Pilot Kleirijperij

WP4.2.1 Voorbereidende laboratoriumproeven

EEMS DOLLARD 2050

EcoShape

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Pilot Kleirijperij: Voorbereidende laboratoriumproeven

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Deltares

Pilot Kleirijperij: Voorbereidende laboratoriumproeven

wp 4.2.1

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Deltares

Titel
Pilot Kleirijperij: Voorbereidende laboratoriumproeven

Project	Kenmerk	Pagina's
11201344-000	11201344-000-ZKS-0006	11

Trefwoorden
Kleirijperij, laboratoriumonderzoek, Seepage Induced Consolidation (SIC), bezinkingsproeven, Eems-Dollard slib, consolidatie

Samenvatting

De rivier Eems-Dollard kampt met zeer hoge concentraties slib die leiden tot negatieve effecten voor de ecologie. Dit slib bezinkt o.a. in de haven van Delfzijl en de polder bij Breebaart en is daar overtollig. Tegelijkertijd is in dit gebied een grote behoefte aan klei ten behoeve van dijkversterking. De centrale vraag voor de Kleirijperij is: "Hoe kan het gebaggerde slib uit Delfzijl en Breebaart worden omgevormd tot bruikbare klei?". Er worden proefvelden aangelegd waarop het gebaggerde slib (met een hoog watergehalte) wordt gedeponeerd. Hier kan het materiaal sedimenteren (bezinken) en consolideren (compacteren). Vervolgens krijgt verdamping de overhand en begint het rijpingsproces. Hierdoor neemt het watergehalte geleidelijk verder af totdat de gerijpte klei geschikt is om te worden gewonnen. De kleirijperij is een pilot studie waarbij verschillende technieken worden getest om de rijpingscompartimenten te vullen, het slib te laten consolideren en rijpen (drogen) en om de kwaliteit van de gewonnen klei te verbeteren.

Het project Kleirijperij is één van de Eems-Dollard-projecten binnen het programma EemsDollard 2050. De projecten zijn er op gericht om om te komen tot nuttige toepassingen voor het overtollige slib.

Dit document beschrijft het voorbereidende laboratoriumonderzoek voor de pilot Kleirijperij. Het doel van het onderzoek is om het bezinkings- en consolidatiegedrag van de het slib van de twee bronlocaties te beschrijven. Daarbij wordt tevens de invloed van het opmengen van zout slib met zoet in deze fases beoordeeld. Hiervoor zijn kolomproeven uitgevoerd waarin gestart is met lage slibconcentraties in zoet- en zoutwater. Tevens is een Suction Induced Consolidation (SIC) proef uitgevoerd, om de effecten op langere termijn en onder hogere spanningen te beoordelen als input voor het mathematische modelleringsonderzoek binnen de pilot. De slibkarakterisatie draagt bij aan de keuze van de proefvakken en dient als startpunt voor de mathematische modellering. De conclusie van het opmengen van zout slib met zoet water is dat het zoutgehalte in de geteste range weinig invloed heeft of bezinking en de eerste fase van consolidatie. Dit maakt zoutverwijderen door opmengen met zoetwater een interessante onderzoeksoptie in de pilot. Met een dergelijke strategie kan naar verwachting makkelijker aan de norm voor het maximale zoutgehalte in dijkenklei worden voldaan, zonder dat het negatieve effecten heeft in de eerste fase van het project.

Versie	Datum	Auteur	Paraaf	Review	Paraaf	Goedkeuring	Paraaf
	mei. 2018	Ebi Meshkati Shahmirzadi		Thijs van Kessel	TvK	Frank Hoozemans	
		Bernadette Wichman		Wouter van der Star			
		Jill Hansen					

Status
concept
Dit document is een concept en uitsluitend bedoeld voor discussiedoeleinden. Aan de inhoud van dit rapport kunnen noch door de opdrachtgever, noch door derden rechten worden ontleend.

Inhoud

1 Introductie

De Eems-Dollard wordt gekenmerkt door een hoog slibgehalte en daardoor een verminderde ecologische waarde. In 2015 (Economie en Ecologie Eems-Dollard in balans, 2015) is voorgesteld dat de structurele jaarlijkse verwijdering van 1 miljoen ton slib (ds) uit het systeem een positief effect zou hebben op de ecologie. Deze winning biedt een nieuwe materiaalstroom op het land die nuttig kan worden toegepast. Binnen het programma Eems-Dollard 2050 worden –op pilotschaal– diverse van deze methoden beproefd.

Een van deze methoden is de Pilot Kleirijperij. In deze pilot wordt slib in 3 jaar tijd gerijpt en omgezet in dijkenklei. De klei zal worden toegepast in een dijkversterking (de Brede Groene Dijk), en –gedeeltelijk- als ophoogmateriaal worden ingezet.

Deze pilot wordt uitgevoerd door Rijkswaterstaat, het Groninger Landschap, Groningen Seaports, Rijkswaterstaat Noord Nederland, Waterschap Hunze en Aa's en stichting Ecoshape.

De pilot wordt uitgevoerd met twee slibbronnen uit de Eems-Dollard (de haven van Delfzijl en de polder Breebaart) en ligt op twee locaties: Kleirijperij Delfzijl ligt binnendijks direct naast het havenkanaal, terwijl de Kleirijperij Kwelder buitendijks ligt op de kwelder, direct voor de locatie waar het materiaal zal worden toegepast in een dijkversterking. Het slib in beide bronnen is marien slib met een hoog zout- en organische stofgehalte.

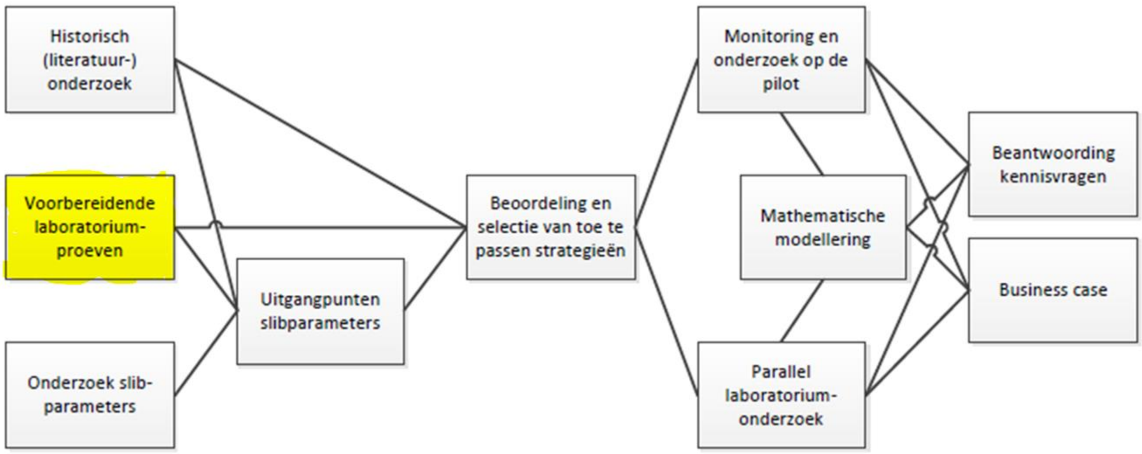
Figuur 1.1 Locatie van de kleirijperij en de slibbronnen.

Beide kleirijperijen zijn verdeeld in proefvakken (15 in Kleirijperij Delfzijl, 10 in Kleirijperij Kwelder) waarin verschillende rijpingsmethodes kunnen worden beproefd. Het doel van de kleirijperij is daarbij tweeledig:

- Kennis opdoen voor opschaling onder optimale condities (en beoordelen of dat haalbaar is)
- Het leveren van voldoende geschikte dijkenklei voor de Pilot Brede Groene Dijk.

Voor een optimale keuze van bewerkingsmethodes en parameters, is voorbereidend laboratoriumonderzoek naar het bezinkings- en consolidatiegedrag uitgevoerd (deze rapportage), zijn de eigenschappen van het slib op beide winlocaties bepaald (Van den Heuvel, 2017) en is literatuuronderzoek uitgevoerd (RHDHV en Deltares, 2018).

Een overzicht hoe het onderzoek past binnen de overall kleirijperij-activiteiten is weergegeven in figuur Figuur 1.2.



Figuur 1.2 Relatie tussen de verschillende onderdelen van de Pilot Kleirijperij

De klei dient uiteindelijk te voldoen aan de eisen gesteld in de rapportage Klei voor Dijken (TAW 1996) en beschreven in het Projectplan Kleirijperij (2015):

1. **De gerijpte klei moet minimaal voldoen aan erosieklasse II of I. Voor erosieklasse II betekent dat: ligging boven de zogenaamde A-lijn.**
2. **De gerijpte klei dient een consistentie-index te hebben van minimaal 0,6.**
3. Afkomstig van een op natuurlijke wijze afgezet materiaal.
4. Zandgehalte (> 63 μm) is maximaal 40%.
5. **Minder dan 5% organisch materiaal volgens de waterstofperoxidebehandeling methode.**
6. Minder dan 25% gewichtsverlies bij de HCl-behandeling (kalkgehalte).
7. **Zoutgehalte (NaCl g/l bodemvocht) is minder dan 4%.**
8. Geen significante bijmenging van puin, grind en dergelijke.
9. Weinig heldere (rode, bruine en gele, soms blauwe) verkleuringen.

Gezien de eigenschappen van de in-situ genomen slibmonsters (Kwaliteitsplan parameters slib, 2017) is de verwachting dat de punten 4, 6, 8 en 9 zonder verdere behandeling worden gehaald. De mate waarin criterium 3 wordt gehaald (natuurlijk afgezet) is in deze pilot niet aan de orde: hoewel de vorming natuurlijk is, is het inbrengen van het slib wellicht niet zo te kwalificeren.

De nadruk ligt in de kleirijperij daarom op de ontwikkeling van de (vetgedrukte) parameters 1, 2, 5 en 7.

1.1 Onderzochte processen

In de pilot worden verschillende technieken getest om de slibcompartimenten te vullen en het slib te laten bezinken, consolideren en rijpen (onder invloed van lucht) en de kwaliteit van de geproduceerde klei te laten voldoen aan de eisen die gesteld worden aan dijkenklei. De processen worden in Figuur 1.3 schematisch weergegeven.

Figuur 1.3 : v.l.n.r. bassin gevuld met slib sedimentatie slib, consolidatie slib waarbij het water steeds wordt afgelaten en rijpen van het slib. (groen: tussenkades, bruin, slib/klei, blauw: water).

Zoals hierboven beschreven moet het slib eerst sedimenteren en consolideren voordat het kan rijpen. Deze eerste twee processen zijn mede bepalend voor de uitvoeringsmethode, de doorlooptijd van het project en kwaliteit van de klei. Erbestaan gestandaardiseerde methodes om in relatief korte periodes te beoordelen hoe deze fases verlopen. Voor de fase van rijping zijn de onzekerheden en de doorlooptijden van de proeven groter en zijn proeven op grotere schaal nodig (zoals de kleirijperij zelf). Laboratoriumproeven naar rijping vinden parallel aan de uitvoering van de pilot plaats onder meer gecontroleerde condities dan in de pilot zelf. Bij het ontwerp is de kennis over rijpingsmethodes verkregen door middel van literatuuronderzoek waarin historische cases worden beschreven.

Om daar meer inzicht in te krijgen in de fase voorafgaand aan rijping zijn bezinkproeven en Seepage Induced Consolidation (SIC) proeven uitgevoerd. De bezinkproeven geven informatie over het slib in de fasevan bezinken (sedimenteren) en de eerste fase van consolidatie (korte termijn). Bovendien is met de bezinkproeven bekeken wat de effecten zijn van verzoeting op bezinking en consolidatie: in de kleirijperij wordt van zout slib immers zoete klei gemaakt, en ook tijdens de uitvoering is het opmengen met zoet water een van de opties. Met deze proeven wordt inzicht verkregen over (positieve of negatieve) effecten van een minder zout milieu op bezinking en consolidatie. De SIC proeven gebeuren onder een bovenbelasting waardoor het consolidatieproces wordt versneld. De proef geeft informatie over consolidatie voor langere tijdschaal en/of van diepere grondlagen.

1.2 Opzet rapport

Dit rapport is opgedeeld in twee hoofdstukken:

- 2.1 Bezinkkolommen; de sedimentatie en consolidatie van het slib wordt gemonitord om de doorlatendheid, sterkte en fractale dimensie af te leiden. De proeven zijn in detail beschreven in Bijlage A.
- 2.2 Seepage induced consolidation (SIC); het materiaal onder belasting wordt gemonitord om de doorlatendheid, samendrukbaarheid en de hoeveelheid water af te leiden. De proeven zijn in detail beschreven in Bijlage B.

In Hoofdstuk 3 worden de conclusies beschreven.

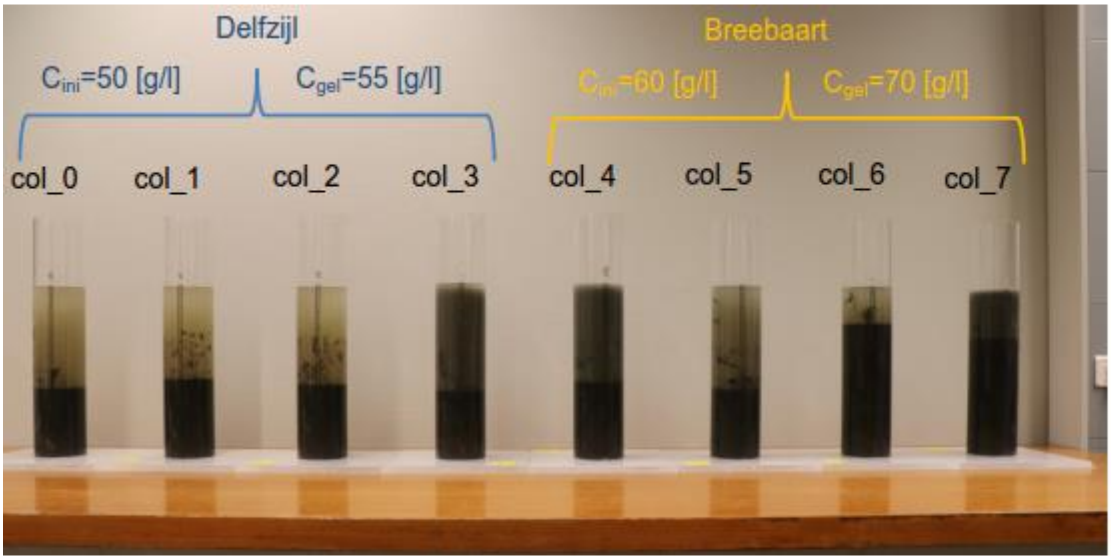
2 Laboratorium proeven

Het slib van polder Breebaart en de haven van Delfzijl is in het laboratorium van Deltares getest.

Het zoutgehalte zal gedurende de pilot langzaam afnemen door contact met regenwater. Bovendien kan door de verschillende locatie van het gewonnen materiaal, en de verschillende uitvoeringsmethodes ook het initiële zoutgehalte variëren. Het zoutgehalte (ook saliniteit genoemd, en vaak uitgedrukt als elektrische geleidbaarheid, EC) is een parameter die invloed kan hebben op de materiaaleigenschappen en op het bezinkings, consolidatie (en rijpings)proces. Het doel van de kolomproeven is om het effect van de verschillende zoutgehaltes op sedimentatie en consolidatie van het slib te bepalen.

2.1 Laboratorium proeven: Bezinkkolommen

Het materiaal van de haven van Delfzijl en polder Breebaart is getest in 8 verschillende kolommen (hoogte 55 cm, diameter 10 cm) gedurende 3 maanden zoals te zien in Figuur 2.1. Per locatie is het slib aangelengd met kraanwater (zoet water) en/of zoutwater (met hetzelfde zoutgehalte als de locatie) om het effect van het zout te toetsen. Allereerst is de gelling concentratie (C_{gel}; concentratie slib waarbij het materiaal niet meer in suspensie is en een bed vormt) getoetst. Deze is hoger voor Breebaart. Op basis hiervan is per locatie de initiële concentratie sediment dat getoetst wordt in de kolommen (C_{ini}) gekozen (dit dient lager te zijn dan C_{gel}). In Bijlage A worden de test en de resultaten in detail beschreven.



