strength of a clayey soil needs to be known. Subsection 2.1.5 shows that the undrained shear strength actually correlates well throughout different soil types at the same liquidity index. For this the Atterberg limits and the plasticity index are used as a means of classifying different clayey soils, all with different mineralogy, based on their strength.

![Yield Strength Contour Plot](image)

Figure 18: A plot of water mass fraction of the slurry versus clay (bentonite) mass fraction of solids. Numbers next to points denote the yield stress in Pa of the bentonite-sand-water mixtures. Contours of the yield strength are also displayed. The open data point indicates that the experimentally determined yield strength is uncertain, as segregation of sand particles occurred. Comparable to Figure 14, showing test results with kaolinite. From (Marr et al., 2001).

The Atterberg limits may also help in defining the role of silt, which is found to be problematic on a pure particle size distribution basis in the previous subsection. This is because the Atterberg limits are usually measured on fine soils, rich in clay and silt. The relationship between strength and liquidity index from Subsection 2.1.5 should then apply over the full range of fine particle size distributions as well.
Locat and Demers (1988) put this idea into practice in their investigations on the effect of pore fluid salinity on the viscosity, yield stress and remoulded shear strength of high-water-content soils. They noted that for sediments of higher plasticity, the rheology of the soils becomes more shear thinning than is the case for less plastic soils. For the six clay types (Table 2) that they investigated, two different types of behaviour were found: one type was very shear thinning, the other showed hysteresis effects. They decided to model the rheology of the pronouncedly shear thinning materials as Casson-type fluids, while modelling the rheology of the other materials as Bingham-type fluids. Examples of both types are given in Figure 19.

Their results, presented in Figure 20, show that there is also a distinct difference in results between the two different rheological models. For both models, the results fall mostly along the same line. The two lines however, deviate significantly for liquidity indices of 2 and higher. This raises questions on the applicability of the method for finding the yield stress. This doubt is further strengthened by the large range over which the shear rate was varied.

### Table 2: Material properties of Locat and Demers (1988)

<table>
<thead>
<tr>
<th>Site</th>
<th>Depth (m)</th>
<th>$c_{uy}$ (kPa)</th>
<th>$S_i$ (%)</th>
<th>$w'$ (%)</th>
<th>$w_L$ (%)</th>
<th>$I_p$ (%)</th>
<th>SS (m²/g)</th>
<th>CF (%)</th>
<th>$S$ (g/L)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Berthierville</td>
<td>6.1</td>
<td>1.38</td>
<td>25</td>
<td>47</td>
<td>44.2</td>
<td>21</td>
<td>49</td>
<td>36</td>
<td>0.3</td>
</tr>
<tr>
<td>Québec</td>
<td>—</td>
<td>2.04</td>
<td>—</td>
<td>41</td>
<td>52.4</td>
<td>28</td>
<td>62</td>
<td>60</td>
<td>5.6</td>
</tr>
<tr>
<td>Saint-Alban-1</td>
<td>2.3</td>
<td>0.20</td>
<td>39</td>
<td>71</td>
<td>42.2</td>
<td>20</td>
<td>49</td>
<td>49</td>
<td>0.5</td>
</tr>
<tr>
<td>Saint-Alban-2</td>
<td>2.3</td>
<td>—</td>
<td>—</td>
<td>—</td>
<td>55.0</td>
<td>32</td>
<td>59</td>
<td>59</td>
<td>30.2</td>
</tr>
<tr>
<td>Saint-Hyacinthe</td>
<td>4.3</td>
<td>0.77</td>
<td>8</td>
<td>82</td>
<td>62.8</td>
<td>37</td>
<td>96</td>
<td>75</td>
<td>1.6</td>
</tr>
<tr>
<td>Saint-Wenceslas</td>
<td>3.6</td>
<td>0.52</td>
<td>82</td>
<td>35</td>
<td>27.4</td>
<td>11</td>
<td>22</td>
<td>19</td>
<td>0.3</td>
</tr>
</tbody>
</table>

**Note:** SS stands for specific surface area; CF, clay-size faction; and $S$, salinity.

Locat and Demers (1988) put this idea into practice in their investigations on the effect of pore fluid salinity on the viscosity, yield stress and remoulded shear strength of high-water-content soils. They noted that for sediments of higher plasticity, the rheology of the soils becomes more shear thinning than is the case for less plastic soils. For the six clay types (Table 2) that they investigated, two different types of behaviour were found: one type was very shear thinning, the other showed hysteresis effects. They decided to model the rheology of the pronouncedly shear thinning materials as Casson-type fluids, while modelling the rheology of the other materials as Bingham-type fluids. Examples of both types are given in Figure 19.

Their results, presented in Figure 20, show that there is also a distinct difference in results between the two different rheological models. For both models, the results fall mostly along the same line. The two lines however, deviate significantly for liquidity indices of 2 and higher. This raises questions on the applicability of the method for finding the yield stress. This doubt is further strengthened by the large range over which the shear rate was varied.

**Figure 19:** Example rheograms of Saint-Alban-1 (a-c) at $I_L=3.0$ and Québec (d-f) at $I_L=2.4$. From left to right the 3 different tests are: a dynamic response (gradually increasing shear rate), a constant shear rate, and a hysteresis test (shear rate from high to low and back again). Note high shear rates for which apparent yield stress is determined in panel d. (Locat & Demers, 1988)
Figure 19 clearly shows an important consequence of using two different models, namely the effect of the chosen rheological model on the indirectly measured yield stress. The indirectly measured yield stress of the Casson model appears to be much higher than the measured stress at a near-zero shear rate. I conclude that the authors have overestimated the yield stress of the Casson-type fluids compared to the Bingham-type fluids (or conversely underestimated the Bingham-type fluids). Whichever of the two models or yield stresses should be adopted depends on the application: clearly the Casson-type is more applicable for higher shear rates than for near-zero shear rates.

Figure 20: Liquidity index plotted against yield stress. The two lines indicate the upper and lower boundaries. The upper boundary is approached by all Casson-type fluids, the lower by the Bingham-type fluids. (Locat and Demers, 1988)
3.2.3 Summary and conclusion

From the qualitative analysis above, it is concluded that the yield stress of a slurry is likely a function of the water content, the amount of fine sediment, and the mineralogy of the fine sediment in the slurry. For an accurate analysis of the yield stress, the slurry should be analysed as yield stress fluid consisting of ‘fines’ and water, and a ‘coarse’ solid fraction. The preliminary analysis suggests that the silt size fraction should not be treated as fines in this context, but as coarse (non-colloidal) material.

The role of clay appears to be quite well defined: the more clay and the more active the clay, the higher the yield stress of the slurry. The role of sand also appears clear: at low concentrations, the sand content has a negligible effect on the yield stress. When the structure of slurry becomes sand dominated, yield stress increases rapidly: between about 60% and 80% of volume, the effect is most pronounced. This corresponds to the analysis on non-colloidal particles in a yield stress fluid.

The role of the silt size fraction is still unclear: it appears that silt has a similar effect as sand. However, for the same increase in silt volumetric concentration and sand concentration, the silt addition leads to a greater increase in yield stress. A lack of data on silt-rich slurries makes it impossible to draw any conclusions.

Use of the Atterberg limits to be able to incorporate the sediment mineralogy, as well as any effects of silt into the analysis of yield strength seems promising. The method links the water content, and two index values, the plastic and liquid limits, to a strength value. This method has been proven effective for liquidity indices between 0 and 1. Experiments by Locat and Demers (1988) have shown that this relationship between liquidity index and strength can probably be extended to higher liquidity indices as well.

To be able to relate slurry composition to slurry yield stress, one fundamental question remains unanswered:

What is the influence of silt content on the yield stress?

Furthermore, the subject of yield stress estimation based on a small set slurry parameters has come up. This lead to the following question:

Can the slurry yield stress accurately be modelled as a function of the Atterberg limits and water content?

To answer these questions, a set of experiments on materials with different silt contents and different Atterberg limits is proposed.

The materials used in the experiment are mixed from two known base materials: one is rich in clay, the other rich in silt. The particle-size distributions of these materials are known, so the particle size distributions of the mixtures can be inferred from the mixing ration between the two constituents.

Of these mixtures, the Atterberg limits need to be known. For determining the plastic limit, the thread roll test is used. The liquid limit can be determined using the fall cone test. As this test is also used to determine the shear strength of mixtures (see below), the liquid limit is measured using the fall cone test.
The shear strength of each of the mixtures is then tested at a wide range of moisture contents. At low moisture contents, and thus high strength, the fall cone test is used. In this test the undrained shear strength is measured. At high moisture contents, and thus low strength, a viscometer is used to measure the shear resistance at a range of shear rates. From these measurements the shear strength at zero shear can be inferred, the yield stress.

Because the shear stress depends very much on the sample moisture content, the moisture content is determined directly by drying some of the sample. A schematic overview of the performed measurements is presented in Figure 21.

By comparing the results of the yield stress measurements to the silt content of the mixture or to the Atterberg limits, the analysis can answer the two questions above.

