The use of additives to stabilise dredged material
The use of additives to stabilise dredged material

By:

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SUMMARY

A study regarding the use of additives to stabilise dredged material has been conducted. The research objectives were to find additives, describe the effects the additives have on the dredged material, explain these effects and the necessary properties to deposit the stabilised material at a high rate, being 2000 m$^3$ per hour. All these objectives have the overriding goal of creating a material that has an undrained shear strength of at least 75 kPa, which equals an unconfined compressive strength of 150 kPa. Equipment used on such a site exerts a maximum pressure on the surface of 75 kPa. Areas that lack sand or containment sites for polluted dredged material can benefit from this method.

A literature study and an experimental study have been conducted to investigate the effect of additives on the strength of dredged material. The literature study showed that the most important additives that have been used historically to stabilise weak soils are lime and cement. Various studies have been conducted with regard to other additives, which are mostly secondary additives to cement or lime, such as fly ash and ground-granulated blast furnace slag. Plaster has also been found during the literature study as an additive that can function on its own, but was deemed too weak. Air foam and EPS beads can be used to decrease the density of cement-stabilised dredged materials, in order to decrease settling. Of all the additives, cement increases the undrained shear strength the most and is least expensive. Cement is therefore the preferred additive. The secondary additives to cement can decrease the price significantly. Air foam and EPS beads can be used to decrease the density to counter the density increasing effect of the cement.

In the experimental study, the change in water content and Atterberg limits caused by these additives has been investigated. Adding a dry material to a slurry has two effects: 1) it lowers the water content and 2) it changes the consistency limits. Two additives have been used, i.e. Portland cement and bentonite and a combination of both. The additives are added to three different types of mud. They are added in such quantities, that the expected water content is well below the expected liquid limit. Furthermore, the change in liquidity index is correlated to the increase in undrained shear strength.

A dataset, established during the literature study, and the data found in the experimental study are combined to investigate the effect of different additives on the undrained shear strength. A trend line, relating the undrained shear strength to the additive content, is developed in this study, which has been benchmarked against the results for the literature and experimental studies.
The observed behaviour is explained as follows: cement causes the clay particles in a slurry to aggregate, after which the clay particles and flocs are cemented. At the same time the cement causes a hydration reaction with water. The reaction also creates a cement matrix, which causes the increase in strength and binds water. The presence of pozzolanic clays, such as kaolinite and bentonite, can increase the strength over curing time. Because cement binds water, the liquid and plastic limits are increased. At a higher cement content, the increase in plastic limit stalls and the liquid limit starts to decrease. The latter process is caused by the reaction of superfluous cement on the surface of the aggregates formed, causing the surface activity to decrease. This in turn decreases the liquid limit.

It is shown that the effect of the cement addition on the liquidity index is correlated with the increase in undrained shear strength, regardless of the slurry used or the presence of bentonite. The undrained shear strength at liquid limit does not correspond with the formulas derived by Leroueil, Tavenas & Le Bihan (1983) or Muir Wood (1990), but is approximated at 30 kPa. This is caused by curing of the cement, in which a structure is developed.

Adding bentonite only to a slurry does not increase the undrained shear strength sufficiently, when economical quantities are considered. The increase in undrained shear strength follows relationships of Locat & Demers (1988) and Leroueil, Tavenas & Le Bihan (1983). The undrained shear strengths in the experimental study only reach approximately 5 kPa. After a review of the data found in the literature and experimental studies, a formula is proposed to describe the increase in liquid limit of the mixture.

Knowing the initial Atterberg limits of the slurry and bentonite and the water content of the slurry, an estimate of the undrained shear strength can thus be calculated. Although adding bentonite does not result in sufficient strength development, it can be used in a “building with nature” setting, when the site needs to be accessible by people or animals only and a limited increase in bearing capacities is required.

A combination of cement and bentonite is also tested. When comparing the cement-bentonite mixture to pure cement, the cement-bentonite mixture gives a lower increase in undrained shear strength at the same total additive content. As bentonite can be three times more expensive than cement, this mixture is not beneficial when compared to pure cement.

To be able to deposit the mixture at a high rate, the pumps and pipelines must be able to cope with the mixed material. It is shown that pipe pressure losses can be mitigated by installing a pipe with a larger diameter, which requires a lower flow velocity. Pumps have a maximum yield stress they can overcome. Common industrial centrifugal pumps can overcome a yield stress of
250 Pa. The yield stress can be correlated to the liquidity index (Locat & Demers, 1988). The liquidity index that correlates to this yield stress is 2.2. This means that slurries that have a high liquid limit may not be suitable for this procedure, as the liquidity index might drop too fast upon addition of solid additives. Additives that show sufficient curing are the only additives suitable for a wet in-line mixing method. Upon addition, the yield stress should remain below 250 Pa, but after curing the undrained shear strength should at least be 75 kPa.
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TABLE OF CONTENTS

Summary ........................................................................................................................................................................... iii
Acknowledgements ........................................................................................................................................................... vii
Figures ........................................................................................................................................................................... xiii
Tables ............................................................................................................................................................................. xvi

1 Introduction ............................................................................................................................................................ 1
  1.1 Background .................................................................................................................................................... 1
  1.2 Problem definition ........................................................................................................................................ 2
  1.3 Research objective ........................................................................................................................................ 3
  1.4 Research questions ....................................................................................................................................... 4
  1.5 Methodology ................................................................................................................................................... 4

2 Reference projects .................................................................................................................................................. 7
  2.1 Summary ....................................................................................................................................................... 7
  2.2 Port of Valencia .............................................................................................................................................. 7
  2.3 Central Japan Airport .................................................................................................................................... 11
  2.4 Tokyo International Airport runway D ............................................................................................. 13
  2.5 Vuosaari Harbour ........................................................................................................................................ 17
  2.6 Vlassenbroek ................................................................................................................................................ 18

3 Theoretical background .................................................................................................................................... 21
  3.1 Dredged material behaviour ......................................................................................................................... 21
  3.2 Additives ........................................................................................................................................................ 22
    3.2.1 Portland cement ........................................................................................................................................ 22
    3.2.2 Ground granulated blast furnace slag .............................................................................................. 23
    3.2.3 Quicklime ............................................................................................................................................... 23
    3.2.4 Fly ash .................................................................................................................................................... 24
    3.2.5 Cement kiln dust .................................................................................................................................... 25
    3.2.6 Gypsum ............................................................................................................................................... 25
    3.2.7 Biomass ash ........................................................................................................................................... 26
    3.2.8 Air foam & EPS beads ....................................................................................................................... 26
    3.2.9 Bentonite .............................................................................................................................................. 27
    3.2.10 Inert filler ............................................................................................................................................. 27
  3.3 Strength .......................................................................................................................................................... 28
  3.4 Atterberg limits ............................................................................................................................................... 29
  3.5 Pipe pressure loss ........................................................................................................................................... 31
3.6 Environmental considerations ................................................................................................. 33
4 Literature study .......................................................................................................................... 35
  4.1 Additives used .......................................................................................................................... 35
  4.2 Economic considerations ........................................................................................................ 35
  4.3 Fundamental behaviour ......................................................................................................... 37
  4.4 Strength .................................................................................................................................. 44
  4.5 Atterberg limits ...................................................................................................................... 54
  4.6 Hydraulic conductivity ........................................................................................................... 56
  4.7 Equipment .............................................................................................................................. 58
  4.8 Preliminary conclusions based on the literature study .......................................................... 63
5 Experimental study .................................................................................................................... 65
  5.1 Introduction ............................................................................................................................ 65
  5.2 Methodology ........................................................................................................................ 65
  5.3 Results ................................................................................................................................... 70
    5.3.1 Strength .............................................................................................................................. 70
    5.3.2 Atterberg limits .................................................................................................................. 75
6 Discussion .................................................................................................................................... 85
  6.1 The effect of cement on dredged material ........................................................................... 85
    6.1.1 Strength .............................................................................................................................. 85
    6.1.2 Atterberg limits .................................................................................................................. 87
  6.2 The effect of bentonite on dredged material ......................................................................... 92
    6.2.1 Atterberg limits .................................................................................................................. 92
    6.2.2 Strength .............................................................................................................................. 93
  6.3 The effect of combining cement and bentonite ................................................................. 94
    6.3.1 Atterberg limits .................................................................................................................. 94
    6.3.2 Strength .............................................................................................................................. 96
  6.4 The relationship between the undrained shear strength and the atterberg limits .............. 97
  6.5 Test methods used in the experimental study ..................................................................... 99
    6.5.1 Additives used .................................................................................................................... 99
    6.5.2 Fall cone test ...................................................................................................................... 100
    6.5.3 Unconfined compressive test ............................................................................................ 101
    6.5.4 Pipe pressure loss .............................................................................................................. 101
  6.6 General applicability of the method ..................................................................................... 103
7 Conclusions and recommendations ........................................................................................ 105
  7.1 Conclusions ........................................................................................................................... 105
FIGURES

Figure 1: the project site (Burgos, Samper, & Alonso, 2006) ................................................................. 7
Figure 2: land reclamation method at the Port of Valencia (Burgos, Samper, & Alonso, 2006) ..... 9
Figure 3: stabilization using in-situ deep mixing (Burgos, Samper, & Alonso, 2006) ......................... 10
Figure 4: development of the unconfined compressive strength of the dredged material with CEM II/B-V (Burgos, Samper, & Alonso, 2006) ......................................................................................... 11
Figure 5: location of the Central Japan Airport project (Kitazume & Satoh, 2003) ............................ 11
Figure 6: unconfined compressive strength on land (a) and under seawater (b) (Kitazume & Satoh, 2003) .................................................................................................................................................................. 12
Figure 7: schematic overview of the mixing and deposition system (Kitazume & Satoh, 2003). 13
Figure 8: detailed view of the mixing method (Kitazume & Satoh, 2003) ............................................... 13
Figure 9: overview of the Tokyo Airport, with runway D in the forefront (Watabe, Advanced Prediction Methods of Consolidation Settlement in Land Reclamation, 2015) ......................... 14
Figure 10: schematic overview of the area filled with cement treated dredged material (Watabe, Noguchi, & Mitarai, Use of Cement-Treated Lightweight Soils Made from Dredged Clay, 2012) 15
Figure 11: joint section of the pier and the reclamation section (Watabe, Noguchi, & Mitarai, Use of Cement-Treated Lightweight Soils Made from Dredged Clay, 2012) ........................................ 15
Figure 12: pneumatically mixed dredged material being deposited (a) and the location of this picture (b) (Watabe, Noguchi, & Mitarai, Use of Cement-Treated Lightweight Soils Made from Dredged Clay, 2012) ........................................................................................................................................ 16
Figure 13: the mass stabilization technique in Niinilahti bay (Havukainen, Piispanen, & Leppänen, 2011) .............................................................................................................................................................. 17
Figure 14: the unloading platform (Van Nederkassel, Van Zele, Van Renterghem, Vermeersch, & Quaeyhaegens, 2015) ......................................................................................................................... 19
Figure 15: the dyke under construction (Van Nederkassel, Van Zele, Van Renterghem, Vermeersch, & Quaeyhaegens, 2015) ................................................................................................................. 20
Figure 16: the relationship between the unconfined compressive strength and the undrained shear strength (Dominic Trani) ....................................................................................................................... 28
Figure 17: graph depicting that, to be able to move a Bingham plastic liquid, the yield stress must be overcome first .......................................................................................................................... 32
Figure 18: the waste hierarchy. Cold immobilisation is a recycling technique (Stuart Young) .... 34
Figure 19: conceptual model of the workings of the diffuse double layer (Winterwerp, 2014) .. 38
Figure 20: the difference between the water in an untreated and a treated clay slurry (Zhu, Zhang, & Chiu, 2007) ........................................................................................................................................ 40
Figure 21: the increase and decrease in different water contents versus the cement content (Zhu, Zhang, & Chiu, 2007) ......................................................................................................................... 40
Figure 22: the water retention curve for untreated and cement-treated soil (Zhu & Chiu, 2009) ................................................................................................................................. 42
Figure 23: the cement content versus the Atterberg limits (Chew, Kamruzzaman, & Lee, 2004) 43
Figure 24: the cement content versus the pH (Chew, Kamruzzaman, & Lee, 2004) ......................... 44
Figure 25: the lime content versus the unconfined compressive strength for lime stabilised clay after data from Bell (1996) .................................................................................................................. 45
Figure 26: the cement content versus undrained shear strength for the Bergen samples, after Sparrevik & Kvennås (2008) .................................................................................................................. 46
Figure 27: the cement content versus undrained shear strength for the Bergen samples, after Sparrevik & Kvennås (2008) .................................................................................................................. 46
Figure 28: the cement content versus undrained shear strength for the Bergen samples, after Sparrevik & Kvennås (2008) ............................................................................................................................................................................................... 47
Figure 29: errors when using the Horpibulsuk formula on the Sparrevik & Kvennås data......................................................................................................................... 49
Figure 30: the water-cement ratio versus undrained shear strength for the Bergen samples, after Sparrevik & Kvennås (2008) ............................................................................................................................................................................................... 50
Figure 31: the water-cement ratio versus undrained shear strength for the Grenland samples, after Sparrevik & Kvennås (2008) ............................................................................................................................................................................................... 50
Figure 32: the water-cement ratio versus undrained shear strength for the Sandvika samples, after Sparrevik & Kvennås (2008) ............................................................................................................................................................................................... 50
Figure 33: the failure envelope of a dredged material treated with 100 kg/m³ cement after 7 days (Chiu, Zhu, & Zhang, 2008) ............................................................................................................................................................................................... 51
Figure 34: the effective pressure versus the void ratio, showing that the cement treated samples possess something similar to a pre-consolidation pressure (Chiu, Zhu, & Zhang, 2008) ............................................................................................................................................................................................... 52
Figure 35: the normalised change in bound water content correlated to the yield strength for three different dredged materials (Chiu, Zhu, & Zhang, 2008) ............................................................................................................................................................................................... 53
Figure 36: the normalised change in bound water content correlated to the cohesion for three different dredged materials ............................................................................................................................................................................................... 54
Figure 37: the plastic limit development for three different cement contents, after Federico et al. (2015) ............................................................................................................................................................................................... 55
Figure 38: the liquid limit development for three different cement contents, after Federico et al. (2015) ............................................................................................................................................................................................... 55
Figure 39: the plastic limit development for three different lime contents, after Federico et al. (2015) ............................................................................................................................................................................................... 56
Figure 40: the liquid limit development for three different lime contents, after Federico et al. (2015) ............................................................................................................................................................................................... 56
Figure 41: the cement content versus the hydraulic conductivity of the Bergen samples, after Sparrevik & Kvennås (2008) ............................................................................................................................................................................................... 57
Figure 42: the cement content versus the hydraulic conductivity of the Grenland samples, after Sparrevik & Kvennås (2008) ............................................................................................................................................................................................... 58
Figure 43: the cement content versus the hydraulic conductivity of the Sandvika samples, after Sparrevik & Kvennås (2008) ............................................................................................................................................................................................... 58
Figure 44: the pneumatic flow principle (Kitazume & Satoh, 2003) ............................................................................................................................................................................................... 59
Figure 45: the mixing ratio versus unit air speed diagram (Kitazume & Satoh, 2003) ............................................................................................................................................................................................... 60
Figure 46: the air pressure versus transport distance (Kitazume & Satoh, 2003) ............................................................................................................................................................................................... 61
Figure 47: the set of barges that makes up the pneumatic flow mixing system (Kitazume & Satoh, 2003) ............................................................................................................................................................................................... 62
Figure 48: the Casagrande chart of the VE-KA, MM2 and MM5 soils............................................................................................................................................................................................... 66
Figure 49: the grain size distribution of the used clays............................................................................................................................................................................................... 67
Figure 50: the undrained shear strength of the cement stabilised samples versus the cement content ............................................................................................................................................................................................... 72
Figure 51: the undrained shear strength of cement stabilised samples versus the water-cement ratio ............................................................................................................................................................................................... 73
Figure 52: the undrained shear strength of bentonite stabilised samples versus the bentonite content ............................................................................................................................................................................................... 74
Figure 53: the undrained shear strength of cement-bentonite stabilised samples versus the cement content ............................................................................................................................................................................................... 75
TABLES

Table 1: the characteristics of the untreated mud (Burgos, Samper, & Alonso, 2006) ....................... 8
Table 2: the constituents of Portland cement........................................................................................................ 22
Table 3: the typical constituents of GGBS........................................................................................................ 23
Table 4: the typical constituents of fly ash....................................................................................................... 24
Table 5: the typical constituents of cement kiln dust (Adaska & Taubert, 2008) .............................. 25
Table 6: reference projects with additives, cost, method and target strengths .................................. 36
Table 7: approximate raw material prices for additives................................................................................. 37
Table 8: the Atterberg limits for all materials used ...................................................................................... 66
Table 9: the determined viscosities using the FANN viscometer............................................................... 67
Table 10: the pipe pressure losses at different diameters and velocities..................................................... 68
Table 11: the target additions for the tests on the different clays ................................................................ 69
Table 12: the cement contents, bentonite contents and Atterberg limits for curing times of 7 and 28 days for the VE-KA samples................................................................................................. 81
Table 13: the cement contents, bentonite contents and Atterberg limits for curing times of 7 and 28 days for the MM2 samples........................................................................................................ 82
Table 14: the cement contents, bentonite contents and Atterberg limits for curing times of 7 and 28 days for the MM5 samples........................................................................................................ 82
Table 15: a summary of the $\alpha$ factors, $c_u(0)$ and $R^2$ for the trend lines shown in Figure 63 and Figure 64........................................................................................................................................................................... 87
Table 16: the values for constants in the equations (30) and (31) for the different clays, as well as $R^2$ for equation (31), using the constants found......................................................................................... 90
Table 17: the difference between the superposed change in liquid limit and actual liquid limit 95
Table 18: the difference in liquid limits between a curing time of 7 and 28 days at cement and bentonite contents of approximately 18% .................................................................................. 95
Table 19: the results from phase 2 of the project by Sparrevik & Kvennås (2008), combined with the formula of Horpibulsuk et al. (2003) ......................................................................................... 114
Table 20: results of tests on lightweight material treated soils (Tsuchida, Porbaha, & Yamane, 2001) ........................................................................................................................................................................ 114
Table 21: the data describing the material properties of the soils in the vibrocores........................... 117
Table 22: the data with regard to MM2 and MM5, as extracted from the datasheet provided........ 118
1 INTRODUCTION

1.1 BACKGROUND

The aim of this thesis project is to find a procedure in which dredged material is employed to create a land reclamation site that can be used as a container or bulk terminal, using additives to make the dredged material suitable for that purpose. Applications of methods using dredged material in land reclamation sites show that using these methods is only economically viable when other materials, such as sand, are not available in the vicinity of the land reclamation site. When sand is available but dredged material from nearby sites cannot be disposed of elsewhere, due to contaminations in the sludge or regulations that forbid the deposition elsewhere, using stabilised dredged material in land reclamation sites can be a good and economical alternative to storing it elsewhere. Factors that influence such a decision are disposal fees, the available sites for disposal, the transport distance and greenhouse gas emissions.

Dredged material is not a standardised material, since its properties differ, depending mostly on the location of origin of the sludge. However, dredged material in general has some properties that can be found in the majority of cases. It usually has a low undrained shear strength, high water content, high liquid limit, high clay content, a low sand content and a high compressibility. These characteristics are issues the use of additives and ground improvement techniques have to overcome.

As dredged material often originates from busy waterways and ports, often including bulk terminals, oil and gas terminals and refineries, the sludge can be contaminated with hydrocarbons or heavy metals. Deposition in a land reclamation site can be a proper way to immobilise these contaminations. Tribulytin contaminated dredged material was converted into a container terminal at Vuosaari harbour recently (Havukainen, Piispanen, & Leppänen, 2011).

In certain areas of the world the material of choice for land reclamations, sand, is scarcely available or not available at all. Cases have been reported where layers of sand and layers of clay are placed alternately (Lee, Karunaratne, Yong, & Ganeshan, 1987) in order to save costs. In places where sand is absent, rocks offer an alternative to sand. Usually this material cannot be found near the shoreline and it thus has to come from the hinterland, which can be quite expensive. In such places using dredged material with additives can be a cost saving solution when used together with mountainous material as a base layer (Kitazume & Satoh, 2003).
Increasing environmental awareness globally increases the demand for immobilising contaminants found in dredged material. In first and second world countries the popular resistance against waste heaps and depots is growing as well, making reuse of waste materials a more acceptable alternative. Otherwise unusable dredged material depots can, using this technique, be used as an area with a useful purpose for society.

1.2 Problem Definition

Although dredged material, including the contaminated variant, can be made useable for reclaiming land, there are several difficulties to overcome. It usually has a high water content, above the liquid limit of the material (Federico, Vitone, & Murianni, 2015) and a very low undrained shear stress (Sparrevik & Kvennås, 2008). Other than that, it differs from site to site what the actual constituents and thus properties of the dredged material are. It is a mixture of clay, silt, sand, organic material and possibly contaminants, organisms and gas (Winterwerp & Van Kesteren, 2004). The behaviour of dredged material is dominated by clay and usually a large portion of a dredged material is clay. The amounts of other constituents present varies largely.

These differing characteristics have caused researchers to be descriptive rather than predictive with regard to the geotechnical properties of dredged material. Various cases are discussed, of which a selection can be found in chapter 2. Projects mainly rely on intensive field investigations and the use of best practices. Experience with practices involving in-situ mixing using lime and cement is mainly gained through clay stabilisation projects in the Nordic countries and Japan (Teunissen, 2005). There has, however, been one novel method created in Japan that uses pneumatic mixing with additives (Kitazume & Satoh, 2003), which is described further in chapter 2 and chapter 3. This technique is in normal operation only capable of processing 300 m$^3$ of dredged material per hour. A rate of 2000 m$^3$ per hour is reached in common dredging operations.

Self-weight consolidation of dredged clays is a very slow process (Hendriks, 2016). Clay particles tend to form flocs, which capture large amounts of water. Due to these flocs and the fact that the pores are small, the permeability is very low as well. The natural reduction of excess pore pressures is therefore a slow process. Because of the flocculation process, the clay particles can show relatively large decreases in volume when loaded. The practical end of self-weight consolidation results in a very weak slurry, with undrained shear strengths well below 1 kPa.

The use of drainage and preloads can help to enhance the process of consolidation. Preloads increase the pressure in the pores, causing the water to flow out into drains, which are highly
permeable and can transport water at high rates. The amount of preload and drains has to be determined per project site.

To install these drains or a preload, the surface of the land reclamation site must be suitable for these kinds of operations. A crust must be formed to allow for these loads to be imposed on the surface. Dredged material cannot sustain such loads by itself, as self-weight consolidation and natural crust formation do not dewater the material enough to gain sufficient strength. This is where the additives and knowledge of their behaviour are imperative.