Figuur 2.1 : Opstelling bezinkkolommen; linkerhelft materiaal uit Delfzijl, rechterhelft materiaal uit Breebaart.

2.1.1 Metingen en analyse

Door middel van foto's is de sedimentatie, vorming van een bed en consolidatie van het bed gemonitord. Voor beide locaties geldt dat initieel de sedimentatie wordt beïnvloed door het zoutgehalte. Dit effect is echter alleen zichtbaar gedurende de eerste dag, daarna wordt de consolidatie niet meer beïnvloed door het zoutgehalte. Dit proces verloopt sneller bij slib van Delfzijl dan bij slib van polder Breebaard.

De foto's zijn geanalyseerd om de consolidatieparameters en fractale dimensie af te leiden door middel van de methode van (Merckelbach, 2000). Deze parameters bepalen de doorlatendheid en effectieve korrelspanning van het slib als functie van de porositeit. De doorlatendheid wordt immers beïnvloed door het poriëngetal (de volume ratio tussen holle ruimte en de vaste stof) van het slib. De mate van invloed verschilt voor Delfzijl en Breebaart. Voor beide locaties geldt echter dat het zoutgehalte vrijwel geen effect heeft op de relatie. Voor beide locaties is de relatie tussen de korrelspanning en poriëngetal berekend. Het slib uit Delfzijl heeft voor een gelijk poriëngetal een hogere korrelspanning (+/- 1 ordegrootte) dan het materiaal uit Breebaart. Voor Delfzijl heeft het zoutgehalte vrijwel geen invloed heeft op de korrelspanning. Voor Breebaart heeft het zoutgehalte wel een kleine invloed wanneer het poriëngetal groter is dan 10.

Na de bezinkproef is op verschillende hoogtes van het gevormde bed en voor verschillende zoutgehaltes (electrische geleidbaarheid) de sterkte en dichtheid van het slib gemeten, respectievelijk met een rotoviscometer en een akoestische UHCM (Ultrasonic High Concentration Meter). De dichtheid van het materiaal van Delfzijl is lager dan van het slib van Breebaart. In alle kolommen neemt de dichtheid toe met de diepte zoals verwacht. De relatieve toename van de dichtheid neemt af met de diepte. Het slib uit Delfzijl met een lagere elektrische geleidbaarheid heeft uiteindelijk een kleinere bedhoogte en als gevolg een hogere dichtheid. Daarnaast is de sterkte ook groter. Voor slib uit Breebaart geldt dat wanneer het wordt gemengd met een medium zoutgehalte (kraanwater + zoutwater) de bodemhoogte het laagst is en de dichtheid en sterkte het hoogst.

De elektrische geleidbaarheid (EC) van het slib van de verschillende monsters is eveneens gemeten vijf minuten na aanvang van de kolomproeven en aan het eind van de proeven. De verandering van de geleidbaarheid is berekend op basis van beide meetmomenten. Voor beide locaties geldt dat de geleidbaarheid toeneemt, wat duidt op uittreden van zout van het slib naar het water. De toename is groter voor monsters die gemengd zijn met kraanwater en hierbij treedt dus meer zout uit.

2.2 Laboratorium proeven: Seepage Induced Consolidation (SIC)

SIC-proeven zijn proeven waarmee de doorlatendheid en korrelspanning na een langere periode van consolidatie kan worden nagebootst op kleine schaal in een relatief korte tijd (dagen in plaats van maanden/ jaren). Een relatief klein monster (hoogte +/- 5 cm) wordt in het SIC apparaat Figuur 2.2 geplaatst.

Figuur 2.2 : SIC opstelling bij Deltares.

Het SIC apparaat onttrekt gecontroleerd water uit het monster onder een vaste bovenbelasting door middel van een pomp. De drukgradiënt en het pompdebiet worden gemonitord om de doorlatendheid van het monster te bepalen. Door de onttrekking comprimeert het monster en kan een variatie in de hoogte van het monster gemeten worden. De belasting wordt stapsgewijs verhoogd om daadwerkelijke belastingen in het veld te simuleren. Met deze proef kan de doorlatendheid en korrelspanning als functie van het poriëngetal worden berekend voor langdurige consolidatie en hoge belasting in de diepere grondlagen. Deze parameters vormen de basis voor een vervolgstudie binnen de kleirijperij (Mathematisch modellering) waarbij met numerieke modellen (mogelijk FSCONGAS (Geodelft, 1999) en DELCON) de consolidatie op grotere tijd en ruimteschalen kan worden berekend.

2.2.1 Metingen en analysis

Voor de SIC proeven is alleen het slib uit Delfzijl getest, omdat de uitvoering zeer tijdsintensief is. Dit slib is gemengd met zand[1]. Voor en na de SIC proeven is de sterkte van het monster bepaald door middel van een spindel (vane) test. Uit deze metingen blijkt dat de schuifsterkte toeneemt met een factor 6 tot 7 door de consolidatie in de SIC.

Tijdens de SIC proeven is het monster stapsgewijs belast (van 100 Pa tot 6 kPa) en is water onttrokken aan het monster (0,2 – 0,8 ml/s). De totale testperiode bedroeg 23 dagen. In deze periode is de hoogte van het monster afgenomen van 54,0 mm tot 32,7 mm.

Op basis van de resultaten van slib met zand, de karakteristieken van het slib (met en zonder zand) en data uit eerde projecten met slib uit Delfzijl, is een analyse gemaakt van de doorlatendheid en korrelspanning van Delfzijl slib zonder zand. De effectieve korrelspanning

[1] Toen het onderzoek startte was naast de invloed van saliniteit ook de vraag hoe zand het slib kan beïnvloeden en werd de mogelijkheid opgehouden om het slib op te mengen met lokaal zand. Om deze reden zijn de SIC proeven uitgevoerd met monsters met zand. Voortschrijdend inzicht leidde ertoe dat opmengen niet is toegepast bij de Kleirijerpij pilots. Bij de interpretatie is daarom een vertaalslag gemaakt naar uitvoering zonder zand.

voor een zelfde poriëngetal is voor slib zonder zand groter dan met zand. De doorlatendheid is voor slib zonder zand kleiner (factor 10 – 100) en dit verschil neemt toe met een afnemend poriëngetal. Bij een poriëngetal van 5 bedraagt de voor het slibmengsel geschatte effectieve spanning 10 Pa en de doorlatendheid is dan 10^{-7} m/s. In de in de buurt van de maximale compressibiliteit (poriëngetal is 1,15) is de effectieve spanning 100 kPa en de doorlatendheid 10^{-11} m/s.

Met deze materiaaleigenschappen zijn de parameters bepaald voor het FSCONGAS model voor het slib uit de haven van Delfzijl. Verdere uitwerking van deze modelstudie zal in de rapportage betreffende (werkpakket 4.3).

3 Conclusies

Het doel van de laboratoriumproeven is het slib te karakteriseren buiten de in het Kwaliteitsplan Parameters Slib genoemde parameters. Tevens is gekeken naar de invloed van het zoutgehalte op bezinking en (begin van) consolidatie. Op deze manier wordt belangrijke input geleverd voor de mathematische modellering (start in 2018) en voor het ontwerp en indeling van de vakken van de pilot.

Het slib dat als uitgangsmateriaal genomen is voor de kleirijperij is beproefd met bezinkproeven. Op basis van de bezinkproeven kan geconcludeerd worden dat op de korte tijdschalen het zoutgehalte geen grote invloed heeft op de snelheid van sedimentatie, de eerste fase van de consolidatie, de dichtheid en de sterkte van het materiaal. Een hogere saliniteit leidt tot een hogere bodemhoogte (minder compactie) en lagere dichtheid. Het materiaal uit Breebaart consolideerde meer (in de proefperiode) in vergelijking met het materiaal uit Delfzijl. In Tabel 3.1 van Bijlage A staan de consolidatieparameters die als basis dienen voor het ontwerp van de pilotstudie in de vervolgstap. Wat het zoutgehalte betreft betekent dit, dat indien een of meer proefvakken vooraf verdund zouden worden met zoet water, dit geen effecten heeft op de bezinking en het begin van consolidatie (niet positief, maar ook niet negatief), terwijl daarmee wel meteen een significante hoeveelheid zout kan worden verwijderd. Door deze maatregel kan dus wellicht sneller worden voldaan aan de norm voor het maximale zoutgehalte.

De suction induced consolidation (SIC) test beschrijft het gedrag bij grotere spanningen en is daarmee een versnelde weergave van het consolidatiegedrag. De verkregen gegevens kunnen worden ingezet bij de modellering van deze fase. Daarbij wordt bij een verlaging van het poriengetal van 5 naar 1.2 een wijziging in doorlatendheid vastgesteld van 4 grootte-ordes.

Bij deze proeven is het onverzadigd gedrag nog niet in beeld gebracht, omdat dit langer duurt, en omdat juist de vertaling van laboratoriumresultaten naar veldresultaten onderwerp van onderzoek is. Op dit onderdeel zal dan ook de nadruk liggen bij het laboratoriumonderzoek dat parallel aan de pilot zal worden uitgevoerd.

4 Literatuur

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A Bijlage A: Kolomproeven bij variërend zoutgehalte

Deltares

Enabling Delta Life

Settling and consolidation behaviour of mud from Delfzijl and Breebaart at varying salinity

Settling and consolidation behaviour of mud from Delfzijl and Breebaart at varying salinity

Laboratory experiments Kleirijperij

Dr. Ebi Meshkati Shahmirzadi

11201344-000

Title
Settling and consolidation behaviour of mud from Delfzijl and Breebaart at varying salinity

Client	**Project**	**Reference**	**Pages**
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Keywords
Building with mud, Consolidation, Strength, Salinity, Delfzijl, Breebaart

Summary
The project Kleirijperij is a pilot study to assess the suitability of dredged mud as a source for (ripened) clay used as construction material in dikes. Using mud in dikes comes with specific requirements and for that reason it is vital to examine whether a mud type can be transformed into suitable dike clay or not. To answer this question several parameters should be studied such as consolidation behaviour of mud, the erosion class of final bed, final water and solid content, percentage of organic material, final salinity and contaminant levels. The objective of the present report is to study (parametrize) the consolidation behaviour of mud from Delfzijl and Breebaart at different salinity levels and to study (on a laboratory scale) the effect of salinity on consolidation and strength of these two mud types; i.e. whether salinity level could affect the strength of dredged mud after it is let to settle and consolidate.

An experimental study is conducted to understand the settling and consolidation behaviour of mud from Delfzijl and Breebaart, which are the sources of mud to be used in the pilot. The consolidation behaviour of these two mud types for three different salinity levels (including the original salinity level of each mud type) is parametrized; through calculating the following material parameters: the coefficient of permeability, the coefficient of effective stress and the fractal dimension. The effect of salinity on these material parameters is investigated. The density profile and strength of the final bed at the end of experimental period (93 days after the initiation of the experiment) are measured. The results show that:

1- Salinity (within the studied salinity range of 2-30 mS/cm and on the short term) played a minor role in the consolidation behaviour of both mud types.

2- Consolidation behaviour of mud from Delfzijl is less sensitive to change in salinity in comparison with mud from Breebaart.

3- For both mud types, higher salinity levels resulted in lower bed strength.

4- Flux of salt (leaching) from bed to supernatant water continued over the experimental period for both mud types but at different rate.

5- Salt flux from Delfzijl mud is slightly higher than salt flux mud from Breebaart.

6- Mud from Breebaart is more compact at the end of experimental period than mud from Delfzijl.

Deltares

Title
Settling and consolidation behaviour of mud from Delfzijl and Breebaart at varying salinity

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Rijkswaterstaat Noord Nederland	11201344-000	11201344-000-ZKS-0004	23

In conclusion, on the short term (initial three months), for both mud types under study no significant changes in consolidation properties as a result of salinity reduction are expected.

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	Dec. 2017	Ebi Meshkati Shahmirzadi		Bernadette Wichman		Frank Hoozemans	
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Contents

1 Introduction

The project Kleirijperij is a pilot study to assess the suitability of dredged mud as a source for (ripened) clay used as construction material in dikes. Using mud in dikes comes with specific requirements and for that reason it is vital to examine whether a mud type can be transformed into suitable dike clay or not. To answer this question several parameters should be studied such as consolidation behaviour of mud, the erosion class of final bed, final water and solid content, percentage of organic material, final salinity and contaminant levels. The objective of the present report is to study the consolidation behaviour of mud from Delfzijl and Breebaart at different salinity levels and to study (on a laboratory scale) the effect of salinity on settling, consolidation and strength of these two mud types; i.e. whether salinity level could affect the strength of dredged mud after it is let to settle and consolidate.

To tackle this research question, consolidation column tests are performed that aimed at better understanding the settling and consolidation properties of two mud types under study (dredged mud from Delfzijl and Breebaart) at different salinity levels. This report describes the experimental setup and the results obtained from the consolidation column tests.

2 Material and methods

2.1 Mud types

Two mud types are investigated in this study, namely mud from Delfzijl and Breebaart. Delfzijl mud samples are taken from the port of Delfzijl during the dredging campaign of spring 2017 directly from the port. Breebaart mud samples are taken at various locations of the Breebaart polder in spring 2017. The mud samples were delivered to Deltares in plastic buckets and immediately stored in the cold room. Since the delivered mud samples of both mud types were dense with very little supernatant water (original pore water) in the buckets, additional representative water from Delfzijl and Breebaart had to be supplied that is used for creating dilutions with different salinity levels from mud types under study. This representative water, so called native water hereafter, was taken in August 2017 from the port of Delfzijl and polder Breebaart. The electric conductivity (EC) of the native water and the supernatant water formed in the buckets were fairly close (Table 2.1)

Table 2.1 The electric conductivity (EC) of native water (used for dilution) and supernatant water (original water from pore)

Location	Type	Density [t/m3]	EC at 20 °C [mS/cm]
Delfzijl	Native water	1.008	31.4
Breebaart	Native water	1.008	29.1
Delfzijl	Supernatant water in bucket (D2*)	-	26.8
Breebaart	Supernatant water in bucket (B5*)	-	26.5

* Label on the bucket

A number of properties of both mud types under study are listed in the table below. These values are not produced by Deltares but reported by partners in the project.

Table 2.2 Characteristics of mud types under study.

Type	Clay fraction [w%]	Sand [w%]	Liquid limit	Plastic limit	Plasticity index	Density [t/m3]	Organic matter [w%]
Delfzijl	60	1	149	54	95	1.18	12-14
Breebaart	44	13	100	40	60	1.35	6-10

2.2 Research question

Standard consolidation experiments were performed with the settling columns, aiming at determining consolidation parameters of mud with change in the salinity. These experiments are based on the work of Dankers and Winterwerp (2007), Merckelbach and Kranenburg (2004) a and b, and Merckelbach (2000). Essentially, the experiments consist in letting settle and consolidate a relatively high-concentrated mud suspension in standard consolidation columns. By following the settling interface, called water – mud interface hereafter, the following parameters can be derived:

- Gelling concentration C_g (Dankers and Winterwerp, 2007);

- Permeability K_k and fractal dimension D (Merckelbach & Kranenburg, 2004 (a and b); Merckelbach, 2000; Winterwerp & van Kesteren, 2004);

- Effective stress coefficient K_{σ}, only when final bed level height is achieved (Merckelbach and Kranenburg, 2004 (a and b); Merckelbach, 2000).

The (consolidation) column tests are carried out in two separate batches, each serving a different purpose. These batches of tests and their purposes are as follow:

- Preliminary column tests: to roughly estimate the gelling concentration C_g of the mud types under study (i.e. mud from Delfzijl and Breebaart). The estimated gelling concentration from this batch of test is used for the design of the main column tests, so as to assure that the initial concentrations in the main columns are sufficient but well below the gelling concentration. Meeting this criterion for the main column tests is necessary as it guarantees that both settling and consolidation processes will be captured during the tests.

- Main column tests: to parametrize the consolidation behaviour of mud types under study and through this to answer the main research question of this study i.e. /how salt content affects the consolidation properties of the mud types under saturated condition?