Figure 21: Schematic representation of the mixing of sediment and water, and the tests performed on the different samples. The bottom row in the schematic represents the different quantities that are compared with one another in Chapter 5.
4. Experimental procedure and results
In the previous chapter, it is hypothesized that the influences of silt and of the clay mineralogy on
the yield stress of a sediment slurry can be captured using the Atterberg limits. In this chapter, the
hypothesis is tested by performing a set of experiments. The experiments consist of strength
measurements on soils and slurries, in a range of moisture contents. The soils used to create the
slurries each have a different ratio of clay content to silt content.

4.1 Material preparation
Four different sediment mixtures were used in the experiment. As a base material, a common
modelling clay was used (Ve-Ka K-10000). To this material, a silica flour (Lieben Minerals LM25) was
added to create three new mixtures with different silt contents. The particle size distributions of the
clay, silt, and the three mixtures are displayed in Figure 22. Note that the silica flour also contains
some clay-sized material (<2 μm). The particle size distribution of the clay was determined by
Mulder (2013, pers. comm., Appendix C), while that of the silt material was provided by the
manufacturer (Lieben Minerals, 1997, Appendix B). The particle size distributions of the mixtures
were determined as a weighted average of those of the comprising materials.

![Figure 22: Particle size distributions of the tested materials.](image)

The modelling clay and silt materials were mixed manually, using two spatulas to gradually fold the
silt material through the clay material, see also Figure 23. Water was added to make the clay more
plastic and thus ease the mixing. The ratios of modelling clay to silica flour mixed are presented in
Table 3.
Table 3: Mixture composition and approximate particle size distribution of the tested materials.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Modelling Clay fraction (dry)</th>
<th>Silica flour mass fraction</th>
<th>Clay mass fraction</th>
<th>Silt mass fraction</th>
<th>Sand mass fraction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay</td>
<td>100%</td>
<td>0%</td>
<td>39%</td>
<td>55%</td>
<td>6%</td>
</tr>
<tr>
<td>1</td>
<td>83%</td>
<td>17%</td>
<td>35%</td>
<td>60%</td>
<td>5%</td>
</tr>
<tr>
<td>2</td>
<td>69%</td>
<td>31%</td>
<td>31%</td>
<td>65%</td>
<td>5%</td>
</tr>
<tr>
<td>3</td>
<td>60%</td>
<td>40%</td>
<td>28%</td>
<td>67%</td>
<td>4%</td>
</tr>
</tbody>
</table>

The different water contents needed for the relationship with the water content were created by adding slightly salted water of 1g/L NaCl to the mixture during the experiment, in between measurements. During the experiments, only a 1 g/L NaCl solution in demineralized water was used.

![Figure 23: Mixing the silica flour through the modelling clay.](image)

*Figure 23: Mixing the silica flour through the modelling clay. A: The silica flour is thinly spread over the clay. B: The clay is folded to mix in the silica. After the material has been homogenised (no more –white– flour is visible), the steps are repeated until all the flour is mixed through.*

The solution was chosen to reflect the typical salinity of tap water in the western Netherlands.
4.2 Experimental procedure
To be able to relate the yield stress to the undrained shear strength and thus the existing relationship between liquidity index and undrained shear strength, as suggested in the previous chapter, the strength of the material was measured both at high moisture contents – where the material behaves as a liquid – and the low range of moisture contents – where the material behaves as a plastic. The boundary between these ranges is by definition the liquid limit.

In the low moisture content range, the undrained shear strength was measured using the fall cone test. In the high moisture content range, the yield stress was measured using a viscometer test. Care was taken to leave some overlap in the moisture contents tested in both experiments, as this could be used to relate the two strengths to each other. To calibrate the results of the cone test, mixture 0 was also subjected to a vane shear test. While the vane shear test is the definitive method for determining undrained shear strength, the cone test requires less material and is quicker to perform. Hence the cone test was preferred for the other mixtures.

The Atterberg limits of each material were also determined. To determine the liquid limit, the fall cone results could be used, as described in BS1377:2 (1990a). To determine the plastic limit, the thread roll test was performed.

4.2.1 Vane shear test
The vane shear test is used to determine the undisturbed and remoulded shear strengths of a soil. As in this experiment only remoulded material was available, only the remoulded strength could be measured. The vane shear test is performed using a special device, which consists of a four-bladed vane that can be rotated through the sample using a crank. The vane is connected to a torsional spring. The spring is connected to a dial, allowing for a read-out of the maximum deflection of the spring, from which the applied shear stress can be calculated. Figure 24 presents a photograph of the used shear vane device.

![Figure 24: The used vane shear test apparatus, with close up of the vane and sample set-up.](image)
Procedure

After testing the sample in the fall cone test, the sample, still in its container, was tested using the vane shear test. The vane was first inserted approximately 1 cm below the surface of the sample and then rotated at 1 revolution per minute until the pointer on the dial indicated that the resistance fell. Over the course of all the tests, two different vanes were used. Their dimensions are listed in Table 4.

Table 4: Dimensions of vanes used in the vane shear test.

<table>
<thead>
<tr>
<th>Vane</th>
<th>D (mm)</th>
<th>H (mm)</th>
<th>K (mm³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5 inch</td>
<td>12,70</td>
<td>12,70</td>
<td>4290</td>
</tr>
<tr>
<td>0.75 inch</td>
<td>12,70</td>
<td>19,05</td>
<td>5898</td>
</tr>
</tbody>
</table>

Each sample was tested three times, with the results being averaged. Between each measurement the entire sample was removed from its container, and replaced with fresh sample material, as partial replacement of the sample led to widely varying results.

Analysis

The shear strength (in kPa) of the material can be calculated using (Verwaal and Mulder, 2006):

$$c_u = \frac{M_{peak}}{K_{vane}} \times 1000 \tag{22}$$

In this equation, M is the total torque applied, and $K_{vane}$ is the vane constant. These can be calculated using the spring stiffness ($K_{spring}$) and the maximum deflection angle on the dial ($\delta$), and the vane dimensions (D,H) respectively, see also Equations (23) & (24) below.

$$M = K_{spring} \times \delta \tag{23}$$

$$K_{vane} = \pi D^2 \left(\frac{H}{2} + \frac{D}{6}\right) \tag{24}$$

Over the course of all the experiments, four different springs were used, and – as mentioned – two vanes. Table 4 shows the vane constants used and Table 5 shows the stiffness of the springs.

Table 5: The stiffness of springs used in the vane shear test.

<table>
<thead>
<tr>
<th>Spring #</th>
<th>Stiffness (N mm/ deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.925</td>
</tr>
<tr>
<td>2</td>
<td>1.67</td>
</tr>
<tr>
<td>3</td>
<td>3.13</td>
</tr>
<tr>
<td>4</td>
<td>6.25</td>
</tr>
</tbody>
</table>
4.2.2 Fall cone

The fall cone test is a procedure in which a metal cone of known mass and with a known tip angle is allowed to fall into a remoulded soil sample. The soil sample is contained in a small container of prescribed dimensions according to the standard used. The penetration depth of the cone is a measure of strength.

The fall cone test is generally only used to determine the liquid limit. In that case the goal is to find the moisture content at which a certain penetration depth is achieved, as described in BS1377:2 (1990a). However, because this fall cone test is essentially a measure of strength, the fall cone apparatus can also be used to measure the strength of a material at a given moisture content.

The fall cone apparatus – sometimes also referred to as drop cone penetrometer – consists of a base plate, on which a standard is mounted. This standard supports a hollow rod, from which hangs the cone. A dial is also mounted on the standard and can be used to measure the penetration depth. Figure 25 shows the fall cone apparatus used. The cone that was used has a tip angle of 30° and combined with the hollow rod has a mass of 80.0 g.

Procedure

The sample was inserted into the container using the spatulas, making sure not to trap any air bubbles. The top of the sample was flattened, at an equal height to the top of the container. The container was then positioned below the cone, with the cone tip positioned directly above the centre of the sample. The cone was lowered until the tip just touches the surface of the sample. The dial was turned to zero, and the cone was released by pressing and holding the release button for 5 seconds. Afterwards the measurement was taken by turning the dial.

Figure 25: Schematic representation (left) and photograph (right) of the used fall cone apparatus.
Each sample was subjected to three measurements or more when the first three did not fall within a maximum range of 1 mm. At the end of the test, the results were averaged. Between each measurement the disturbed zone of the sample was removed, and replaced with fresh sample material.

After three measurements, some sample material was set aside for water content determination. More water was then added to the remaining sample and thoroughly mixed using the spatulas. The new sample was then tested according to the same procedure. This process continued until not enough sample material remained to perform a test, or until the penetration depth exceeded 23 mm.

**Analysis**

To go from penetration depth to undrained shear strength, the analysis of Koumoto and Houlsby (2001) is followed. The key assumption in this analysis is that the weight of the cone is fully supported by the strength of the material. As the cone penetrates further into the material, the area along which the material strength acts is increased, thus the resisting force increases as well. When the resisting force equals the driving force (the cone weight), the cone is assumed to stop.

From the above it is clear that the key components in calculating the undrained shear strength are the cone weight \( Q \), the penetration depth \( h \), and the cone geometry. Hansbo (1957) captured the cone geometry in a single factor \( K_{cone} \), called the cone factor. The resulting relationship is then:

\[
c_u = \frac{K_{cone}Q}{h^2}
\]

The cone factor is generally determined empirically as it differs from cone to cone, although Koumoto and Houlsby (2001) also devised a method of estimating it based on other empirical relationships. For a 30° cone they estimate a typical value of 1.33. An empirical value of 0.85 is provided by Wood (1985).