Summarising, the main problems concerning the use of dredged material in a land reclamation site are threefold. Firstly, because of the very slow self-weight consolidation, the high water content of the material poses a problem. Secondly, the low undrained shear strength of the material causes it to be useless for industrial purposes when no further action is taken. Thirdly, the rates at which stabilised dredged material can be deposited are quite low when compared to common dredging disposal rates.

1.3 Research Objective

As mentioned in the previous paragraph, the past investigations regarding additives to stabilise dredged material have focussed on describing the occurring changes in strength and water content. There have been little investigations into the theoretical background of these changes. A deeper understanding of the fundamental behaviour of dredged material combined with these additives is therefore one of the objectives of this research.

Higher rates can be achieved by finding a balance between dredged material properties and the properties and quantity of additives. Very large, costly materiel is involved in these kind of operations. Cutting hours by multiplying the rates at which the stabilised dredged material can be deposited can save costs, even if that means increasing the quantity of additives. Therefore, an objective of this research is to find a balance between the properties of dredged material and those of additives such that they can be processed at a high rate, being approximately 2000 m$^3$ per hour.

In the two objectives mentioned above, the objective of adding the additives to the dredged material must be kept in mind. This objective is increasing the undrained shear strength and lowering the water content in such a manner, that ground improvement material can work on the improved material. Throughout this research, this is the overriding objective. Ground improvement materiel can exert a pressure of 75 kPa. This is equal to the pressure a large vertical drainage installing machine exerts. For safety reasons, an unconfined compressive strength is assumed of 150 kPa, which is in line with the available literature (Burgos, Samper, &
Alonso, 2006). A safety factor of at least 2 should be used as well, when comparing the strength of samples thoroughly mixed in a laboratory and expected field strength. When other safety factors are considered as well, a total safety factor larger than 3 is to be used. (Watabe, Noguchi, & Mitarai, Use of Cement-Treated Lightweight Soils Made from Dredged Clay, 2012).

1.4 RESEARCH QUESTIONS

Based on the research objectives stated in the previous paragraph, the main research question is:

*How can additives be used to increase the strength of dredged material, such that the deposition rate is high and the unconfined compressive strength is at least 150 kPa in the field?*

This main research question can be divided into the following sub-questions:

- What kind of additives are available on the market at an economically acceptable price?
- What is the influence of these additives on the strength parameters, water content and Atterberg limits of these dredged materials?
- How can these effects be explained?
- What is the required composition of dredged material such that the material can be deposited at a high rate?

1.5 METHODOLOGY

To investigate the behaviour of dredged material and additives, the research is divided into a desk study and an experimental study. The focus of the desk study lies with literature describing research conducted in the past and the gathering of data. Using the results gained from the literature study, an experimental study is set up. Additionally, reference projects will be investigated in order to give a full picture of the current applications of additives to dredged material.

Desk study:

- Literature review on what additives are available and what their effect is on dredged material and clays;
- Literature review on the correlation between the undrained shear strength and the Atterberg limits;
- Data gathering on the effect additives have on dredged material with regard to strength parameters and the Atterberg limits.
The experimental study is set up to link the undrained shear strength with the additive content and the Atterberg limits.

Experimental study:

- Perform tests on undisturbed samples with varying additive content to determine their undrained shear strength, whilst determining the water content;
- Determine the liquid limit and plastic limit of the samples, whilst determining the remoulded undrained shear strength;
- Derive parameters of established formulae.
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2 REFERENCE PROJECTS

2.1 SUMMARY

<table>
<thead>
<tr>
<th>Project</th>
<th>Additives</th>
<th>Mixing method</th>
<th>Desired end strength [kPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Port of Valencia</td>
<td>CEM II/B-V 42.5 R</td>
<td>In-situ</td>
<td>150</td>
</tr>
<tr>
<td>Central Japan Airport</td>
<td>Cement</td>
<td>Pneumatic flow mixing</td>
<td>157</td>
</tr>
<tr>
<td>Tokyo International Airport Runway D</td>
<td>Cement, air foam</td>
<td>Pneumatic flow mixing</td>
<td>360</td>
</tr>
<tr>
<td>Vuosaari Harbour</td>
<td>CEM II/A-M (S-LL)</td>
<td>In-situ</td>
<td>70</td>
</tr>
<tr>
<td>Vlassenbroek</td>
<td>Cement, fly ash</td>
<td>Ex-situ</td>
<td>35</td>
</tr>
</tbody>
</table>

2.2 PORT OF VALENCIA

Land reclamation has been used at the Port of Valencia to build a container terminal at the south side of the port. When building the container terminal, a lagoon was created with hydraulic filling. The lagoon, depicted in Figure 1 in its state before soil improvement, was left in place until expansion of the port was deemed necessary (Burgos, Samper, & Alonso, 2006). Mud in the lagoon has a low plastic limit, liquid limit and a water content around the liquid limit. It can be classified as either clayey silt or silty clay, with a low amount of sand and organic matter. The undrained shear strength of the material is low and ranges from 3 to 25 kPa, implying that the soil is in its plastic state. The characteristics can be seen in Table 1.

Figure 1: the project site (Burgos, Samper, & Alonso, 2006)
<table>
<thead>
<tr>
<th>Characteristics</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Organic matter [%]</td>
<td>1 – 2</td>
</tr>
<tr>
<td>Calcium content [%]</td>
<td>15 – 18</td>
</tr>
<tr>
<td>Sand (&lt;63 μm) [%]</td>
<td>&lt;10</td>
</tr>
<tr>
<td>Silt (&gt;2 μm, &lt;63 μm) [%]</td>
<td>40 – 60</td>
</tr>
<tr>
<td>Clay (&lt;2 μm) [%]</td>
<td>40 – 60</td>
</tr>
<tr>
<td>Liquid limit [%]</td>
<td>20 – 45</td>
</tr>
<tr>
<td>Plasticity index [%]</td>
<td>5 – 25</td>
</tr>
<tr>
<td>Water content [%]</td>
<td>30 – 60</td>
</tr>
<tr>
<td>Dry unit density [kN/m³]</td>
<td>12 – 14</td>
</tr>
<tr>
<td>Void ratio [-]</td>
<td>0.9 – 1.3</td>
</tr>
<tr>
<td>Compression index [Cₗ]</td>
<td>0.20 – 0.25</td>
</tr>
<tr>
<td>Vertical consolidation coefficient (Cᵥ) [cm²/s]</td>
<td>4 * 10⁻⁴</td>
</tr>
<tr>
<td>Coefficient of horizontal consolidation (Cₕ) [cm²/s]</td>
<td>8 * 10⁻⁴</td>
</tr>
<tr>
<td>Undrained shear strength (cₜ) [kPa]</td>
<td>3 – 25</td>
</tr>
</tbody>
</table>

Table 1: The characteristics of the untreated mud (Burgos, Samper, & Alonso, 2006)

A crust of approximately 0.5 metres had dried before the project commenced, making the surface suitable to work on. The work order can be seen in Figure 2. First, the mud is stabilised, using in-situ deep mixing, in which cement is mixed with the mud. Calculations using Plaxis, a finite element method computer program and the classical Morgenstern-Price and Bishop methods, have shown that a crust of four metres is sufficient, giving a safety factor of at least 2.4. The design equivalent field unconfined compressive strength was set at 150 kPa, which is a 2.5 times as much as the equivalent load the layer will have to bear, which is 60 kPa according to the Spanish standard ROM 4.1.-94. For mud that has to be driven on by machinery within three days, 110 kg/m³ cement was used. Mud that had to be driven on after seven days was treated with 90 kg/m³ cement.

After stabilisation and curing of the cement, vertical drains were installed. These were installed every two square metres to a depth of 15 metres. As the mud below the stabilised layer still has a high water content and the water cannot drain via another route, the drains are the only way the water can be removed once a load is applied during the consolidation phase. On top of the stabilised layer a drainage blanket, consisting of 0.5 metres of gravel and geotextile sheets on the top and bottom of this gravel layer, is applied, as well as drainage ditches. From these ditches the water is directed to wells, from where it is pumped into the sea.

When the drainage system is installed, a preload of 9.5 metres (195 kPa) is applied, which constitutes a 30% greater load than the service load of 150 kPa. The materials used as a preload
are from excavations carried out in the city of Valencia, but could in any given project also be ordinary sand or construction waste. The preload was applied for eleven months, after which the overburden was removed. The settlements inside the lagoon area were approximately 120 – 160 centimetres, whereas the settlements near the embankments were approximately 60 – 80 centimetres. The total amount of water pumped out during the consolidation phase was approximately 33,000 m$^3$, which constitutes a loss of volume of the mud, corresponding to a settlement of 4 mm/day. These values have been measured during consolidation in the field as well.

![Diagram](image)

**Figure 2: land reclamation method at the Port of Valencia (Burgos, Samper, & Alonso, 2006)**

The work method used is in-situ deep mixing. A mixer can mix a cell of 4 metres deep, 4.5 metres long and 3.2 – 3.8 metres wide at one time, which takes approximately 60 to 90 minutes per cell. This roughly corresponds to a progression rate of 50 to 70 m$^3$ per hour. Mixing was
performed by multiple mixers, but this method has a rate an order of magnitude smaller than the rate proposed in the requirements of this study. The work method is depicted in Figure 3.

![Figure 3: stabilization using in-situ deep mixing (Burgos, Samper, & Alonso, 2006)](image)

A binder was chosen out of three options:

- CEM I 42.5 R SR cement, which is mainly Portland cement
- CEM II/B-V 42.5 R cement, which is Portland cement with 21% - 35% fly ash
- CEM I 42.5 R with 40% fly ash added to it

All types of binder were tested under lab conditions. The main selection criterion was the unconfined compressive strength. The first and second binder showed approximately the same strength after 90 days, with the first binder showing a faster reaction time than the second. The third binder had a significantly lower end strength after 90 days and was therefore discarded. The second binder was chosen due to economic reasons. The laboratory behaviour of the samples is depicted in Figure 4. What can be observed is that the 100 kg/m³ sample reaches 450 kPa after seven days. This value was taken as a design standard, as the EuroSoilStab (2000) standard dictates a factor three difference between laboratory strength and equivalent field strength for stabilised soils.
Figure 4: development of the unconfined compressive strength of the dredged material with CEM II/B-V
(Burgos, Samper, & Alonso, 2006)

2.3 CENTRAL JAPAN AIRPORT

The Central Japan Airport was constructed in the Ise Bay near Tokoname, as Nagoya Airport, the airport serving the region, became too small. Since there was no suitable land available, it was decided to build a near shore artificial island on which to build the new airport, a method that has also been deployed in Tokyo and Hong Kong. The area of the new airport comprises 580 ha and is built 2 – 3 kilometres off-shore. Its location can be seen in Figure 5. The amount of soil needed to build the artificial island is approximately $7 \times 10^7 \text{ m}^3$, but due to the unavailability of sand in the area, on-shore as well as off-shore, the options were either to use dredged material or haul material from mountainous areas in the hinterland. For the Central Japan Airport, it was decided to partly use dredged material from the Nagoya port, using a method called the pneumatic flow mixing method. The majority of the materials used is still rock from mountainous areas, which constitutes $5.8 \times 10^7 \text{ m}^3$ of material. The dredged material has a volume of $1.2 \times 10^7 \text{ m}^3$ (Kitazume & Satoh, 2003).

Figure 5: location of the Central Japan Airport project (Kitazume & Satoh, 2003)
Combining the requirements for the land reclamation site, the target average field strength was set at 157 kPa. In this project it was decided to set the target laboratory strength to twice the value of the target average field strength, being 314 kPa. Note that this is different from the project at the Port of Valencia, where the laboratory strength was set at three times the equivalent field strength. The origin of the dredged material differs, as it is dredged for ten different sites in the Nagoya port. Material from every site was tested and for all the optimal mixture was determined.

Figure 6 shows that as long as the cement content is higher than approximately 50 kg/m$^3$, adding cement has an effect. Near 60 kg/m$^3$ the target laboratory strength has been reached. For dredged material from most sites a cement content of 53 to 57 kg/m$^3$ has been used, but for one site, where 87 kg/m$^3$ had to be used.

![Figure 6: unconfined compressive strength on land (a) and under seawater (b) (Kitazume & Satoh, 2003)](image)

Depositing the dredged material is done using the pneumatic flow mixing method, from which a schematic overview can be found in Figure 7. A barge with dredged material from one of the ten sites is docked at a pneumatic barge, where the material is loaded in a hopper. From there it is pumped hydraulically into a pipeline, where compressed air is added. At the cement supply barge the cement is added to the plugs formed, from where a pneumatic pipeline takes it to the placement barge. At the placement barge a cyclone is used to release the air pressure and places the mixture in the reclamation site by gravity. A detailed drawing of the working principle can be seen in Figure 8. Note that plug flow is a prerequisite for this method to work, as is explained in 4.7. The maximum capacity of one such system is 1000 m$^3$/hour. In this project three of these machines were used with a combined average capacity of 25,000 m$^3$/day.
The Tokyo International Airport or Haneda Airport has been evolving over time. Before an offshore expansion project, the site at which the airport is situated now partially was a disposal site for dredged material. Airport extension was done on top of the reclamation site, thus making room for Runway C. Plans for another runway, Runway D, commenced in the early 21st century, with the works on the extension starting in 2007 and lasting until 2010. The new runway was partially built as a piled pier and partially as a land reclamation, which is shown in the aerial photograph in Figure 9. The pier section was developed to ensure a stable outflow of the Tama River. The site needed to be 17.1 metres above sea level, because airplanes need to land as ships pass by. Sea depth at the site is approximately 20 metres, making the reclamation site 37.1 metres high (Watabe, Noguchi, & Mitarai, Use of Cement-Treated Lightweight Soils Made from Dredged Clay, 2012).
Embankments were created on all sides (see Figure 10), except for the side of the piled pier, where a sheet pile wall was used (see Figure 11). Sand compaction piles were used at the embankment site to increase the consolidation rate of the clay layers at the sea bottom. In this project, an unconfined compressive strength of 300 kPa was required. A safety factor of 1.2 was applied for the target field strength, setting the target at 360 kPa. After correction for a coefficient of variation and a defective percentage of 25%, the mean value of an assumed normal distribution was set to be 471 kPa, being the field strength finally used to design the reclamation site. The laboratory strength was to be, as was in the Central Japan Airport project, twice as high as the target equivalent field strength and thus 942 kPa. As there was a high construction time, the curing time used to calculate the properties of the soil was not 28 days, but 91 days.
The dredged material was only used to heighten the land reclamation site to 2.5 metres. The pneumatic mixing method as was discussed before in 2.3 was used for Runway D as well. The execution of this method can be seen in Figure 12. After the mixture was applied, air foam treated lightweight soil was applied at the side of the piled pier. An alternative to using this method is applying ordinary sand, but the lateral earth pressures induced by sand would be too high for the sheet pile wall to hold and therefore lightweight soil was used.
The cement admixed soils had cement contents ranging from 78 kg/m$^3$ to 109 kg/m$^3$. This is considerably higher than the cement contents at the Central Japan Airport project, where the laboratory strength was almost three times lower. However, the cement content is not three times higher at this particular project, implying that cement content and unconfined compressive strength do not have a linear relationship. This will be shown later on in 4.4, when data is presented. The average field strength reached by this method is approximately 750 kPa.

The air foam treated lightweight soil is designed to be as lightweight as possible. A practically reachable weight is 8 kN/m$^3$, but to ensure durability the bulk density levels were set to be between 10 and 11.5 kN/m$^3$. The design unconfined compressive strength here was 200 kPa, with a target equivalent laboratory strength of 440 kPa, meaning a safety factor of 2.2. Cement contents varied from 68 kg/m$^3$ to 103 kg/m$^3$ and volumetric percentages of air foam varied between 7.6% and 20.5%, reaching the design strength of 200 kPa in most cases, with the minimum UCS around 100 kPa and the maximum UCS around 1000 kPa.

Figure 12: pneumatically mixed dredged material being deposited (a) and the location of this picture (b) (Watabe, Noguchi, & Mitarai, Use of Cement-Treated Lightweight Soils Made from Dredged Clay, 2012)
2.5 Vuosaari Harbour

The new Vuosaari Harbour was built to remove cargo terminals from a location closer to the Helsinki city centre. There was no room for expansion at those sites and the presence of these terminals caused heavy traffic in some areas. At the new site in Vuosaari, east of Helsinki, elevated levels of tribultytin were found in the clay from the seabed of the proposed port area. Tribultytin or TBT is a hazardous compound that was used in antifouling paints. Formerly a shipyard was located at the site, which used a large amount of this type of paint to paint its ships. Terramare, a Boskalis subsidiary, built the quay walls, the four jetties and did most of the dredging work involved in this project.

Before building commenced, an environmentally sound concept had to be developed for the disposal of the TBT-contaminated sediments. The total area contaminated was found to be 75 hectares. This area was isolated from the sea by embankments and a silt curtain. The deposition area was determined to be the Niinilahti bay, a site that needed to be filled as part of the harbour construction. The Niinilahti bay was first filled with sand to an elevation of -4 metres. Approximately 0.45 million m$^3$ of TBT-contaminated dredged material was disposed in the Niinilahhti bay. The dredged material was subsequently stabilised, using 130 kg/m$^3$ cement using the mass stabilisation in-situ technique, which can be seen in Figure 13 (Havukainen, Piispanen, & Leppänen, 2011).

![Figure 13: the mass stabilisation technique in Niinilahti bay (Havukainen, Piispanen, & Leppänen, 2011)](image)

The cement used to stabilise the dredged material is CEM II/A-M (S-LL), a cement that contains 80% or more Portland cement, GGBS and lime. The required unconfined compressive strength was set at 200 kPa in the laboratory, which was estimated to be 70 kPa in the field. The large
difference can be explained by homogeneity issues when mixing in-situ. This field to laboratory strength ratio is comparable with the project in the Port of Valencia, described in 2.2. After performing the tests, it is noted by Havukainen et al. that the compressive strength of the material is a function of the water content and the amount of cement used.

The contaminated dredged material was deposited in the Niinilahti bay in a layer of approximately 5 metres on the sand layer. After in-situ stabilisation the layer was covered with a filter fabric and a preload embankment of 1 metre. After 90 days of preloading the preload was enlarged to increase the rate of consolidation. The preload was removed after 6 months, but in some cases only after 14 months. The last preload was removed 1.5 years after the project had commenced. It was required that the hydraulic conductivity of the layer was less than 5 * 10^{-9} m/s, due to the contaminants present in the dredged material. This requirement has been met. A \( c_v \) of 5 * 10^{-7} m^2/s was approximately reached during the consolidation phase.

### 2.6 Vlassenbroek

Vlassenbroek is a polder in Flanders, Belgium near Dendermonde and next to the river Scheldt. As part of a flood protection programme, a flood protection dyke was to be constructed in the polder. The polder would partially become a wetland and partially an overflow area. The dyke was designed to separate these two areas. Waterwegen en Zeekanaal, the Flemish execution institute for the waterways in Flanders, initiated this project as a test case to see whether or not it was possible to build a dyke out of dredged material. The contract was eventually won by Envisan, a subsidiary of the Jan De Nul Group (Van Nederkassel, Van Zele, Van Renterghem, Vermeersch, & Quaeyhaegens, 2015).

The project was executed as a continuous process, aligned with maintenance dredging of the river Scheldt. This means that dredged material was directly used as a construction material for the Vlassenbroek polder dyke. The additive used in this case was cement with fly ash. Dredged material and the cement were mixed ex-situ on the river Scheldt in a barge. The mixture was from there driven to the construction site.

Dredged material from the maintenance works in the river Scheldt arrived at the construction site of the dyke in a barge. The barges were moored to an unloading platform on which a long reach transhipment crane was mounted, which can be seen in Figure 14. The crane unloaded the dredged material from the barge into a screen, which was used to filter large objects from the sludge. From there it was pumped to a mixing station on land, next to the construction site. After mixing, the mixture was transported by conveyor belt into a hopper, from where it was loaded into dump trucks. The mixture was dumped at the place where construction took place at that point in time, where backhoes put it in place. The dyke was constructed in layers,
allowing for drying and curing time of the mixture. The construction process can be seen in Figure 15.

The strength requirements were considerably less than for the other projects. The required undrained shear strength was set to 35 kPa. The main difference between this project and the others is that the dyke does not need to bear any permanent loads induced on the surface. Vane tests showed that the undrained shear strength of the material at the surface was 76 ± 29 kPa and 55 ± 20 kPa at the base of the dyke. Consolidated undrained triaxial tests showed that the cohesion varied between 8 and 30 kPa. The angle of friction varied between 34 and 40 degrees. Cone penetration tests conducted at the site showed that over the height of the dyke the requirements were met.

Figure 14: the unloading platform (Van Nederkassel, Van Zele, Van Renterghem, Vermeersch, & Quaeyhaegens, 2015)
Figure 15: the dyke under construction (Van Nederkassel, Van Zele, Van Renterghem, Vermeersch, & Quaeyhaegens, 2015)
3 THEORETICAL BACKGROUND

3.1 DREDGED MATERIAL BEHAVIOUR

Dredged material is all material that is or can be dredged. This loose definition includes all kinds of materials not included in this study, such as sand and gravel. The material that is actually dealt with in this study is the dredged material that is removed from rivers, canals, ports, harbours and such in order to make these waterways navigable. Even when just considering this type of material, the properties of two types of dredged material from two different sites can differ greatly.

All dredged material that is considered in this study is cohesive sediment. This is also simply called mud. Its formal definition is as follows:

*Mud is a mixture of clay, silt, sand, organic material, water and sometimes gas. Besides, contaminants and organisms may also be present in mud* (Winterwerp & Van Kesteren, 2004).

Gravel is not present in mud. Very coarse particles can be present, but are often found to be debris or inappropriately disposed waste. Mud can contain a rather large sand fraction, but it is established by Winterwerp & Van Kesteren (2004) that the clay portion governs the physical properties of the mud. However, the other constituents do matter and can alter these properties.

The mineral composition of the cohesive sediments varies greatly and has influence on the behaviour of the dredged material. As the sediments are deposited by rivers and other waterways, the mineral composition of the material depends on the mineralogy of the hinterland. The difference in mineralogy also applies to the sand and silt fractions. Organic matter comes from various sources as well and can have a significant contribution to the behaviour of the dredged material. Organic matter does not seem to influence the behaviour of the material when lower than 3% - 4% (Federico, Vitone, & Murianni, 2015).

Taking the above into account, it is fair to say that “mud” is not so much a defined material, but more a category of materials. There are however some properties that mud usually has, such as a high water content, a high liquidity index and a low undrained shear strength when compared to other soils. Important properties of mud are the Atterberg limits, which are discussed in 3.4. The liquid limit and plastic limit of dredged material can vary greatly from site to site. In stabilising dredged material, these limits can have a large effect on the methods and additives that need to be used. Dredged material is, during dredging or right after dredging, always in its liquid state. After stabilisation it should be at least in its plastic state. It is therefore necessary to
know with how much the liquid limit should be raised or the water content lowered before this state is reached.