2.3 Preliminary column tests

At first, using small scale columns (with 20 cm height and 5 cm diameter) pre-settling tests are carried out to roughly approximate a value for the gelling concentration of the mud types under study. The gelling concentration is known as the structural density at the transition between (hindered) settling and consolidation (C_g). For this, each mud type is diluted with its corresponding native water to four different initial concentrations (i.e. 20, 40, 60 and 80 [g/l]). The lowering of the water-mud interface of the diluted mud samples in the small columns is measured manually at every minute in the beginning with an increasing time interval as time progressed. These measurements are used to estimate the gelling concentration of each mud type (for detailed description see 3.1). The obtained gelling concentration of each mud type is an important parameter for design of the main column tests. For the main column tests the initial concentration in each column has to be smaller than gelling concentration of corresponding mud type in a column. This allows a wider spectrum for investigation of consolidation behaviour of mud. As mentioned earlier, meeting this criterion for the main column tests is necessary as it guarantees that both settling and consolidation processes will be captured during the tests. This is described in details in the following sections.

2.4 Main column test

The main consolidation column tests are performed using transparent polycarbonate columns with a height of 55 cm and an inner diameter of 10 cm. Settling and consolidation processes can be influenced by the inner diameter of the columns. Mieghem et al. (1997) recommended a minimal diameter of 10 cm for consolidation experiments. Hence, we can assume that an inner diameter of 10 cm induces only a minimal wall effect on the obtained results. The tests are carried out in a temperature-controlled room at 20 ± 1°C. Table below presents the experimental design for the main column. As can be seen, the main column tests are carried out with the initial concentrations of 50 and 60 [g/l] for mud from Delfzijl and Breebaart, respectively. The reason for selecting these initial concentrations is explained in the following sections. Three different salinity levels are provided for each mud type through combining native water with tap water (drinking water). Table below provides an overview of main the consolidation column tests designed for current study as well as the specific density of solids for each mud type.

Table 2.3 The experimental design for the main column tests

Test id	Mud type	Initial height of mud [m]	Initial Con. [g/l]	Gelling Con. [g/l]	Specific density of solids [t/m3]	Salinity type
col_0	D.*	0.4	50	55	2.54	native + tap water
col_1						native + tap water
col_2						native water only
col_3						tap water only
col_4	B.**		60	70	2.5	tap water only
col_5						native + tap water
col_6						native water only
col_7						tap water only

* Delfzijl **Breebaart

For each mud type, four columns are assigned. In three out of these four columns, a given weight of dry matter (for Delfzijl: 542.5 [g], for Breebaart: 603 [g]) and a given volume of water (water was added into the columns till the height of diluted mud in the columns reached to 40 cm i.e. total initial volume of 3.14 [litre]) with varying salinity are added. Three different levels of salinity are considered, namely high salinity (by adding only native water to mud sample in a column), medium salinity (by adding a combination of native water and tap water to mud sample), and low salinity (by adding only tap-water to mud sample). As for each mud type the initial weight of dry matter in columns is the same, the initial solid concentrations in all three columns is also the same and the only varying parameter is salinity. The electric conductivity (EC) of supernatant water in the columns is measured at the beginning of each test just after the initiation of the experiment and at the end of experimental period. This value is used as an indication for the salinity after dilution in the columns. In this study an attempt is made to find out if there is a relationship between EC and consolidation behaviour of mud types under study.

For each mud type, one set of conditions is considered as a duplicate test to study the repeatability of the results. Therefore, in total, 8 column tests are conducted. In all columns, the mud samples are diluted to a concentration below the gelling concentration obtained from the preliminary column tests. At the beginning of the experiment, the sediment-water mixture in the columns is gently stirred to get a uniform distribution over the settling column. Over time, the sediments settle in the column and an interface between the water-sediment mixture and the clear water above becomes visible. In the meantime, a bed starts to form at the bottom of each column. This situation is referred to as the hindered settling phase. After a while, the interface between clear water and the water-sediment mixture merges with the bed and phase I of consolidation starts. From here onwards, the bed starts to consolidate.

In the first phase of settling, before a consolidating bed is formed, sediments settle in the hindered settling regime. During this phase, the settling velocity and gelling concentration can be computed from the observations. When a consolidating bed is formed, the sediment / water interface coincides with the bed / water interface. Analysis of this interface allows the determination of consolidation parameters (see in section 3.3)

2.5 Measurement techniques

Basically three parameters were measured: water-mud interface with time, bed strength and density of final bed.

2.5.1 Water-mud interface

A camera (Canon EOS 800D) was used to take pictures of the columns at an increasing time interval, to be able to determine the position of the water-mud interface over time. At the beginning, every 30 seconds 1 image was taken but as time progresses the time interval between two successive images logarithmically increased until it reached a constant time interval of 16.66 minutes between two images. An example image is shown in figure below.

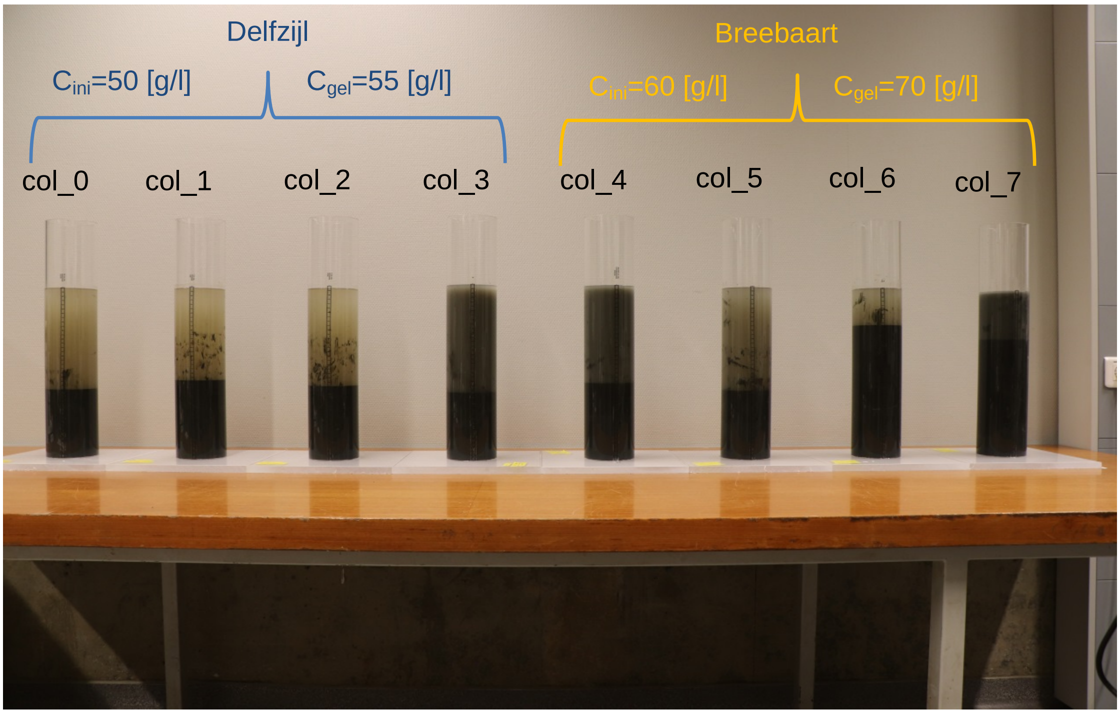

Figure 2.1 shows eight columns used for the main column tests in this study; in this image column number is in black; the initial concentration and gelling concentration of mud from Delfzijl and Breebaart is in blue and orange, respectively. Image is taken at August 21, 2017, 17:49:17, about an hour after the initiation of the experiment.

2.5.2 Density profile of final bed

Bulk densities of the bed in the segmented columns were measured with an Ultrasonic High Concentration Meter (UHCM) that is designed to measure high concentrations of solid particles in liquids [mk II, Delft Hydraulics]. The measuring principle is based on the attenuation of ultra sound by particles suspended between a pair of acoustic transducers. The acoustic principle is based on measuring the transmission of acoustic energy (ultra sound) through the measuring volume between the transmitter and the receiver probes (Berkhout, 1994). Before starting the measurement the device was calibrated in a sediment-water mixture of which the density was known. For better accuracy the calibration was done for 4 known densities. Calibrating the device for multiple points increases the accuracy. For calibration same material (mud type) was used, as particle size may influence the measurement. For the measurement the probes are lowered in the consolidated bed. A measurement was taken at a set interval of 1 cm over the vertical. This will give a density over height, a density profile.

2.5.3 Bed strength of final bed

Rheological measurement, peak shear stress (or failure strength) and residual shear stress (or remoulded strength) are determined with a vane test using a rotoviscometer [Haake Mars]. A vane was placed into the mud segment in each column (2.5 cm below the surface of final bed height) and rotated with a constant angular velocity of 0.5 RPM. The resistance of the sample was measured by a torque transducer between the rotating vane and motor. The protocol used in this study consists of a constant rotation rate of 0.5 RPM that is applied to the vane for 5 minutes. From this test two rheological parameters were determined, i.e. peak shear stress (the maximum shear during the test) and residual stress (the average shear during the last 2 minutes. The peak strength is a function of the rotation speed, stress history and the sample preparation and known as the highest rigidity of material at failure (transition point between elasticity and plasticity). The residual shear stress is a material property at given water content of the sample (Winterwerp & van Kesteren, 2004) and is defined as the residual stress in the bed after failure of a sample. A schematic view of shear stress development with time due to an applied constant rotating rate is shown in Figure 2.2.

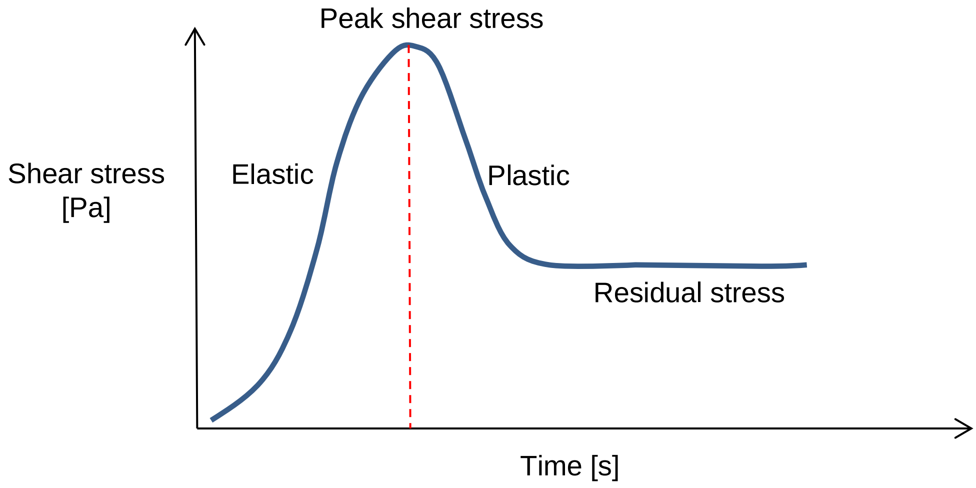

Figure 2.2 A schematic view of shear stress development with time.

2.6 Consolidation parameters

Consolidation behaviour of a mud type can be parameterized using the approach proposed by Merckelbach (2000). He classified the settling process of soft sediment into three categories as follows: first, hindered settling that occurs e.g. at $t < t1$, which is a process that is affected by the initial sediment concentration. Second, the first phase of consolidation, that occurs at *t1*<t<*t2* and is assumed to be only dominated by permeability. Third, the final phase of consolidation, in which the effective stress begins to play role and the influence of permeability fades out gradually (for *t*>*t2*). Merckelbach and Kranenburg (2004a) suggested that after a time period T_∞ (T_∞>*t2*) from the initiation of the experiment the second phase of consolidation reaches its final stage where the bed is at its final height (h_∞) and the effect of permeability can be neglected. Following Merckelbach (2000) theory, one can use the Gibson equation (Eq. 1), which is widely used to describe the consolidation behaviour of soft sediment, to parameterize the consolidation behaviour of a mud type during the first phase (assuming effective stress is negligible) and at the final stage of consolidation at T_∞ (assuming hydrostatic pore pressure). The assumptions for simplification of the Gibson equation and the resultant simplified equations will be presented in the following sections. The Gibson equation is:

$$\frac{\partial \Phi}{\partial t} - \rho_s' \frac{\partial}{\partial z}\left[k\Phi^2\right] - \frac{1}{\rho_w g}\frac{\partial}{\partial z}\left[k\Phi\frac{\partial \sigma_v'}{\partial z}\right] = 0 \tag{1}$$

Here Φ is the particle volume fraction, which can be estimated using $\Phi = \frac{\rho-\rho_w}{\rho_s-\rho_w}$, with ρ_s the density of solids, ρ is the bulk density and ρ_w the density of pore water. Furthermore, σ_v' denotes the effective (vertical) stress, $\rho_s' = (\rho_s - \rho_w)/\rho_w$ is the relative density, k is the permeability, t is time and z represents the vertical coordinate. Eq. 1 has three dependent variables: k, σ_v' and Φ. To solve this equation, two constitutive equations are needed. The following constitutive relations are suggested in Merckelbach and Kranenburg (2004a):

$$k = K_k \Phi^{-\frac{2}{3-D}} \tag{2}$$

$$\sigma_v' = K_\sigma \Phi^{\frac{2}{3-D}} + K_{\sigma,0} \tag{3}$$

where K_k is the permeability coefficient and D is the fractal dimension. In Eq. 2, K_σ is the effective stress coefficient and $K_{\sigma,0}$ is the creep. In this study, $K_{\sigma,0}$ is considered to be 0 (Merckelbach and Kranenburg, 2004b)

Note that, as suggested in Merckelbach (2000), K_k, K_σ and D are the consolidation parameters. These consolidation parameters are material specific. Obtaining these three parameters one can eventually estimate the permeability and effective stress development in a mud type (e.g. for given Φ values). In the following sections, it is described how to drive these parameters.

2.6.1 Permeability coefficient (K_k) and fractal dimension (D)

Merckelbach and Kranenburg (2000) proposed a method based on the temporal evolution of the mud-water interface to estimate K_k and D. This method assumes that in the initial phase of the consolidation, permeability is the only parameter that governs the consolidation processes; thus, the effect of effective stress is negligible σ_v'=0. Using Eq. 2 and the Gibson Equation (Eq. 1), the relationship between mud-water interface and time, during *t1* <*t*<*t2*, can be described as follows (de Boer et al, 2007):

$$h(t) = \left(\zeta\frac{2-n}{1-n}\right)^{\frac{1-n}{2-n}} \left[\rho_s'(n-2)K_k t\right]^{\frac{1}{2-n}} \tag{4}$$

where h is the mud-water interface, n is a dimensionless empirical exponent (see Eq. 7) and ζ is the material height, or Gibson height, which can be computed by:

$$\zeta = \frac{\rho_0 - \rho_w}{\rho_s - \rho_w} h_0 \qquad (5)$$

where ρ_0 is the initial density and h_0 is the initial height of mud in the column. Since very little sand was observed in the mud samples, the correction of Gibson height due to presence of sand is not applied in this study.

Plotting $h(t)$ versus t on a double logarithmic figure, a linear line can be fitted through the data points from which the effective permeability coefficient K_k and the fractal dimension D can be retrieved. Figure 2.3 illustrates the temporal evolution of the mud-water interface $h(t)$ on a double logarithmic scale for the mud from Delfzijl (test id: col_0). The figure clearly reveals the linear relation (on a double logarithmic plot) between h and t from time-points *t1* to *t*2. *t*1 is at the inflection point at the end of the hindered settling phase (when the straight line starts) and t2 is approximately at the point where the consolidation curve starts to deviate from the straight line. Choosing time-points *t1* and *t*2 is based on expert judgment. The linear line (on the double logarithmic scale) is highlighted in red in Figure 3.5. In order to parameterize the consolidation in this phase, by retrieving K_k and D from the first phase of consolidation where the process is dominated by permeability only, a linear fit between $h(t)$ and t (Eq. 4) on a double logarithmic figure is derived. Through this fit, the coefficients of permeability K_k and fractal dimension D are obtained. For detailed explanation of the method, we refer to Merckelbach (2000).