The liquid limit is defined as the water content at which the cone penetrates 20mm as per BS1377:2 (1990a)

**4.2.3 Viscometer**

A viscometer is used to investigate the rheology of fluids. It does this by measuring the torque required to shear a fluid between a (rotating) rotor and a (static) stator. For these experiments the Haake VT550 viscometer was used, as seen in Figure 26A. This viscometer uses a variety of rotor/stator combinations, also known as sensors, most of which are cylindrical. In all cylindrical sensors, the inner cylinder is rotated, while the outer one is fixed in place. The sensor is of Searle type, meaning that the torque is measured at the rotor: the inner cylinder is connected to the motor via a spring, and the deflection of the spring between the rotor and the motor is a measure of the torque applied to the rotor (Schramm, 1994). A schematic representation of the viscometer is given in Figure 26B.

To be able to measure the full range of yield stress from the liquid limit to 10 Pa, three different sensors were used. The sensors are listed in Table 6. Two of these sensors have a smooth surface, while one has a ribbed surface. The idea behind the ribbed surface is preventing the cylinder from slipping and giving false (lower) stress values. During slip, the shear rate would actually be (near-) zero, as there is no shear stress transferred from the cylinder to the fluid. However, because the
rotation rate does not change, the computer registers the same shear rate as before, but with a lower shear stress than expected. In the higher stress range unfortunately no ribbed sensors were available. Preventing or recognising slip is an important part of rheometry (Barnes, 1999).

Table 6: The dimensions and operational ranges of the sensor used in the experiments.

<table>
<thead>
<tr>
<th>Sensor</th>
<th>Ri (mm)</th>
<th>Ra (mm)</th>
<th>H (mm)</th>
<th>Surface</th>
<th>Minimum stress (Pa)</th>
<th>Maximum stress (Pa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MV2-P</td>
<td>18.4</td>
<td>21.0</td>
<td>60.0</td>
<td>Ribbed</td>
<td>76.8</td>
<td>230</td>
</tr>
<tr>
<td>SV1</td>
<td>10.1</td>
<td>11.55</td>
<td>61.4</td>
<td>Smooth</td>
<td>253</td>
<td>759</td>
</tr>
<tr>
<td>SV2</td>
<td>10.1</td>
<td>11.55</td>
<td>19.6</td>
<td>Smooth</td>
<td>768</td>
<td>2304</td>
</tr>
</tbody>
</table>

To test if the relationship for granular material in yield stress fluids proposed by Chateau et al. (2008) also holds for silt in clay-water slurries, an additional 3 mixtures with high silt contents (but the same clay/water ratios) were tested with the viscometer. The mixtures were made in accordance with section 4.1. The mixture compositions are given in Table 7.

Table 7: The compositions of the high-silt content mixtures used in the additional tests.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Modelling Clay fraction (dry)</th>
<th>Silica flour fraction</th>
<th>Clay mass fraction</th>
<th>Silt mass fraction</th>
<th>Sand mass fraction</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>49%</td>
<td>51%</td>
<td>22%</td>
<td>75%</td>
<td>3%</td>
</tr>
<tr>
<td>5</td>
<td>30%</td>
<td>70%</td>
<td>16%</td>
<td>81%</td>
<td>3%</td>
</tr>
<tr>
<td>6</td>
<td>24%</td>
<td>76%</td>
<td>15%</td>
<td>83%</td>
<td>2%</td>
</tr>
</tbody>
</table>

The Haake VT550 also contains a temperature control vessel that can be connected to a heating or cooling unit. In these experiments this option has not been used. Instead, the temperature of the surroundings has been monitored and found not to deviate more than 1 degree from an average temperature of 23°C.
Procedure

The sample material was prepared according to Section 4.1, and further wetted to a moisture content near the liquid limit. The material was carefully smeared into the stator, and the rotor was placed in the viscometer. The stator with the sample was then inserted into the holder on the viscometer. During this insertion, the fluid entered the annulus. Any excess fluid flowed over into the top recess of the cylinder, where it could not be sheared during the test. This insertion also exerted some torque on the rotor, hence a relaxation period was observed until the torque read-out stabilised.

Two different test were done on the samples. In between these tests the stator was removed from and reinserted into the holder to allow the spring to relax. At the same time, some of the remaining sample material was used to determine the moisture content.

The first test was a full hysteresis loop. In this test, the rotation rate started out at 0 and was continuously increased over a period of 120s to a value of 60 rpm. This rotation rate was then maintained for 30s, after which it was continuously lowered back to 0.

The second test was a stepwise increase in shear rate. The rotation rate was increased stepwise in a non-linear way. The measured rotation rates are: 1, 2, 4, 6, 8, 10, 20, 30, 40 & 60, all in rpm. At every step the shear was maintained for 15s, and the average shear stress over that period was registered.

After both tests, the sample was removed from the sensor. Both of the sensors components were cleaned and dried. Water was added to the remaining sample material, and mixed with spatulas, or stirred when possible. This material was then tested as described above.

The Haake VT550 was connected to a computer during the entire experiment, and the torque and rotation speed data were gathered digitally using the Rheowin 4.00.0002 software.

Analysis

Just as with the shear vane test, the shear stress can be calculated from the applied torque and the sheared surface. Because the recess in the bottom of the inner cylinder used in viscometer tests prevents any contact between the bottom of the cylinder and the rest of the material, the shear stress equation is slightly different than in the shear vane test, and is given in Equation (27). The shear rate is calculated from the cylinder rotation rate and the sensor inner and outer diameters ($R_i$ & $R_a$), see Equation (26).

\[
\dot{\gamma} = N \times \frac{\pi R_i^2}{15 R_a^2 - R_i^2}
\]  \hspace{1cm} (26)

\[
\tau = \frac{2\pi L R_i^2}{0.01}
\]  \hspace{1cm} (27)

All torque and rotation data were gather by Rheowin and converted to shear rates and shear stresses before being presented.
4.2.4 Thread roll test
The thread roll test is used to determine the plastic limit of a soil. The test makes use of heat radiating from a person’s hands to evaporate any pore water by kneading the sample manually, thus slowly drying the soil sample. In-between periods of kneading, the strength of the sample is tested by rolling it between the fingers and a plate to a thread. When the thread starts cracking, the thread’s moisture content is at the plastic limit.

Procedure
The procedure that has been followed is almost the same as that for the plastic limit determination in BS1377:2 (1990a). For each material, a sample was kneaded until small cracks began to appear. The sample was then split in three. Each of the three parts was then rolled into a thread with a thickness of 3 mm. When longitudinal cracks were observed in the thread at 3 mm thickness, the plastic limit was reached, see Figure 27. At this point the moisture content of each of the cracking threads is determined individually.

Analysis
The plastic limit is calculated as the average of the three moisture contents of the cracked threads.

4.2.5 Moisture content determination
The moisture content is defined as the moisture mass divided by the solids mass. The moisture content of a sample is determined by drying out the sample in an oven, and comparing the wet mass to the dry mass.

Procedure
First a dry and empty container or dish was weighed (W1). Then the sample was added to the container, and weighed (W2). The container with the sample was then put in the oven at 105 degrees for 24h. After this, the container and sample were weighed again (W3).

Analysis
The moisture content can then be calculated using Equation (28).

\[
w = \frac{W_2 - W_3}{W_3 - W_1} \times 100\% \tag{28}
\]
4.3 Results

4.3.1 Vane shear test
A total of 17 samples were tested using the vane shear test. Only samples of the pure modelling clay were tested using the shear test. These are compared to the results of fall cone testing the modelling clay. A $K_{\text{cone}}$-value of 0.85, as provided by Wood (1985), provides an adequate fit of the two values ($R^2 = 0.92$), see also the cross plot in Figure 28. The shear strength data as a function of moisture content is presented together with those of the fall cone test in Figure 29 in the next subsection.

![Cross-plot of the vane test results and the cone test results with $K_{\text{cone}} = 0.85$.](image)

4.3.2 Fall cone test
Besides the pure clay, also all three mixtures of clay and silica flour were tested using the fall cone. The penetration depths are converted to undrained shear strengths using the $K_{\text{cone}} = 0.85$, as derived in the previous subsection. In total, 43 samples were tested using the cone: 19 samples of the pure modelling clay, and 7, 8, and 9 samples of mixtures 1, 2, and 3 respectively. The results are presented in Figure 29.
4.3.3 Atterberg limits

The Atterberg limits of the four tested materials are given in Table 8. The liquid limit is defined as the water content at a 20mm penetration depth is measured. Hence the liquid limit is calculated by interpolating the results from the fall cone tests. Given $K_{\text{cone}} = 0.85$, the undrained shear strength at the liquid limit is 1.7 kPa. This value is in agreement with literature values (Sharma and Bora, 2003; Wood, 1985) thus further strengthening the estimated $K_{\text{cone}}$ value.

Table 8: Results of cone and thread roll tests: Atterberg limits and plasticity index of tested mixtures.