3.2 ADDITIVES

3.2.1 PORTLAND CEMENT

Portland cement has been widely used as an additive in soil stabilisation works since the 1960’s, especially in Japan and Sweden (Teunissen, 2005). It is the binding constituent of concrete, where it is used together with water, sand and gravel, and mortar, where only water and sand are used. Cement is made from clinker, a product made from limestone and clay. The first provides calcium carbonate, whereas the latter is a source of silica and aluminium. Other sources such as shale, sand, fly ash, iron ore, bauxite or slag can be used, depending on the availability at the geographical location of the cement kiln. The mixture is calcined first at a temperature of 600 °C, after which it is sintered at a temperature of 1,450 °C. The main constituents of clinker are tricalcium silicate, dicalcium silicate, tricalcium aluminate, tetracalcium aluminoferite and gypsum. The latter is added after the sintering process and is used to obtain the desired setting qualities of the material. The Portland clinker is ground, obtaining the final product, being Portland cement.

<table>
<thead>
<tr>
<th>Constituents</th>
<th>Chemical formula</th>
<th>% by weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tricalcium silicate</td>
<td>(CaO)(_3) · SiO(_2)</td>
<td>45% - 75%</td>
</tr>
<tr>
<td>Dicalcium silicate</td>
<td>(CaO)(_2) · SiO(_2)</td>
<td>7% - 32%</td>
</tr>
<tr>
<td>Tricalcium aluminate</td>
<td>(CaO)(_3) · Al(_2)O(_3)</td>
<td>0% - 13%</td>
</tr>
<tr>
<td>Tetracalcium aluminoferite</td>
<td>(CaO)(_4) · Al(_2)O(_3) · Fe(_2)O(_3)</td>
<td>0% - 18%</td>
</tr>
<tr>
<td>Gypsum</td>
<td>CaSO(_4) · 2H(_2)O</td>
<td>2% - 10%</td>
</tr>
</tbody>
</table>

Table 2: the constituents of Portland cement

Due to its wide availability and known characteristics it has been widely used in soil stabilisation. Recent applications in large land reclamation sites include Runway D of the Tokyo Airport (Watabe, Advanced Prediction Methods of Consolidation Settlement in Land Reclamation, 2015), a terminal extension in the Port of Valencia (Burgos, Samper, & Alonso, 2006) and the Central Japan International Airport (Kitazume & Satoh, 2003). The data shows that Portland cement is the most effective additive in terms of strength parameters improvement, which will be discussed in a separate paragraph. The main advantage of Portland cement is, that next to being water consuming, it also glues the particles together, as it does in concrete. Furthermore, the cement forms a matrix in which it can trap water and contaminants.
3.2.2 Ground granulated blast furnace slag

Ground granulated blast furnace slag or GGBS is a by-product of steelmaking via the blast furnace process. It is commonly used as a partial replacement for Portland cement in cement. Up to 90% of Portland cement can be replaced by GGBS (CEM III/C), which is slightly less expensive than Portland cement. Slag floats on top of the iron in a blast furnace and is diverted, after which it is cooled down rapidly to stop crystallisation processes, which would render it useless. It is granulated afterwards, creating a hydraulically active mixture, mainly consisting out of quick lime, silicon dioxide, aluminium oxide and magnesium oxide, as can be seen in Table 3. The reaction time of GGBS and thus cement with GGBS is slower than that of Portland cement, which increases its workability. Adding GGBS to the mixture usually results in a denser slurry.

<table>
<thead>
<tr>
<th>Constituent</th>
<th>% by weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>CaO</td>
<td>30% - 50%</td>
</tr>
<tr>
<td>SiO₂</td>
<td>28% - 38%</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>8% - 24%</td>
</tr>
<tr>
<td>MgO</td>
<td>1% - 18%</td>
</tr>
</tbody>
</table>

Table 3: the typical constituents of GGBS

GGBS is available in areas with blast furnace operations and therefore the inexpensive availability of GGBS is limited to these geographical locations. Its properties are well known and its cementing and water consuming nature could contribute to stabilising dredged material. There are, however, a few amount of cases reported to have used GGBS or blast furnace cement (Federico, Vitone, & Murianni, 2015). The available literature is limited to reports of lab tests using GGBS to stabilise clay or dredged material, despite its favourable properties.

3.2.3 Quicklime

Quicklime, which is the trivial name of calcium oxide, has been used widely to improve soft soils, including clays (Bell, 1996), (Al-Mukhtar, Khattab, & Alcover, 2012). It has also been tested on dredged material (Grubb, Chrysochoou, Smith, & Malasavage, 2010), (Grubb, Malasavage, Smith, & Chrysochoou, 2010). It can be made from limestone, seashells or other calcium carbonate containing material. The production process of quicklime mainly consists of burning calcium carbonate at a temperature of 825 °C, in which calcium oxide and carbon dioxide are formed. If the calcium oxide is not packed airtight, it will react with carbon dioxide in the air to convert back to calcium carbonate.

The working principle of quicklime in clays and dredged material is the hydration reaction upon contact with water. Quicklime and water together create an exothermic reaction, in which
temperatures can reach 150 °C, to form calcium hydroxide. On itself it improves the properties of clay or dredged material, but it can also be used with other additives, such as cement. This results appear to result in more consistent behaviour than cement or quicklime alone (Federico, Vitone, & Murianni, 2015). Both the liquid limit and plastic limit of the material show the highest increase when using both.

3.2.4 Fly ash

Fly ash is a by-product of coal combustion and therefore its composition varies, due to variation in the composition of coal. Plants that use anthracite and older bituminous coal produce class F fly ash, which is pozzolanic, meaning that it reacts with water and calcium hydroxide to produce a cementitious material. Class C fly ash does not need any other agent and is self-cementing, because it has a higher amount of calcium oxide, which can be seen in Table 4. Class C fly ash is, under normal conditions, a by-product of lignite plants. However, the quality between any given two types of fly ashes can differ greatly and therefore fly ash needs to be beneficiated before use in cement. Untreated fly ash can also be used, it will however in that case only act as a filler material. Normally, fly ash is used in combination with cement and can replace up to 50% of Portland cement. Under the Eurocode fly ash can be used in CEM II, where it can replace as much as 35% of Portland cement.

<table>
<thead>
<tr>
<th>Constituent</th>
<th>Bituminous % by weight</th>
<th>Subbituminous % by weight</th>
<th>Lignite % by weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO₂</td>
<td>20% - 50%</td>
<td>40% - 60%</td>
<td>15% - 45%</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>5% - 35%</td>
<td>20% - 30%</td>
<td>20% - 25%</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>10% - 40%</td>
<td>4% - 10%</td>
<td>4% - 15%</td>
</tr>
<tr>
<td>CaO</td>
<td>1% - 12%</td>
<td>5% - 30%</td>
<td>15% - 40%</td>
</tr>
<tr>
<td>LOI</td>
<td>0% - 15%</td>
<td>0% - 3%</td>
<td>0% - 5%</td>
</tr>
</tbody>
</table>

Table 4: the typical constituents of fly ash

Fly ash cement (CEM II/B-V) has been successfully used to stabilise the top layer at the Port of Valencia (Burgos, Samper, & Alonso, 2006). When used in Bangkok clay as a replacement for Portland cement it does not appear to improve or deteriorate the unconfined compressive strength of the clay when compared to stabilisation with Portland cement only (Horpibulsuk, Rachan, & Suddeepong, Assessment of strength development in blended cement admixed Bangkok clay, 2011). However, fly ash does seem to enhance the dispersion of the cement in clay (Horpibulsuk, Phojan, Suddeepong, Chinkulkijniwat, & Liu, 2012). It has been shown that fly ash does work on dredged material in the presence of a calcium hydroxide creating or
containing agent and does not work well when such an agent is not present (Grubb, Chrysochoou, Smith, & Malasavage, 2010).

3.2.5 **Cement Kiln Dust**

A waste product of the cement industry is cement kiln dust or CKD. Much like fly ash, it is a by-product that is left at the outlet of the factory. CKD is often fed to the kiln again, but this is not always the case. It is often used as a filler material, but, depending on the composition of the CKD, it does not have to be inert. Its main component is calcium carbonate, which normally constitutes approximately half of the mass of the CKD. Other components include silicon dioxide, quicklime, potassium sulphate, calcium sulphate, aluminium oxide, ferric oxide, potassium chloride, magnesium oxide, sodium sulphate and potassium fluoride. The components are shown in Table 5.

When having the right composition, it can be a useful material in stabilising dredged material (Grubb, Chrysochoou, Smith, & Malasavage, 2010). As it is a cement residue it shares some properties with cement, including the crystallisation of water. It can also be cementitious, largely depending on the chemical composition.

<table>
<thead>
<tr>
<th>Constituent</th>
<th>% by weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>CaCO$_3$</td>
<td>55.5</td>
</tr>
<tr>
<td>SiO$_2$</td>
<td>13.6</td>
</tr>
<tr>
<td>CaO</td>
<td>8.1</td>
</tr>
<tr>
<td>K$_2$SO$_4$</td>
<td>5.9</td>
</tr>
<tr>
<td>CaSO$_4$</td>
<td>5.2</td>
</tr>
<tr>
<td>Al$_2$O$_3$</td>
<td>4.5</td>
</tr>
<tr>
<td>Fe$_2$O$_3$</td>
<td>2.1</td>
</tr>
<tr>
<td>KCl</td>
<td>1.4</td>
</tr>
<tr>
<td>MgO</td>
<td>1.3</td>
</tr>
<tr>
<td>Na$_2$SO$_4$</td>
<td>1.3</td>
</tr>
<tr>
<td>KF</td>
<td>0.4</td>
</tr>
<tr>
<td>Other</td>
<td>0.7</td>
</tr>
</tbody>
</table>

Table 5: the typical constituents of cement kiln dust (Adaska & Taubert, 2008)

3.2.6 **Gypsum**

Gypsum in its pure form is calcium sulphate, a mineral which can be calcined. The calcined form of gypsum is better known as plaster of Paris, a common building material, which hardens when water is added to it. Completely calcined calcium sulphate is also known as anhydrite. Gypsum
is a dihydrate, meaning that it can contain up to two water molecules as water of crystallisation. Construction plaster, on average, has approximately crystallised half a water molecule on average, leading to the following reaction upon hydration:

\[ \text{CaSO}_4 \cdot \frac{1}{2} \text{H}_2\text{O} + \frac{3}{2} \text{H}_2\text{O} \rightarrow \text{CaSO}_4 \cdot 2\text{H}_2\text{O} \]

Its hydrating qualities have been used in studies where the improvement of peat was researched, leading to positive effects on the strength of the material (Timoney, McCabe, & Bell, 2012).

3.2.7 Biomass Ash

A few cases of studies where biomass ash has been used to investigate the possibilities of using it to stabilise soft soils (Horpibulsuk, Phojan, Suddeepong, Chinkulkijniwat, & Liu, 2012). Biomass ash can differ much in composition, but in general the chemical composition partially overlaps with other ashes commonly used, such as fly ash. The silicon dioxide content of the biomass ash used in the study mentioned was higher than fly ash, with very low aluminium oxide, ferric oxide and calcium oxide contents when compared to fly ash and Portland cement. Biomass ash has not been tested on dredged material according to literature, but on non-liquid clays only.

Biomass ash has not been tested on its own (without cement or lime), as its qualities were deemed to be comparable to fly ash. Tests have shown that when used as a clay improvement agent together with cement, a 25% addition factor by weight of cement added creates an optimal dispersion effect (Horpibulsuk, Phojan, Suddeepong, Chinkulkijniwat, & Liu, 2012).

3.2.8 Air Foam & EPS Beads

Air foam has been used in the Tokyo Runway D project (Watabe, Noguchi, & Mitarai, Use of Cement-Treated Lightweight Soils Made from Dredged Clay, 2012), the Kobe Port Island project (Watabe, Saeguse, Shinsha, & Tsuchida, 2011) and the Central Japan Airport project (Kitazume & Satoh, 2003). Its main application is not, as is with the other additives, to improve the strength parameters of the soil, but to decrease the lateral earth pressure, consolidation settlement and lateral soil movement. Air foam is therefore useable when using dredged material in a land reclamation site. The foaming agents used are commonly those used in the concrete industry. These agents are designed for applications involving cement and are therefore suitable for use with cement-stabilised dredged material as well. Due to its weight and nature, air foam is added as a percentage of the volume of dredged material, rather than the mass.
EPS beads are used in the same fashion and for the same purpose as air foam. As with air foam, the use of EPS beads has been inspired by their use in concrete. A disadvantage of EPS beads when comparing it with air foam is the fact that EPS beads are solids that tend to float, whereas air foam is initially a liquid.

3.2.9 Bentonite

Bentonite is a high plasticity clay, being known mostly for its ability to keep boreholes open. In that case, the bentonite is brought into suspension. Its large swelling capacity is used to stop the borehole from caving in. However, it is also a soil that has an exceptionally high liquid limit. It can be mixed with dredged material, in order to increase the liquid limit of the mixture. Dredged material usually has a liquid limit much lower than that of bentonite and a water content far above its liquid limit. When the liquid limit can be improved using bentonite, the liquid limit of the mixture and the water content will at a certain bentonite content coincide. Bentonite is a thixotropic material, which means that its strength decreases when an external force is applied above a certain threshold. When the external force is removed, it will take some time before the bentonite will return to its original strength. This means that when mixing bentonite to dredged material, an apparent curing time effect will be present. The settling properties of bentonite have been subject of investigation in the past. It has been shown that the influence of an alkaline environment on the settling behaviour of bentonite is small (Hendriks, 2016) and therefore it could be well used in combination with cement, GGBS and lime. Bentonite would decrease the permeability of the dredged material mixture, which can be advantageous when contaminants present in the dredged material need to be immobilised.

Bentonite occurs as several types of clay mineral, amongst which sodium bentonite and calcium bentonite are the prevalent types. The former has a higher liquid limit than the latter, which is caused by the higher ionic strength of calcium. This causes tighter bonds between the particles, leaving less room for water. Elaboration on this subject can be found in 4.3. Bentonite, acting as the negative ion, is a mix of mainly montmorillonite and some other minerals.

3.2.10 Inert filler

Inert fillers, such as sand, can also be used as an additive to dredged material. The working principle of these additives is quite straightforward: it increases the amount of dry mass in the material and therefore lowers the overall water content. Coarse particles do however tend to lower the liquid limit of the material, as these particles are mostly non-cohesive. Due to these properties, large quantities are needed to stabilise the dredged material. Reducing the water content below the liquid limit of the combined material implies that the only way to use this
technique is either via in-situ mixing or via mechanical transportation, using for example backhoes.

### 3.3 Strength

Strength is the most important property of the stabilised dredged material. In this thesis the undrained shear strength and the unconfined compressive strength are frequently used. They are related according Mohr’s circle, as follows:

$$\sigma_1 - \sigma_3 = 2c_u$$  \hspace{1cm} (1)

This relationship is used frequently. Field cores and cylindrical lab samples can be tested quite simply using a hydraulic press. Using this relationship, the undrained shear strength is known as well. Caution has to be taken, however. These relations are applicable in theory and only when the conditions for the use of Mohr’s circle are met.

The relationship is pictured in Figure 16. $\sigma_2$ is equal to zero, as there is no confining pressure present in an unconfined compressive test. $\sigma_1$ is equal to the normal stress applied by the hydraulic press used. It can be seen in the figure that this method only works when the test is performed under fully saturated, undrained conditions, as $\phi$ is equal to zero.

![Figure 16: the relationship between the unconfined compressive strength and the undrained shear strength (Dominic Trani)](image)

The undrained shear strength is a justifiable measure of strength. It can be measured by various test devices and is related to the unconfined compressive strength. It is also empirically related to the liquidity index. The behaviour of the dredged material, once the additives have caused the material to solidify, can be described as undrained. Due to the high water content of the dredged material the material is fully saturated. The hydraulic conductivity of the material itself is very low. As will be discussed in 4.3 and 4.6, the hydraulic conductivity does change under influence of additives, but will not create a drained setting. Therefore, the pore water cannot dissipate when large pressures are exerted on the soil.
3.4 Atterberg Limits

The Atterberg limits were introduced by Atterberg (1911) in order to establish a method that describes the water contents at which a fine-grained soil starts to show liquid, plastic or shrinking behaviour. These water contents are different for every kind of fine-grained soil and are named the liquid limit, plastic limit and shrinkage limit, respectively.

The plastic limit describes the water content at which the soil starts to behave plastically. Under this limit the soil is assumed to be non-plastic. Plastic behaviour is, in this case, the preferred type of behaviour, as the soil is a solid but does not behave brittle and hence does not crack. The soil behaves plastically in the range between the liquid limit and the plastic limit, called the plasticity index. A clay with a high plasticity index is also called a highly plastic clay, of which an example is bentonite.

At the water content where the soil stops to shrink, the shrinkage limit has been reached. The volume of the soil stops decreasing at this point and the void ratio will not decrease, but the water content will. As it is not expected that the shrinkage limit will be reached in stabilised dredged material, the emphasis is put on the plastic limit and liquid limit.

Tests to determine the liquid limit and plastic limit have been standardised in various codes, like ASTM D4318 and BS1377-2. Casagrande (1932) developed a method to determine the liquid limit of a soil that is nowadays known as the Casagrande cup method. Another method, which is more commonly used in Europe and leaves less room for interpretation, is the fall cone test. In this test a soil sample of a predefined size of 40 millimetres deep and 55 millimetres in diameter, is put under a stainless steel cone of 80 grams. The cone has a 30 degrees angle and is placed right on top of the sample. The cone is released for five seconds. The liquid limit is the water content for which the cone reaches into the sample for 20 millimetres. This value is found by interpolation.

The plastic limit is determined by rolling a thread on a flat surface, like a table. The surface has to be non-porous. If the thread keeps its shape, the behaviour is plastic. After remoulding the sample this test has to be repeated, until due to evaporation the water content has dropped to the plastic limit. At this point the thread will start to show cracks. The water content of the sample can then be determined by oven drying.

The plasticity index has been mentioned before as the difference between the liquid limit and the plastic limit:

$$I_p = LL - PL$$  \hspace{1cm} (2)
A measure used to describe the water content as a function of the liquid and plastic limit and the plasticity index, is the liquidity index:

\[
I_L = \frac{w - PL}{L_p} = \frac{w - PL}{LL - PL}
\]  

(3)

The water content does not tell much about the physical properties of the soil. A high water content does not necessarily mean that a soil is in its liquid state or even in its plastic state. If only the water content is used, the definition of “high” is left to interpretation. The liquidity index does not leave room for interpretation on that matter when the liquid limit and plastic limit have been determined properly. The state of the soil is captured in the liquidity index. A liquidity index of 0 means that the soil is at its plastic limit, whereas a liquid limit of 1 means that the soil is at its liquid limit. The soil is in its liquid state if the liquidity index is higher than 1.

This knowledge is imperative when dealing with stabilised dredged material. In order for the stabilising constituent to work properly, it has to verified that the stabilised material has a liquidity index well below 1.

Various researchers have shown that there is a link between undrained shear strength and the Atterberg limits. The most recent elaborate discussion of these relations has been conducted after a review of 641 fall cone tests on 101 soil samples from 12 countries (Vardanega & Haigh, 2014). Earlier an equation that relates the undrained shear strength to the liquidity index has been proposed (Wroth & Wood, 1978):

\[
c_u = 170 \exp(-4.6I_L) = 1.7 \cdot 10^{2(1-I_L)} \text{ [kPa]} \quad (0 < I_L < 1)
\]  

(4)

Note that this means that at \( I_L = 0 \) the undrained shear strength is assumed to be 170 kPa and at \( I_L = 1 \) to be 1.7 kPa. The 1:100 ratio has been observed in an earlier review research (Schofield & Wroth, 1968) of vane shear test data (Skempton & Northey, 1952). An alternative was found, valid for higher liquidity indices (Leroueil, Tavenas, & Le Bihan, 1983):

\[
c_u = \frac{1}{(I_L - 0.21)^2} \text{ [kPa]} \quad (0.5 < I_L < 2.5)
\]  

(5)

The formula fits the data on which it is based quite well, but as can be seen by the formula's characteristics, it cannot be extrapolated beyond a liquidity index of 0.5. At a liquidity index of
0.21, the undrained shear strength would reach infinity, for example. Another formula was proposed for liquidity indices in the liquid phase (Locat & Demers, 1988):

\[ c_u = \left( \frac{19.8}{I_L} \right)^{2.44} \text{[Pa]} \quad (1.5 < I_L < 6.0) \]  

(6)

A generalised version of (4) has been proposed in order to make the equation more applicable to different kinds of soils (Muir Wood, 1990):

\[ c_u = c_L R_{MW}^{1-I_L} \]  

(7)

In which \( c_L \) is the undrained shear strength at the liquid limit \( (I_L = 1) \) and \( R_{MW} \) is a function of the clay mineralogy. The data used by Vardanega & Haigh (2014) shows little correlation between \( R_{MW} \) and the clay mineralogy, but the data used shows values for \( R_{MW} \) ranging from 7 to 152. Based on their observations they propose to use a standard value of 35 for \( R_{MW} \). Looking at (7), it can be seen that \( R_{MW} \) times \( c_L \) is equal to the undrained shear strength at plastic limit. It is known that the undrained shear strength at liquid limit is equal to 1.7 kPa for remoulded clays, but such a fixed value applicable to stabilised clays at plastic limit has not been derived yet.

### 3.5 Pipe Pressure Loss

Important to whether or not the dredged material can move through a pipeline is the pipe pressure loss. The material has to be pumped through pipelines as a liquid by a dredging pump. Pumps have a certain power and must use that power to overcome the pressure loss induced by friction between the dredged material and the pipe.

The pressure loss can be calculated by means of the Darcy-Weisbach equation:

\[ \frac{\Delta p}{L} = f_D \frac{\rho v^2}{2D} \]  

(8)

With \( p \) being the pressure, \( L \) the pipe length, \( \rho \) the density, \( v \) the flow velocity, \( D \) the pipe diameter and \( f_D \) the Darcy friction factor. The Darcy friction factor can be established as follows, when a friction factor formula is wanted that spans all flow regimes (Churchill, 1977):

\[ f_D = 8 \left[ \left( \frac{8}{Re} \right)^{12} + \left( -2.457 \ln \left( \frac{7}{Re} \right) + \frac{0.27\varepsilon}{D} \right)^{16} + \left( \frac{37530}{Re} \right)^{16} \right]^{\frac{1}{12}} \]  

(9)

This formula includes \( \varepsilon \), which is the pipe roughness in metres. It also includes Reynolds’ number, which is defined as follows:
\[ Re = \frac{\rho v D}{\mu} \]  

(10)

In which \( \rho \), \( v \) and \( D \) represent the same quantities as in the Darcy-Weisbach equation and \( \mu \) represents the dynamic viscosity, which can be determined experimentally. These formulae apply to Newtonian fluids, which are perfect fluids in which pressure loss due to yielding does not appear.