2.6.2 Effective Stress Coefficient

The Gibson Equation (Eq. 3) at the end of consolidation i.e. T_∞, can be written as follows (Merckelbach (2000)):

$$h_\infty = \frac{nK_\sigma}{(n-1)(\rho_s - \rho_w)g} \left(\frac{\rho_s - \rho_w}{K_\sigma} g\xi\right)^{\frac{n-1}{n}} \qquad (6)$$

Using the equation above and n (which is a function of D and can be calculated using Eq. 7) found for the first phase of consolidation described in Eq. 4, K_σ can be estimated from the final bed of water-mud interface. This means no density measurement is required to estimate K_σ, assuming that n at the end of consolidation equals to n for *t1 <t<t2*.

$$n = \frac{2}{3-D} \qquad (7)$$

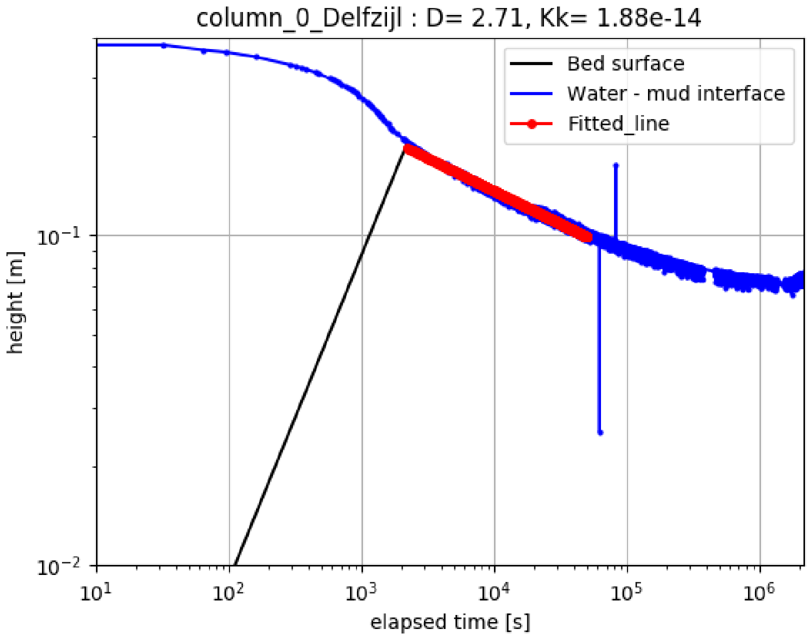

Figure 2.3 Temporal evolution of settling curve for test id col_0 (mud from Delfzijl) on a log-log scale.

3 Results

3.1 Gelling concentration

The preliminary column tests started on 16 August 2017 for the mud type from Delfzijl and on 17 August 2017 for the mud type from Breebaart. A total of eight tests are carried out in small columns: four tests for the mud from Delfzijl and four tests for the mud from Breebaart. Each mud type was diluted with its corresponding native water to four different initial concentrations. We aimed to have these initial concentrations below the approximated gelling concentration; however eventually we found that some of the tested initial concentrations were above the gelling concentration. For instance, in case of mud from Delfzijl, the initial concentrations of 60 and 80 [g/l], no significant reduction (about 1 mm) in the water - mud interface happens over the first 10 and 20 minutes after the initiation of the experiment. We believe this is because these initial concentrations are higher than the gelling concentration of mud from Delfzijl. Thus, for this mud type only those small scale column tests with the initial concentrations of 20 [g/l] and 40 [g/l] are used to approximate the gelling concentration. Similarly, for mud from Breebaart, it is found that the initial concentration of 80 [g/l] is above or very close to the gelling concentration as during the first 10 minutes of the experiment only 1 cm lowering of the water - mud interface was observed. For this reason, only those small column tests with the initial concentrations of 20, 40 and 60 [g/l] are used to identify the gelling concentration of mud from Breebaart. For both mud types, no interface between the hindered settling phase and the bed could be observed. In order to determine the gelling concentration, the settling curve is divided in two definite parts, first part from the beginning of the settling curve (*t*=0) till the knee of the settling curve (*t*=t1) that is characterized by a steep slope; and second part from the knee of the settling curve onwards that is characterized by a gentle slope. We fitted two linear lines through these two parts. The intersection of these two lines is a point (height) at which the gelling concentration occurs. Then, using the equation below the gelling concentration can be calculated:

$$C_g = \frac{C_{initial} * H_{initial}}{H_{intersection}} \tag{8}$$

where $C_{initial}$ is the initial concentration [g/l], $H_{initial}$ is the initial mud height in the column and $H_{intersection}$ is the mud height corresponding to the gelling concentration; as above mentioned, this height is the intersection point of two fitted lines through the steep part of the settling curve and the more gentle part of the settling curve. The gelling concentration of each mud type is then taken as the average value of the gelling concentrations obtained from the small column tests. The results of small column tests showed that the gelling concentrations of mud from Delfzijl and Breebaart are about 55 and 70 [g/l], respectively. This leads us to choose the initial concentration of 50 and 60 [g/l] for the main consolidation tests on mud from Delfzijl and Breebaart, respectively.

3.2 Temporal evolution of water – mud interface

The evolution of the mud - water interface (settling curve) in the main consolidation columns for mud types under study is shown in Figure 3.1 and Figure 3.2. These figures only depict the settling curve within 25 days after the initiation of the experiment. During this period, for both mud types, the (hindered) settling phase and first phase of consolidation is accomplished. Thus, material parameters K_k and D are derived from the settling curve obtained in first 25 days. In these figures both horizontal and vertical axis is logarithmic to profound the difference between the settling curves. Each curve in these figures represents a salinity level that can be found in the legend.

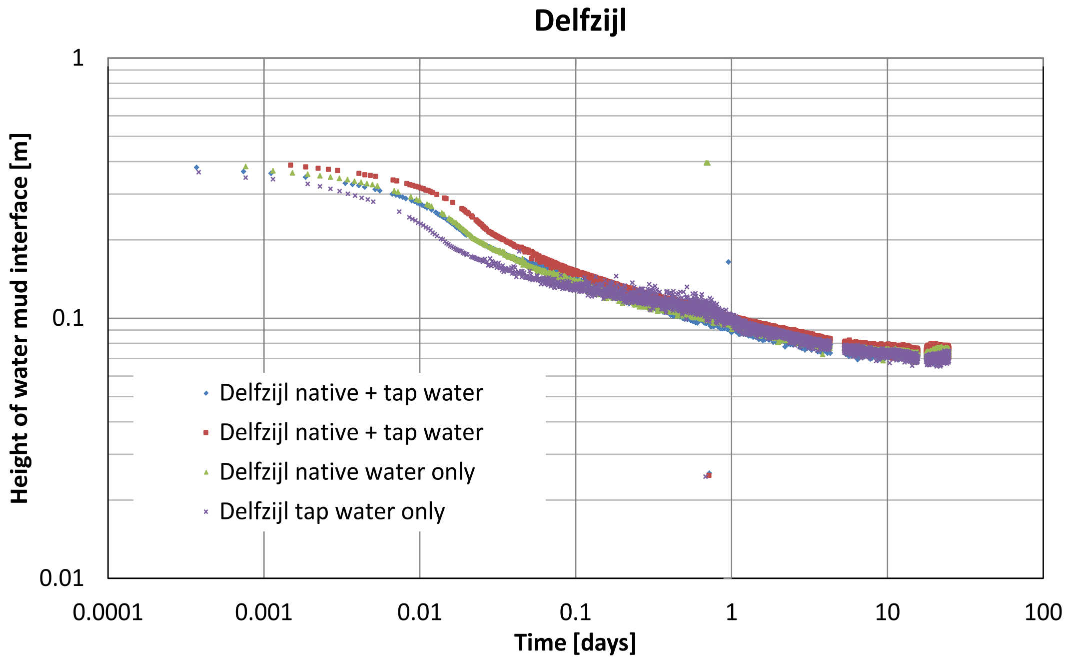

Figure 3.1 The settling curves of mud from Delfzijl from the initiation of the experiment until day 25 for different salinity levels on a log-log scale.

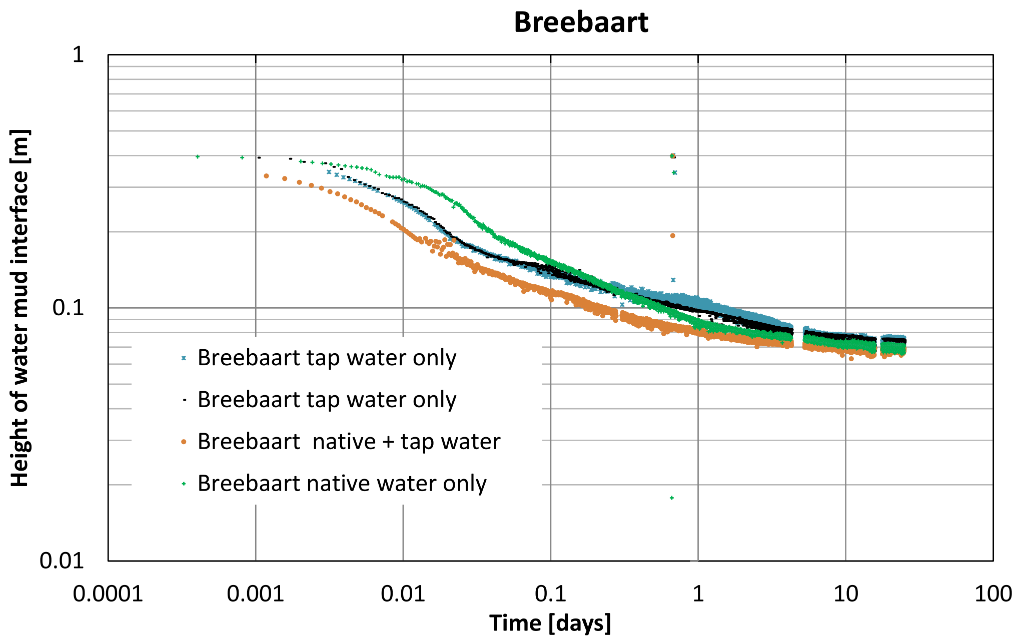

Figure 3.2 he settling curves of mud from Breebaart from the initiation of the experiment until day 25 for different salinity levels on a log-log scale.

3.3 The parameterization of consolidation behaviour of mud types

Table below provides the results from parameterization of the consolidation behaviour (K_k, D and K_σ) of mud types under study as well as the final bed height (h_∞) in the column at the end of experimental period i.e. days 93. Final bed height was manually measured through direct measurement.

Table 3.1 The results of parameterization of the consolidation behaviour of mud types under study.

Test id	Mud type	C_0 [g/l]	Salinity type	h_∞ [m]	K_k	D	K_σ
c0l_0	Delfzijl	50	native + tap water	0.074	1.88E-14	2.71	5.22E+08
col_1			native + tap water	0.075	1.81E-14	2.70	4.28E+08
col_2			native water only	0.074	4.87E-14	2.70	2.41E+08
col_3			tap water only	0.073	1.30E-14	2.72	1.02E+09
col_4	Breebaart	60	tap water only	0.073	1.36E-15	2.76	2.18E+09
col_5			native + tap water	0.070	9.44E-14	2.72	1.38E+08
col_6			native water only	0.073	4.74E-13	2.67	2.13E+07
col_7			tap water only	0.071	1.32E-15	2.76	1.73E+09

3.4 Effect of salinity on permeability

Using the consolidation parameters K_k and D, provided in Table 3.1, for a given range of void ratio (e) the permeability of mud sample in each column is estimated. This relationship for both mud types under study i.e. Delfzijl and Breebaart is depicted in Figure 3.3 and Figure 3.4. The permeability is strongly determined by e, and salinity appears to have a minor influence only. In case of mud from Delfzijl, moreover, the variation in permeability between duplicate tests is as large as the variation in permeability due to salinity. Hence, no clear trend in change in permeability due to salinity can be observed.

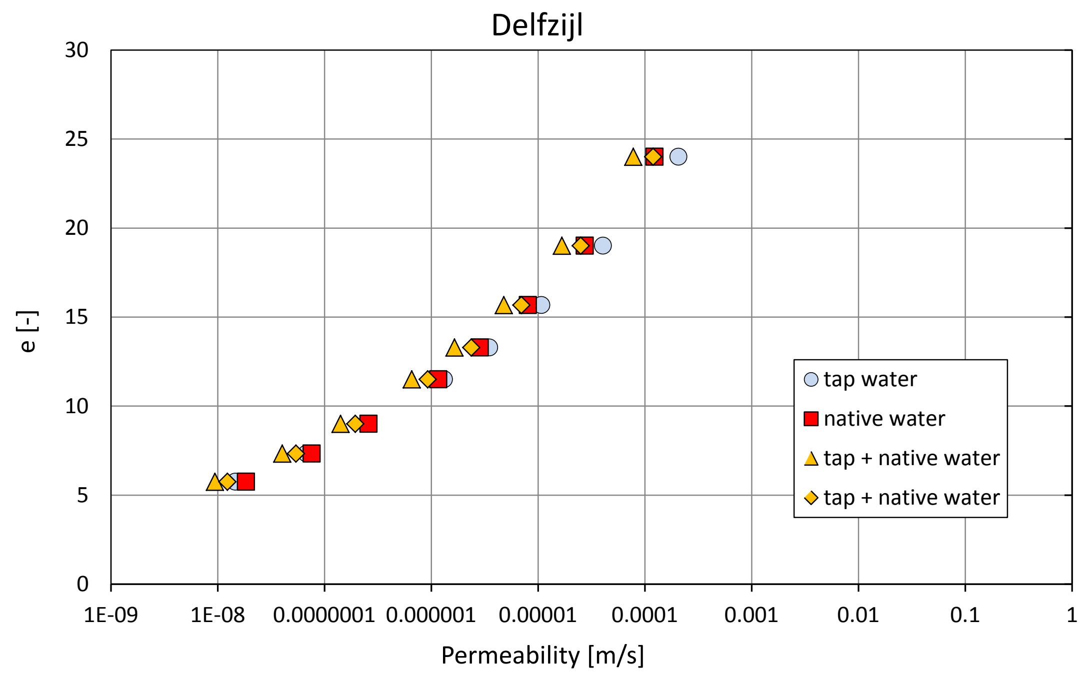

Figure 3.3 Relationship between e and permeability for mud from Delfzijl for different salinity levels.

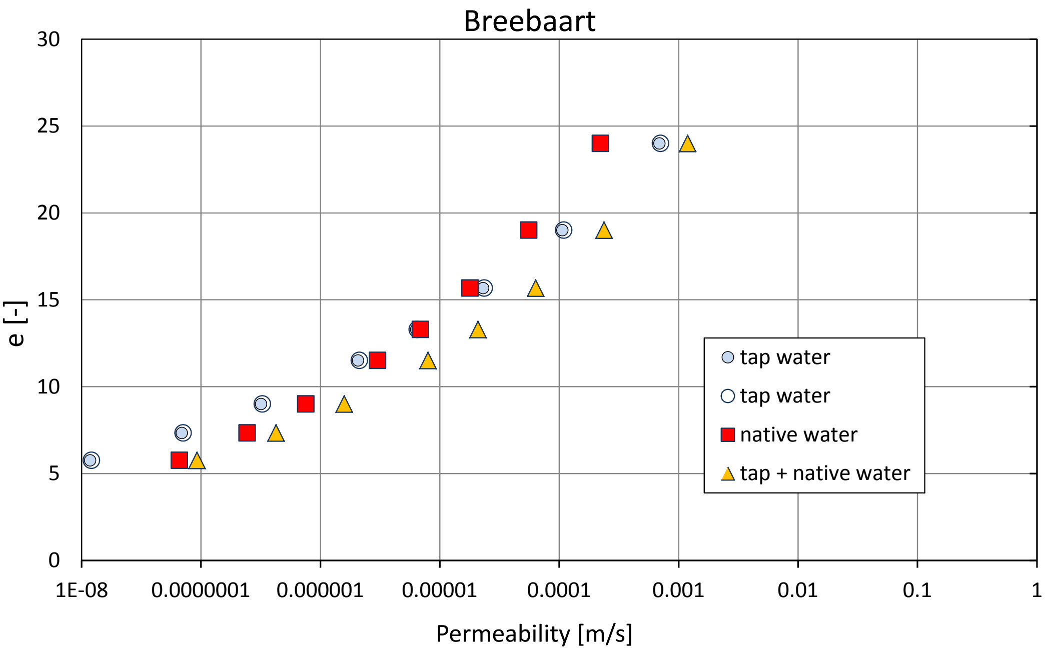

Figure 3.4 Relationship between e and permeability for mud from Breebaart for different salinity levels.