<table>
<thead>
<tr>
<th>Mixture</th>
<th>$w_p$</th>
<th>$w_l$</th>
<th>$I_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modelling Clay</td>
<td>26%</td>
<td>61%</td>
<td>35%</td>
</tr>
<tr>
<td>1</td>
<td>26%</td>
<td>53%</td>
<td>27%</td>
</tr>
<tr>
<td>2</td>
<td>24%</td>
<td>46%</td>
<td>22%</td>
</tr>
<tr>
<td>3</td>
<td>23%</td>
<td>43%</td>
<td>20%</td>
</tr>
</tbody>
</table>

Figure 30 shows the four mixtures plotted in a plasticity chart. The modelling clay and all the mixtures just fall on the clay side of the A-line. The modelling clay and mixture 1 are highly plastic, while mixtures 2 and 3 are of medium to low plasticity.

Determining the true activity of the clay is impossible, as the mineralogy of the clay-sized fraction changes between the different mixtures. An apparent activity is found instead. Figure 31 shows the plasticity index as a function of the total clay-sized content. The apparent activity is 1.37, but only for mixtures of modelling clay and silica flour in which the total clay content is greater than 14%, corresponding to a modelling clay content of 7%. This means that the silica flour ($\xi_{cl} = 12\%$) likely...
has a plasticity index of 0. This was confirmed experimentally, as no plastic limit of the silica flour was found. The liquid limit is approximately 35%. Due to high sensitivity of the silt strength to water content, the silt becomes liquid at around the same water content as it becomes plastic.

Figure 30: Plasticity chart showing the four tested materials.

Figure 31: Activity plot of the four different mixtures. Only an apparent activity can be determined, because the clay mineralogy changes between materials. The x-intersection indicates that pure silica flour has no plasticity.
4.3.4 Viscometer tests

In total 52 samples from three different materials were tested with the viscometer. Mixture 3 was not tested, because the cone test results do not differ much from that of mixture 2 – see Figure 29. Therefore no new information is expected from testing mixture 3. As an example, the results of some of the sample from mixture 2 are presented in Figure 32. Plots depicting all results are presented in Appendix D.

![Figure 32: Selected rheograms of samples of mixture 2 at different water contents. Increasing water content corresponds to decreasing measured shear stress. Note that the 4 samples with the highest water content were measured with a different sensor (MV2-P instead of SV1) and show different behaviour in the low shear rate range.](image)

Typically a rheogram measured using one of the smooth-walled sensors (e.g. w = 86% in Figure 32) shows a dual-viscosity type behaviour, with a very large viscosity in the lower shear rate range (>10 Pa·s) and a lower (<1 Pa·s) viscosity at higher (>20/s) shear rates. The ribbed sensor measurements (e.g. w = 111% in Figure 32) do not show this behaviour, but rather have only a single viscosity, smaller than 1 Pa·s. Because the main difference between the two types of sensors is the occurrence of slip, the higher viscosity is deemed a slip effect. The effect of slip is limited to the shear rate range at which the friction of a slipping cylinder is smaller than the shear stress needed to shear the slurry at the same rate. For the purposes of interpreting the data, it is assumed that the behaviour associated with the ribbed sensors represents the true flow behaviour of the slurry.

Another striking feature can be seen in the more irregular rheograms, such as w = 79% in Figure 32. While the measurements in this plot are quite ‘spikey’ in an upward direction, meaning that some individual values are much higher than the other values, there are no data spikes in a downward direction. This behaviour is also observed in the other tested mixtures. This is likely caused by the presence of sand grains in all of the tested mixtures. When a (slightly angular) sand grain gets caught between the rotor and stator, stresses increase to overcome the extra friction. When the sand grain rotates slightly – due to the rotation applied by the rotor – the rotor can move freely again, returning the shear stress to its regular value. After some further rotation of the rotor, the differential
rotational velocities in the annulus causes the sand grain to rotate again. The sand grain gets lodged between the rotor and stator, and the whole process begins again.

The rheograms in Appendix D are interpreted assuming a constant Bingham viscosity and a yield stress, meaning the rheology follows a Bingham model. In all interpretations, only data points at shear rates > 20/s are used in a linear regression, from which the yield stress and Bingham viscosity are obtained. Note that a Bingham rheology may not accurately describe the flow behaviour at higher shear rates, as some researchers found that a shear thinning model may sometimes be more applicable for clay slurries (Coussot, 1995; Locat and Demers, 1988). The Bingham model is in this case deemed adequate to describe the behaviour at these low shear rates, and to accurately find the slurry yield stress. The results from this regression analysis are presented in Figure 33 (yield stress).

![Figure 33: Results of the regression analysis on the rheograms generated during the experiments. Data points at shear rates <20/s were not included in the analysis.](image)

The results for the high silt content slurries are presented in Figure 34. These results do not show the same consistency of the results for lower silt contents. While the reference clay-slurry and mixture 4 still show the dual-viscosity behaviour as in the previous figures, mixtures 5 and 6 only have a single viscosity in the measured shear rate range. One explanation for this is that slip occurred at all measured shear rates in these tests. This thought is further strengthened by the observation during testing that mixtures 5 and 6 did not adhere to metal surfaces (such as the mixing spatulas) as well as the more clayey mixtures did. Following the reasoning above, that slip occurred during the entire measurement of mixtures 5 and 6, the measured rheograms cannot be used to determine the yield stress.
Figure 34: Rheograms of the high-silt content mixtures. Note the diversity in behaviour and deviation from the dual-viscosity type of behaviour as silica content increases/clay content decreases.
5. Analysis and discussion of results

In this chapter, the results are analysed to answer the questions from Chapter 3: Can the Atterberg limits be used to estimate a slurry's yield stress and what are the effects of the constituting materials on this yield stress? The answers to these questions are then be applied to the subaqueous deposition concept that is presented in the Introduction. Finally, this concept is discussed in the light of the results from this thesis.

5.1 Experimental results

5.1.1 Integration of experimental results

The combined results of all the fall cone and viscometer tests are presented in Figure 35. This figure shows that the yield stress shows the same behaviour as the undrained shear strength, in that decreasing the amount of clay in the mixture and increasing the water content both lead to a decrease in the mixture strength. More specifically, the results of the viscometer seem to be a continuation of the cone results. This is especially visible in the results of the modelling clay (blue), as there the results of the two overlap slightly around \( w = 65\% \).

![Figure 35: The combined results of the fall cone tests (circles) and the regression analysis on the viscometer results (crosses). Clay percentages in the legend indicate the amount of modelling clay in the mixture. Matching colours indicate matching mixture compositions.](image)

Earlier on, it was noted that the undrained shear strength of a soil and the yield stress of a sediment slurry might be comparable, since both describe the resistance to shear of a mixture of sediment and water in what can be considered undrained conditions – if the slurry behaved as drained material.
the whole problem of settling and consolidation would not exist. From this observation and Figure 35 it can thus be concluded that the yield stress can be considered the continuation of the undrained shear strength in the liquid ($w > w_d$) range. From here on in the thesis, the two are considered interchangeable and are denoted by $c_u$.

5.1.2 Shear strength as a function of liquidity index

In this subsection, the results of the fall cone and viscometer tests are compared, to see if the undrained shear strength and yield stress can be considered equal. The figures from the previous subsection are then discussed to find the material property or properties that can best be used to predict a slurry’s yield strength.

In Chapter 3 it is hypothesized that the yield stress of sediment slurry can be related to the sediment Atterberg limits. As the previous subsection showed that the yield stress is in essence a continuation of the undrained shear strength at higher water contents, and because the Atterberg limits are essentially an indexing method for this undrained shear strength, this hypothesis appears to be confirmed. To confirm this visually, Figure 36 presents the measured strength as a function of the slurry liquidity index.

![Figure 36: The combined results of the fall cone tests (circles) and the regression analysis on the viscometer results (crosses) plotted as a function of the liquidity index. Clay percentages in the legend indicate the amount of modelling clay in the mixture. Matching colours indicate matching mixture compositions.]

From Figure 36, it becomes immediately clear that indexing the results using the Atterberg limits is a good method for finding the yield stress over the full range of investigated water contents. While the similarity between the different mixtures is strongest near the liquid limit, mixtures 1 and 2 show a strong similarity down to a liquidity index of 4.
The results of the pure modelling clay do lie somewhat lower than those of the mixtures. This could mean that the plasticity index for the pure modelling clay is overestimated, as this would increase the liquidity index at a given water content. As the results do lie close together near the liquid limit, also for the viscometer, the plastic limit is then probably underestimated. When comparing the plastic limits of the different mixtures, it is striking that the plastic limits of the pure clay and mixture 1 are the same, while the other mixtures have a markedly lower plastic limit. This could mean that $w_P$ for mixture 1 is overestimated, or conversely that the $w_P$ of the pure clay is underestimated.

Two regression analyses were carried out on the base 10 logarithm of the strength, and the liquidity index: a linear and a quadratic regression. The results of these analyses are presented in Figure 37 and Figure 38 respectively. The linear estimation had an $R^2$ of 0.944 and the quadratic an $R^2$ of 0.982, both in the log-lin space. The difference is not considered to be decisive, as the extra term in the quadratic fit should always provide some improvement in $R^2$ (Khan, 2010). Optically, the linear approach is unable to capture the convex nature of the results in the plot. In the high and low $I_L$ ranges, the estimate is too low, in the mid-range the estimate is too high. Clearly the quadratic regression captures this and is therefore considered a better fit. The best fit for the estimation of slurry strength from liquidity index is then:

$$c_u(I_L) = 10^{0.839 I_L + 1.13}$$

A big disadvantage of this quadratic fit over the linear fit is that it overestimates results at values beyond its apex ($I_L = 4.3$). The estimation is therefore limited to $I_L \leq 4.3$. Besides this, both fits do not reproduce the strength of 170 kPa at the plastic limit ($I_L = 0$).