Dredged material that has a liquidity index higher than 1 is regarded as a liquid, but still has an undrained shear strength of 1.7 kPa (at \( I_L = 1 \)) or lower (at higher liquidity indices). This value is related to its yield stress when the liquid soil is considered to be a Bingham plastic liquid (Locat & Demers, 1988). The difference between a Newtonian liquid, which starts to move immediately once stress has been applied, and a Bingham plastic liquid are depicted in Figure 17.

![Figure 17: graph depicting that, to be able to move a Bingham plastic liquid, the yield stress must be overcome first](image)

The term "Bingham plastic liquid" is confusing when compared to the terminology of the Atterberg limits. The terminologies are not related and sensu stricto the definition of when soil can be regarded as a Bingham plastic liquid is unrelated to the Atterberg limits. However, attempts have been made to relate parameters of the Bingham plastic model to the Atterberg limits. The plastic viscosity, the viscosity used in the Bingham plastic model, can be related to the liquidity index of a soil as follows (Locat & Demers, 1988):

\[ \eta = \left( \frac{9.27}{I_L} \right)^{3.33} \quad 1.5 < I_L < 6 \]  

(11)
This formula can be used as a first approximation and proves to be quite accurate.

When the liquidity index is sufficiently low, the yield stress will be significantly high and the equations (8), (9) and (10) have to be replaced by formulae that apply to Bingham plastic liquids. These formulae include the Buckingham-Reiner equation for laminar flow and the Darby-Melson equation for turbulent flow.

3.6 ENVIRONMENTAL CONSIDERATIONS

Dredged material can contain several sorts of contaminants, which can be immobilised when stabilised. From an environmental point of view, the stabilisation of dredged material is the cold immobilisation of contaminants. Another form of immobilisation is thermal immobilisation, where the material is sintered or molten. Other techniques to separate the contaminants in dredged material from the environment are thermal cleaning, wet cleaning, biological cleaning and disposal in depots.

The guideline for repair and maintenance of the soil quality in the Netherlands (in Dutch also known as the Bodemrichtlijn) provides some guidance with regard to cold immobilisation. It states that if possible, cleaning is the preferred method, as opposed to immobilisation. However, this guideline does not have a legal status.

Recycling of dredged material, with or without contaminations, can be placed in a framework known as the waste hierarchy, which can be seen in Figure 18. This hierarchy is used in the European Union in a slightly different form (Directive 2008/98/EC, 2008). The idea behind the waste hierarchy is to minimise waste streams and to handle waste as efficiently as possible. When seen in this light, the recycling of contaminated dredged material is preferred over disposal. Stabilising dredged material and thus immobilising the contaminants present in that material is a recycling technique and not a reuse technique. Reuse implies that the waste can be reused as is, which is not the case.
Cold immobilisation is mainly focussed on heavy metals and cyanide contaminations. The common binders used are inorganic. Organic binders could also be used, but this is an uncommon practice. Cement, lime and fly ash are commonly used inorganic binders. They create a calcium silicate matrix, in which they enclose the contaminants. Other chemicals can be added to the mixture as well, in order to facilitate neutralisation, oxidation, ion exchange, a change in pH or another beneficial process.

Organic matter in the dredged material has a negative effect on the binding capabilities of cement and thus on the cold immobilisation of the contaminants. An organic matter content of 10% appears to be a small problem and even with an organic matter content of 30% cold immobilisation should still be possible. Experiments on peat show that stabilising peat using cement is possible, even at organic matter contents of 99% (Timoney, McCabe, & Bell, 2012). These experiments have, however, not established if the cement also immobilises possible organic contaminants.

Proper investigations should be conducted before cold immobilisation is performed. Site investigations must be performed in order to establish what contaminants are present. The state of knowledge in the field of cold immobilisation is such, that the preferred method is to check for projects in the past, in which the same contaminants were immobilised in similar conditions. If such projects do not exist, an experimental approach to determining the right additive for immobilisation is common.
4 LITERATURE STUDY

4.1 ADDITIVES USED

The additives used in literature are mainly cement, investigated by various researchers, and lime. The latter is used by Bell (1996) on various clays and Grubb et al. (2010). Federico et al. (2015) have used lime on dredged material. Gypsum has been used by Timoney et al. (2012) on peat samples. Fly ash, biomass ash, ground-granulated blast furnace slag (GGBS), cement kiln dust and inert fillers have been used by various authors as a secondary additive to Portland cement, such as Horpibulsuk et al. (2004), Grubb et al. (2010, Horpibulsuk et al (2011), Horpibulsuk et al. (2012), Timoney et al. (2012) and Federico et al. (2015).

Fly ash and GGBS are common additives to Portland cement in different varieties of the Eurocode standardised cements CEM II and CEM III. These types of cement can be regarded as showing similar behaviour as Portland cement. The use of biomass ash appears to have the same effect as fly ash (Horpibulsuk, Phojan, Suddeepong, Chinkulkijniwat, & Liu, 2012). Cement kiln dust can be regarded as being a category rather than a defined material. It is thus not useful to compare experiments performed as part of different research programmes. Gypsum is weaker than cement, but, as can be seen in the next paragraph, comes at the same cost as cement. The emphasis will therefore be put on cement, lime and bentonite.

4.2 ECONOMIC CONSIDERATIONS

As mentioned earlier, the reuse of dredged material is only economically viable if other, more common materials, such as sand and gravel, are unavailable or if the dredged material is contaminated. The costs of additives in all registered cases of ground stabilisation are often more than four times higher than the cost of ordinary sand where sand is widely available. The total cost for placing stabilised dredged material in a land reclamation site are shown in Table 6 (Suzdalev & Rogbeck, 2012).
<table>
<thead>
<tr>
<th>Project</th>
<th>Additives</th>
<th>Amount of additives [kg/m³]</th>
<th>Costs [€/m³]</th>
<th>Estimated additive costs [€]</th>
<th>Method</th>
<th>Target strength [kPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hammarby</td>
<td>Cement (50%), GGBS (50%)</td>
<td>250</td>
<td>€ 43,-</td>
<td>€ 13,25</td>
<td>In-situ</td>
<td>N/A</td>
</tr>
<tr>
<td>Oxelösund</td>
<td>Cement (50%), GGBS (50%)</td>
<td>150</td>
<td>€ 16,70</td>
<td>€ 7,88</td>
<td>In-situ</td>
<td>140</td>
</tr>
<tr>
<td>Vuosaari</td>
<td>Cement</td>
<td>130</td>
<td>€ 21,-</td>
<td>€ 8,45</td>
<td>In-situ</td>
<td>140</td>
</tr>
<tr>
<td>Aurajoki</td>
<td>Cement, GGBS, Fly ash</td>
<td>N/A</td>
<td>€ 45,-</td>
<td>N/A</td>
<td>Process stabilisation</td>
<td>100</td>
</tr>
<tr>
<td>Trondheim</td>
<td>Cement (66%), Fly ash (33%)</td>
<td>180</td>
<td>€ 50,-</td>
<td>€ 10,20</td>
<td>Ex-situ</td>
<td>200</td>
</tr>
<tr>
<td>Kadettangen</td>
<td>Fly ash cement (50%), GGBS (50%)</td>
<td>160</td>
<td>€ 72,-</td>
<td>€ 8,40</td>
<td>In-situ</td>
<td>100</td>
</tr>
</tbody>
</table>

Table 6: reference projects with additives, cost, method and target strengths

In Table 7 the approximate prices for the different additives are shown. It has to be noted that these prices depend on local and global availability. Ordinary Portland cement appears to be inexpensive when compared with the other additives. Lime, which is also used as a primary additive, is one of the more expensive additives. Bentonite proves to be quite expensive as well and it should thus be noted that it should only be used when its properties are superior to the properties of other additives. What makes bentonite superior depends on the requirements of a specific project.
<table>
<thead>
<tr>
<th>Additive</th>
<th>Costs [€/tonne]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Portland cement</td>
<td>€ 65.00</td>
</tr>
<tr>
<td>Sulphate cement</td>
<td>€ 100.00</td>
</tr>
<tr>
<td>Fly ash cement</td>
<td>€ 55.00</td>
</tr>
<tr>
<td>Industrial cement</td>
<td>€ 80.00</td>
</tr>
<tr>
<td>GGBS</td>
<td>€ 40.00</td>
</tr>
<tr>
<td>Fly ash</td>
<td>€ 40.00</td>
</tr>
<tr>
<td>Gypsum</td>
<td>€ 65.00</td>
</tr>
<tr>
<td>Quick lime</td>
<td>€ 100.00</td>
</tr>
<tr>
<td>Biomass ash</td>
<td>€ 40.00</td>
</tr>
<tr>
<td>Cement Kiln Dust</td>
<td>€ 40.00</td>
</tr>
<tr>
<td>Sand</td>
<td>€ 5.00</td>
</tr>
<tr>
<td>Air foam</td>
<td>€ 40.00</td>
</tr>
<tr>
<td>Bentonite</td>
<td>€ 200.00</td>
</tr>
</tbody>
</table>

Table 7: approximate raw material prices for additives

Combining the findings from Table 6 and Table 7, it can be established that the majority of costs comes from transportation, the equipment used and the dredging process itself. These factors are dependent on the context of the project, the equipment used for mixing and dredging and the location of the project. From Table 6 it can be concluded that if sand is widely available and no contaminants need to be immobilised, it is wiser to use regular land reclamation methods.

4.3 **FUNDAMENTAL BEHAVIOUR**

The behaviour of clays in suspension can be explained by colloid chemistry. When the dredged material has a significant clay fraction, the behaviour of the dredged material is governed by its clay fraction (Hendriks, 2016). Clay consists of minerals, having cations and anions. The clay particle itself, a plate-like particle, acts as the anion. The cations are usually a mixture of various metal ions.

In solution, the clay particle and cations rearrange themselves according to Figure 19. The clay particle – in suspension – is negatively charged all the way around its surface. Positively charged cations dissolve in the solution and are attracted to the clay particle. This causes a diffuse, but finite, layer of cations around the clay particle. The size of this diffuse double layer, as it is called, is determined by the type and amount of cations present in the solution.
Because of the build-up of the diffuse double layer, the positively charged cations on the outsides of two clay particles repel one another. This phenomenon causes the particles not to aggregate in solutions with low ionic strength. When moving close to one another, the Van der Waals force overcomes the effect of the double diffuse layer and start to form flocs.

![Diagram of the diffuse double layer](image)

**Figure 19: conceptual model of the workings of the diffuse double layer (Winterwerp, 2014)**

The tendency to move closer to one another is enhanced by a larger ionic strength of the solution. A larger concentration of cations in the solution will decrease the gradient between the cations attracted to the clay particle and the surrounding solution, thus decreasing the size of the double diffuse layer. The double diffuse layer is therefore compressed in a solution with a large number of dissolved minerals. Experiments have shown that there already is a large difference between purified water and ordinary tap water, as tap water contains a lot of calcium cations. Tap water and a sodium chloride solution show similar effects on the settling behaviour of clays in suspension (Van Dijk, 2016).
As was mentioned in 3.2.1, the cations in cement are mainly calcium. The calcium cations exchange with the (weaker) cations present in the clay suspension. That effect and decrease in gradient due to the increased ionic strength cause the diffuse double layer to compress (Chew, Kamruzzaman, & Lee, 2004). This means that the clay particles will start to aggregate. Aggregation caused by cementation enhances the formation of larger cement-clay clusters. Similar behaviour can be attributed to lime, as it also has calcium as the main cation (Locat, Bérubé, & Choquette, Laboratory investigations on the lime stabilization of sensitive clays: shear strength development, 1990).

The primary reactions the cement induces are hydration reactions, that form hydrated calcium silicates, calcium aluminates and hydrated lime. Considering the contents of Table 2 in 3.2.1, these outcomes are to be expected. The hydrated lime and silica and alumina from the clay minerals react with one another, forming calcium silicate hydrates and calcium alumina hydrates. This reaction is a pozzolanic reaction.

The hydration reaction of cement is largely finished after a curing time of 7 days (Chew, Kamruzzaman, & Lee, 2004). However, at that point, the curing of cement has not finished. The much slower, pozzolanic reactions continue. Certain clays, such as kaolinite and illite are pozzolans. Kaolinite is known to be able to completely react in a pozzolanic reaction. At high cement contents, kaolinite can dissolve completely. Pozzolanic reactions will lead to a more open structure and enhance strength. This means that the chemical composition of the clay also matters in terms of reactivity with the cement. As pozzolanic reactions occur when calcium oxide is present, they also occur when lime stabilisation is used instead of cement stabilisation (Rao & Rajasekaran, 1996).

The reaction cement has with water changes the water content of the dredged material. This difference is depicted in Figure 20. In this model (Zhu, Zhang, & Chiu, 2007) the following types of water are proposed:

- Hydration water, $m_{h\text{w}0}$, which is water captured in the soil minerals' crystal lattice;
- Bound water, $m_{b\text{w}0}$, which is water bound to surface of the clay;
- Free water, $m_{f\text{w}0}$, which is water not bound to the clay;
- Pore water, $m_{p\text{w}0}$, which is the sum of the bound and free water, the water in the pores.

These initial water contents can be seen on the left side of Figure 20. On the right side, the changes in water contents can be seen. Because of the hydration reaction of cement with water, more hydration water appears, whereas the free water is reduced with the same amount. Some
water, denoted as $m_o$, evaporates due to the reaction heat. Cement traps water in the formed aggregates and thus the bound water increases as well, decreasing the free water.

![Diagram of water content changes](image)

**Figure 20:** the difference between the water in an untreated and a treated clay slurry (Zhu, Zhang, & Chiu, 2007)

The effect of an increasing cement content on the different water contents can be seen in Figure 21. On the bottom of the graph, the initial bound water content can be seen. On the top, linearly increasing evaporation water and hydration water can be seen. From the starting point, the bound water content then increases inversely exponential with the cement content, decreasing the free water. This process continues until the cement content has reached the point where no free water is available anymore. At that point, the cement starts to hydrate the bound water.

![Diagram of water content changes versus cement content](image)

**Figure 21:** the increase and decrease in different water contents versus the cement content (Zhu, Zhang, & Chiu, 2007)

The following formulae apply to Figure 21, according to Zhu, Zhang & Chiu (2007):

$$\Delta m_{hw} = k_1 a_c$$  \hspace{1cm} (12)
In which \( k_1 \) represents a constant, to be determined experimentally and \( a_c \), the cement content.

\[
m_{pw} = m_{pwo} - k_1 a_c - m_e \tag{13}
\]

And:

\[
m_{bw} = \begin{cases} 
m_{bwo} + k_3 \left(1 - \exp(-k_2 a_c)\right) & a_c \leq a_{c0} \\
m_{pw} & a_c > a_{c0} \end{cases} \tag{14}
\]

In which \( a_{c0} \), is the cement content at the point where the free water content is equal to zero. The constants \( k_2 \) and \( k_3 \) have to be determined experimentally.

Bound water can also be described as the water that has a \( pF \) higher than 3.8 (Lebedev, 1936), in which \( pF \) is defined as follows:

\[
pF = -\log(h) \tag{15}
\]

In which \( h \) is the groundwater head.

What happens to the soil water retention curve when stabilising a soil with cement, can be seen in Figure 22. Because water is bound in the clay-cement clusters, it requires more energy to remove the water from the pores. This means that the soil water retention curve is more shallow in the beginning. Because the soil mixture is still saturated, but the void ratio has decreased because of the addition and reaction of cement, the saturated water content is lower.
The phenomenon of an increasing bound water content has led Chew, Kamruzzaman & Lee (2004) to propose that this is one of the main drivers of the increment in plastic and liquid limit. The presence of cement causes the mixture to flocculate, after which the reaction of the cement with water causes the creation of clay-cement clusters. Between these clusters large voids exist, which should increase the hydraulic conductivity of the mixture.

At high cement contents, a decrease in liquid limit is observed. This is attributed to the deposition of cementitious products on the surface of the clay-cement flocs. This reduces the surface activity of the clay, as it becomes encapsulated. A slight decrease between 7 and 28 days curing time can be observed, as more pozzolanic reactions have taken place after 28 days. Precipitation of CSH and CASH on the clusters due to these pozzolanic reactions decrease the liquid limit further. However, the trend in change of liquid limit depends heavily on the soil type. Experiments on lime-stabilised Louiseville clay show an increase in liquid limit (Locat, Tremblay, & Leroueil, Mechanical and hydraulic behaviour of a soft inorganic clay treated with lime, 1996). Very plastic soils (having a high plasticity index) have been reported to show a decrease in liquid limit (such as montmorillonite), whereas low-plasticity soils show an increase in liquid limit (Brandl, 1981). Chew, Kamruzzaman & Lee (2004) propose that “in soils of high plasticity, the encapsulation of the clay clusters by deposited cementitious products has a dominant effect, leading to lowering of the liquid limit.”
The plastic limit increases with the cement content, before halting at a certain point. This point is associated with the start of deposition of cementitious products on the surface of the clay-cement flocs. As more and more water is trapped in the inter-aggregate pores, the plastic limit, as well as the liquid limit, increases. This implies that the plasticity index should not change significantly after curing, when lower cement or lime contents are used. Experiments performed on dredged material from the Port of Taranto, stabilised with both cement and lime, seem to confirm this hypothesis (Federico, Vitone, & Murianni, 2015).

The statements regarding the liquid limit and plastic limit are confirmed by experiments performed by Chew, Kamruzzaman & Lee (2004). The results can be seen in Figure 23.

As cement, as well as lime, is an alkaline substance, the pH increases with increasing cement content. The pH increases rapidly with cement content, until it stabilises around a pH of 12.5. The results of experiments is captured in the graph in Figure 24. The point where the pH stabilises is around the same point as where the plastic limit stabilises and the liquid limit starts to decrease. This point is the same point as where the flocculation of the clay-cement clusters stop and the deposition of cementitious products on these clusters start. The pH of 12.5 corresponds with that of hydrated lime and thus supports the hypothesis that there is unreacted hydrated lime present in the mixture.
4.4 Strength

The main purpose of additives is an increase in strength. The strength is commonly measured in terms of the unconfined compressive strength or the undrained shear strength.

Bell (1996) has tested two types of lime-stabilised clay. The first is Upper Boulder Clay, which is a till. The second is Tees Laminated Clay. Both were tested in the context of the potential use of clays as subgrades in road construction. Note that this means that the unconfined compressive strength at 0% added lime is therefore higher than the strength of dredged material without additives, because both soils have an initial liquidity index well below one.

The results can be seen Figure 25. A remarkable feature of this graph is the very large spread in the results. At 6% lime content, the unconfined compressive strength of one sample is 222 kPa, only 44 kPa higher than the unconfined compressive strength of an untreated sample. Another sample, however, had an unconfined compressive strength of 880 kPa. The mean value of the UCS at 4% lime content for the Tees Laminated Clay is higher than the mean UCS at 6% lime content. This effect can be attributed to something that does not appear in stabilised dredged material. Clay does not necessarily have to have a high water content. Quicklime is highly hydrating and it thus could have happened that the water content fell below the optimum water content. Another explanation could be that the unconfined compressive tests were not performed under fully saturated conditions, giving inconsistent results. Mixing problems could also be part of the issue, as the unconfined compressive strengths at zero lime content show that the water content is approximately near the plastic limit.
From these data it can be concluded that lime has an effect on clays. The difference between the clays used by Bell (1996) and dredged material is mainly the water content, which can be several times higher in dredged material. Due to this high water content the three problems mentioned are less likely to appear in dredged material. More consistent results are thus expected when experimenting with lime-stabilised dredged material.

Sparrevik & Kvennås (2008) have used cement as an additive to dredged material. The results can (partially) be seen in Figure 26, Figure 27 and Figure 28. The samples selected are the Bergen, Grenland and Sandvika samples, with water contents of 196.4%, 113.5% and 131.9%, respectively. These values are deemed to be representative for dredged material, as dredged material with a density of 1300 kg/m$^3$ has a water content of approximately 160% to 170%, depending on the specific gravity of the material. The in-situ densities of the materials tested are 1.28, 1.38 and 1.38 kg/m$^3$, respectively. Tests to determine the Atterberg limits were not undertaken in this study.

The data shows a non-linear but consistent increase in undrained shear strength with an increasing cement content, regardless of the cement used. These data are therefore well comparable to the experimental study in chapter 5.
Figure 26: The cement content versus undrained shear strength for the Bergen samples, after Sparrevik & Kvennás (2008)

Figure 27: The cement content versus undrained shear strength for the Bergen samples, after Sparrevik & Kvennás (2008)
Horpibulsuk et al. (2003) have found similar results for adding cement to clay. The same researchers have performed similar experiments, including Horpibulsuk, Miura & Bergado (2004), Horpibulsuk, Miura, Koga & Nagaraj (2004), Horpibulsuk, Rachan & Suddeepong (2011), Horpibulsuk, Phoja, Suddeepong, Chinkulkijnwat & Liu (2012), Horpibulsuk, Suddeepong, Chinkulkijnwat & Liu (2012), Horpibulsuk, Rachan & Suddeepong (2013) and Horpibulsuk, Suddeepong, Suksiripattanapong, Chinkulkijnwat, Arulrajah & Miri Disfani (2014). Using the results from these tests, they have found a formula based on Abrams’ Law, which relates the water-cement ratio to the strength of concrete.

The relationship between the strength of concrete and the water-cement ratio has been defined as follows (Abrams, 1918):

$$S = \frac{A}{Bw/c}$$  \hspace{1cm} (16)

In this formula, $S$ is the compressive strength of concrete at a specific age (often 28 days), $w/c$ is the water-cement ratio by weight and $A$ and $B$ are empirical constants. This formula has also been used in researching the water-cement ratio in cement-admixed clays, where $A$ is a parameter used to describe the type of clay, the liquidity index and the rest period and the parameter $B$ is a value between 1.22 and 1.24 (Horpibulsuk, Miura, & Nagaraj, 2003). As described in the study by Horpibulsuk et al. (2003), the factor $A$ can be eliminated in the following way, when the strength of a sample with a different cement content needs to be calculated:
In which the water-cement factor is determined as follows:

\[
\frac{q_{(w_c)_1}^{w_c}}{q_{(w_c)_2}^{w_c}} = \frac{A/\left(w_c^{(w_c)_1}\right)}{B/\left(w_c^{(w_c)_2}\right)} = 1.24\left(w_c^{(w_c)_2}-w_c^{(w_c)_1}\right)
\]

(17)

Horpibulsuk et al. (2003) have shown that this empirical relationship is valid for a range of cement-admixed clays. However, no research to this formula has been done regarding dredged material. To check whether this is the case, data from Sparrevik & Kvennås (2008) is used. They have tested several types of dredged materials on four types of cement, being standard Portland cement, fly ash cement, industrial cement and sulphate resistant cement. For this study, the Horpibulsuk formula is rewritten to:

\[
q_{(w_c)_1}^{w_c} = q_{(w_c)_2}^{w_c} 1.24\left(w_c^{(w_c)_2}-w_c^{(w_c)_1}\right)
\]

(19)

From the Sparrevik & Kvennås (2008) data, only the samples that have a higher water content than 100% are investigated, since these are the samples relevant to this study, because dredged material usually has a very high water content. Since the dredged material samples with lower UCS values can show very unpredictable behaviour at low cement content (50 kg/m\(^3\)), the unconfined compressive strength of the samples is predicted using the sample with the highest cement content (150 kg/m\(^3\)). The results can be seen in Figure 29. The complete table can be found in the appendix.
The mean error in these results is 27.43%, with the maximum error at 56.57% and the minimum error at 1.50%. From these results, it can be concluded that this formula has some predictive value, but does not guarantee that the calculated outcome is nearly correct. At most this formula can be applied as a first estimation.