3.5 Effect of salinity on effective stress

Using the consolidation parameters K_σ and D, shown in Table 3.1 for a given range of void ratio (e) the effective stress of mud sample in each column is estimated. This relationship is depicted in Figure 3.5 and Figure 3.6 for mud from Delfzijl and Breebaart, respectively. It is found that in case of mud from Delfzijl, the estimated effective stresses of dilutions with different salinity levels are very close to each other. While the spread in estimated effective stress for mud from Breebaart is larger, in particular for higher void ratio values. This may indicate that the consolidation behaviour of mud from Delfzijl is less sensitive to change in salinity compared to mud from Breebaart. It is found that, in case of mud from Delfzijl, at the lowest void ratio (around 5 [-]), the estimated effective stress for the column diluted with tap water (low salinity level) is the highest; and for the column diluted with native water (high salinity level) is the lowest. This finding is in an agreement with the results of density profile and strength measurements (see section 3.6). Similar results are obtained for mud from Breebaart, suggesting that the lower the salinity level the higher is the strength of final bed. However, this is not completely approved by the density profile and strength measurements (see section 3.6). In case of mud from Breebaart it is found that column with medium level of salinity (tap + native water) has the largest measured density and strength.

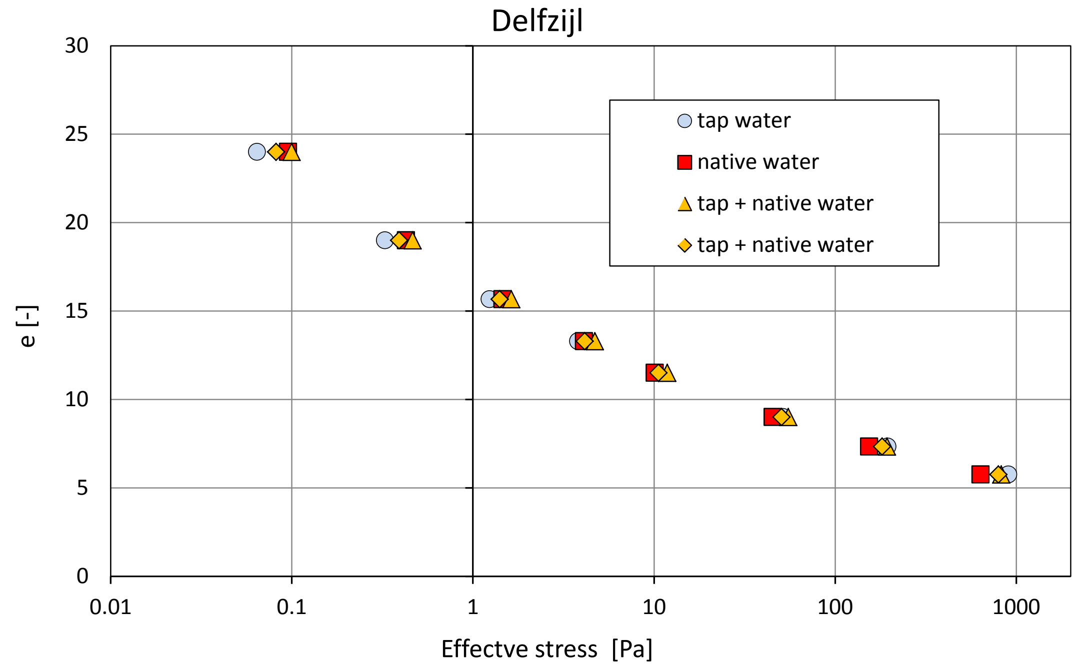

Figure 3.5 Relationship between e and effective stress for mud from Delfzijl for different salinity levels.

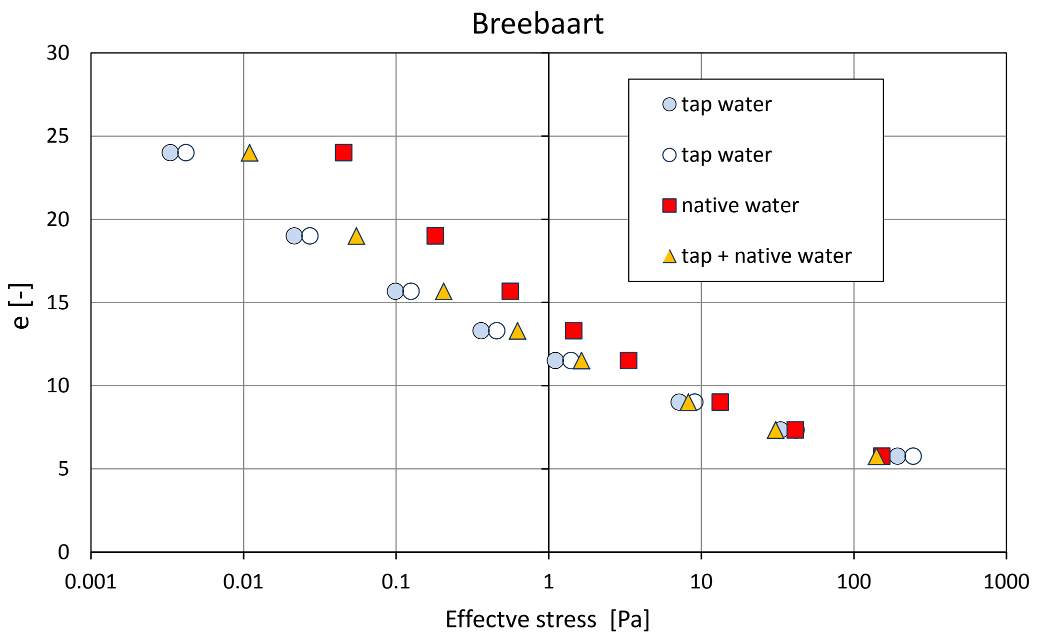

Figure 3.6 Relationship between e and effective stress for mud from Breebaart for different salinity levels.

3.6 Effect of salinity on density profile and strength

The density profile and strength of bed (at depth of 2.5 cm below the interface) of all columns used in the main column tests are measured. For density UHCM is used. For strength, the peak shear stress and residual stress of final bed are measured using a rotoviscometer at 0.5 RPM. Figure 3.7 and Figure 3.8 depict the density profiles along the depth for both mud types under study i.e. Delfzijl and Breebaart, respectively. In overall, it is observed that in all columns, the density increases with depth. In case of mud from Delfzijl, it is found that the column diluted with low level salinity (tap water) ends up with a smaller final bed height (see in Table 3.1), a relatively higher density and strength in comparison with other columns dedicated to mud from Delfzijl (Figure 3.7, Figure 3.9 and Figure 3.11). In case of mud from Breebaart, it is found that the column diluted with medium level salinity (native + tap water) ends up with a smaller final bed, higher density and strength (Figure 3.8, Figure 3.9 and Figure 3.10).

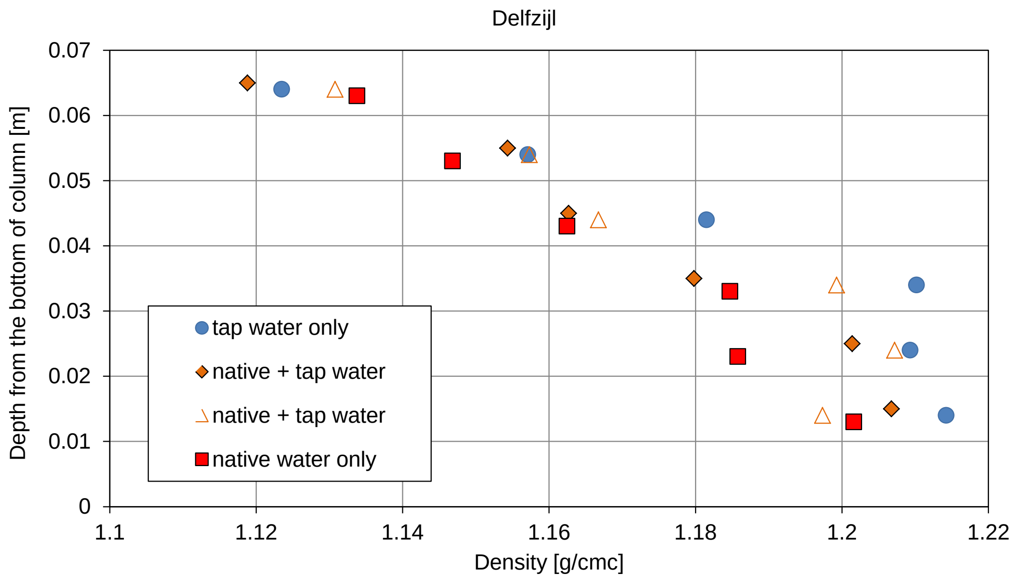

Figure 3.7 Density profile along the depth of those columns with mud from Delfzijl at the end of the experimental period (93 days after the initiation of experiment).

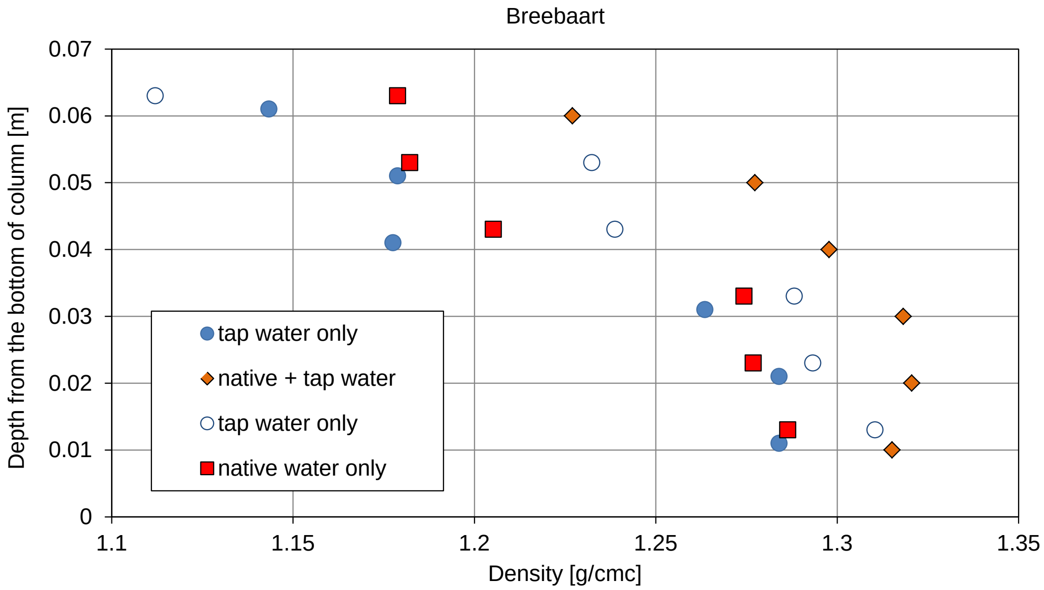

Figure 3.8 Density profile along the depth of those columns with mud from Delfzijl at the end of the experimental period (93 days after the initiation of experiment).

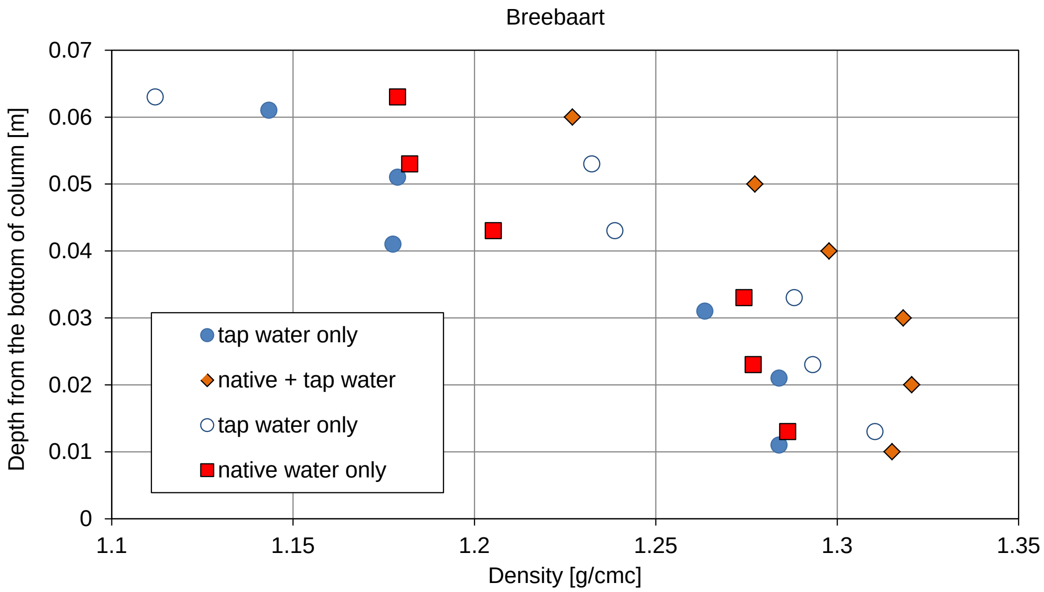

Figure 3.9 Density profile along the depth of those columns with mud from Breebaart at the end of the experimental period (93 days after the initiation of experiment).

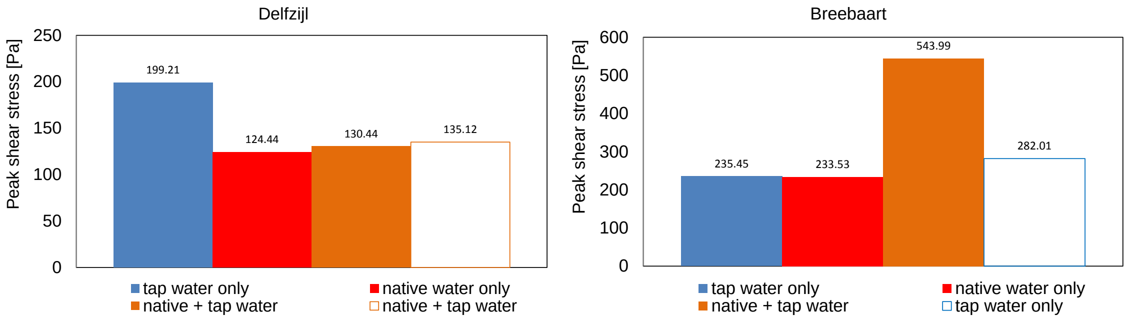

Figure 3.10 Strength (peak shear stress at 0.5 RPM rotation speed) measured at 2.5 cm below the water - mud interface of mud from Delfzijl (graph at left) and from Breebaart (graph at right) at the end of the experimental period (93 days after the initiation of experiment).

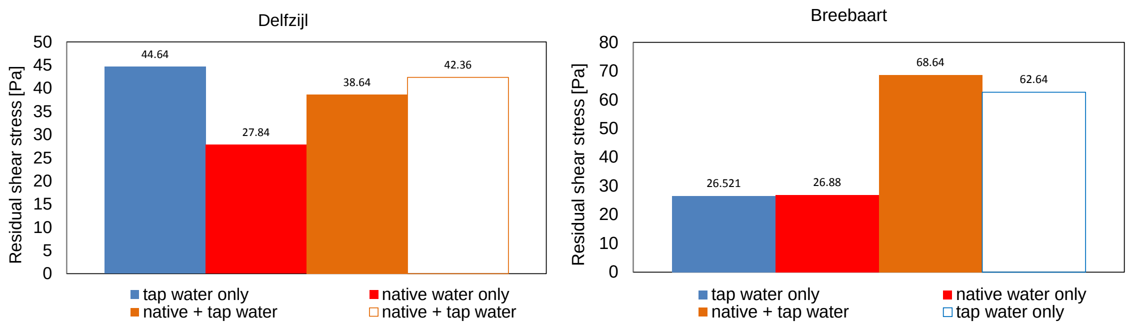

Figure 3.11: Strength (residual shear stress at 0.5 RPM rotation speed) measured at 2.5 cm below the water - mud interface of mud from Delfzijl (graph at left) and from Breebaart (graph at right at the end of the experimental period (93 days after the initiation of experiment).