A special note is devoted to the confidence interval of the selected quadratic fit. At $I_L = 4$, the estimation range of the yield stress is the largest of the range depicted. This could however also be due to boundary effects pertaining to the apex at $I_L = 4.3$. Therefore we look at the estimation range at $I_L = 1$ and $I_L = 2$, which lie near the centre of the estimation range. At both these points, the upper confidence boundary is around 3.2 times the value of the lower confidence boundary, and around
1.8 times the estimated value. This means that the spread is quite large: a slurry which is mixed to spec could have a measured yield stress of only 56% of the expected value, or a measured yield stress of 80% higher than expected. When estimating yield stress, the order of magnitude should thus be more important than the exact value.

5.1.3 Comparison of results to literature

In this subsection, the combined results of the fall cone and viscometer tests are compared to similar results obtained from the literature, specifically those of Locat and Demers (1988) and Merckelbach (2000).

The study by Locat and Demers (1988) is already referred to in Chapter 3, as these authors also studied the relationship between the yield stress and liquidity index of sediments. They found that there are two common relationships between these two parameters, and that the relationship depends on the rheological model. To compare the fitted trend (Equation (29)) to their data, Figure 39 presents these in the same plot.

In Figure 39 it can be seen that the current experimental results show a strong similarity to those of Locat and Demers (1988) that were determined using the Casson-type rheological model. The results obtained by these authors by fitting a Bingham-type model are slightly different from the current results, especially for higher liquidity indices where their results fall outside of the estimation range. As these authors used sensors with smooth walls for their measurements, an explanation for this may be that during measurement of these two clays slip occurred, which was then inadvertently interpreted as part of the rheological model of the slurry. At least the conclusion of Locat and Demers (1988) that the Casson-curve is typical of highly saline samples is premature: the current experiments are conducted using water of low salinity (1 g/L), comparable to their low salinity samples (0.5 g/L).

Another dataset was obtained from Merckelbach (2000), who measured the yield stress of muds after settling and consolidation. In his study, the yield stresses were measured using a vane test.
rather than smooth viscometer sensors. His results are presented together with the current results in Figure 40. His results show distinctly larger yield stresses at a given liquidity index. This is entirely to be expected, as his measurements were done on undisturbed and (lightly) consolidated sediments, while the current experiments are all done on remoulded sediments. However, the distinctly log-linear relationship between yield stress and liquidity index that can be seen here does reinforce the idea that yield stress can be indexed based on only two measured values: the Atterberg limits.

Figure 40: The data from Merckelbach (2000) presented in a $c_u - I_L$ plot together with the trend found in this thesis.

Figure 39: The data from Locat and Demers (1988) presented in a $c_u - I_L$ plot together with the trend found in this thesis.
5.2 The role of sediment composition

While the previous section focuses on the best way to predict a slurry’s yield stress, this section aims to clarify the roles of the different materials that comprise the slurry in achieving this yield stress. The analysis is done visually and qualitatively, as only one type of clay was available, and hence no general quantitative relationships can be derived.

The main question that was posed in Chapter 3, is how the silt content of mixture affects the mixture’s yield stress. One way to investigate this, is to look at how the yield stress differs between the different mixtures – that all have a different clay to silt ratio – when they have equal ratios of the two components that are known to influence yield stress: water and clay. This is depicted in Figure 41; it shows the strength of the mixtures as a function of $\xi_{cl}$, the clay content of the carrier fluid, or the mass of clay-sized particles over the mass of both clay-sized particles and the total mass of water, see also Equation (30).

\[
\xi_{cl} = \frac{m_{cl}}{m_{cl} + m_w}
\]  

Figure 41 shows that for any clay-silt ratio, the yield stress is always the same at a given clay content of the carrier fluid, as the results of the different mixtures lie very close to each other. This means that changing the amount of silt in a slurry without changing the amounts of clay and water does not change the yield stress, at least not in such a degree that it can be noticed on a log-scale, while a small change in the carrier fluid clay content does change the yield stress significantly.

![Figure 41](image-url)

Figure 41: The combined results of the cone and viscometer tests presented as a function of the mass fraction of clay-sized material in the carrier fluid ($\xi_{cl}$). Clay percentages in the legend indicate the amount of modelling clay in the mixture. Matching colours indicate matching mixture compositions.

Apparently, the silt had little to no effect on the overall yield stress. An explanation for this is found in the results of Chateau et al. (2008), which are outlined in Section 3.2. These authors state that the
effect of silt on the yield stress depends strongly on the ‘maximum volumetric silt fraction’, or the volume fraction at which the silt forms a space-filling network, and hence the shear mechanism loses its viscous property and becomes purely frictional. The reasoning for low silt contents is that, because silt particles are unlikely to collide much at low concentrations, the frictional component in the shear mechanism is small. The results of Chateau et al. (2008) show that for a silt volumetric fraction of 75% of the maximum, the yield stress of the mixture is twice that of the carrier fluid. Only at higher fractions does this ratio increase rapidly.

The lack of effect of the silt content on a logarithmic scale is best illustrated with an example calculation (see also Table 9):

Suppose that for a mixture of the modelling clay and water the $\xi_{cl}^{*}$ is 0.5, which is relatively high. This means that the silt volume fraction is also high compared to all the other measurements. For a well sorted granular material a loose packing density leaves about 40% pore space, thus the maximum silt volume fraction is assumed 60%. For the pure modelling clay the silt mass fraction of solids is 55% and the clay mass fraction is 39% (Table 3) (sand is ignored as it is of a different order of particle size). A clay specific gravity (s.g.) of 2.65, a silt s.g. of 2.65 and a water s.g. of 1 are assumed. (The clay density is relatively high, but a lower value would give even lower silt contents, thus a smaller silt influence.) The silt volume fraction is then 33%, over half of the maximum. The dimensionless yield stress (mixture yield stress over fluid yield stress) is then 1.48. The silt mass fraction is then increased to 65%, the same as mixture 2. The silt volume fraction has then been increased to 42%, leading to a dimensionless yield stress of 1.87. The ratio between the two measured yield stresses is then 1.27. On a logarithmic scale this difference of 27% would be hardly visible, and it falls comfortably within the spread of the measurements themselves (see Subsection 5.1.2). Any influences of silt can thus not be discerned from Figure 41.

Table 9: Input parameters and resulting dimensionless yield stress from the example calculation.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Modelling clay</th>
<th>Mixture 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay content</td>
<td>39%</td>
<td>31%</td>
</tr>
<tr>
<td>Silt content</td>
<td>55%</td>
<td>65%</td>
</tr>
<tr>
<td>$\xi_{cl}^{*}$</td>
<td>0.5</td>
<td>0.5</td>
</tr>
<tr>
<td>$\varphi_s$</td>
<td>33%</td>
<td>42%</td>
</tr>
<tr>
<td>$\tau_y / \tau_y^{*}$</td>
<td>1.48</td>
<td>1.87</td>
</tr>
</tbody>
</table>

Because this role of silt is indistinguishable from the presented results, tests with very high silt contents were also performed. Unfortunately, these mixtures present a different challenge, as they do not attach to the smooth cylinder walls and hence only slip is measured. One test at a silt mass fraction of 75% of solids does provide some information (Figure 34 Mixture 4a): the yield stress is approximately 1.38 times the yield stress of the modelling clay mixture (at 55% solids). In both cases $\xi_{cl}^{*}$ is 0.34. This corresponds to a maximum silt volume fraction of approximately 55%, which is low. And even then the factor 1.38 falls within the spread of the measurements in Figure 41.

In Figure 41, all the clay-sized particles in the mixture are considered when calculating $\xi_{cl}^{*}$. This includes the clay sized particles in the silica flour. This is an interesting observation, because it means that all clay-sized particles contribute to the slurry yield stress, not just the clay minerals. Where
with clay minerals, the clay-sized fraction of the modelling clay is meant; it is assumed that all clay-sized particles in the modelling clay are clay minerals, and that all larger particles are not. To confirm this observation, Figure 42 shows the slurry strength as a function of $\xi_{\text{cl-min}}^*$ or the mass of the clay minerals over the combined mass of the clay minerals and the water. There we see that the slurry yield strength does increase with increasing silica flour content. Between the modelling clay and Mixture 2 the increase is up to a factor of 1.6, and thus inexplicable by the method of Chateau et al. (2008). This means that the silica flour contributes to the yield stress, up to a certain particle size: around 2 μm – the boundary between clay and silt size – as shown by Figure 41.

While most authors (Ilstad et al., 2004b; Marr et al., 2001; Mohrig and Marr, 2003) deem that sand does not contribute to a slurry’s yield stress, the theory of Chateau et al. (2008) indicates that any granular material in the slurry – of which sand is a prime example – increases the yield stress. This is important, because the method proposed in Chapter 1 allows for different ratios of fines and sand to be mixed together. Unfortunately, the effects predicted by Chateau et al. (2008) cannot be observed in the conducted experiments.