However, the assumption that the undrained shear strength is correlated to the water-cement ratio appears to be legitimate, when looking at Figure 30, Figure 31 and Figure 32. In these graphs, the undrained shear strength and the water-cement ratio appear to be related. There is however no closure on whether this is due to the cement content or both the cement and water content, as only one water content per sample is tested. Several types of functions can be considered to fit these data. It has been suggested by Horpibulsuk et al. (2003) that this should be an exponential function. Other research suggests that this approach is too simplistic (Chiu, Zhu, & Zhang, 2008).
Figure 30: the water-cement ratio versus undrained shear strength for the Bergen samples, after Sparrevik & Kvennås (2008)

Figure 31: the water-cement ratio versus undrained shear strength for the Grenland samples, after Sparrevik & Kvennås (2008)
Figure 32: the water-cement ratio versus undrained shear strength for the Sandvika samples, after Sparrevik & Kvennås (2008)

The increase in strength also implies an increase in yield strength. Experiments by Chiu, Zhu & Zhang (2008) show that the failure envelope widens as a result of cementation. The results of consolidated undrained triaxial tests can be seen in Figure 33.

Figure 33: the failure envelope of a dredged material treated with 100 kg/m$^3$ cement after 7 days (Chiu, Zhu, & Zhang, 2008)

Cement treated soils could be a reconstituted soil, in which case they would follow the intrinsic normal compression line, as proposed by Burland (1990). This is a line that accurately describes the compressive behaviour under stress of a reconstituted clay. However, it can be seen by tests performed by Chiu, Zhu and Zhang (2008) that the addition of cement results in something like
a preconsolidation pressure. The results are shown in Figure 34. The graph clearly shows that the cemented clays do not behave as if they were a reconstituted clay. It can also be seen in this graph that a larger cement content decreases the void ratio and increases the yield strength. Yield strengths are depicted in this graph with an arrow. It is defined as the point where the void ratio starts to decrease rapidly under increasing effective stress. What is suggested in the graph, is that because the gradient after yielding is larger than the gradient of the intrinsic normal compression line, they will meet at a very low void ratio.

![Figure 34: the effective pressure versus the void ratio, showing that the cement treated samples possess something like a pre-consolidation pressure (Chiu, Zhu, & Zhang, 2008)](image)

The phenomena shown in Figure 33 and Figure 34 were to be expected on the basis of good engineering judgement. Chiu, Zhu & Zhang (2008) chose to describe the yield strength of their samples as a function of the change in bound water content, as described in Figure 21 by Zhu, Zhang & Chiu (2007). The change in bound water content accurately describes the effect cement has on dredged material, as it is a measure of the flocculation process. However, change in bound water content is a measure relative to the solid particles, but not to the other water available in the dredged material. This is an important feature, because dependent on the free water (4.3) available, this will increase or decrease the strength. To relate the change in bound water content to the other water available, the normalised change in bound water content is introduced (Chiu, Zhu, & Zhang, 2008):

\[
\Delta m_{bnw} = \frac{\Delta m_{bw}}{m_{pw} - m_{bw0}}
\]  

The normalised change in bound water content thus is the change in bound water content divided by the free water initially available. The outcome of this equation cannot be higher than
1. It can also be seen that, when a very large amount of water is added to the dredged material, this number will only be significant when a very large amount of cement is added as well.

Chiu, Zhu & Zhang (2008) correlated the yield strength, for which examples are shown in Figure 34, to the normalised change in bound water content. The graph can be seen in Figure 35. What is striking, is that the curve only starts at a value for the normalised change in bound water content of 0.37. This means that the material can still considered to be a liquid at this point, as liquids yield when pressure is applied. The correlation derived by Chiu, Zhu & Zhang (2008) is:

\[
p_y' = 527.37 \Delta m_{bwN}^2 + 617.96 \Delta m_{bwN} - 307 \quad 0.37 \leq \Delta m_{bwN} < 1 \quad R^2 = 0.818 \quad (21)
\]

Below a normalised change in bound water content the yield strength is zero.

\[
\Delta L \quad \square \quad M \quad \bigcirc \quad \text{Best-fit curve}
\]

**Figure 35:** the normalised change in bound water content correlated to the yield strength for three different dredged materials (Chiu, Zhu, & Zhang, 2008)

The cohesion of the cement-treated soils can also be correlated to the normalised change in bound water content, for which the follow formula was proposed:

\[
c' = 925.92 \Delta m_{bwN}^2 + 626.39 \Delta m_{bwN} + 105 \quad 0.37 \leq \Delta m_{bwN} < 1 \quad R^2 = 0.897 \quad (22)
\]

The graph showing the correlation can be seen in Figure 36.
Figure 36: the normalised change in bound water content correlated to the cohesion for three different dredged materials (Chiu, Zhu, & Zhang, 2008)

The curves described in (21) and (22) fit quite well, however, the constants used in the polynomial are chosen to fit the curve. The constants appear to be unrelated to any parameter associated with the materials. A larger database would show the validity of these formulae for other dredged materials. Finding the change in bound water content in the field requires at least a small-scale centrifuge. This increases the amount of tests one has to perform on a sample of dredged material. It can, however, increase the accuracy, as the normalised change in bound water content appears to be a very (but not totally) inclusive parameter to use, especially when comparing it to the water-cement content. The water-cement ratio does not incorporate bound water or the difference in the capability cement has to bind water in different soils.

4.5 ATTERBERG LIMITS

Research on the Atterberg limits of stabilised dredged materials has been performed recently by Federico et al. (2015). The dredged material concerned was dredged in the Port of Taranto. The results are summarised in Figure 37, Figure 38, Figure 39 and Figure 40.

These show that with increasing cement content, the plastic limit increases as well. However, after 28 days, the effect is small for small cement contents. The same is applicable to the liquid limit. The liquid limit, however, appears to decrease over time. This means that the plasticity index decreases over time as well.

The plastic limit of lime stabilised dredged material shows similar behaviour as the cement stabilised dredged material. However, at lower lime contents, the plastic limit does not decrease as much over time, when compared to cement stabilised dredged material. The decrease in liquid limit observed for cement stabilised dredged materials has not been observed in lime
stabilised dredged materials. On the contrary: the liquid limit keeps increasing after 28 days for all lime contents.

These results seem to – independently – confirm the results found by Chew, Kamruzzaman & Lee (2004). On a larger scale the stabilised samples behave in line with the hypothesis of bound water being responsible for the increase in plastic and liquid limit.

Figure 37: the plastic limit development for three different cement contents, after Federico et al. (2015)

Figure 38: the liquid limit development for three different cement contents, after Federico et al. (2015)
4.6 **Hydraulic Conductivity**

Based on the experiences in concrete, it is expected that cement decreases the hydraulic conductivity of the dredged material. Concrete is made of cement, sand and gravel. Sand and gravel have a high hydraulic conductivity. When cemented, this hydraulic conductivity drastically decreases to a near-zero value, several orders of magnitude smaller than the hydraulic conductivity of sand and gravel. It is therefore in the line of expectation that the hydraulic conductivity of stabilised dredged material decreases with an increase in cement content.
However, based on the theory described in 4.3, the hydraulic conductivity should increase with cement content due to flocculation. Test results should be able to show which process has a stronger influence on the behaviour of the cement-stabilised material regarding the hydraulic conductivity.

Sparrevik & Kvennås (2008) have used oedometer tests to determine the hydraulic conductivity of their samples. The results can be seen in Figure 41, Figure 42 and Figure 43. The results appear to be inconsistent. Only in the Grenland samples a consistent decrease in hydraulic conductivity can be seen. The rest of the samples all show, depending on the type of cement used, little change in hydraulic conductivity or even an increasing hydraulic conductivity, followed by a decrease. Based on these data and on further data produced by Sparrevik & Kvennås (2008), it is only possible to state that there is no apparent correlation between the cement content and the hydraulic conductivity of the samples. Sparrevik & Kvennås (2008) do however state that the unconfined compressive strength and the hydraulic conductivity are correlated.

The influence cement has on the hydraulic conductivity of dredged material thus remains vague. There is no apparent prevalent process that either increases or decreases the hydraulic conductivity of the stabilised material.

![Figure 41: the cement content versus the hydraulic conductivity of the Bergen samples, after Sparrevik & Kvennås (2008)](image)
4.7 Equipment

Various pieces of equipment have been discussed already in chapter 2. These can generally be put in three categories:

- In-situ mixing
- Ex-situ mixing from barges
- Pneumatic flow mixing

In-situ mixers use mixers that inject cement, grout or lime at high pressures. This method has been described extensively by Teunissen (2005). The progression rate of this method is
approximately 50 m³/hour per mixer. However, the in-situ mixing method allows for the dredged material to be placed at much higher rates. The progression rate is only that of the stabilisation works. The progression rate is linear with the number of in-situ mixers used. It can thus be said that this method is useable for large land reclamation sites, if the progression rate of the stabilisation does not necessarily have to be large.

Ex-situ mixing from barges is referred to in 2.6. This method uses an in-line mixing tool, making the progression rate of the entire project dependent on the rates the separate pieces of equipment can handle. These pieces of equipment usually include a barge, a backhoe, a sieve, a conveyor belt, a mixing unit and dump trucks. As the throughputs of these pieces of equipment are usually quite low and some efficiency will be lost, this method appears only to be suitable for projects in which time is not a limiting factor. An example of ex-situ mixing from barges can be seen in Figure 14.

Pneumatic flow mixing has been used for various land reclamation sites in Japan, including the cases described in 2.3 and 2.4 (Kitazume & Satoh, 2003). The pneumatic flow mixing principle is schematically drawn in Figure 8. A closer look at the pneumatic flow principle is given in Figure 44. Pneumatic flow mixing uses clay plugs mixed with an additive to produce a soil that is strong enough to meet the requirements. The plugs are created by the compressed air that is inserted at high speeds via a valve in a pipeline that contains solely mud at the starting point.

![Figure 44: the pneumatic flow principle (Kitazume & Satoh, 2003)](image)

The creation of these clay plugs is governed by a set of parameters that together form the mixing ratio. The mixing ratio is described as follows:

\[
M = \frac{\rho_L Q_L}{\rho_A Q_A}
\]

(23)

Where \( \rho_L \) is the unit weight of soil [kg/m³], \( Q_L \) is the flow volume of soil [g/m³], \( \rho_A \) is the unit weight of air [kg/m³] and \( Q_A \) is the flow volume of air [g/m³]. In Figure 45 the unit air speed is depicted versus the mixing ratio. The oval field that is marked with the words “Plug flow” is the area where plug flow will occur. The unit air speed is, as is also shown, related to the pipe...
diameter. Equation (23) holds the soil density parameter, which is an important parameter in dredging.

![Figure 45: the mixing ratio versus unit air speed diagram (Kitazume & Satoh, 2003)](image)

This method can reach up to 1000 m$^3$/hour, but most equipment has a rate of 300 m$^3$/hour. To transport 300 m$^3$/hour of dredged material and 80 kg/m$^3$ of cement, the inlet air pressure has to be 550 kPa, in order to be almost zero 350 metres down the line, as can be seen in Figure 46. A picture of the process at Nagoya Airport is shown in Figure 47. As can be seen in the picture, the soil is delivered to the site on a barge. A backhoe shovels it into a hopper, from where a sand pump pumps the mud to the compressed air inlet. Here it is mixed with air to form the plugs. After that it goes to the stabilising agent supply barge, which mixes the plugs with cement. The cement and dredged material are mixed as a result of the turbulence in the pipeline. Lastly the cemented plugs go to the placement barge where the plugs are deposited into the land reclamation site.
The main advantage of this method is its higher capacity than the other two methods. However, it has not been scaled to a larger version that could potentially handle 2000 m$^3$/hour. The simple solution to this problem is to install multiple units. Theoretically, this type of system could handle any type of additive that can be added as a powder, as long as the density of the mixed substance does not cause the plug to leave the plug flow range as depicted in Figure 45.
Figure 47: the set of barges that makes up the pneumatic flow mixing system (Kitazume & Satoh, 2003)

Novel methods would include in-line wet mixing. If that could be made possible, the additives would have to be inserted directly into the pipeline, where the turbulence of the pipe would mix the additives and soils. The main problem with this approach is the density of the dredged material and additives combined. Pneumatic flow mixing works, because the plugs are pushed forward by compressed air, which can handle higher densities than a fully saturated pipeline. Liquid limit improving additives would pose a problem, as well as highly water consuming additives. Addition would thus have to take place at the end of the pipeline. It would also require that the additive used does not react immediately. Additives such as lime (immediately reacts) or immediately swelling additives are thus not preferred.

Kitazume & Satoh (2003) have also shown that there is a relation between the water-cement content of the dredged material and the unconfined compressive strength. They do not, however, capture this into a general formula, like Horpibulsuk et al. (2003).

When large land reclamation sites would need to be filled with stabilised dredged material, it seems appropriate to use the pneumatic flow mixing method. Another option, if there is quite some time available, is to mix the stabilising agents in-situ with the dredged material.
4.8 PRELIMINARY CONCLUSIONS BASED ON THE LITERATURE STUDY

Economic considerations play a vital role in choosing the right additives. Gypsum appears to be as expensive as cement, but is inferior when the increase in strength is considered. Lime appears to have similar qualities as cement, but is more expensive. As lime is one of the ingredients of Portland clinker, the main raw material used in Portland cement, it is unlikely that a situation will occur in which lime will be less expensive than cement. It is therefore considered to be justifiable to focus on cement in the remainder of this research.

Based on the literature study, it can be concluded that the following additives are secondary additives to cement:

- Fly ash;
- Biomass ash;
- Cement kiln dust;
- Ground-granulated blast furnace slag

The combination of these additives and Portland cement can be regarded as a cement (as is stated in the Eurocode), being CEM II or CEM III when fly ash or GGBS is used.

The potential of bentonite as an additive has, however, not been researched yet. Bentonite has the potential to increase the liquid limit of the dredged material tremendously. Combined with cement, a larger increase in strength could be achieved than with cement only as an additive. This assumption finds its basis in equations (5), (6) and (7), regarding the relationship between the liquidity index and the undrained shear strength. A clear disadvantage of the use of bentonite is its price.

The behaviour of cement and lime-stabilised dredged material can be traced back to the way it interacts with the clay particles and water. As can be explained by the double diffuse layer model, cement and lime cause flocculation. Both crystallise water and induce an exothermic reaction, decreases the pore water. The clusters created by the additives cause an increase in bound water and thus a decrease in free water. At the point where there is no free water available anymore, the cement or lime starts to crystallise bound water. Binding water initially causes an increase in liquid and plastic limit. When more cement or lime is added than is needed for the flocculation process, the plastic limit stabilises and the liquid limit slightly decreases. The decrease in liquid limit is due to a reduced surface activity of the clusters, caused by the precipitation of cementitious products on the clusters. A parameter called the normalised change in bound water content was introduced. It can be used to describe the increase in yield strength and cohesion. These descriptions show that the yield strength and cohesion only start
to increase after a certain water content. The soil needs to gain a solid structure before a yield strength can start to develop.

Strength is the most important parameter in this study. Based on the data produced by Sparrevik & Kvennås (2008), it can be established that, even for the weakest samples (Bergen), the undrained shear strength reaches a satisfactory level when using cement to stabilise the dredged material. What remains unclear, is what exactly governs the increase in strength when using cement as a stabiliser.

There appears to be no relationship between the hydraulic conductivity and the cement content. Whether the cement has any effect on the material regarding the hydraulic conductivity, depends on the material itself. If there is an effect, it can be increasing or decreasing the hydraulic conductivity. The hydraulic conductivities registered by Sparrevik & Kvennås (2008) are so small, that the entire dataset could easily be influenced by relatively large errors. Many insecurities regarding the dataset cause the conclusion to be that nothing explicit can be concluded from the data provided.

Various types of equipment and deposition methods have been used in the past. Lately, a new method has emerged in Japan. This method uses pneumatic mixing and can achieve a rate of 300 m³ per hour. This rate is not sufficient, when the research objectives are considered, unless multiple units would be used.
5 EXPERIMENTAL STUDY

5.1 INTRODUCTION

The experimental study is set up to test the effect of cement and bentonite on dredged material. The dredged material used in these experiments must have such properties that it can be deposited at high rates. This means that it must meet density, yield stress and viscosity requirements for dredging pumps to be able to transport the dredged material over relatively small distances.

Cement is chosen for these experiments as the literature study has shown that it has the most favourable properties of all additives. It is relatively inexpensive and shows the highest increase in strength. Next to that, it is also the additive that is most common in literature and therefore most of the data available regards cement.

Bentonite is chosen because there is little data available about the use of bentonite as an additive to dredged material. A positive quality of bentonite is the high liquid limit of the material. This can cause a dredged material to gain strength at a higher water content when bentonite is added.

Three types of clay are used for this experimental study. One is provided by the Delft University of Technology, two are provided by Boskalis. The first is a clay produced by VE-KA. The two clays provided by Boskalis are samples from the Markermeer (a lake in the Netherlands), named MM2 and MM5 and have been tested by Boskalis Environmental in their laboratory as well. The experimental study was performed entirely at the geotechnical laboratory of the Delft University of Technology.

5.2 METHODOLOGY

The three types of clay were first characterised. Their water contents (as they were), liquid limit, plastic limit and grain size distribution were determined. The liquid limit has been determined using the fall cone test. Both the liquid and plastic limit have been determined according to BS 1377:Part 2:1990. The grain size distribution has been determined by wet sieving of the material. The results can be seen in Table 8 and Figure 49.
As can be seen in Table 8, the liquid limits vary quite a bit. The plastic limits are more consistent, making the plasticity index vary as well.

Based on these properties, the position of the soils in the Casagrande chart can be determined. It can be seen in Figure 48. In this graph, VE-KA and MM2 both classify as a high plasticity clay, whereas MM5 classifies as a high plasticity silt. As the VE-KA clay is an engineered clay, it has a marginal organic matter content. The organic matters content of MM2 and MM5 are 8.5% and 5.3%, respectively (Van Bergeijk, 2015).

<table>
<thead>
<tr>
<th>Clay</th>
<th>LL</th>
<th>PL</th>
<th>Ip</th>
</tr>
</thead>
<tbody>
<tr>
<td>VE-KA</td>
<td>55.8</td>
<td>21.9</td>
<td>33.9</td>
</tr>
<tr>
<td>Bentonite</td>
<td>356.0</td>
<td>41.0</td>
<td>314.0</td>
</tr>
<tr>
<td>MM5</td>
<td>74.4</td>
<td>39.6</td>
<td>34.8</td>
</tr>
<tr>
<td>MM2</td>
<td>112.7</td>
<td>42.8</td>
<td>69.9</td>
</tr>
</tbody>
</table>

Table 8: the Atterberg limits for all materials used

---

Figure 48: the Casagrande chart of the VE-KA, MM2 and MM5 soils

The grain size distribution can be seen in Figure 49. It appears that a higher percentage of clay in a soil correlates to a high liquid limit and a higher plasticity index.
After the determination of the properties of the clays, measurements on the self-weight consolidation were performed. It was tested what the self-weight consolidation density was after 24 hours. Based on those results slurries were created with densities that were tested in the viscometer, together with the self-weight consolidation samples. The results can be seen in Table 9. The viscosities have been determined using the FANN viscometer model 35, using the R1-B1-F1 combination (FANN Model 35 Viscometer Instruction Manual, No. 208878, Revision P, 2016). The manual provides methods to calculate the dynamic viscosity, plastic viscosity and yield point.

<table>
<thead>
<tr>
<th>Clay</th>
<th>Density [kg/m³]</th>
<th>Iₗ [-]</th>
<th>Determined plastic viscosity [mPa s]</th>
<th>Calculated plastic viscosity [mPa s]</th>
<th>Yield point [Pa]</th>
<th>Dynamic viscosity [mPa s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>VE-KA</td>
<td>1159.7</td>
<td>9.9</td>
<td>5.5</td>
<td>N/A</td>
<td>2.9</td>
<td>11.5</td>
</tr>
<tr>
<td>MM2</td>
<td>1126.9</td>
<td>5.3</td>
<td>6.5</td>
<td>6.3</td>
<td>5.3</td>
<td>17.5</td>
</tr>
<tr>
<td>MM5</td>
<td>1143.1</td>
<td>10.3</td>
<td>6</td>
<td>N/A</td>
<td>1.9</td>
<td>10</td>
</tr>
<tr>
<td>VE-KA</td>
<td>1296.4</td>
<td>4.3</td>
<td>12</td>
<td>12.6</td>
<td>18.7</td>
<td>51</td>
</tr>
<tr>
<td>MM2</td>
<td>1209.2</td>
<td>2.9</td>
<td>23</td>
<td>48.3</td>
<td>24.9</td>
<td>75</td>
</tr>
<tr>
<td>MM5</td>
<td>1290.6</td>
<td>3.7</td>
<td>29</td>
<td>21.0</td>
<td>28.3</td>
<td>88</td>
</tr>
</tbody>
</table>

Table 9: the determined viscosities using the FANN viscometer

From Table 9 it can be seen that, as expected, the viscosities and the yield point increase with decreasing liquidity index and increasing density for all samples. The calculated plastic viscosity is calculated using equation (11). As it appears, the formula proposed by Locat & Demers (1988)
works quite well on these samples, with only one value being completely off. It appears that a high liquid limit, combined with a lower liquidity index makes the estimation inaccurate.

If an imperfect vacuum of 80 kPa would be applied to pump the liquids mentioned in Table 9, the yield stresses of these liquids are presumed to be ignorable, due to their small values regarding the pump pressure. It is therefore safe to say that equations (8), (9) and (10) are applicable for an approximation of the pipe pressure losses. The purpose of these calculations is to check whether these materials can actually be pumped through a pipeline and if these pressure losses allow for solids to be added to the material. The results for pipes with a diameter of 0.5 metres, 3 m/s velocity and 1 metre, 1 m/s velocity can be seen in Table 10. These velocities and diameters ensure a flow of approximately 2000 m³ per hour.