3.7 Relative EC increase in supernatant water

Table below presents the experiments along with the initial measured EC (electric conductivity) of supernatant water in each column about 5 minutes after the initiation of the experiment (EC_{start}) and the final measured EC of supernatant water (EC_{end}) in each column at the end of experimental period (93 days after the initiation of the experiment). The last column of this table indicates the relative EC increase [-] from the consolidating mud over the experimental period. The rate of salt diffusion is calculated the following equation:

$$Relative\ EC\ increase = \frac{(EC_{end} - EC_{start})}{EC_{start}} \tag{9}$$

It is found that, in general, the relative EC increase in supernatant water of mud from Delfzijl is larger than mud from Breebaart. Also, the relative EC increase in diluted mud with lower salinity levels is larger than those of with higher salinity levels.

Table 3.2 The experimental design for the main column tests along with the measured initial EC.

Test id	Mud type	Init. Con. [g/l]	Salinity type	At the beginning of exp.		After 93 days		Rel.EC increase [-]
				EC [mS/cm]	Temp. [C]	EC [mS/cm]	Temp. [C]	
c0l_0	D.*	50	native + tap water	17.50	19.3	25.6	19.9	0.46
col_1			native + tap water	17.78	19.3	25.5	19.9	0.43
col_2			native water only	29.60	19.1	42.4	19.9	0.43
col_3			tap water only	4.06	19.1	6.93	20.1	0.71
col_4	B.**	60	tap water only	2.19	19.6	3.57	20.0	0.63
col_5			native + tap water	14.24	19.3	19.7	19.9	0.38
col_6			native water only	26.40	19.1	36.2	19.9	0.37
col_7			tap water only	2.108	19.6	3.34	20.0	0.58

* Delfzijl **Breebaart

4 Conclusions

In this study, the consolidation parameters of mud from Delfzijl and Breebaart at different salinity levels are obtained. It is found that:

1- Salinity (within the studied salinity range and on the short term) played a minor role in the consolidation behaviour of both mud types under study.

2- Consolidation behaviour of mud from Delfzijl is less sensitive to change in salinity in comparison with mud from Breebaart.

3- For both mud types columns with higher salinity levels resulted in a weaker bed (lower final bed strength).

4- For both mud types, salt leaching from bed to supernatant water in the columns continued over the experimental period.

5- Relative EC increase in the supernatant water in mud from Delfzijl is higher than mud from Breebaart.

6- Mud from Breebaart is more compact at the end of experimental period than mud from Delfzijl.

In conclusion, on the short term (about 3 months), for both mud types no significant changes in consolidation properties as a result of salinity reduction are expected.

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B Bijlage B: Suction Induced Consolidation Test (SIC)

Deltares
Enabling Delta Life

Seepage induced consolidation test

On Delfzijl mud with 47% sand content (46% added)
Project kleirijperij

Seepage induced consolidation test

On Delfzijl mud with 47% sand content (46% added)
Project kleirijperij

dr. B.G.H.M. Wichman

1210724-000

Deltares

Title

Seepage induced consolidation test

Project	**Reference**	**Pages**
1210724-000	1210724-000-ZKS-0004-gbh	24

Keywords
SIC, consolidation, parameter determination

Summary
A suction induced consolidation test (SIC) has been performed to obtain a reference for the consolidation parameters of Delfzijl mud. This mud will be used in ripening fields in the Kleirijperij project. Parameter relations have been obtained to be used in FSCongas- or Delcon- consolidation models that are based on the finite strain consolidation theory. Also time dependent effects that are influencing the consolidation process are discussed.

Version	Date	Author	Initials	Review	Initials	Approval	Initials
	nov. 2017	dr. B.G.H.M. Wichman		dr. G. Greeuw		ir. L. Voogt	

State
draft
This is a draft report, intended for discussion purposes only. No part of this report may be relied upon by either principals or third parties.

Contents

Nederlandse samenvatting

In het kader van het project Kleirijperij zullen er in het voorjaar van 2018 testvelden worden aangelegd. Binnendijks, nabij de Haven van Delfzijl, zullen er velden worden aangelegd met sediment uit de haven van Delfzijl.
In het proces van kleirijping, zal het gebaggerde sediment eerst sedimenteren, wat relatief snel gaat, en daarna treedt er geleidelijk zetting op ten gevolge van consolidatie. In de zomermaanden kan zich aan de bovenkant al een rijpingslaag vormen, terwijl de onderste sliblagen nog consolideren. Deze rijpingslaag kan gezien worden als een bovenbelasting in de consolidatieberekening. Als er voldoende scheuren in deze rijpingslaag ontstaan zal de bovenzijde goed draineren. Dit alles bevordert de ontwatering van de onderste lagen.
Verkennende berekeningen en praktijkervaringen laten zien dat consolidatie een belangrijke factor is in het verkrijgen van gerijpte klei met de juiste consistentie.
Er is nagedacht over zand bijmenging, om de consolidatiesnelheid te vergroten. Ook veranderen hiermee de Atterbergse grenzen, en mogelijk ook de erosieklasse. Door rijping kunnen de Atterbergse grenzen ook veranderen. Consolidatie sec, heeft geen effect op de Atterbergse grenzen.
Om de juiste consistentie van de klei te verkrijgen, zal er ook landfarming moeten worden toegepast. Dit vergt een begaanbare laag klei. Consolidatie vergroot de ongedraineerde schuifsterkte en daarmee de begaanbaarheid. Rijping speelt hierbij ook een rol.

In dit rapport wordt verslag gedaan van een speciale laboratoriumproef, met begeleidende metingen, de zogenaamde Seepage Induced Consolidation Test (SIC). Hierin is het gebaggerde slib belast tot een effectieve belasting van 1,5 meter sliblaag. In de SIC kan de consolidatie van zacht materiaal bij dergelijke lage belastingen goed worden bemeten. Stapsgewijs wordt de belasting verhoogd, waarna het monster consolideert en bij iedere stap wordt de doorlatendheid bepaald. We verkrijgen zo een serie punten met effectieve spanning als functie van het poriegetal (= volume poriewater/volume droge stof) en ook een serie punten met doorlatendheid als functie van het poriegetal. De consolidatie programma’s FSCONGAS en DELCON gebruiken fits verkregen uit deze data, die de gewenste consolidatie parameters geven.

Het materiaal dat in de SIC is gebruikt, komt uit de beun van een baggervaartuig, dichtbij het inlaatpunt, tijdens het baggeren van vak 3 in de haven van Delfzijl. Er is zand bijgemengd, tot een zandgehalte van 47%. Dit zand komt ook uit de haven van Delfzijl.
Er is gebruik gemaakt van de Deltares slibdatabase, om de consolidatie parameters uit de SIC te valideren, en om consolidatieparameters af te leiden voor het slib zonder zand-bijmenging, met zandgehalte van 1%. In dit rapport zijn de FSCONGAS- en DELCON-parameters, zoals verkregen uit de SIC-test, gegeven.

Er is ook aandacht besteed aan tijdsafhankelijke processen die de consolidatie beïnvloeden. In de SIC-test is aandacht besteed aan effecten die duiden op zwel.

1 Objective

A Seepage Induced Consolidation test (SIC) has been carried out by Deltares on mud from Bucket 3 from the Delfzijl Harbour, where 46% of sand by dry weight was added, leading to a sand content of 47% (expected value, grain size distribution of total has still to be determined). The sand was obtained from the Delfzijl harbour too. This will give insight in the improvement of consolidation properties when adding sand. In section 5.2 also Delfzijl mud without addition of sand is treated. Here the consolidation properties are evaluated and estimated, using the SIC results.

The consolidation parameters from section 5.2, i.e. without adding sand, will be used in the analysis of the test sections. This will be reported separately.

The primary objective of the SIC test is to measure the consolidation properties of very soft sediment with strength lower than about 5 kPa. The SIC test differs from the standard oedometer consolidation test in that it accounts for two consolidation mechanisms: (1) loading by a constant discharge through the sample, which results in consolidation primarily at the lower boundary; and (2) loading by an external load, which results in consolidation over the full height of the sample. This procedure is ideal for determining the consolidation properties of very soft soils with high water content. In the present research the discharge option (1) has not been used for consolidation, but for determination of the permeability.

The test was limited to the determination of the consolidation properties of the mud, excluding gas specific testing. The results of this test can be utilized for assessing and modelling the consolidating and strength properties of the mud.

Specifically, the SIC test is run to determine the:
1. Compressibility of the sediment as a function of the void ratio.
2. Permeability of the sediment as a function of the void ratio.
3. Amount of water released during consolidation.

On the basis of the SIC-data and the ‘Deltares mud database’ consolidation parameter relations for mud with varying sand content have been considered. These insights can be used in simulations of the field situation.

2 Experimental methods

2.1 Experimental Setup

The SIC test setup, shown in a photo on Figure 2.1 and flow diagram in Figure 2.2, includes two main mechanical components:

1. The flow controlled piston pump.
2. The triaxial cell, which contains a brass ring sample holder (diameter: 151 mm, height: 60 mm) with filter stones and filter paper on both ends.

The piston pump generates a precisely controlled downward flow rate through the sample, imposing a suction force to the lower drain system. In the drain system the hydraulic pressure is measured with respect to the cell pressure. The triaxial cell is connected to a motor that accurately controls the displacement of the sample, imposing a precise load. The settlement is measured with a displacement gauge. On top of the sample different boundary conditions can be applied. The top plate has a chamber in which different liquids can be placed or used for gas or bitumen migration testing.

External load steps are applied to induce consolidation, while suction is utilized for measuring permeability.

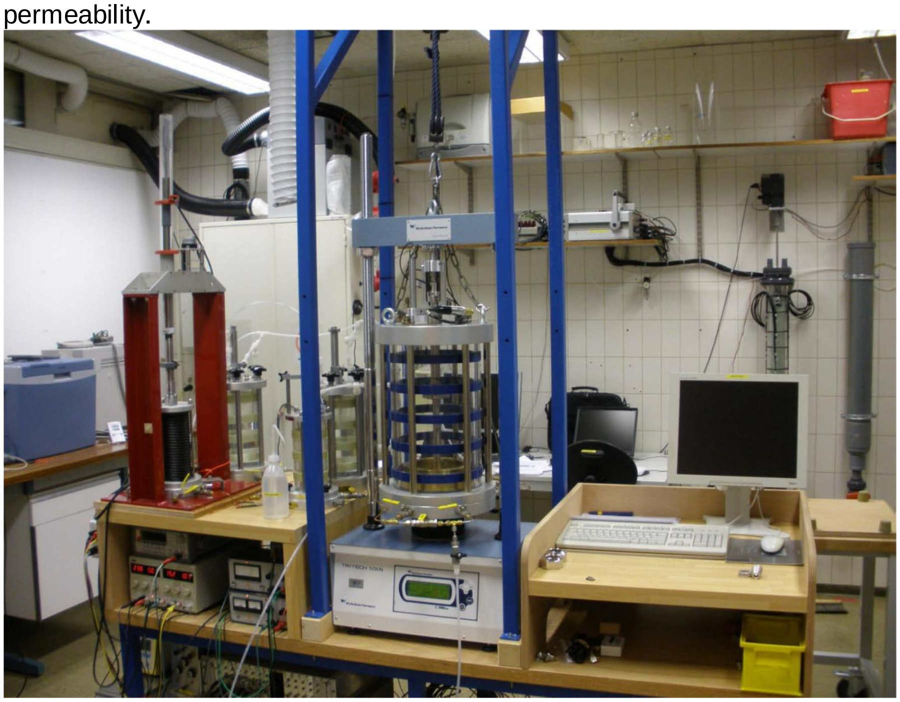


Figure 2.1 Photo of the SIC apparatus at Deltares. The sample holder is in the lower part of the cell. Underneath the cell, the motor controls the imposed external load by controlled displacement of the sample against the fixed piston inside the cell. To the left of the cell the automatic pump is for pore water suction and to the right the computer for test control and data storage

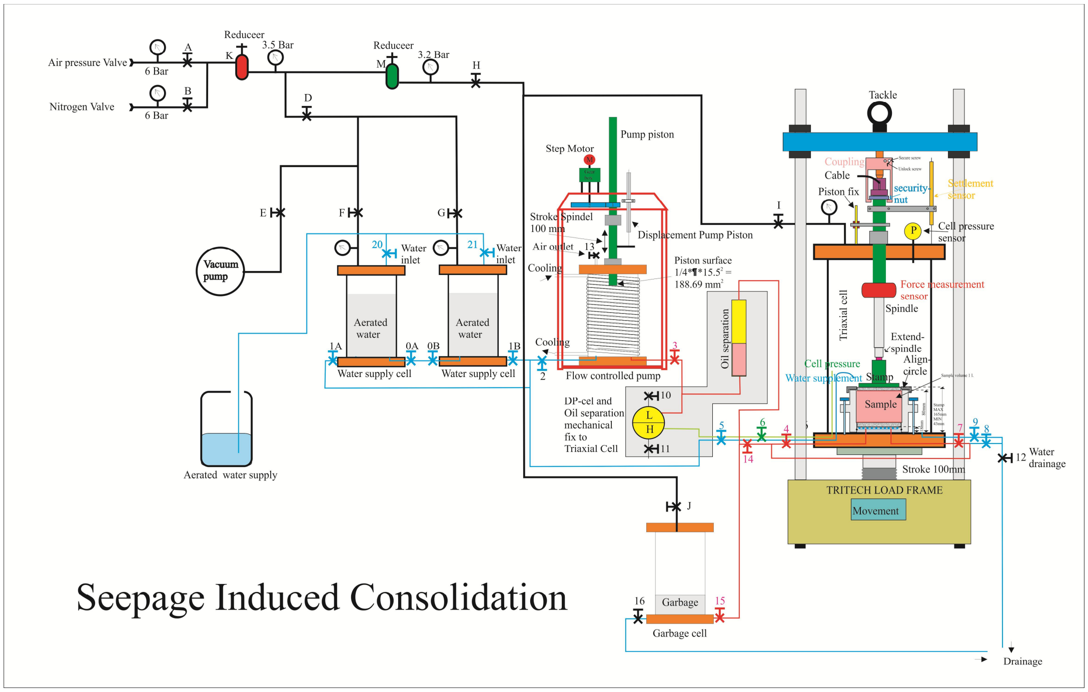


Figure 2.2 Schematic flow diagram of the SIC apparatus

When an external load is applied to consolidate the sample, the lower drain is generally not allowed to drain pore water. As a result, expelled pore water is drained exclusively through the top of the sample. However, to decrease test duration, it is possible to execute the test with dual top and bottom drainage.
When active, the flow controlled pump generates a negative pressure in the drain system that forces pore fluid to flow downwards (seepage). In between consolidation steps, the permeability as a function of void ratio can be determined measuring the pressure difference resulting from different flow rates.
At all times, pore water pressure and settling of the sample are monitored until re-equilibration to determine the end of each consolidation and permeability step, and ultimately the duration of the test.

2.2 Test procedure

2.2.1 Sample preparation

The mud of the large Bucket 3, originating from the Delfzijl Harbour section 3 (taken from the vessel while dredging), has the properties reported in Table 2.1 and Table 2.2. The overlying water in the bucket was removed before mixing the mud.
The mud from Bucket 3, without the supernatant water was mixed with dry sand in a proportion of 200/800 of total bulk mass, leading to a sand content of 47%, i.e. with 46% extra sand. This will give insight in the possible improvement of consolidation speed due to addition of sand. The resulting grain size distribution is given Table 2.2. This mud was used in the SIC. The water was taken from the mud buckets, being the supernatant water. This water is typical salt water, with a specific density of 1020 kg/m^3. Later, it was decided to study Delfzijl mud without addition of sand. Parameters for that are estimated in section 5.2.