In conclusion, the yield stress is chiefly governed by the clay content in the carrier fluid ratio, where clay means clay-sized content (<2 μm). Silt – and sand – content most likely only secondarily governs the yield stress. The effects of sand and silt are not observed during the experiments. The sand content is never large enough to have any effect, while the expected effect of the silt content always fell within the natural spread of the results. However, as it is inconceivable that at high sand concentrations the strength of the material is unaffected by the grain structure, the supposed effects of the sand materials are included in the prediction method.
5.3 Integration of results into the method

In this section, the results of the experiments and the literature study are combined to provide a graphical method for estimating a slurry’s flow behaviour from the mixture parameters.

5.3.1 Slurry mixing

Now that the origins and a method for estimating a slurry’s yield stress have been devised, the yield stress of a mixture of fines, water and sand for a specific tailings stream may be estimated. This subsection presents an example calculation, using the modelling clay from the experiments as the basis for the mixture. For any application to any other material, the relevant properties of that material need to be known first.

In the mining and processing operation, the clay and silt are always treated together as fines, and thus silt contents are not likely to differ from those of the in-situ material. For mixing purposes, these two size fractions should then also be treated as one component, namely fines. This idea is reinforced by the results of the previous section, where it is shown that while a slurry’s silt content probably impacts its yield stress, this effect is not significant on a logarithmic scale. Further, when estimating the yield stress based on the liquidity index, a part of the silt effect is also already captured in the Atterberg limits. It is therefore decided to treat the fines together, and to calculate the yield stress of fines slurry’s based on the Atterberg limits of the fines.

Note that the approach in the previous paragraph means that for mixing purposes, the Atterberg limits of pure fine sediment (<63 μm) should be known, thus that any coarse material should be sieved from the test sample beforehand. Alternatively, the clay activity can be used to calculate those limits when combined with Equation (8) and a plasticity chart.

To reiterate, the effect of the water content and fine sediment (the modelling clay) can be estimated as a function of the liquidity index using Equations (31) & (32) below. The first of these calculates liquidity index from the water content and the Atterberg limits, in this case \( w_L = 61\% \) and \( w_P = 26\% \);

\[
I_L = \frac{W - W_P}{I_P}
\]

\[
c_u = 10^{(0.172 I_L^2 - 1.48 I_L + 1.60)}, \quad 0 \leq I_L \leq 4.3
\]

When sand is added to the wet fines, the yield stress increases. The increase of the yield stress is quantified using the dimensionless yield stress, which is defined in Equation (21). A key parameter in this equation is the maximum volume fraction of sand. For this example, this value is assumed 60%, which means the sand has a porosity of 40% when loosely packed.

The equations above and Equation (30) are combined to calculate the yield stress for any given mixture of fines, sand and water. The axes are chosen such that the mixture components can easily be calculated. The mixing process starts with a given fines slurry, to which sand can then be admixed. Therefore the x-axis is \( \xi_{fr} \), or the solids (fines) mass fraction of the slurry (Equation (33)). The y-axis is the mass fraction of sand over sand plus fines slurry \( \xi_{sa} \), and thus denotes the fractions of sand and slurry to be mixed. The results are presented graphically in Figure 43.
The coloured area in Figure 43 shows the application range of the estimation method. At higher fines contents the material becomes predominantly elastic, while at higher water contents Equation (32) is no longer applicable. The sand content is limited by its volumetric maximum (60%). The intervals on the labelling are structured in a 30–100–300–1000 style, except for the label 25 Pa. This last label denotes the minimum estimated yield stress for this method.

\[
\xi_{yi}^* = \frac{m_{cl} + m_{sl}}{m_{cl} + m_{sl} + m_w}
\]  

(33)

From Figure 43 it can once again be deduced that the water content of the fines is the deciding factor for the yield stress of a slurry. Sand content does have an influence on the yield stress, but this is mainly in the dilute fine suspensions at high sand contents (>40% of total mass).

### 5.3.2 Slurry mixture optimisation

To find the optimum mixture, the optimum yield stress needs to be known. Because a single optimum yield stress value would be too specific, considering the natural spread in the experimental results, a range of values between which the yield stress could be considered ideal is presented instead.

Ideally, the yield stress would be such that slurry can be deposited using the method described in Chapter 1, with as little loss of material due to resuspension as possible. This means that the slurry should fulfil at least the following requirements: it should show coherent behaviour when flowing underwater; and it should be pumpable, preferably using a centrifugal pump as these have a relatively high capacity.
From the results of Marr et al. (2001), we know that the slurries with yield stresses higher than ±35 Pa stay coherent during flow down shallow (<6°) slopes. Cooke (2007) and Wennberg and Sellgren (2007) did tests on the efficiency of centrifugal pumps when pumping tailings pastes. They found that for yield stresses between 100 and 200 Pa the pump efficiency is impacted, but by no more than 10%. Pumping pastes with yield stresses higher than 200 Pa did lead to strong (>20%) reductions in efficiency. A slurry with \( \tau_y = 200 \) Pa is thus considered to be pumpable. This gives an ideal yield stress range of between 35 Pa and 200 Pa.

The mixture ratios that these values correspond to are depicted in Figure 44. Clearly the sand content can be used to impact the strength at lower clay concentrations. However, it may be questionable if the sand in the mixture actually has an impact on the coherency of a slurry, as it was found that sand mainly increases the internal friction of a slurry, but has no cohesive properties of its own. It may well be that sand only increases the resistance of a slurry to flow, but has no effect on the erodibility of the fine sediment. The optimal mixture would then be defined along the x-axis of Figure 44 only, with sand content only coming in when the flow dynamics are simulated. This question of erodibility should be subject to further research, preferably including lab-or even pilot-scale testing.

5.3.3 Additional behaviour of optimal mixture

With the range of yield stresses and related mixture compositions is known, the behaviour of the tailings stream flowing into the dredge pond can be fully characterised. For this the relationships between the material properties and behavioural characteristics that were identified in Chapter 2 are used.

The tailings are continuously deposited on the edge of the pond, forming a small stack until the yield height (~20 cm at \( \tau_y = 200 \) Pa) is overcome, and the material slumps out down towards the pond.

![Figure 44: The optimal yield stress range (depicted in blue) for the deposition of slurried sediments on shallow (<6°) slopes. The red area depicts the mixtures at which slurries are even stronger, and thus are strong enough for deposition, but too strong for centrifugal pumps.](image-url)
Every time the yield thickness is reached, the flow slumps out again, until the water surface is reached. Because the flow body is thinner near the edges of the stream, and thus immobile, the material mainly flows through self-formed channels.

When the material reaches the water, it is initially immobile, as the yield height is larger in water than in air. After this new yield height is reached, slumping occurs again. When a slump releases a lot of energy, e.g. due to the mass involved, or a local steepening of the slope, a portion of the tailings stream may reach such a high velocity that it starts to hydroplane. This block can then slide all the way to the opposite end of the pond, unless it is stopped beforehand, for instance because the slope flattens, causing the flow to run out of energy (see also Mohrig et al. (1999)).

The suspended sand particles are not likely to segregate in the suspension. When the slurry is static, this is definitely the case, as was shown in Subsection 2.1.6: the yield stress of the optimal mixture is orders of magnitude larger than the 2 Pa needed to suspend sand particles of 1 mm. Note that this yield stress should be a property of the carrier fluid alone, which can be considered the case when the sand content is not near its maximum. However, when the mixture is sheared, the entire mixture is fluidised. This means that the sand grains can settle to the bottom. The free settling rate can be calculated using Equation (18). Assuming the flow velocity is low (as at high velocities hydroplaning sets in), the shear rate is assumed low (<10/s) meaning that the apparent viscosity is high (3.5 Pa·s to >20 Pa·s), and thus the settling rate is in the order of 0.01 mm/s for 1 mm grains. This seems low enough to prevent a significant redistribution of sand through the slurry. The added consideration here is that when sand grains form a significant part of the slurry, settling is hindered (see Section 2.1.3), further slowing the settling rate, or even preventing it when the concentration is near the maximum.

To completely understand the flow of the tailings and thus be able to predict the flow velocities and any reactivation of underlying sediment, ideally a numerical model should be developed. Numerical models for debris flows with a Bingham rheological model do currently exist, but are only applicable in subaerial conditions (Imran et al., 2001). Hydroplaning and resuspension of sediment in a subaqueous environment are currently not included in these models.
5.4 Application in dredge mining

5.4.1 Considerations in slurry production

In Section 2.1 the process of strength accretion in fine sediments was described. The current subsection aims to link the results of the slurry mixture optimisation to the processes described there to come to a discussion on the mixture preparation, including such practical considerations such as mixture preparation time.

As the fines slurry coming from the concentrator, where the dredged material is separated into sands and fines, is typically dilute, some dewatering (or thickening) is be required to get the fines slurry up to the desired yield stress. Figure 44 shows that even for the highly plastic modelling clay used in this study the fines concentrations need to be high to achieve the required yield stress: a concentration between 40% and 50% fines is considered optimal. In Section 2.1 it was stated that the unaided settling and consolidation of fine sediment on a large scale could take between months and years. Although these numbers were for achieving reclaimable land, settling and consolidation time is nevertheless a major factor in the slurry production process.