<table>
<thead>
<tr>
<th>Clay</th>
<th>Density [kg/m³]</th>
<th>Self-weight consolidation time [days]</th>
<th>Viscosity [mPa s]</th>
<th>Pressure loss [Pa/m] D=0.5, v=3</th>
<th>Pressure loss [Pa/m] D=1, v=1</th>
</tr>
</thead>
<tbody>
<tr>
<td>VE-KA</td>
<td>1159.7</td>
<td>1</td>
<td>11.5</td>
<td>374.0</td>
<td>17.2</td>
</tr>
<tr>
<td>MM2</td>
<td>1126.9</td>
<td>1</td>
<td>17.5</td>
<td>366.7</td>
<td>17.1</td>
</tr>
<tr>
<td>MM5</td>
<td>1143.1</td>
<td>1</td>
<td>10</td>
<td>368.0</td>
<td>16.9</td>
</tr>
<tr>
<td>VE-KA</td>
<td>1296.4</td>
<td>14</td>
<td>51</td>
<td>436.1</td>
<td>21.1</td>
</tr>
<tr>
<td>MM2</td>
<td>1209.2</td>
<td>14</td>
<td>75</td>
<td>418.1</td>
<td>20.8</td>
</tr>
<tr>
<td>MM5</td>
<td>1290.6</td>
<td>14</td>
<td>88</td>
<td>449.37</td>
<td>22.5</td>
</tr>
</tbody>
</table>

Table 10: the pipe pressure losses at different diameters and velocities

From the results it can be seen that the denser slurries show some increase in pressure loss, when compared to the less dense slurries. According to these results, the pressure loss would be around 40 kPa over 100 metres for the pipe with a diameter of 0.5 metres and a pressure loss of only 2 kPa over 100 metres for the pipe with a diameter of 1 metre. The higher densities are therefore considered to be suitable for dredging equipment and thus acceptable for testing.

For the experimental study, a target density of 1300 kg/m³ was set for the VE-KA and MM5 clays. A target density of 1200 kg/m³ was set for the MM2 clay. The targeted properties are listed in Table 11.

<table>
<thead>
<tr>
<th>Clay</th>
<th>Target density [kg/m³]</th>
<th>Target water content [%]</th>
<th>Target Iₜ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>VE-KA</td>
<td>1300</td>
<td>169%</td>
<td>4.3</td>
</tr>
<tr>
<td>MM2</td>
<td>1200</td>
<td>254%</td>
<td>2.9</td>
</tr>
<tr>
<td>MM5</td>
<td>1300</td>
<td>166%</td>
<td>3.7</td>
</tr>
</tbody>
</table>

Table 11: the targeted properties of the slurries
Additional information in regarding the clays used can be found in the appendix.

The target additions are listed in Table 12. The cement additions were estimated based on the effect the cement and bentonite would have on the water content and the liquid limit. The aim was to lower the liquidity index of the majority of samples below 1. This ensured that almost all of the samples were solid and had a sufficient undrained shear strength after curing. Compared to the samples used by Sparrevik & Kvennås (2008), the cement contents are rather high. There are two reasons for this difference. Firstly, the water contents of these samples are higher than the samples used by Sparrevik & Kvennås (2008). Secondly, the cement contents were exaggerated on purpose to obtain more data. In the progress of the study it was however found that lower contents than used in this study would have ensured the liquid limit would have been reached.

<table>
<thead>
<tr>
<th>VE-KA</th>
<th>MM2</th>
<th>MM5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement [kg/m³]</td>
<td>Bentonite [kg/m³]</td>
<td>Cement [kg/m³]</td>
</tr>
<tr>
<td>200</td>
<td>0</td>
<td>250</td>
</tr>
<tr>
<td>300</td>
<td>0</td>
<td>350</td>
</tr>
<tr>
<td>400</td>
<td>0</td>
<td>450</td>
</tr>
<tr>
<td>500</td>
<td>0</td>
<td>500</td>
</tr>
<tr>
<td>100</td>
<td>100</td>
<td>50</td>
</tr>
<tr>
<td>200</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>200</td>
<td>200</td>
<td>100</td>
</tr>
<tr>
<td>300</td>
<td>200</td>
<td>200</td>
</tr>
<tr>
<td>0</td>
<td>100</td>
<td>0</td>
</tr>
<tr>
<td>0</td>
<td>150</td>
<td>0</td>
</tr>
<tr>
<td>0</td>
<td>200</td>
<td>0</td>
</tr>
<tr>
<td>0</td>
<td>300</td>
<td>0</td>
</tr>
</tbody>
</table>

Table 12: the target additions for the tests on the different clays

After the slurries were mixed with the cement and bentonite, they were placed in cups. These cups have a height of 66.9 millimetres and a diameter of 51.66 millimetres. The cups have a lid and can thus be sealed off. The size of the cups is comparable to the size of the cups used in the fall cone test and are therefore used as such. The samples that involved cement were made twice, to be able to test these samples at a curing time of 7 days and a curing time of 28 days.

At the start of the experimental study, it was thought that the samples could all be tested for the undrained shear strength using the fall cone test, based on NPR-CEN-ISO/TS 17892-6:2004,IDT.
However, quite a lot of samples had a penetration too low to derive the undrained shear strength. Therefore, same samples are tested using the unconfined compressive test. The liquid limits have been determined using the fall cone test. Both the liquid and plastic limits have been determined according to BS 1377:Part 2:1990.

The results of the fall cone test are converted into the undrained shear strength as follows:

$$c_u = cg \frac{m}{i^2}$$ (24)

Where $c$ is a constant, depending on the tip angle and whether a remoulded or undisturbed sample is used. Because all tests are performed with a 30° cone, the constant is equal to 0.8 for remoulded samples and 1.0 for undisturbed samples. The factor $m$ is the mass of the cone in grams, which is standardised at 80 grams. If weight is added to the cone, it must be added to $m$. The penetration is represented by $i$ in millimetres. The outcome, $c_u$, is in kPa. The weights added are one kilogramme and 0.5 kilogrammes.

The undrained shear strength for the undisturbed sample is to be corrected as follows:

$$c_{u(\text{corr})} = c_u \cdot \mu$$ (25)

With:

$$\mu = \left( \frac{0.43}{L} \right)^{0.45} \quad 0.5 \leq \mu \leq 1.2$$ (26)

Because of the length-to-diameter ratio of the samples, the unconfined compressive strength found, has to be converted to an equivalent unconfined compressive strength with a length-to-diameter-ratio of 2. The following empirical formula is used (Güneyli & Rüsen, 2016):

$$UCS_{\text{corr},2} = \frac{UCS}{1.83 - 0.39 \frac{L}{D}}$$ (27)

After which equation (1) can be applied.

5.3 RESULTS

5.3.1 STRENGTH

The fall cone test was used to find the undrained shear strength for the undisturbed samples. In this case, “undisturbed” means that the cement stabilised samples were not remoulded after curing. NPR-CEN-ISO/TS 17892-6:2004,IDT defines the penetration for which the standard is
valid to be between 5 and 20 millimetres. This poses a problem, as most penetration values of the undisturbed cement stabilised samples are below 5 millimetres. After this was discovered during the tests on the samples that had cured for 7 days, it was decided to use an unconfined compressive test instead. However, not all samples could be released from their cups without disturbing the samples. As this was not anticipated, no lubricant had been applied to the cups. Some data is therefore missing for the cement stabilised samples and the cement-bentonite stabilised samples.

The results of the tests to determine the undrained shear strength of the cement stabilised samples can be seen in Figure 50. The VE-KA samples and MM2 samples that have cured for 28 days have multiple useable data points. From the graph one might think that the relationship between the cement content and the undrained shear strength is nearly linear, but it can easily be observed that this cannot be the case. At a cement content of zero, the undrained shear strength is very small. Therefore, the relationship between the cement content and the undrained shear strength cannot be linear. It can be described as an exponential, power law or polynomial equation.

The research objective states that an unconfined compressive strength in the field of 150 kPa needs to be reached. Most sources, including EuroSoilStab, agree that a laboratory strength of at least three times the field strength needs to be achieved (Kitazume & Satoh, 2003). Using equation (1), it can easily be determined that this means the field undrained shear strength needs to be 75 kPa, meaning that the laboratory undrained shear strength needs to be 225 kPa. This value is reached for a cement content of 200 kg/m$^3$ after 28 days, but in most cases not after 7 days for this cement content. The cement contents used in this experimental study can are higher than those used in the literature study. The contents were chosen to ensure that the resulting mixtures would have a water content below their liquid limits. The effects on the Atterberg limits and undrained shear strength were higher than expected, resulting in excessively high strengths at higher cement contents.
The undrained shear strength of the cement stabilised samples has been plotted versus the water-cement ratio, as well, in Figure 51. This form of representation is analogous to the form proposed by Horpibulsuk et al. (2003). The water-cement ratio as an input variable is meant to create a function with two variables (initial water content and cement content), as the water content is certainly of influence on the resulting strength. Dividing the initial water content by the cement content gives the same water-cement ratio as is common practice in the concrete industry. As the water content was not varied in the course of the experimental study, the initial water content is not a variable in this function. Therefore, the use of the water content as an additional variable cannot be evaluated in this manner. The graph in Figure 51 is therefore very similar to the graph in Figure 50.
In Figure 52 the undrained shear strengths of the bentonite stabilised samples are shown. The bentonite stabilised samples could be tested using the fall cone test, as the penetration was sufficient, without adding weight to the cone. It can be observed that bentonite in itself is not nearly sufficient to gain an undrained shear strength remotely close to the desired undrained shear strength. The different clays react differently to the same amount of bentonite. The VE-KA clay and MM5 clay start off with approximately the same water content (168.85% and 166.01%, respectively). The VE-KA clay has a lower liquid limit than the MM5 clay, being 55.76% and 74.4%, respectively. The VE-KA clay thus has a higher liquidity index before the bentonite is added, but the undrained shear strength increases (and thus the liquidity index decreases according to equations (5), (6) and (7)) rapidly when more bentonite is added.
Figure 52: the undrained shear strength of bentonite stabilised samples versus the bentonite content

The results of the strength tests on the cement-bentonite stabilised samples are shown in Figure 53. As with the cement stabilised samples, not all data could be retrieved, because the fall cone test proved to be an unreliable testing method for determining the undrained shear strength of these samples. Intact cement-stabilised samples become more rock-like than clay-like. The method is devised for clays and thus the fall cone test does not work properly for these samples. The undrained shear strength is plotted as a function of the cement content and not as a function of the total additive content. It has been observed in Figure 52 that the effect bentonite has on the undrained shear strength is very small when compared to the effect cement has on the undrained shear strength. Including bentonite and using the total additive content versus the undrained shear strength would give a scattered graph. The cement-bentonite samples give somewhat lower undrained shear strengths at the same cement contents than the samples that were stabilised with cement only. This can be explained by the addition of bentonite. Bentonite is also a soil and adding bentonite to the mixture thus decreases the cement to soil ratio.
5.3.2 Atterberg limits

5.3.2.1 Cement

As was observed in the literature study, the additives impose a change in Atterberg limits on the slurries. The liquid limit of the cement stabilised samples can be observed in Figure 54. In this graph, the liquid limit decreases when the cement content increases. At first glance, the relationship between the liquid limit and the cement content is linear, but this cannot be true. The liquid limit at zero cement content must be equal to the liquid limit found during the characterisation tests, summarised in Table 8.

The decrease in liquid limit for higher cement content is assumed to be caused by the decrease in surface activity of the clay-cement clusters, as mentioned in 4.3. This makes the samples with a higher cement content less cohesive and thus the liquid limit decreases. Before the liquid limit decreases from a cement content of 200 kg/m$^3$ onwards, it must have increased from the level stated in Table 8. It is quite remarkable that all slurries appear to have a similar increase in liquid limit at similar cement contents. From these data, a curing time dependent effect cannot be observed.
Figure 54: the liquid limit of the cement stabilised samples versus the cement content

The results of the experiments determining the plastic limit of the cement stabilised samples can be seen in Figure 55. A curing time effect cannot be observed for these data as well. There appears to be an increase in plastic limit before a cement content of 200 kg/m$^3$ is reached. After that, the plastic limit shows marginal discrepancies from the plastic limit at 200 kg/m$^3$. It appears that, as was discussed in 4.3, the plastic limit does not show larger increases or decreases anymore at higher cement contents.

Figure 55: the plastic limit of the cement stabilised samples versus the cement content
The plasticity index of these samples can be seen in Figure 56. As can be seen in equation (2), the plasticity index is the difference between the liquid limit and the plastic limit. As the liquid limit declines with increasing cement content and the plastic limit does only marginally increase or decrease, the decrease in liquid limit governs the decrease in plasticity index.

![Figure 56: the plasticity index of cement stabilised samples versus the cement content](image)

The liquidity indices for the cement stabilised samples can be seen in Figure 57. As can be seen in equation (3), the liquidity index of a soil is the difference between the water content and the plastic limit, divided by its plasticity index, which in turn is the difference between the liquid limit and the plastic limit, as per equation (2). It can be seen that, although the liquid limit is decreasing, the water content is decreasing faster, as the liquidity index decreases as well. This is caused by the addition of solids, being cement, and the hydration reaction of the cement.
5.3.2.2 Bentonite

The results for the liquid limit of the bentonite stabilised samples can be seen in Figure 58. Unsurprisingly, the clay with the lowest liquid limit, shows the largest increase when bentonite is added. The effect caused by the bentonite is quite straightforward: it has a higher liquid limit than the slurries used and will therefore, upon addition, raise the liquid limit of the clays. This process appears to be nonlinear, however.

The results for the plastic limit of the bentonite stabilised samples can be seen in Figure 59. There appears to be little to no change with increasing bentonite content. This was expected for
the MM2 and MM5 clays, since their plastic limit is similar to the plastic limit of the bentonite used. On the contrary: the plastic limit of the MM5 clay appears to have dropped by 10% (percentage point). This can also be caused by measurement errors. The plastic limit of the VE-KA clay and the MM2 clay neither increases nor decreases. This is odd, as the bentonite over solids ratio for the VE-KA clay is 0.87 for a bentonite content of 298 kg/m$^3$. A change in plastic limit was expected.

![Graph showing plastic limit of the bentonite stabilised samples versus the bentonite content]

Figure 59: plastic limit of the bentonite stabilised samples versus the bentonite content

Because of the results for the plastic limit, the results for the plasticity index of the bentonite stabilised samples, which can be seen in Figure 60 look much like the results for the liquid limit. This means that the VE-KA clay shows a relatively fast increasing plasticity, whereas especially the MM2 clay has a slowly developing plasticity, due to a shallow increase in liquid limit.
The liquidity index, as per equation (3) a function of the water content, plastic limit and plasticity index, shows the type of behaviour one would expect after reviewing the graphs for the liquid and plastic limit and the plasticity index. The results can be seen in Figure 61. Since the plastic limit does not appear to change with an increasing bentonite content, the liquidity index is dependent on the change in water content and the change in liquid limit. The former decreases due to the added dry bentonite. The latter increases due to the change in liquid limit the bentonite induces.
5.3.2.3 Cement and Bentonite

Combining cement and bentonite as additives to dredged material implies that it will combine the properties of both additives. It has been observed that both bentonite and cement increase the liquid limit of the material. Cement increases the plastic limit of the material, whereas bentonite appears not to.

Results for the use of cement and bentonite on the VE-KA clay can be seen in Table 13. It can be observed that the plastic limit has increased approximately 5% more than is the case with the cement stabilised samples. However, this effect only occurs at a curing time of 7 days and is therefore possible that this effect can be contributed to measurement errors. The increase in liquid limit is larger for both curing times. The plastic limit is slightly larger after 28 days of curing, which is caused by further hardening of the cement, but could also be contributed to measurements errors.

<table>
<thead>
<tr>
<th>Curing time</th>
<th>Cement content [kg/m³]</th>
<th>Bentonite content [kg/m³]</th>
<th>Liquid limit [%]</th>
<th>Plastic limit [%]</th>
<th>Plasticity index [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>7 days</td>
<td>99.2</td>
<td>103.0</td>
<td>124.6</td>
<td>81.5</td>
<td>43.1</td>
</tr>
<tr>
<td></td>
<td>201.9</td>
<td>102.1</td>
<td>119.0</td>
<td>90.1</td>
<td>28.9</td>
</tr>
<tr>
<td></td>
<td>201.4</td>
<td>205.1</td>
<td>120.6</td>
<td>86.1</td>
<td>34.5</td>
</tr>
<tr>
<td></td>
<td>300.6</td>
<td>198.2</td>
<td>105.9</td>
<td>85.0</td>
<td>20.9</td>
</tr>
<tr>
<td>28 days</td>
<td>103.2</td>
<td>102.0</td>
<td>139.3</td>
<td>83.1</td>
<td>56.2</td>
</tr>
<tr>
<td></td>
<td>200.5</td>
<td>101.3</td>
<td>117.5</td>
<td>94.3</td>
<td>23.1</td>
</tr>
<tr>
<td></td>
<td>199.5</td>
<td>198.6</td>
<td>114.7</td>
<td>91.4</td>
<td>23.3</td>
</tr>
<tr>
<td></td>
<td>298.6</td>
<td>197.9</td>
<td>104.6</td>
<td>83.9</td>
<td>20.7</td>
</tr>
</tbody>
</table>

Table 13: the cement contents, bentonite contents and Atterberg limits for curing times of 7 and 28 days for the VE-KA samples

The test results for the tests on the MM2 samples can be seen in Table 14. The plastic limit of these samples are lower than the cement stabilised samples with the same curing time at a curing time of 7 days. At a curing time of 28 days, they appear to be similar. For both curing times, the liquid limit is somewhat higher.
The test results of cement-bentonite stabilised MM5 clay can be seen in Table 15. Note that the different cement and bentonite contents are there because the samples for 7 and 28 days are two different samples. The results seem to confirm what has been seen for the VE-KA clay. The plastic limit of the MM5 clay is higher as well at a curing time of 7 days than when only cement is used to stabilise the MM5 clay. Where this effect seemed to disappear after 28 days for the VE-KA clay, the effect remains in the case of the MM5 clay. After both 7 and 28 days, there appears to be little difference in liquid limit between the cement stabilised MM5 clay and the cement-bentonite stabilised MM5 clay. The addition of bentonite next to cement thus appears to have little effect on the liquid limit of the MM5 clay.

<table>
<thead>
<tr>
<th>Curing time</th>
<th>Cement content [kg/m³]</th>
<th>Bentonite content [kg/m³]</th>
<th>Liquid limit [%]</th>
<th>Plastic limit [%]</th>
<th>Plasticity index [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>7 days</td>
<td>53.0</td>
<td>51.2</td>
<td>170.8</td>
<td>92.3</td>
<td>78.6</td>
</tr>
<tr>
<td></td>
<td>102.3</td>
<td>102.2</td>
<td>163.9</td>
<td>96.8</td>
<td>67.1</td>
</tr>
<tr>
<td></td>
<td>103.1</td>
<td>153.9</td>
<td>164.4</td>
<td>85.9</td>
<td>78.5</td>
</tr>
<tr>
<td></td>
<td>202.9</td>
<td>100.4</td>
<td>149.8</td>
<td>94.7</td>
<td>55.1</td>
</tr>
<tr>
<td>28 days</td>
<td>69.0</td>
<td>50.4</td>
<td>178.2</td>
<td>99.9</td>
<td>78.3</td>
</tr>
<tr>
<td></td>
<td>102.4</td>
<td>103.0</td>
<td>184.2</td>
<td>94.1</td>
<td>90.2</td>
</tr>
<tr>
<td></td>
<td>101.7</td>
<td>153.0</td>
<td>172.2</td>
<td>95.7</td>
<td>76.5</td>
</tr>
<tr>
<td></td>
<td>202.9</td>
<td>100.9</td>
<td>147.5</td>
<td>109.3</td>
<td>38.3</td>
</tr>
</tbody>
</table>

Table 14: the cement contents, bentonite contents and Atterberg limits for curing times of 7 and 28 days for the MM2 samples

The results for the liquidity index for the cement-bentonite stabilised samples, to be found in Figure 62, are very similar to Figure 57. Again, the liquid limit seems to decrease slower than the water content is decreasing.
Figure 62: the liquidity index of the cement-bentonite stabilised samples versus the cement content
6 DISCUSSION

6.1 THE EFFECT OF CEMENT ON DREDGED MATERIAL

6.1.1 STRENGTH

The strength increment caused by adding cement to the clay slurries can be seen in Figure 50. The cement content is expressed in kg/m³, as is common in literature. To derive proper trend lines, it is easier to ensure the main variable, the cement content, is dimensionless. This can be achieved by defining the cement content in the same fashion as the water content, as follows, in which \( m_s \) is the initial solids mass, before cement addition:

\[
a_c = C = \frac{m_c}{m_s}
\]  

This relation could also have been deduced from equation (18). Not only is it analogous to that equation, it is also in line with other literature (Chew, Kamruzzaman, & Lee, 2004). Zhu, Zhang & Chiu (2007) use \( a_c \) to denote the cement content, whereas Horpibulsuk et al. (2003) uses \( C \). As the initial water content was not varied during the experiments, the initial water content is not a variable in the analysis of the results. From the data, it can be deduced that a polynomial of the following form fits the data:

\[
c_u = aC^2 + c_u(0)
\]  

This equation has been used on the cement stabilised VE-KA samples and the cement stabilised MM2 samples, which both had cured for 28 days. The results can be seen in Figure 63. To check whether the equation applies to data from other sources, the data from Sparrevik & Kvennås (2008) is used. For all the samples discussed in 4.4, the results for the standard cement and sulphate cement are taken and plotted in Figure 64. Trend lines in the form suggested in equation (29) are drawn through the results.

The factors \( a \) for both the results from the experimental study and the literature study, as well as the factors \( R^2 \) and the initial undrained shear strengths, can be seen in Table 16. The factors \( a \) have been determined experimentally. The undrained shear strengths at zero cement content have been found in the data provided by Sparrevik & Kvennås (2008) for the data from the literature study. For the VE-KA and MM2 clay slurries they have been estimated by using equation (6).

What combination of parameters can describe the factor \( a \) is unclear. As the cement content is a dimensionless parameter, the factor \( a \) must have the dimension of a stress, like the undrained
shear strength at zero cement content. It is clear from Figure 64 that the type of cement used is one of the factors that plays a role. The strength of the clay itself is also a factor of significance, as achieved undrained shear strengths differ largely between clays. The amount of water that the cement used binds in combination with the dredged material is also a large factor of influence. In both graphs it is shown that the undrained shear strength of 75 kPa, which is wanted, is reached at different cement contents for different slurries.