Table 2.1 Some properties of the mud (with sand added)

Property	Value [unit]
Loss on ignition (organic content) after adding sand.	6.6 [%ds, i.e. with respect to dry solid mass]
CaCO3 content.	9.8 [%ds, i.e. with respect to dry solid mass]
Bulk density (without supernatant water) before adding the sand.	1235 [kg/m^3]
Bulk density after adding sand (200/800 of total mass added).	1378 [kg/m^3]
Specific density of solid particles after adding sand.	2593 [kg/m^3]
Specific density of the water used.	1020 [kg/m^3]

Table 2.2 Grain size distribution for mud as supplied in Bucket 3 (location Harbour section 3) and the grain size distribution after 46% (mass by dry solids) of sand had been added

mud type	grains size distribution		
	< 2 µm	< 16 µm	> 63 µm
	[%]	[%]	[%]
Delfzijl mud section 3	61.4	94.9	1
Delfzijl mud from SIC	33	51	47

The overlying water in the buckets was removed before mixing the mud. The mud was put into the buckets at a higher water content than what remained after settling. Removing water, i.e. modifying the initial concentration may influence consolidation at very low effective stress when consolidation starts above gelling concentration. The overlying water, than had been expelled from the dredged mud, was used in the SIC, as the chemical properties of the water around the SIC sample should be similar as for the pore water in the sample.
Before introducing the sample into the SIC apparatus, the sample was homogenized and the water content was determined. Also the undrained shear strength was measured with a laboratory vane, see Section 2.2.2. The sample was placed in the brass ring of the SIC apparatus.
The sample in the SIC was left in the sample ring for 3 days before loading. In that period the mud gains strength (due to thixotropy, see section 5.3.2), which will also occur in the field situation where consolidation is slower than in the SIC. No further mixing of the sample was performed before loading. In these tests the strength of the remaining mixed mud in the bucket was determined at the time the SIC sample was loaded with the first load, i.e. 3 days after mixing.

2.2.2 Vane tests

Before and after the four SIC tests, peak and remoulded shear strength was measured with a laboratory vane. The peak shear strength and the remoulded shear strength of a muddy soil can be determined with a vane test. A vane (Figure 2.3, right) is placed in at some depth in the soil, with the marker just at the surface. The vane test is a strain rate controlled test, with a rotation rate of the spindles of 0.512 RPM. The resistance of the sample is measured by a torque transducer between the rotating vane and motor. The recorded signal (Figure 2.3, left figure) is multiplied by the A-factor, which is dependent on the properties of the vane. The peak strength is a function of the rotation speed, stress history and the sample preparation.

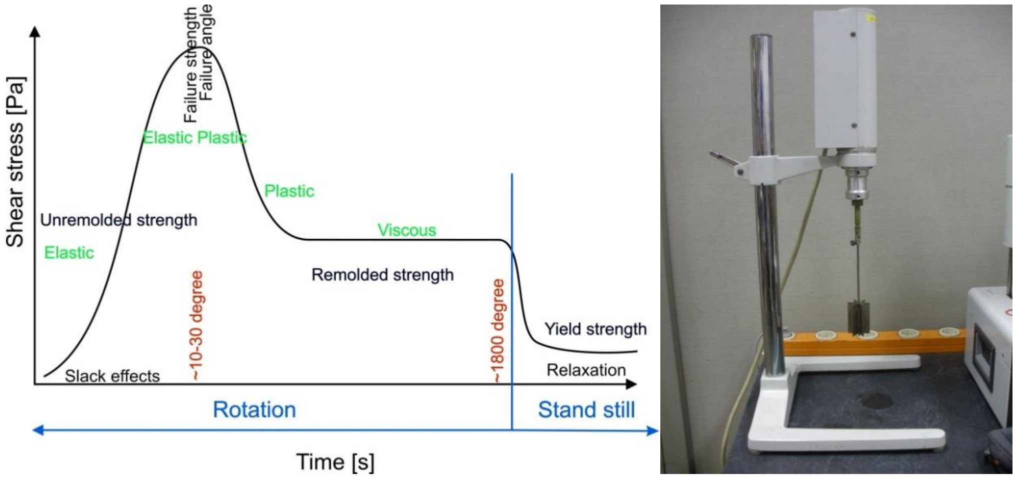


Figure 2.3 Vane results schematization (left) and Vane apparatus (right)

The remoulded shear strength is defined as the residual strength in the bed after failure of a sample.
The vane measurements were performed with the FL100 element (i.e. a large vane suitable for low shear strength) on the sample before execution of the SIC test. Vane measurements after the SIC test were performed with the FL1000 element (i.e. a small vane suitable for high shear strength). Thus an appropriate sensitivity was obtained.

2.2.3 SIC test

The SIC test was run with subsequent consolidation and permeability steps. Each step was characterized by an incremental load imposed on the sample, from about 100 Pa to 6 kPa with six logarithmic incremental steps (see Table 2.3). An effective stress range up to 6 kPa is sufficient to simulate a layer thickness of about 1.5 meter, which is the maximum in the field situation.
The consolidation steps were undrained at the bottom of the sample, which enables measurement of the excess pore water pressure in time.
Initially, the sample in the triaxial cell was submerged into a few centimetres of water that was taken from the mud buckets. This ensures that no air was entrained into the sample during the suction steps. Subsequently, consolidation was induced by application of constant external load, followed by a constant suction force.

Table 2.3 Consolidation and seepage steps for test SIC

Phase	Load [N]	Load [Pa]	Discharge [mm^3/s]
Consolidation	2	112	
Consolidation	5	279	
Permeability			0.2
Permeability			0.5
Permeability			0.8
Consolidation	10	558	
Permeability			0.2
Permeability			0.5

Phase	Load	Load	Discharge
Permeability			0.8
Consolidation	20	1117	
Permeability			0.5
Permeability			0.8
Consolidation	50	2809	
Permeability			0.5
Permeability			0.8
Consolidation	100	5584	
Permeability			0.3
Permeability			0.5

After equilibration or after a pre-determined time period (-each new loading step was let consolidate for typical three days), a permeability test was performed. Some loading steps lasted longer and will show more creep. The permeability of the sample was measured imposing suction at the bottom of the sample during a short period, followed by a re-equilibration phase. During the permeability step, the external load was maintained. The imposed discharge rate is adjusted to induce a clear differential pressure, being necessary to establish the permeability parameters. Generally, permeability is lower as the load increases, and therefore the imposed discharge rate should be lower to avoid too high pressure gradients and thus sample disturbance. When the sample returned to the equilibrium pore water pressure, the subsequent permeability step was carried out. The permeability was measured at two or three discharges. At times, a third permeability test was carried out to verify accuracy of the data. At 2N no permeability tests were carried out, because the sample did not consolidate at that stage, but seemed to have a tendency to swell (see sections 4.3.3 and 4.3.4).

The same procedure of consolidation and permeability steps was repeated until the end of the test; see Table 2.3.

After loading to 2 N, several trials have been performed to test the fabric of the sample. This fabric seemed to have a strong tendency to resist the imposed load, i.e. the sample height did not decrease. There was a gradual increase of the measured load. As the sample height is only regulated when the force drops below the imposed value, the sample height will stay constant if the (resisting) force increases. This increase in force that was possibly due to the tendency of the sample to swell. All this might be influenced by wall friction too.

At the end of the test, the sample was taken from the SIC and its dimensions, weight and water content were determined. Also vane tests were performed on this sample.

3 Data processing and calculations

3.1 Data collection

During each test the following data were collected:

1. Differential pore water pressure, in the drain below the sample.
2. Vertical displacement of piston pump.
3. Vertical displacement of the triaxial cell piston at the bottom.
4. Absolute pressure outside cell.
5. Pressure inside cell.
6. Temperature.

The first three measurements are used for the analysis and allow calculating effective stress and permeability as function of void ratio. Before and after the SIC tests the water content and undrained shear strength (peak and remoulded) were also measured with a laboratory vane (see Section **Error! Reference source not found.**).

3.2 Data processing

The data measured with the SIC test can be fitted (using least squares) with an exponential or a power law model for both compressibility and permeability as a function of void ratio. The program FSCONGAS uses exponential relations; DELCON uses power law functions.

3.2.1 FSCONGAS parameters

The compressibility function reads:

$$\sigma'(e) = \sigma_0 \exp(m1 + m2 \cdot e) \tag{3.1}$$

Where *e* is void ratio, σ' is vertical effective stress [kPa], m1 and m2 coefficients; σ_0 = 1 kPa.

The permeability function reads:

$$k(e) = k_0 \exp(n1 + n2 \cdot e) \tag{3.2}$$

Where *e* is void ratio, k is vertical permeability [m/s], n1 and n2 coefficients; k_0 = 1 m/s.

These relations give a straight line on single log-scale.

3.2.2 DELCON parameters

The compressibility function reads:

$$e = A(\sigma' + Z)^B \tag{3.3}$$

Where *e* is void ratio, σ' is vertical effective stress [kPa], *A* and *B* coefficients of the power law model and *Z* is coupled to the void ratio at zero effective stress. To simplify the fitting we have assumed that Z = 0; in this case we get a linear relation on a log-log scale with slope *B*.

The permeability function reads:

$$k = C e^{D} \tag{3.4}$$

Where e is void ratio, k is permeability [m/s], C and D are coefficients of the power law model.

These relations give a straight line on double log-scale.

4 Results

4.1 Results from the SIC tests

4.1.1 DELCON parameters

The data collected with the SIC tests allow for determination of the DELCON parameters A, B, C and D defined in Section 3.2, which are the soil specific parameters. Figure 4.1 shows the power law fit for permeability versus effective stress for sample SIC1. Note that in some cases we have two or three permeability values at the same effective stress, corresponding to different discharge rates. These are all reported to highlight possible parameters variation. In addition, the permeability can be correlated with the void ratio in Figure 4.2. Note that for the permeability at the respective flow rates, two values are given (k_high and k_low). These are upper and lower limits due to the variation in the differential pressure over the sample before and after a suction test. Void ratio as a function of effective stress is plotted in Figure 4.3. Also the values of the undrained shear strength Cu, as obtained from vane-tests, have been added, see Section 4.2. Note that the void ratio is derived from the settlement at the end of the load step, including the gas that might have been produced, see section 4.3. Since the load steps are not equal in time, the amount of creep settlement may differ over the various steps. In order to make a better comparison, error margins on the effective stress were added that indicate the residual pore pressure left at the moment the void ratio was calculated, see Section 4.3.

All these DELCON parameter relations show clear correlations using a single power function, against total void ratio.

The DELCON parameter relations are in terms of total void ratio (water and gas). The obtained values for the fit parameters are given in Table 4.1.

Table 4.1 Soil Parameters for SIC by fitting the experimental data (DELCON model)

Sample	A	B	C	D
SIC on Delfzijl mud	2.3979	-0.215	2.91 E-09	2.37

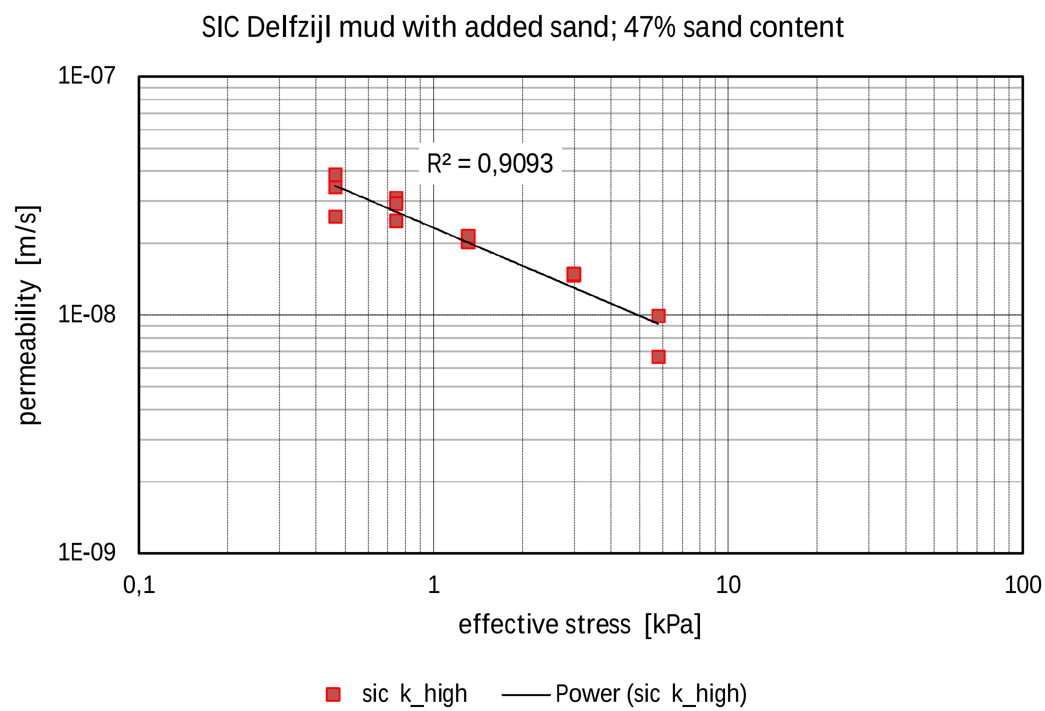


Figure 4.1 Permeability (upper limit) as function of effective stress for sample SIC

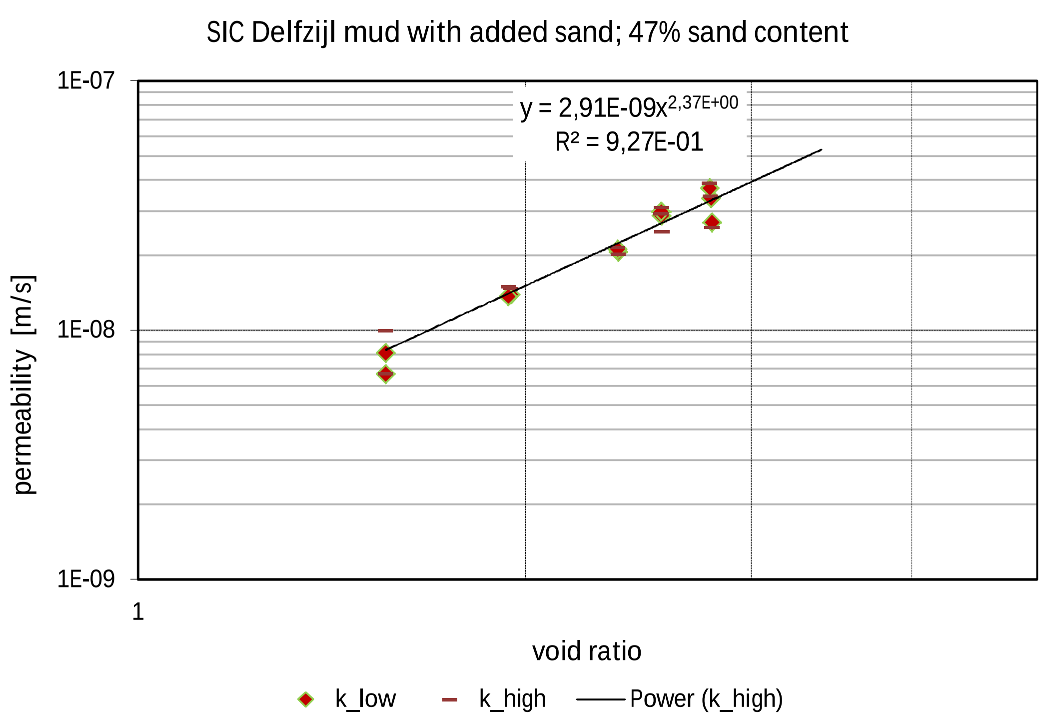


Figure 4.2 Permeability as function of void ratio for sample SIC

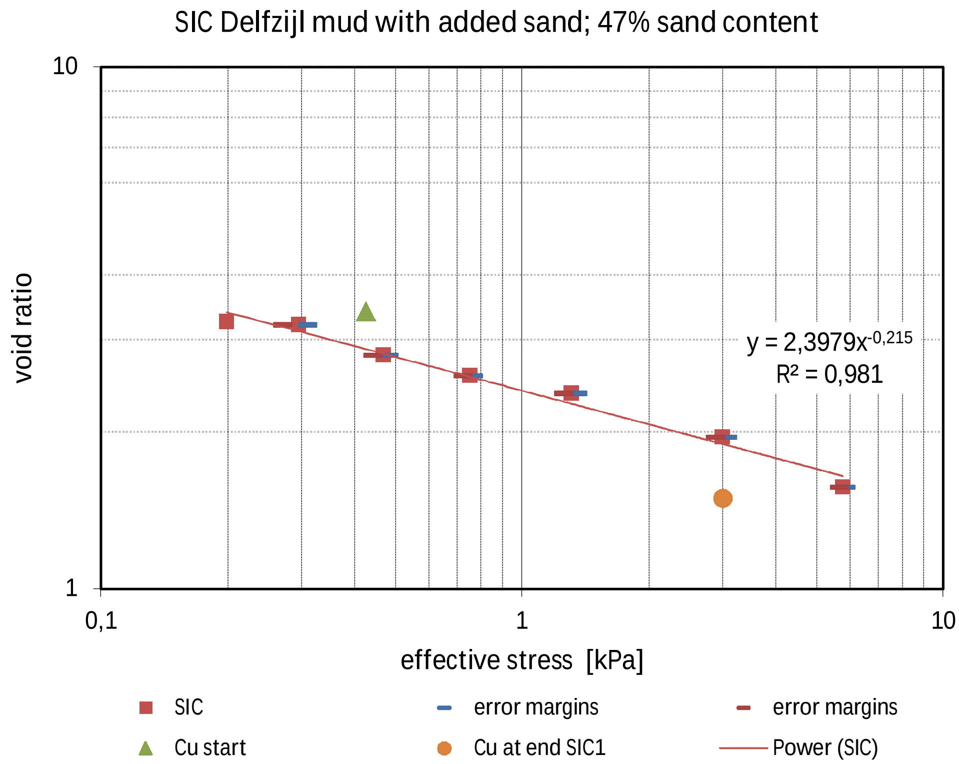


Figure 4.3 Void ratio as function of effective stress for sample SIC

4.1.2 FSCONGAS parameters

The exponential relations (used in FSCONGAS) are shown in Figure 4.4 and Figure 4.5. The Cu-values have been added, see Section 4.2.