A number of measures can be taken to decrease the settling and consolidation time, a few of these are discussed below.

**Settling**

Two methods of settling time reduction are discussed: reducing the settling depth, and the use of additives.

Decreasing the settling depth decreases the time needed for mud flocs to settle. However, it also requires a larger settling area to retain the same volume capacity. Moreover, the reduced settling depth merely shortens the depth range at which large flocs—with relatively high settling velocities—settle. The settling time reduction is thus not linear with settling depth reduction; the actual effect is much smaller. Hence decreasing settling depth is not as straightforward a method as it may sound.

Another method of reducing the settling time is using additives such as flocculants and coagulants. These can significantly decrease the flocculation time, and thus cut a large part off the total settling time. Another benefit of using these additives is that they increase the cohesion of the thickened slurry. This effect can be compared to increasing the clay activity of the fines: the plasticity of the fines increases, thus at the same water content, liquidity index decreases. As liquidity index is a measure of the slurry strength, this means that the strength is also increased for the same slurry solids concentration. Adding to this effect, some additives may have a self-hardening effect, comparable to cement. These materials harden over time, and can eventually cement the sediment together, forming a rock-or concrete-like material (as for example in (Drost, 2012)). This is obviously a benefit for reclamation, as the backfilled areas become available much quicker than they would through self-weight compaction only.

The quantitative effects of different additives on the strength and behaviour of fines should be investigated separately when considering their use. There is a possibility that these additives not only increase the strength of the fines slurry, but also significantly alter their rheology. This altered rheology in turn affects the pumpability of the material, and impacts the flow patterns. When flow patterns are predicted from simulations, care should be taken to include the altered rheology. Flow
patterns predicted from experience should be ignored, as the altered rheology renders this experience useless.

**Consolidation**

Consolidation rate can be increased by increasing either one of the drivers for consolidation: applied (self-)weight and hydraulic conductivity.

Increasing the applied weight is quite straightforward: by increasing the consolidation column, the maximum applied weight is increased, and thus the product of the consolidation column is of increased density. A drawback is that the sediment retention time is also increased, thus a larger capacity (volume) is required. A taller consolidation column thus does not increase capacity, but does increase ultimate density.

Increasing the hydraulic conductivity is more difficult to achieve. There is a possibility that it is a side-effect of the use of additives. As additives increase the slurry strength, a slurry may have a larger strength at the same solids concentration. Hence for the same strength the slurry has a lower solids concentration, thus a higher void ratio. Since hydraulic conductivity is primarily a function of this void ratio (Subsection 2.1.4), additives increase the hydraulic conductivity at a constant slurry strength.

**Sand-mixing**

The sand can be mixed with the carrier fluid at three points in the slurry production process: before, during, and after settling and consolidation of the fine sediment.

![Diagram](image)

Figure 45: Mixing of sand and mud to form a homogeneous mass during the settling process. While the mud bed accretes at the mud-water boundary, sand accumulates at the sand-supportive boundary.

Mixing the sand before and during settling likely results in the same behaviour. The sand settles more quickly than the fine sediment, falling down onto and into the consolidating bed. If the sand addition is done continuously this leads to a homogeneous distribution of the sand throughout the slowly accreting bed: as the sand settles, so does the mud. The top of the bed accumulates strength,
and at some point it is able to support the settling sand. A schematic representation of this process is presented in Figure 45.

However, as the sand settles, it disturbs its surroundings in the same ways that are considered in hindered settling (Subsection 2.1.3). As the mud is more susceptible to disturbances than sand, there is a real possibility that the settling of sand reduces the settling rate of mud, an unwanted effect. Manning et al. (2013) also reported that in sand-rich environments, the flocs that form are smaller than in clay-rich environments, this could lead to even longer mud settling times. It is therefore preferred to mix the sand with the mud after consolidation, when the mud can readily support the sand. Mixing should then be done mechanically, making sure the process does not take more than a few minutes to prevent the sand settling out of the liquefied mud in the same way as was considered for segregation during flow at the end of the previous section.

While the mixture composition of a non-segregating slurry is pretty straightforward (see Figure 43), the actual preparation of such a mixture from a dilute fines slurry is more complex. Fortunately, a lot of research has been done on the subject of mud consolidation, and the knowledge for a successful slurry production is readily available. It is recommended here to investigate the precise effect of adding a generic flocculant to the slurry on the relationship between strength and mixture composition. This leads to a better understanding of the effects and possible benefits of adding the flocculant.

5.4.2 Discussion of the proposed deposition method

As a conclusion to this chapter, and the thesis, the different results of this thesis, from both literature and experiments, the deposition method that is described in the Introduction is discussed. The emphasis of this subsection is to identify any considerations that are not yet identified in the Introduction.

As mentioned in subsection 5.3.3, the hydroplaning effect that is identified for debris flows also has consequences within the setting of the confined dredge pond. When blocks of fine sediment regularly flow out toward the production face, the goal of the method – preventing sediment recirculation – is defeated. Therefore a trapping mechanism, that prevents these mobile flows from reaching too far into the pond, is needed. Such a trapping mechanism should contain a slope break, and can be in the form of a barrier or of a depression in the bottom of the pond. The capacity of such a trap can be a fraction of the total deposited volume, as the outrunner blocks observed in natural debris flows typically constitute only a small part of the total mud flow volume (Ilistad et al., 2004a).

The addition of sand was considered as a means to increase the consolidation rate of the tailings material after deposition. The results from this thesis indicate that the addition of sand to the tailings may also be beneficial for the proposed deposition method: as the deposited material becomes more resistant to flow, the erodibility of that material also decreases.

From the results presented in Figure 44, it follows that consolidation already plays a large role in the production of the slurry, as is mentioned in the previous subsection. It is therefore suggested to further investigate and develop methods to increase self-weight consolidation rates. While it is known that adding sand to a mud bed increases the self-weight, and thus speed up the consolidation process, it is interesting to know what the effect of adding sand on the material functions is,
especially as adding sand also changes the void ratio of the overall material. Further research on this topic is needed.

A subject which is remains untouched in this thesis is the shape of the submerged tailings stack, which is critical when it comes to designing a mining operation or deposition facility. To determine this, smaller scale experiments may find some use. Fitton (2007) noted that very small, or lab scale models are not suitable. Ultimately a (numerical) model is required to incorporate the tailings deposit into the pond design. Numerical models are already mentioned briefly in the previous section, but their absence deserves another mentioning here: currently are no numerical models available that accurately model the behaviour of mud or debris flows under water. But looking at the general shape of subsea debris flows, it should be expected that the tailings stack has a form which is strongly elongated in the direction of flow, perpendicular to the pond edge.

Finally, but crucially, while submerged deposition is interesting from a land use perspective, it eventually results in longer consolidation times than when tailings are deposited subaerially, as the process cannot benefit from atmospheric effects such as evaporation. This trade-off should always be at the forefront when deciding on the deposition method.

In the end, any attempt at submerged deposition of tailings has to be preceded by small scale and ultimately full scale trials, as a model cannot even predict the different turbulence regimes in a pond. Practical considerations prevent a purely theoretical analysis. Nevertheless I do hope that this thesis brings a fundamental understanding of the forces at work in such an operation, and shortens the experimental phase of any future projects regarding this topic.
6. Conclusions and recommendations

6.1 Conclusions

It is concluded that in determining if a slurry is suitable for stable deposition in a dredge pond the clay-sized fraction of the slurry material plays a decisive role. The material property that was found best suited as a measure of a slurry’s coherence when deposited under water, the yield stress, was found to be mostly a result of the clay-sized material fraction. The magnitude of the clay-sized fraction is critical for determining which water contents result in a stable slurry. The silt and sand fractions are found to be of minor relevance. A convenient method of estimating a slurry’s yield stress, and thus suitability for deposition, uses the liquidity index of the fines fraction.

The answers to the secondary research questions are as follows:

1.1 Fine sediment is resuspended in the dredge pond by erosion of the slurry or dilution of the slurry by entrainment of water. These processes form as a results of the dynamic water pressure – a function of the flow velocity – becoming larger than the cohesive forces holding the material together. The key material property to estimate coherence of the flow is the yield stress.

1.2 The flow mobility is mainly governed by the slurry’s ability to stay intact – or coherence. When a coherent slurry achieves a high flow velocity it may hydroplane, reducing friction and thus further increasing the flow velocity and mobility.

1.3 Consolidation processes are governed by the material functions, describing the relationship between the material void ratio on one side, and its permeability and effective stress carrying capacity on the other. The consolidation rate and maximum consolidation state are furthermore controlled by the load which is applied to the material. A way to increase the load in self-weight consolidation is to increase density: if the material is denser, the self-weight applied is larger.

2.1 For the method to be feasible, the fine material should not be resuspended in the dredge pond. Furthermore it is desirable to have a low material mobility, a high consolidation rate and no segregation of any added sand.

2.2 Because the slurry yield stress is considered the material property of prime importance for estimating the feasibility of the method, limits were only established for this parameter. As a minimum, yield stress should be 35 Pa for a slurry to remain coherent at low slope angles. As a maximum, the pumpability with a centrifugal pump was used. This can be guaranteed up to a value of 200 Pa. However, for different pumps different values apply.