Another remark is that the proposed equation cannot be expected to work at higher cement contents than are used in this study. A very high cement content would cause the undrained shear strength to rapidly increase to extremely high levels in this equation, which at some point would become unrealistic. Furthermore, the equation is not suitable to describe just cement, without any soil, as that would mean that the cement content would reach infinity.

![Graph showing the undrained shear strength versus the cement content with trend lines for the VE-KA and MM2 clay slurries](image)

**Figure 63:** The undrained shear strength versus the cement content with trend lines for the VE-KA and MM2 clay slurries
Figure 64: The undrained shear strength versus the cement content for the Bergen, Grenland and Sandvika samples with trend lines.

<table>
<thead>
<tr>
<th>Sample</th>
<th>$\alpha$ [kPa]</th>
<th>$c_0(0)$ [kPa]</th>
<th>$R^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bergen standard cement</td>
<td>0.028</td>
<td>0.1</td>
<td>0.991</td>
</tr>
<tr>
<td>Bergen sulphate cement</td>
<td>0.07</td>
<td>0.1</td>
<td>0.999</td>
</tr>
<tr>
<td>Sandvika standard cement</td>
<td>0.09</td>
<td>0.1</td>
<td>0.999</td>
</tr>
<tr>
<td>Sandvika sulphate cement</td>
<td>0.16</td>
<td>0.1</td>
<td>0.990</td>
</tr>
<tr>
<td>Grenland standard cement</td>
<td>0.146</td>
<td>0.1</td>
<td>0.922</td>
</tr>
<tr>
<td>Grenland sulphate cement</td>
<td>0.31</td>
<td>0.1</td>
<td>0.999</td>
</tr>
<tr>
<td>VE-KA 28 days</td>
<td>0.11</td>
<td>0.041</td>
<td>0.993</td>
</tr>
<tr>
<td>MM2 28 days</td>
<td>0.085</td>
<td>0.109</td>
<td>0.999</td>
</tr>
</tbody>
</table>

Table 16: A summary of the $\alpha$ factors, $c_0(0)$ and $R^2$ for the trend lines shown in Figure 63 and Figure 64.

6.1.2 Atterberg Limits

As can be seen in Figure 57, Figure 61 and Figure 62 the objective stated in 5.2 to make sure all samples have a water content below the liquid limit (a liquidity index of 1 or smaller), has been achieved. It does necessitate rather high cement contents, but at lower cement contents the mixture would be in danger of liquefying upon remoulding.

When the theory regarding the Atterberg limits of Chew, Kamruzzaman & Lee (2004), which can be found in 4.3, is followed, the liquid limit should reach a peak and then start to decrease with increasing cement content. The plastic limit should reach a peak level, after which it maintains that particular level. As can be seen in Figure 65, the change in liquid limit with respect to the original liquid limit of the soil is, for these cement contents, already in its degrading part. That
means that cementitious products have already started to precipitate on the clay-cement clusters (Chew, Kamruzzaman, & Lee, 2004).

Figure 65: The change in liquid limit for cement stabilised samples with respect to the original liquid limit of the soil in percentage points.

That would mean that the change in plastic limit is constant for these cement contents. As it appears in Figure 66, this seems to be the case. The change in plastic limit varies somewhat, but this can also be explained by measurement errors, as the method to determine the plastic limit leaves quite some room for these errors.

Figure 66: The change in plastic limit for cement stabilised samples with respect to the original plastic limit of the soil in percentage points.

It can be derived from Figure 23 in 4.3 that at the point the difference in plasticity index with regard to the original plasticity index becomes negative, the liquid limit starts to decline,
whereas the plastic limit shows little increase (or decrease). To spot that point in the samples used in this experimental study, the change in plasticity index is plotted in Figure 67. Following the theory proposed by Chew, Kamruzzaman & Lee (2004), the decline in liquid limit starts at roughly 40% cement content for all samples after 28 days of curing. This is significantly higher than the 10% found by Chew, Kamruzzaman & Lee (2004). The experiments performed for this study have a significantly higher water content, being a possible explanation for this deficit.

![Figure 67: The change in plasticity index for cement stabilized samples with respect to the original plasticity index of the soil in percentage points](image)

The liquid and plastic limit are measures of how much water a clayey soil can bind before reaching a certain state. Thus, when cement binds water in the cement matrix, the liquid and plastic limit rise. Therefore, the increase in liquid and plastic limit are actually an increase in bound water content and will henceforth follow equation (14) in 4.3. The change in liquid and plastic limit can thus be approximated as the change in bound water content, as follows:

\[ \Delta LL \approx \Delta PL \approx \Delta m_{bw} = k_3(1 - \exp(-k_2a_c)) \quad 0 \leq a_c \leq a_{c,eq} \quad (30) \]

With \( a_{c,eq} \) being the equilibrium cement content, the point where the liquid and plastic limit stop increasing and the liquid limit starts to decrease. The decrease in liquid limit, being caused by the deposition of cementitious products on the clay-cement clusters, reducing the surface activity, is a process different from the change in bound water content. It thus can be described with a different equation. Curing time does not seem to affect the change in liquid and plastic limit as much. This implies that the amount of pozzolanic materials in the clays used is quite low.
The decreasing liquid limit with cement contents higher than the equilibrium cement content can be described as a declining linear function of the following form:

\[ \Delta LL = -Aa_c + B \quad a_c > a_{c,eq} \]  

(31)

Fitting this linear function and a form of equation (30) results in the graph in Figure 68. For \( k_3 \) and \( k_2 \) values similar to those found by Zhu, Zhang and Chiu (2007) are used. For \( k_3 \) the value found for \( \Delta P_{eq} \) is used, as that is the value the plastic limit will eventually reach. As can be seen in the graph, there is no way to verify if equation (30) and the constants assumed actually apply to the materials used. This is hypothesised and is therefore a point of discussion left open. The values \( k_2, k_3, A, \) and \( B \) are listed in Table 17, as well as the values \( R^2 \) for the linear parts, being equation (31).

![Figure 68: the trend line for the samples and a form of equation (30) for the change in liquid limit](image)

<table>
<thead>
<tr>
<th>Clay</th>
<th>( k_2 [-] )</th>
<th>( k_3 [-] )</th>
<th>( A [-] )</th>
<th>( B [-] )</th>
<th>( R^2 [-] )</th>
</tr>
</thead>
<tbody>
<tr>
<td>VE-KA</td>
<td>0.12</td>
<td>59.24</td>
<td>0.38</td>
<td>76.07</td>
<td>0.81</td>
</tr>
<tr>
<td>MM2</td>
<td>1</td>
<td>57.32</td>
<td>0.67</td>
<td>64.02</td>
<td>0.98</td>
</tr>
<tr>
<td>MM5</td>
<td>0.09</td>
<td>40.82</td>
<td>0.44</td>
<td>59.91</td>
<td>0.90</td>
</tr>
</tbody>
</table>

Table 17: the values for constants in the equations (30) and (31) for the different clays, as well as \( R^2 \) for equation (31), using the constants found

The process increasing the liquid limit and the process decreasing the liquid limit are two different processes, justifying the use of two different equations. However, equation (31) can only be used for this particular range as the liquid limit cannot perpetually decline. It is therefore believed that this equation is actually the linear part of an asymptotic curve that has its asymptote at the equilibrium plastic limit. This means that, theoretically, when the mass of
the cement is several orders of magnitudes higher than the amount of soil, the liquid limit will reach the equilibrium plastic limit, as cement is non-plastic.

The equilibrium plastic limit is that plastic limit at which it does not change or only changes marginally, as was discussed in 4.3. Marginal changes, being both increases and decreases, can be caused by measuring errors, mostly due to the testing method, which is prone to errors. The assumed equilibrium plastic limit is plotted in Figure 69, along with equation (30), which describes the (assumed) change in bound water content.

These models for the liquid and plastic limit are based on a combination of literature and this experimental study. It is recommended that these models are tested and verified in a follow-up study, using the following equations:

\[
\Delta LL = \begin{cases} 
    k_3 (1 - \exp(-k_2 a_c)) & 0 \leq a_c \leq a_{c,eq} \\
    -Aa_c + B & a_c > a_{c,eq}, \: LL \geq PL
\end{cases}
\]

(32)

\[
\Delta PL = k_3 (1 - \exp(-k_2 a_c))
\]

(33)

It has to be emphasised that this is hypothetical and a point open for discussion and further research. It is based on earlier research on the change in bound water content, combined with research on the liquid limit and plastic limit change when cement is used as an additive to dredged material.
6.2 The Effect of Bentonite on Dredged Material

6.2.1 Atterberg Limits
Research on the development of the change in liquid limit when bentonite is added to soils has been performed by Sivapullaiah & Sridharan (1985). They defined the bentonite content of a soil as being the mass solids of bentonite divided by the total mass solids, being:

\[ Be = \frac{m_{bentonite}}{m_{total}} \quad 0 \leq Be \leq 1 \quad (34) \]

When using this relation to describe the amount of bentonite in the mixture, they found that the liquid limit does not increase linearly with increasing bentonite content. When the results from the experimental study are described in the same way, a similar non-linear increase can be observed, as can be seen in Figure 70.

![Figure 70: the liquid limit versus the bentonite content, expressed as the mass bentonite over mass total solids, including trend lines, for the samples from the experimental study](image)

The results found by Sivapullaiah & Sridharan (1985) are thus reproducible to a certain extent. Results comparable to Figure 59 were also found in their study. It shows that at bentonite contents below 40%, little change in plastic limit can be observed. The discrepancy of the plastic limit of MM5 with the original plastic limit is most likely caused by measurement errors, as there is no other viable explanation to what could have caused the drop in plastic limit. Note that a measurement error can also have occurred when characterising MM5.

The trend lines are composed as follows:

\[ LL(Be) = (LL_{bentonite} - LL_{soil})Be^{1.1\frac{LL_{soil}}{100}} + LL_{soil} \quad 0 \leq Be \leq 1 \quad (35) \]
With a value of $R^2$ of 0.88.

This trend line equation can also be used on the data from Sivapullaiah & Sridharan (1985). The results can be seen in Figure 71. The overall value for $R^2$ is 0.88 as well. The trend line formula seems to estimate the liquid limits of the different soils mixtures quite well, only being off slightly at higher bentonite contents for Silt I. It has to be noted that this trend line has only been tested on soil mixtures with two different bentonites and nothing – at this point - can be said with certainty about its applicability when not bentonite, but a clay with an entirely different liquid limit is used.

![Figure 71: the liquid limit versus the bentonite content, expressed as the mass bentonite over mass total solids, including trend lines, for the samples from Sivapullaiah & Sridharan (1985)](image)

The trend line appears to work on two independent sets of experimental data. As more confirmation of the applicability of such a trend line is wanted, it is recommendable to perform more tests on soil mixing with bentonite.

6.2.2 **STRENGTH**

The strength increase of the samples is mainly due to a decrease in water content and an increase in liquid limit. This was the original reason to add bentonite to the dredged material. As water content and liquid limit form the liquidity index together with the plastic limit, it would be expected that the increase in undrained shear strength is related to the decrease in liquidity index. This idea is supported by the graph shown in Figure 72. The Leroueil formula, equation (5), is also plotted in this graph to compare it to the results found in the experimental study. The value for $R^2$ is rather low, 0.40, which is primarily caused by the high discrepancy with the values found for the VE-KA slurry.
The correlation provided by Leroueil appears to be useful as a good estimation. However, liquidity indices of different mixtures for these tests had to be determined separately, which means determining the water content, the liquid limit and the plastic limit for every mixture. Using equations (35) the liquid limit can easily be estimated, when the liquid limits of the original material and the bentonite used are known. The water content can be calculated based on the initial water content and the amount of solids (bentonite) added to the mixture. Based on Figure 59, it can be said that the plastic limit increases or decreases marginally with regard to the original plastic limit of the soil, up to a bentonite content of 50%. The plastic limit can, for an estimation, thus be assumed to be constant.

6.3 THE EFFECT OF COMBINING CEMENT AND BENTONITE

6.3.1 ATTERBERG LIMITS
As was discussed in 6.2.1, the liquid limit of bentonite stabilised dredged material increases with bentonite content. A slightly more complex relationship with the liquid limit with regard to cement has been discussed in 6.1.2. If these processes were independent, it would be expected that the effects of both additives can be added up. However, these processes do not appear to be independent. In Table 18, it can be seen that these processes are indeed not independent, as the liquid limit is between the expected values of cement and bentonite. The expected values are taken from the separate experiments on cement and bentonite. When taking the sum of the two expected changes, the liquid limit would be twice as high as it actually is. In this table, the cement content is the mass of cement over the mass of solids, including the bentonite, as bentonite is a clay as well. The bentonite content is the same as described in equation (34).
Not being able to superpose the effects the additives separately have on the liquid limit has multiple reasons, among which are the following:

- The effect of cement on pure bentonite, being highly plastic, is that its liquid limit declines (Chew, Kamruzzaman, & Lee, 2004);
- Sodium bentonite is a known pozzolan and thus reacts in combination with cement (Wong, Hashim, & Ali, 2013);
- As mentioned in 4.3, the precipitation of cementitious products on clay clusters decreases the liquid limit at higher cement contents, because of a decreased surface activity. This will also decrease the surface activity of bentonite.

The increase in liquid limit does not follow the trend lines proposed in Figure 68. As has been mentioned before, these trend lines are very much open for discussion. However, it implies that the addition of bentonite has a liquid limit increasing effect at lower cement contents than those stated in Table 18. These increases are in the order magnitude of 10 percentage point. At a curing time of 28 days, however, a significant increase in liquid limit is spotted for a cement and bentonite content of both 17% This difference can be observed in Table 19.

| Clay (curing time) | | | | | | |
|--------------------|---|---|---|---|---|
| VE-KA (7)          | 45%| 29.64%| 50.47%| 45.16%| 95.63%| 50.09%|
| VE-KA (28)         | 45%| 29.61%| 58.45%| 45.09%| 103.54%| 48.81%|

Table 18: the difference between the superposed change in liquid limit and actual liquid limit

| Clay (curing time) | | | | | | |
|--------------------|---|---|---|---|---|
| VE-KA (7)          | 45%| 18.0%| 58.1%| 68.9%|
| VE-KA (28)         | 18.0%| 17.8%| 83.5%|
| MM2 (7 days)       | 17.9%| 17.9%| 51.1%|
| MM2 (28 days)      | 17.9%| 18.0%| 71.5%|
| MM5 (7 days)       | 18.0%| 18.1%| 47.5%|
| MM5 (28 days)      | 18.0%| 17.8%| 62.1%|

Table 19: the difference in liquid limits between a curing time of 7 and 28 days at cement and bentonite contents of approximately 18%

Such an effect cannot be observed for other cement and bentonite contents. Marginal differences are observed between a curing time of 7 and 28 days, but these can also be due to a slightly different composition or measurements errors.
The effect appears to be a curing time effect, which can be attributed to the pozzolanic nature of sodium bentonite. For the change in plastic limit this effect is not present. With the data and literature available no conclusive statement can be made as to why this effect only appears at these cement and bentonite contents, nor can such a statement be made with regard to the absence of such an increase in the plastic limit.

With regard to the Atterberg limits it can be concluded that cement is the dominant additive, with bentonite slightly increasing the liquid limit, especially at cement contents lower than 40%.

### 6.3.2 Strength

The strength increment can be viewed from two different angles. When considering the effect of the cement, it is interesting to regard the bentonite as part of the soil. When considering economics, bentonite should be regarded as a cost and the undrained shear strength should be taken as a function of the total additive content.

One could argue that adding bentonite to a soil essentially creates a different soil. If that argument is followed, the undrained shear strength increases as a function of the cement content of the “new” soil. This gives the graph shown in Figure 73.

![Figure 73: the cement-bentonite stabilised samples added to the graph showing the undrained shear strength as a function of cement content](image)

It shows that the samples that are also stabilised with bentonite have a higher undrained shear strength at a similar cement content. This effect can largely be attributed to the decrease in initial liquidity index.
The values for \( \alpha \) from equation (29) are 0.19 and 0.17 for VE-KA with bentonite and MM2 with bentonite, respectively. The values for \( R^2 \) are 0.988 and 0.939, respectively.

From a financial point of view, it is much more interesting to look at the total effect of the additives. As bentonite can be more than three times as expensive as cement, it will only be used to replace cement when the resulting undrained shear strength is higher than when just cement is used. As can be seen in Figure 74, bentonite is a weaker additive than cement and thus in terms of increasing the undrained shear strength, cement would be preferred as the sole additive from a financial point of view.

![Figure 74: the undrained shear strength as a function of the total additive content](image)

6.4 THE RELATIONSHIP BETWEEN THE UNDRAINED SHEAR STRENGTH AND THE ATTERBERG LIMITS

The soil mixtures using bentonite only can be, as they are remoulded soils, described with the Leroueil formula, equation (5). The cement stabilised samples, however, cannot be described using this formula. As was mentioned in 4.3 and shown in Figure 34, cement stabilised soils cannot be described as remoulded soil, as they have an apparent overconsolidation pressure. As can be seen in Figure 75, the found values for four different kinds of samples can however be described by a trend line. The trend line is analogous to equation (7) and is of the following form:

\[
c_{\text{u}} = 30 \cdot 6.2^{1-l_{c}} - 1 \leq l_{c} \leq 1
\]  

(36)

\( R^2 \) for this formula and the found data points is 0.807. It is valid in the range described and only
for non-remoulded cement-stabilised clay or silt slurries. It has been tested only for the soils in the experimental study and using one type of cement. Further confirmation of the applicability of this trend line is therefore wanted. Equation (7), using the constants proposed by Vardanega & Haigh (2014) itself is also plotted in the graph, to show that it deviates from the trend line found. Equation (5), found by Leroueil et al. (1983), is also plotted for comparison with Figure 72. The constants proposed by Vardanega & Haigh (2014) for equation (7) as well as equation (5) are meant to be used on remoulded clays. The cemented samples can be considered to be non-remoulded and thus the proposed trend line deviates from the trend lines proposed by Leroueil et al (1983) and Vardanega & Haigh (2014).

![Graph: Liquidity Index versus Undrained Shear Strength](image)

Figure 75: The liquidity index versus the undrained shear strength, for both the cement stabilised as the cement-bentonite stabilised samples

It has to be noted that the undrained shear strength at liquid limit is thus 30 kPa. This means that upon remoulding, the undrained shear strength will drop to 1.7 kPa. Stabilised dredged material with a liquidity index around 1 is therefore unfavourable. When, accidentally, a load larger than the permitted load is applied, the structure will break and the soil will liquefy.

According to the proposed trend line, the desired undrained shear strength of 75 kPa is reached at a liquidity index of 0.5.
6.5 TEST METHODS USED IN THE EXPERIMENTAL STUDY

6.5.1 ADDITIVES USED
Cement has shown to be serving its purpose well in this context. Above all, it increases strength rapidly. It also increases the plastic and liquid limits quite fast. It is relatively cheap, when compared with other additives, and is widely available, as it is used in the construction industry worldwide. It is thus very much fit for purpose. The cement industry is, however, responsible for 5% of all carbon dioxide emissions worldwide. The lime used in cement kiln has to be calcined first, in order to turn calcium carbonate into calcium oxide. This reaction releases one mole of carbon dioxide into the atmosphere per produced mole of calcium oxide. The same is thus applicable to the use of quicklime. As discussed in 4.3, the pH rises rather quickly with increasing cement content, until it stabilises at a pH of 12. This thus causes the stabilised soil to be very alkaline. Most plants cannot grow on such an alkaline soil. As a preload will most likely be used, nothing will need to grow on such a layer. However, it can also raise the pH of the groundwater. Measures must thus be taken to ensure that this water does not come into contact with drinking water sources or water sources used for agricultural purposes.

Ideally, lower cement contents would have also been used in the experimental study. This would have improved the understanding of the working of cement as an additive, especially with regard to the Atterberg limits. It was expected that the chosen cement contents would deliver results that were somewhat lower in terms of undrained shear strength than they actually turned out to be. It is therefore recommendable to involve larger ranges of cement contents in future studies.

Bentonite as the sole additive to slurries is not useful if a land reclamation site has to be built that must be able to bear container terminal-like structures. However, for land reclamations that only need to be accessible on foot, this kind of additive can be useful. Artificial islands that have the sole purpose of bird and wildlife preservation, like the Marker Wadden in the Netherlands, can be constructed using bentonite as an additive to dredged material. For these kind of applications, bentonite has the advantage that it is a natural material and does not change the pH of the soil drastically. Cement does and can increase the pH to around 12, as can be seen in 4.3. The behaviour of bentonite as an additive is also very predictable, as was discussed in 0. Knowing the liquid and plastic limits of the dredged material and the bentonite and the water content of the dredged material is sufficient to give a good estimate of the resulting undrained shear strength. However, bentonite is a very expensive material. It should also be noted that the type of bentonite that has a high liquid limit is a specific type of bentonite,
being sodium bentonite. Calcium bentonite usually has a lower liquid limit of approximately 150%. This effect can be attributed to the thinner diffuse double layer, discussed in 4.3.

6.5.2 FALL CONE TEST
The fall cone test has been used to determine two properties, being the undrained shear strength and the liquid limit. As these properties are actually linked (Houlsby, 1982) and standardised, this use of the fall cone test for those purposes is justified.

The fall cone test is used on undisturbed samples as well, in the manner described in NPR-CEN-ISO/TS 17892-6:2004,IDT. These results were however not always useable, as the penetration would be below the minimum penetration value of 5 millimetres for the majority of tests. A weight of approximately one kilogram was used to enhance the penetration, but this turned out to be insufficient. A larger weight cannot be used on the fall cone test apparatus, as the shaft can barely hold the weight used. The samples that had a value below 5 millimetres gave very unrealistic, large results. Undrained shear strengths up to approximately 440 kPa could be measured. It should however be noted that tests on undisturbed samples is for indicative purposes only.

When the results of the unconfined compressive tests, performed on a minority of the samples, are reverse-calculated into fall cone penetrations, it appears that the samples should have a penetration higher than 5 millimetres. These results raise the question whether or not a fall cone test can actually be executed on cement stabilised samples. Multiple doubts can be raised regarding the use of the fall cone test on cemented samples, including:

- The fall cone test is meant for use on clays, whereas undisturbed cemented samples are, like concrete, more of a rock-like material.
- Cemented samples develop a crust on top of the sample, which cannot be penetrated well and might have a higher strength than the rest of the sample.

Using the fall cone test on the bentonite stabilised samples, however, worked quite well. Bentonite is a weak clay and therefore not a significantly strengthening additive. At the same liquidity index, bentonite stabilised samples are actually weaker than the clays the bentonite is meant to stabilise. For the tests on the undisturbed bentonite stabilised samples, it was not necessary to add any weight to the cone. Therefore, the fall cone test is suitable for tests on bentonite stabilised dredged material.