3.1 The yield stress of a fluid is mainly dependent on the amount of clay-sized material in suspension. Any coarser material such as silt and sand only has a noticeable impact at very high concentrations.

4.1 It is investigated if the liquidity index is suitable as a quick method of yield stress estimation for fine sediment slurries. The yield stress of a slurry can then be estimated without having to know the precise relationships between particle size distribution, clay mineralogy and yield stress. It is found that the yield stress can be estimated using the liquidity index.

5.1 The mixture compositions corresponding to the limits of objective 2 depend heavily on the sediment composition. As an example, the mixture composition limits for the clay used in the experiments is given in Figure 44. The main parameter affecting yield stress is the water content of the primary fines slurry.
It was furthermore concluded that:

The yield stress of a slurry of fine sediment can be considered as the continuation of the undrained shear strength of the constituent sediment in the liquid (w>\(w_L\)) range.

The yield stress and thus also the undrained shear strength of a fine sediment slurry can be modelled as a function of Atterberg limits, through the liquidity index. The accuracy of this method is not very high, but the range of expected results is small enough to allow suitable application in submerged deposition. This type of estimation method allows the estimator to forgo any considerations pertaining to clay mineralogy or the grain size distribution of the fines.

Adding sand to a yield stress fluid (such as a concentrated fines slurry) increases the yield stress of the overall fluid. There was however no indication of whether the process of fines resuspension is controlled by the overall fluid yield stress, or by the carrier fluid yield stress. This difference influences the decision to mix sand with the fines.

The total increase in yield stress caused by adding silica flour cannot be explained by the theoretical effect of the silt content alone, indicating that the clay-sized silica also increases the yield stress.

Because the submerged weight of a slurry is less than its weight on dry land, the same yield stress allows for a steeper slope of the subaqueous tailings stack than it does for a subaerial one. However, debris flow analogues indicate that this angle is likely still not steeper than a few degrees.
6.2 Recommendations
To prevent recirculation of fines during submerged deposition of tailings, it is recommended to increase the strength of the fine tailings before deposition. This can be done for instance by dewatering the tailings, or by using additives such as flocculants. Methods for increased dewatering should therefore be investigated with the purpose of adding strength to the material.

Flocculants affect the Atterberg limits of the material, thus impact the relationship between mixture composition and strength. When flocculants are used, the effects of these flocculants on the Atterberg limits should be investigated. If the Atterberg limits are corrected, the liquidity index is likely suitable again to estimate the slurry strength using the original relationship.

Further research should be directed to investigating the role of sand in both the yield stress increase of the overall slurry, and its effect in particle segregation and fines erosion. The question that arose during the study was if the increase in slurry yield stress caused by the addition of sand actually affects the minimum force required for resuspension of the fine sediment during flow at all, or if it only changes the flow dynamics of the overall slurry.

It is recommended that future research into slurries consisting of both fine and coarse sediment the slurry is treated as a two-phase flow. One phase is the carrier fluid, which has a non-Newtonian rheology and consists of water and clay-sized sediment, or of water and fines when the Atterberg limits are used in the analysis. The other phase is a solid phase, which consists of the sand and silt particles.

To accurately separate the effects of the clay-sized and silt-sized fractions of the silica flour, another test should be done where the silica flour added consists of only silt-sized particles. This should isolate the effect of silt-sized material. The theory that silt particles should be treated as coarse material, and thus as part of the solid phase of the slurry, can then be validated.

Highly coherent tailings slurries are also highly mobile, meaning they flow out over large distances. This may allow these slurries to flow towards the opposite end of the tailings pond. A trapping mechanism such as a sand barrier is then needed to prevent recirculation in this matter.

To come to an accurate design of the tailings stack, including expected slope and run-out distance, numerical models incorporating hydroplaning effects and the consolidation and erosion of underlying sediment should be developed.
7. References

Literature


BSI, 1990a. BS 1377 Methods of test for soils for civil engineering purposes, Part 2: Classification tests, London.


**Personal communication**


Mulder, A., 2014. Hydrometer test of veka clay (first batch), Delft [e-mail] (23 April, 2014)
Appendix A – Company overview

Royal IHC (IHC) is one of the world’s leading manufacturers for the specialist maritime industry. Headquartered in Sliedrecht in the Netherlands, they provide advanced dredging and mining vessels and equipment, as well as innovative ships and systems for the offshore construction industry. IHC has over 3000 employees and has offices all over the world. The operations of IHC are structured along three main product lines: dredging, offshore, and mining (IHC Mining).

IHC has a long history in providing mining equipment. In fact, after six shipyards in the Netherlands decided to work together (under the name Industriele Handels Combinatie, or IHC) one of the first combined projects was the production of the large vessels used in mining tin ore off the coast of Indonesia (see also the figure below).

Today IHC Mining focuses on integrated systems for wet mining applications in onshore, near-shore and deep sea environments. IHC is able to deliver all the equipment for a mining operation: mining dredges, slurry transport systems, and mineral separation plants. In addition to equipment, IHC also provides a number of consultancy services. They can assist prospective miners with their operation during every stage, from the planning stage to mine closure. In the onshore environment these services can help a client decide whether or not dredge mining is a viable option, and how this method compares economically to other methods, usually dry open cast mining.
Appendix B – Analysis of silica flour by Lieben Minerals

Granulometrie: Type LM25

Controle nat - zeying, Retsch DIN-ISO 3310-1
Norm zeefrest 63μ : 0,30 %
Tolerantie : + 0,20 %

Controle luchtstraalzeef Alpine H&B DIN-ISO-3310-1
norm controle zeef 40μ : 1,50 %
Tolerantie : 0,50 %

Ultrason analyse ( Cilas ) :
D-50 waarde : 11 - 15μ ( zie bijlage )

Gemiddelde chemische analyse:

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</table>

Physische eigenschappen:

pH : 7,00 +/- 0,50
Soortelijk gewicht : 2,65 g/cm³
Hardheid : 7 Moh’s

Lieben Minerals Laboratorium

04/01 '01 DON 14:49 [TX/RX NR 5874] #002
### Particle Size Measurement CILAS 715

**Material**: E9 IM25 S21  
**Comment**:  
**Suspension fluid**: WATER  
**Wetting agent**:  
**Ultrasonic**: 60 sec  
**Sample density**: 2.650 g/cm³  
**Concentration**: 169  
**File name**: 0812801

#### Particle Size Volume Distribution

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<th>Cumul. greater /%</th>
<th>Diff. /%</th>
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</table>

\[ D(100 / 90.0 %) = 1.84 \text{ um} \]
\[ D(50.0 / 50.0 %) = 13.81 \text{ um} \]
\[ D(90.0 / 10.0 %) = 39.01 \text{ um} \]

**Surface area**: 1.37 m²/cm³  
**Corresponding to**: 0.52 m²/g (spherical shape assumed)

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**M.Sc. Thesis**

M.C. Ensing
Appendix C – Particle size analysis of Ve-Ka clay

**Hydrometer test**

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<th>Engineering Geology Lab.</th>
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<td>Scale factors</td>
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</table>

**Input parameters**

- Sample: 23
- Temperature, T [°C]: 23
- Dynamic viscosity, η [mPas]: 0.04000
- Particle density, ρs [Mg/m³]: 2.65
- Density reading, R0 [g/mL]: 1.000
- Initial hydrometer reading, R 0: 1
- Start mass of the sample, m0 [g]: 37.23
- Meniscus correction, Cm: 0.5

**Sieve analysis**

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<th>Passed %</th>
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**Hydrometer analysis**

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<th>Hydrometer reading</th>
<th>Corrected hydrometer reading</th>
<th>Modified hydrometer reading</th>
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*M.Sc. Thesis*  
M.C. Ensing
Appendix D – Results of viscometer tests

Hysteresis loop tests

Figure A: Modelling clay viscometer hysteresis loop test results, part 1 of 3.

Figure B: Modelling clay viscometer hysteresis loop test results, part 2 of 3.
Figure C: Modelling clay viscometer hysteresis loop test results, part 3 of 3.

Figure D: Mixture 1 – 83% modelling clay and 17% silica flour – viscometer hysteresis loop test results, part 1 of 2.
Figure E: Mixture 1 – 83% modelling clay and 17% silica flour – viscometer hysteresis loop test results, part 2 of 2.

Figure F: Mixture 2 – 69% modelling clay and 31% silica flour – viscometer hysteresis loop test results, part 1 of 2.
Figure G: Mixture 2 – 69% modelling clay and 31% silica flour – viscometer hysteresis loop test results, part 2 of 2.

**Stepwise averaged measurements**

The first 7 samples, corresponding to Figure A, were not tested using this stepwise measurement.

Figure H: Modelling clay viscometer stepwise measurement test results, part 1 of 2.
Figure I: Modelling clay viscometer stepwise measurement test results, part 2 of 2.

Figure J: Mixture 1 – 83% modelling clay and 17% silica flour – viscometer stepwise measurement test results, 1 of 2.
Figure K: Mixture 1 – 83% modelling clay and 17% silica flour – viscometer stepwise measurement test results, 2 of 2.

Figure L: Mixture 2 – 69% modelling clay and 31% silica flour – viscometer stepwise measurement test results, 1 of 2.
Figure M: Mixture 2 – 69% modelling clay and 31% silica flour – viscometer stepwise measurement test results, 2 of 2.