The large difference between bentonite and cement for this particular purpose is that the bentonite behaves like a clay, because it is, and cement does not, because it is not. The fall cone test is meant to be used for clays. Undisturbed cemented samples appear not to behave as clays.
It is therefore recommended not to use the fall cone test as a means to determine the undrained shear strength of strongly cemented samples.

Determining the liquid limit and the undrained shear strength of the remoulded samples using the fall cone test was done without any irregularities.

6.5.3 UNCONFINED COMPRESSIVE TEST

The unconfined compressive test was used as an alternative to the fall cone test to determine the undrained shear strength. It was determined that the fall cone test results were unreliable after the test were performed on the cement samples that had cured 7 days.

The use of the unconfined compressive test was thus not anticipated. If this type of test was included in the initial programme, the length-to-diameter ratio of the samples would have been set at 2, as to avoid the use of the empirical formula derived by Güneyli & Rüsen (2016). The existence of such formulae is however useful. It is thus recommended to use such a formula when using samples with a small length-to-diameter ratio.

The unconfined compressive test has been used by other authors to determine the strength of cement stabilised soil (Burgos, Samper, & Alonso, 2006), (Kitazume & Satoh, 2003). In order to be able to compare results to those of different authors, it is recommendable to use the same type of test. Other than having to use equation (27), the use of the unconfined compressive test was satisfactory.

6.5.4 PIPE PRESSURE LOSS

When adding cement or bentonite to the slurry, the density increases. Cement does not immediately alter the liquid limit, but bentonite does. Even still, both do alter the liquidity index quite much. At very high cement contents, the liquidity indices become so low that pipe pressure loss per meter due to wall shear is not the governing problem anymore, but the yield stress that has to be overcome. The liquidity index drops below 1 when 500 kg/m³ of cement is added to the MM2 slurry. The full results can be seen in Figure 76.
Figure 76: the direct decrease in liquidity index as a function of the cement content

Two types of pumps are generally used in industrial applications, being the centrifugal pump and the positive displacement pump. The former can handle yield stresses of approximately 250 Pa, whereas the latter can handle yield stresses up to 500 Pa, when the hydraulically actuated piston pump type is used (Cooke, 2008).

Yield stress and liquidity index are correlated (Locat & Demers, 1988), as can be seen in Figure 77. The red arrows indicate the range of liquidity indices in which the yield stress reaches 250 Pa, the maximum yield stress of centrifugal pumps. Centrifugal pumps are commonly used in dredging applications. The minimum liquidity index appears to be between 1.5 and 2.2. This means that most of the cement contents in the experimental study have been too high. However, valuable trends have been found using these cement contents.
Generally speaking, a mixture should not have a liquidity index below 2.2, as that would cause the pump to stall. When high strengths and thus high cement contents are needed, some clays and silts cannot be stabilized using this method, because their liquid limit is too high. As can be noted from 3.5 and 5.2, problematically high pipe pressure losses can be mitigated by using larger pipe diameters. Mitigation of high pressure losses and an increase in yield stress due to the addition of cement or other additives should be done by placing the mixing station close to the end of the pipe. It must however have a sufficient distance to the end of the pipeline to cause mixing by turbulent flow.

6.6 GENERAL APPLICABILITY OF THE METHOD

Based on the research objectives, the reference projects, literature study and experimental study, the working method should be as shown in (Burgos, Samper, & Alonso, 2006). This means applying a stabilised crust on top of non-stabilised dredged material. The layer in this figure is 4 metres thick, which was considered a safe thickness in the Port of Valencia, after bearing capacity calculations. After applying the cover layer and curing, a vertical drainage system should be applied. The next step is to apply a preload, after which consolidation begins. When consolidation has reached its practical end, the preload can be removed to the desired level.
When the expected density of the stabilised material is higher than the non-stabilised dredged material, it has a very real risk of sinking. In such cases, air foam or EPS beads should be used, as suggested in 3.2.8. Air foam is preferred, as EPS beads might float on the material, rendering them useless. Densities as low as 1100 kg/m$^3$ can be achieved using air foam. An example of cement and air foam or EPS beads stabilised samples can be found in Table 21 in the appendix.

However, before it can all come to that, further research on this subject is required. A proper pilot test needs to be designed. Deeper investigations into what governs the increase in undrained shear strength is required as well. Further investigations into the hypothesis that the increase in plastic limit and liquid limit follow the increase in bound water content are required too. There is a strong indication that there is the relationship between the liquidity index and undrained shear strength also exists for non-remoulded cement-stabilised samples. Further research could reveal if that is so and how that all correlates to the factor $\alpha$ from equation (29).
7 CONCLUSIONS AND RECOMMENDATIONS

7.1 CONCLUSIONS
The goal of this thesis is to find a method in which an unconfined compressive strength of 150 kPa can be achieved using liquid dredged material and additives. Deposition has to take place at a high rate. The study aims to use material properties to explain the effects of the additives on the liquid dredged material. The behaviour of cement and bentonite have been evaluated in the literature and experimental study. Maximum yield stress and pipe pressure loss have been investigated as well.

Cement catalyses the flocculation process of clay clusters. It glues these clusters together, which increases the strength of the stabilised dredged material. An increase of strength is also gained because of two processes involving the water present in the material. The first is the reaction of cement with water, crystallising the water. The second is the increase in bound water content. Water is bound in the cement matrix formed. The latter process also increases the liquid and plastic limit of the material. A descriptive formula that describes this process has been found in literature (Zhu, Zhang, & Chiu, 2007). The applicability of this formula is suggested in this study, but has not been proven. Due to precipitation of cementitious products on the cluster surfaces at higher cement contents, the surface activity of the cluster decreases, decreasing the liquid limit. The plastic limit appears to have a horizontal asymptote. At higher cement contents than were found in the literature study and used in the experimental study, the liquid limit will decrease towards this asymptote.

Many formulae have been proposed in the past by various researchers to describe the increase in undrained shear strength with increasing cement content. These formulae are all highly empirical and case sensitive. In this study, a formula is proposed that matched the data from the experimental study and the data from a large dataset found in the literature study. This formula is equation (29). It involves the factor $\alpha$. Based on this study, no closure can be given on what parameters can be used to derive this factor. It is believed that the following factors are of influence:

- The particles present in the dredged material and their mineralogy;
- The initial liquidity index;
- The type of cement used;
- The presence of pozzolanic substances in the dredged material itself.

The results with regard to the Atterberg limits support these points. The change in liquid and plastic limit is different for all materials tested. The decrease in liquid limit at high cement
contents is different for each material tested, but can in all cases be approximated as a linear process. This is shown in equation (31), Figure 68 and Table 17. The change in plastic limit with respect to the asymptote of the plastic limit at high cement contents is different for each material as well. It can be observed in Figure 69.

A trend has been observed in the relationship between the non-remoulded undrained shear strength of the cement-stabilised samples, including those with added bentonite, and their liquidity indices. This relationship is shown in equation (36) and Figure 75. The change in liquidity index largely incorporates the four points that the factor $\alpha$ is believed to be dependent on.

Bentonite has also been tested as an additive. The increase in undrained shear strength caused by the addition of bentonite largely follows the Leroueil formula, as stated in equation (5). The increase in undrained shear strength is too low to be able to function as an additive on its own when an undrained shear strength of 75 kPa is wanted. Little increase in plastic limit has been observed with increasing bentonite content. An increase in liquid limit has been observed, for which a trend line was developed, equation (35). It has been tested on an independent dataset, that has used a different type of bentonite and different soils.

Addition of bentonite in combination with cement to a slurry does not have significant benefits over just adding cement. Cement is more than three times less expensive than bentonite. Bentonite, on the other hand, does not increase the undrained shear strength as much as cement. It appears that it does increase the liquid limit at lower cement contents.

Bentonite, as the sole additive, can be used in other applications, however. It can be used in a Marker Wadden-like setting, where islands are created that need to be accessible by geese. The increase in undrained shear strength that can be achieved by bentonite is sufficient to let such animals walk the surface. Since bentonite has a low hydraulic conductivity and can hold high amounts of water, it will decrease consolidation effects and thus hinder large settlements.

Whether or not a dredging pump can transport stabilised dredged material over longer distances is dependent on the maximum pumping pressure and the maximum yield stress the pump can overcome. Decrease in pumping pressure can be reduced by reducing the pipe pressure loss. This can be done by increasing the diameter, decreasing the velocity and decreasing the pipe roughness. Common centrifugal pumps can overcome a yield stress of 250 Pa, which is correlated to a liquidity index of 2.2. This means that slurries with a high liquid limit that need a high cement content in order to be stabilised are not suitable for this method.
Turbulent flow can mix the additives to the slurry. As long as an inflow of a sufficient amount of additive is possible, the method is theoretically viable.

7.2 RECOMMENDATIONS FOR PRACTICAL APPLICATIONS
Cement-stabilised samples should be tested using an unconfined compressive test and not a fall cone test when the undrained shear strength is wanted. The fall cone is designed for clays, which a cement-stabilised slurry effectively is not anymore. It resembles concrete, which is more of a rock-like material. The fall cone test can, however, be used on remoulded cement-stabilised samples. When the unconfined compressive test is used, a length to diameter ratio of at least 2 should be used, as to avoid the use of empirical correlations.

Derivations for the factor $\alpha$ of equation (29) have not been made yet, but the formula can be used when three or more samples have been tested. After testing, curve-fitting should provide a value for this factor, after which the cement content needed to reach the desired undrained shear strength can be determined.

Predictions for the liquid limit, plastic limit and undrained shear strength of bentonite-stabilised layers can be made. As mentioned before, these layers are very weak and by no means capable of bearing the loads that were set as a goal in this thesis. There might however be other applications. When the water content of the original material is known and a known mass of dry bentonite is added, the new water content can be calculated. Knowing the Atterberg limits of the dredged material and the bentonite, the new liquid limit can be calculated using equation (35). Assuming the plastic limit remains constant, the liquidity index can be calculated. The undrained shear strength can then be estimated using equation (5).

7.3 RECOMMENDATIONS FOR FURTHER RESEARCH
A pilot test should be conducted in order to confirm that this method is actually useable. Such a test should include trying to pump slurries with a high liquid limit and a high cement content, decreasing the liquidity index to the corresponding yield stress the pump used can still handle. This can confirm the applicability of Figure 77. Varying pipe diameters and velocities will show how much the pipe pressure loss in practical applications is. For such a pilot test, a suitable way of adding the cement to the pipeline must be designed. It should be ensured that when adding cement to the pipeline, a sufficient rate can be reached.

Research on the undrained shear strength increase with increasing cement content should be performed as well. Initial liquidity indices should be varied, as to see what the effect of a changing water content and liquidity index is. The effect of using different types of cements should be investigated. It is believed that the final compressive strength of the cement
influences the effect cement has on the undrained shear strength of cement-stabilised materials. However, as only one type of cement has been used, this has not been confirmed.

It has been shown that the liquid limit decreases at high cement contents. However, in the experimental study, it was not confirmed that the liquid and plastic limit follow the formula proposed by Zhu, Zhang & Chiu (2007), equation (30). Research should be performed to establish whether or not this formula is applicable. It should also be investigated what governs the constants used in this equation. In this study, the type of additive, the liquidity index and the type of clay used should be varied. Lower cement contents should be used as well. The clay should be analysed for pozzolans, such as kaolinite. Tests on pure kaolinite should give better insights into these pozzolanic reactions. In this testing programme, attention should also be given to the relationship between the liquidity index and the undrained shear strength, as a correlation has been found between those two parameters for non-remoulded cement-stabilised samples. When the factors that govern the increase in liquid and plastic limit are known and a stronger correlation between the liquidity index and the undrained shear strength has been established, basic input material parameters could be used to estimate the resulting undrained shear strength.

As mentioned, bentonite can be used as an additive in a “building with nature” setting. In a Marker Wadden-like case, stabilising the crust with bentonite could help strength development. Bentonite could be promoted as a natural additive. Further investigation to crust formation when adding bentonite should be done before it is used.
REFERENCES


## APPENDIX

### TABLES

<table>
<thead>
<tr>
<th>Sample type</th>
<th>Cement type</th>
<th>Cement content [kg/m³]</th>
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<th>UCS [kPa]</th>
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Table 20: the results from phase 2 of the project by Sparrevik & Kvennås (2008), combined with the formula of Horpibulsuk et al. (2003)

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</tr>
<tr>
<td>B1</td>
<td>789</td>
<td>EPS</td>
<td>161</td>
<td>100</td>
<td>261</td>
<td>Dry</td>
<td>1191</td>
<td>559</td>
</tr>
<tr>
<td>B2</td>
<td>789</td>
<td>EPS</td>
<td>161</td>
<td>100</td>
<td>261</td>
<td>Underwater</td>
<td>1187</td>
<td>676</td>
</tr>
<tr>
<td>B3</td>
<td>714</td>
<td>EPS</td>
<td>206</td>
<td>200</td>
<td>236</td>
<td>Dry</td>
<td>1217</td>
<td>1601</td>
</tr>
<tr>
<td>B4</td>
<td>714</td>
<td>EPS</td>
<td>206</td>
<td>200</td>
<td>236</td>
<td>Underwater</td>
<td>1183</td>
<td>1564</td>
</tr>
</tbody>
</table>

Table 21: results of tests on lightweight material treated soils (Tsuchida, Purbaha, & Yamane, 2001)
# Nomenclature

## Parameters

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Parameter used in equation (31)</td>
</tr>
<tr>
<td>$a_C$</td>
<td>Cement content as used by Zhu, Zhang &amp; Chiu (2007)</td>
</tr>
<tr>
<td>B</td>
<td>Parameter used in equation (31)</td>
</tr>
<tr>
<td>C</td>
<td>Cement content</td>
</tr>
<tr>
<td>$c'$</td>
<td>Cohesion</td>
</tr>
<tr>
<td>$c_L$</td>
<td>Undrained shear strength at liquid limit</td>
</tr>
<tr>
<td>$c_u$</td>
<td>Undrained shear strength</td>
</tr>
<tr>
<td>D</td>
<td>Diameter</td>
</tr>
<tr>
<td>$f_0$</td>
<td>Darcy friction factor</td>
</tr>
<tr>
<td>$I_L$</td>
<td>Liquidity index</td>
</tr>
<tr>
<td>$I_P$</td>
<td>Plasticity index</td>
</tr>
<tr>
<td>h</td>
<td>Groundwater head</td>
</tr>
<tr>
<td>k</td>
<td>Hydraulic conductivity</td>
</tr>
<tr>
<td>$k_1, k_2, k_3$</td>
<td>Parameters used in equations (12), (13), (14), (32) and (33)</td>
</tr>
<tr>
<td>L</td>
<td>Length</td>
</tr>
<tr>
<td>LL</td>
<td>Water content at the liquid limit</td>
</tr>
<tr>
<td>M</td>
<td>Mixing ratio</td>
</tr>
<tr>
<td>m</td>
<td>Geotechnical water content as used by Zhu, Zhang &amp; Chiu (2007)</td>
</tr>
<tr>
<td>p</td>
<td>Pressure</td>
</tr>
<tr>
<td>PL</td>
<td>Water content at the plastic limit</td>
</tr>
<tr>
<td>Q</td>
<td>Flow volume</td>
</tr>
<tr>
<td>$q_{\text{subscript}}$</td>
<td>Unconfined compressive stress (always appears with subscript)</td>
</tr>
<tr>
<td>q</td>
<td>Deviatoric stress (always appears without subscript)</td>
</tr>
<tr>
<td>$R_{MW}$</td>
<td>Muir Wood material parameter</td>
</tr>
<tr>
<td>S</td>
<td>Compressive strength of concrete</td>
</tr>
<tr>
<td>w</td>
<td>Geotechnical water content</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>Constant used in equation (26)</td>
</tr>
<tr>
<td>$\gamma'$</td>
<td>Specific bulk weight</td>
</tr>
<tr>
<td>$\gamma_w$</td>
<td>Specific weight of water</td>
</tr>
<tr>
<td>$\varepsilon$</td>
<td>Pipe roughness</td>
</tr>
<tr>
<td>$\eta$</td>
<td>Plastic viscosity</td>
</tr>
<tr>
<td>$\mu$</td>
<td>Dynamic viscosity</td>
</tr>
<tr>
<td>$\rho$</td>
<td>Density</td>
</tr>
<tr>
<td>$\sigma'$</td>
<td>Effective stress</td>
</tr>
<tr>
<td>$\varphi'$</td>
<td>Friction angle</td>
</tr>
</tbody>
</table>

## Abbreviations

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>CKD</td>
<td>Cement kiln dust</td>
</tr>
<tr>
<td>GGBS</td>
<td>Ground granulated blast-furnace slag</td>
</tr>
<tr>
<td>UCS</td>
<td>Unconfined compressive strength</td>
</tr>
</tbody>
</table>
MATERIALS USED

ADDITIVES

The bentonite used has the following production characteristics:

<table>
<thead>
<tr>
<th>Producer</th>
<th>Fischer Scientific</th>
</tr>
</thead>
<tbody>
<tr>
<td>Product name</td>
<td>Bentonite powder</td>
</tr>
<tr>
<td>Packing date</td>
<td>06/02/10</td>
</tr>
<tr>
<td>Code</td>
<td>B/1300/60</td>
</tr>
<tr>
<td>Batch</td>
<td>0943566</td>
</tr>
<tr>
<td>Formula</td>
<td>EINECS 215-108-5</td>
</tr>
</tbody>
</table>

The cement used has the following characteristics:

<table>
<thead>
<tr>
<th>Producer</th>
<th>ENCI</th>
</tr>
</thead>
<tbody>
<tr>
<td>Product name</td>
<td>Portland cement CEM I</td>
</tr>
<tr>
<td>Strength class</td>
<td>42.5 N</td>
</tr>
<tr>
<td>Initial strength (2 days)</td>
<td>≥ 10,0 MPa</td>
</tr>
<tr>
<td>Normative strength (28 days)</td>
<td>≥ 42.5 MPa</td>
</tr>
<tr>
<td>Soundness</td>
<td>≤ 10 mm</td>
</tr>
<tr>
<td>Mass percentage of Portland clinker</td>
<td>95% - 100%</td>
</tr>
<tr>
<td>Other constituents</td>
<td>0% - 5%</td>
</tr>
<tr>
<td>Applicable standard</td>
<td>EN 197-1, including adaptation A1</td>
</tr>
<tr>
<td>Production date</td>
<td>26/05/2016</td>
</tr>
<tr>
<td>Production location</td>
<td>Lage Kanaaldijk 115, Maastricht, the Netherlands</td>
</tr>
</tbody>
</table>

VE-KA, MM2 AND MM5

VE-KA clay is a commercially produced clay, commonly used at the Delft University of Technology for experimental purposes. Its actual purpose is to be used in pottery. It contains quartz (50.2%), vermiculite (21%), muscovite (16.2%), anorthite (6.8%) and calcite (5.8%). Clay constitutes approximately 42% of the soil, silt 54.9% and sand 3.1% (Jommi, Valimberti, Tollenaar, Della Vecchia, & Van Paassen, 2016).

MM2 and MM5 are clays from the Markermeer. They are mixtures of multiple vibrocore samples. The vibrocore samples were taken from the Markermeer in the context of the "Marker Wadden" project. The research was ordered by Boskalis Nederland B.V., sampling was performed by Wiertsema & Partners. Laboratory investigations were done by Boskalis Dolman B.V. (trading as Boskalis Environmental).

Both clays are Holocene clays. The MM2 mixture was created from vibrocores drilled at the edges of a planned sand deposit. The MM5 mixture was created from vibrocores drilled in a proposed sludge trench. The top layer of the vibrocores existed of clayey sand. In the MM2 and MM5 mixtures, these soils have been left out, as can be seen in Figure 79. As can be seen, MM2 is made out of parts of vibrocores VC1 to VC4 and VC7. MM5 is made out of parts of VC5 and VC6.
Table 22: the data describing the material properties of the soils in the vibrocores

<table>
<thead>
<tr>
<th>Vibrocore</th>
<th>Clay type</th>
<th>Organic matter content</th>
<th>Liquid limit</th>
<th>Plastic limit</th>
<th>Plasticity Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>VC01</td>
<td>Organic</td>
<td>N/A</td>
<td>193%</td>
<td>86%</td>
<td>106%</td>
</tr>
<tr>
<td>VC01</td>
<td>Non-organic</td>
<td>N/A</td>
<td>140%</td>
<td>50%</td>
<td>90%</td>
</tr>
<tr>
<td>VC02</td>
<td>Organic</td>
<td>15.5%</td>
<td>211%</td>
<td>92%</td>
<td>119%</td>
</tr>
<tr>
<td>VC02</td>
<td>Non-organic</td>
<td>9.5%</td>
<td>131%</td>
<td>54%</td>
<td>78%</td>
</tr>
<tr>
<td>VC03</td>
<td>Organic</td>
<td>N/A</td>
<td>160%</td>
<td>69%</td>
<td>92%</td>
</tr>
<tr>
<td>VC03</td>
<td>Non-organic</td>
<td>N/A</td>
<td>88%</td>
<td>38%</td>
<td>50%</td>
</tr>
<tr>
<td>VC04</td>
<td>Organic</td>
<td>11.0%</td>
<td>130%</td>
<td>56%</td>
<td>74%</td>
</tr>
<tr>
<td>VC04</td>
<td>Non-organic</td>
<td>6.9%</td>
<td>101%</td>
<td>47%</td>
<td>55%</td>
</tr>
<tr>
<td>VC05</td>
<td>Organic</td>
<td>16.2%</td>
<td>270%</td>
<td>108%</td>
<td>162%</td>
</tr>
<tr>
<td>VC05</td>
<td>Non-organic</td>
<td>N/A</td>
<td>47%</td>
<td>27%</td>
<td>20%</td>
</tr>
<tr>
<td>VC06</td>
<td>Organic</td>
<td>24.1%</td>
<td>101%</td>
<td>36%</td>
<td>65%</td>
</tr>
<tr>
<td>VC06</td>
<td>Non-organic</td>
<td>N/A</td>
<td>83%</td>
<td>32%</td>
<td>51%</td>
</tr>
<tr>
<td>VC07</td>
<td>Organic</td>
<td>16.4%</td>
<td>210%</td>
<td>99%</td>
<td>111%</td>
</tr>
</tbody>
</table>

Figure 79: the composition of the mixtures
<table>
<thead>
<tr>
<th>Clay mixture</th>
<th>Organic matter content</th>
<th>Liquid limit</th>
<th>Plastic limit</th>
<th>Plasticity Index</th>
<th>Density after &gt;14 days</th>
</tr>
</thead>
<tbody>
<tr>
<td>MM2</td>
<td>8.5%</td>
<td>116%</td>
<td>47%</td>
<td>69%</td>
<td>1195</td>
</tr>
<tr>
<td>MM5</td>
<td>5.3%</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>1275</td>
</tr>
</tbody>
</table>

Table 23: the data with regard to MM2 and MM5, as extracted from the datasheet provided