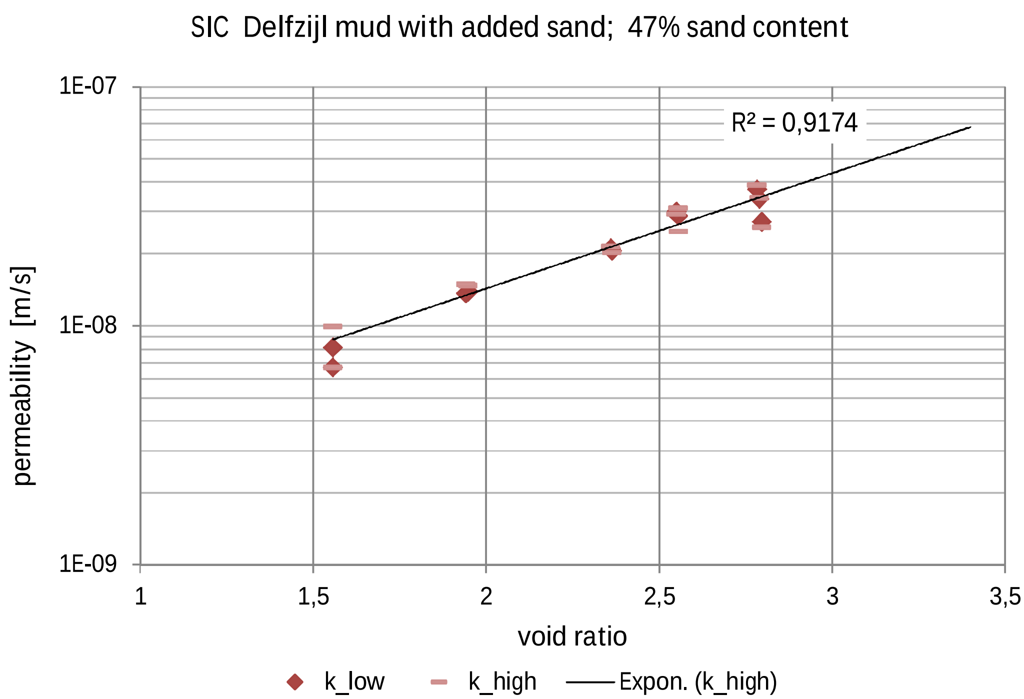


Figure 4.4 Permeability as function of void ratio for sample SIC

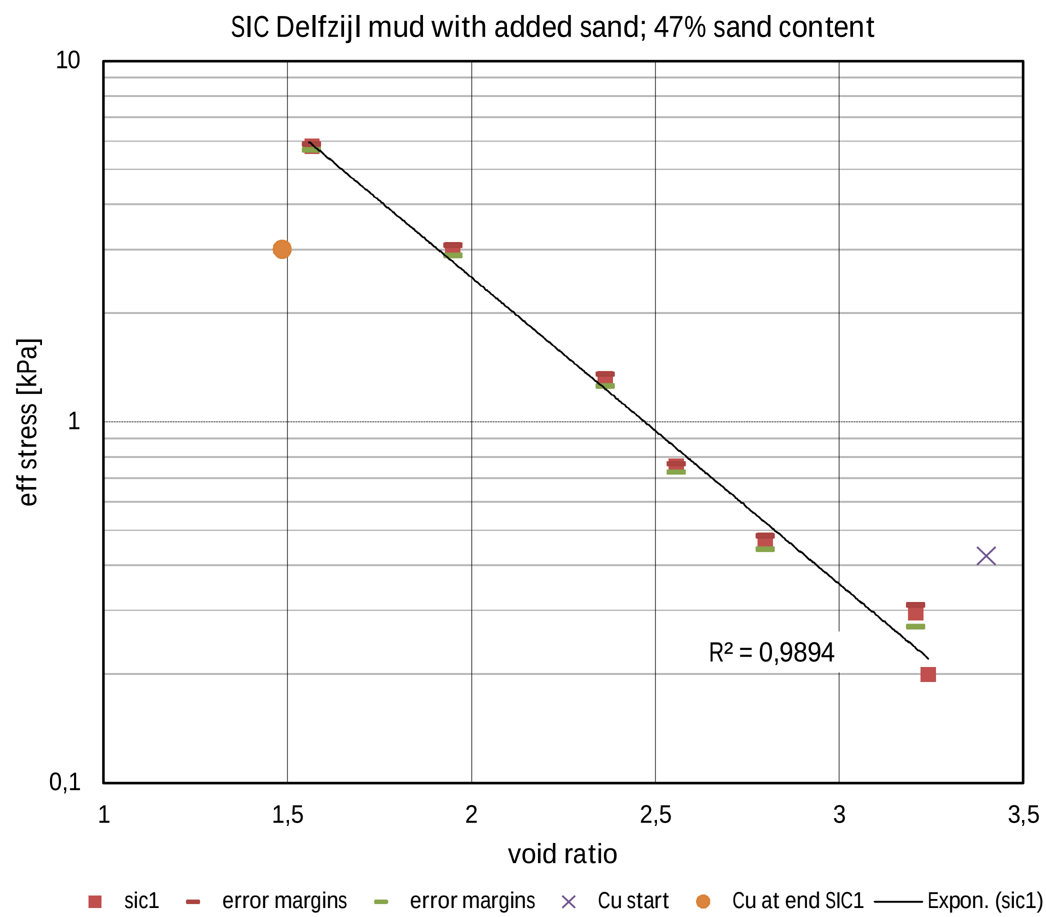


Figure 4.5 Effective stress as function of void ratio for sample SIC

The FSCONGAS parameter values are given Table 4.2.

Table 4.2 Soil Parameters for SIC by fitting the exponential relations (FSCONGAS)

Sample	m1	m2	n1	n2
SIC Delfzijl mud	4.8339	-1.9565	-20.401	1.156

4.2 Vane tests

Before and after the SIC test, peak and remoulded shear strength was measured with a laboratory vane. The sample was left in the SIC ring for 3 days before loading. The strength of the remaining mixed mud in the bucket was determined at the time the sample was loaded with the first load, i.e. 3 days after mixing. The shear strength of the mud directly after mixing with the sand was 250 Pa. The (standard) position of the vane was at the top of the mud layers.

The experimental results are shown in Figure 4.6 and Figure 4.7.

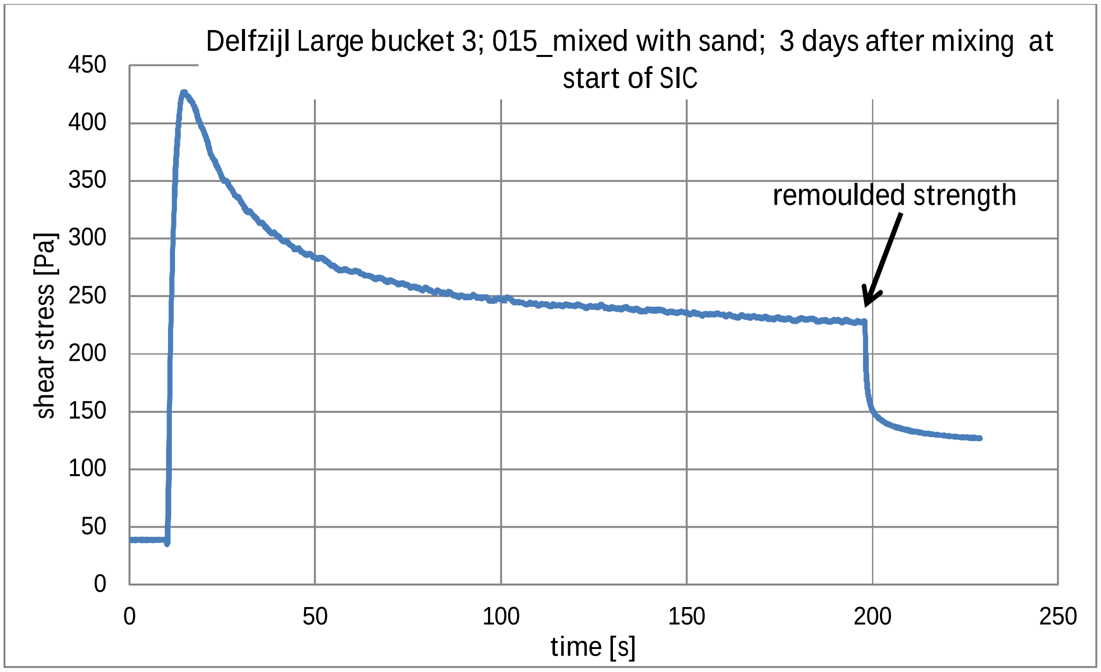


Figure 4.6 *Results from vane test that was performed 3 days after mixing, i.e. at the time the loading started for the SIC*

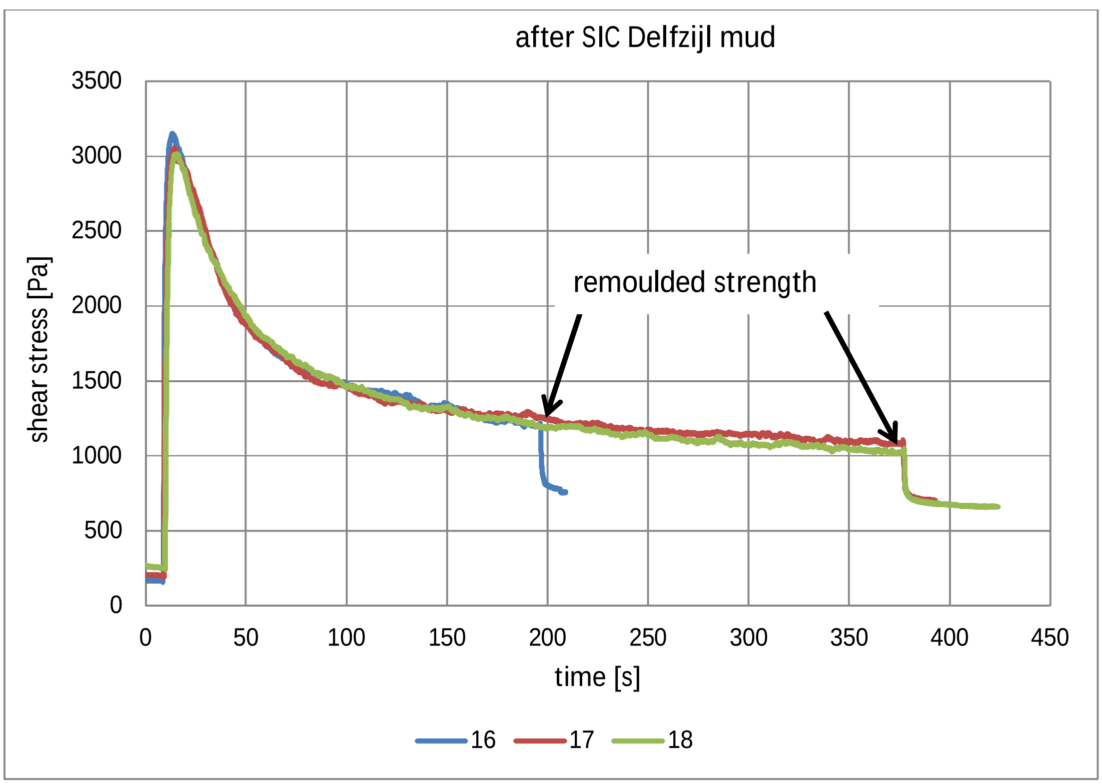


Figure 4.7 *Results from the vane tests performed on the sample after the SIC was finished*

The peak and remoulded strength values are reported Table 4.3.
The undrained shear strength Cu is defined as the peak value of the stress curve.

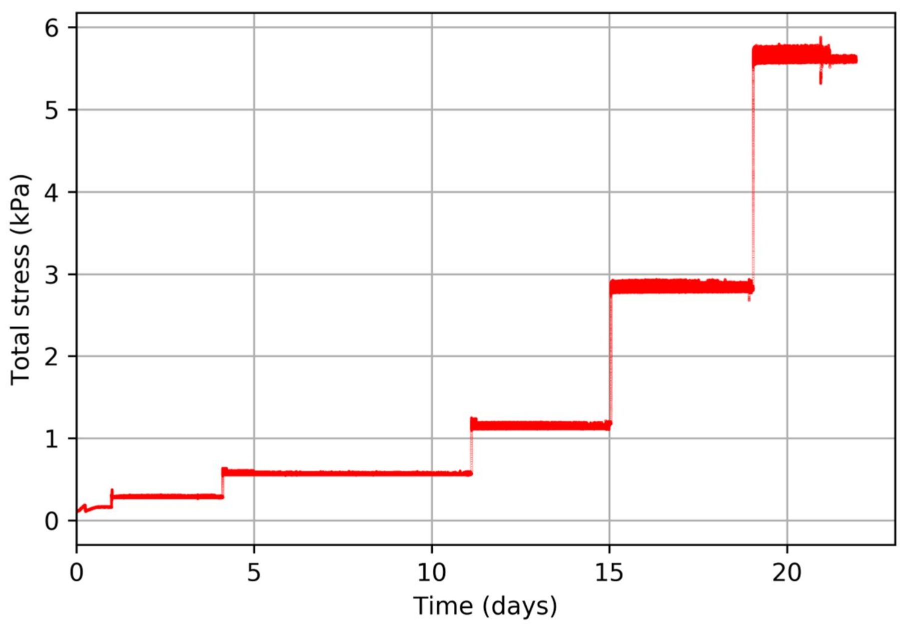
Table 4.3 Shear strength (peak) measured with the vane before and after SIC1and SIC2.

Sample	Before SIC; peak [kPa]	Before SIC; remoulded [kPa]	After SIC: peak [kPa]	After SIC; remoulded [kPa]
SIC Delfzijl mud	0.425	0.23	3.0	1.1- 1.25

4.3 Measurements SIC

4.3.1 Settlement

Figure 4.8 gives the respective loading steps (above) and the resulting settlements as measured by a lvdt (below).



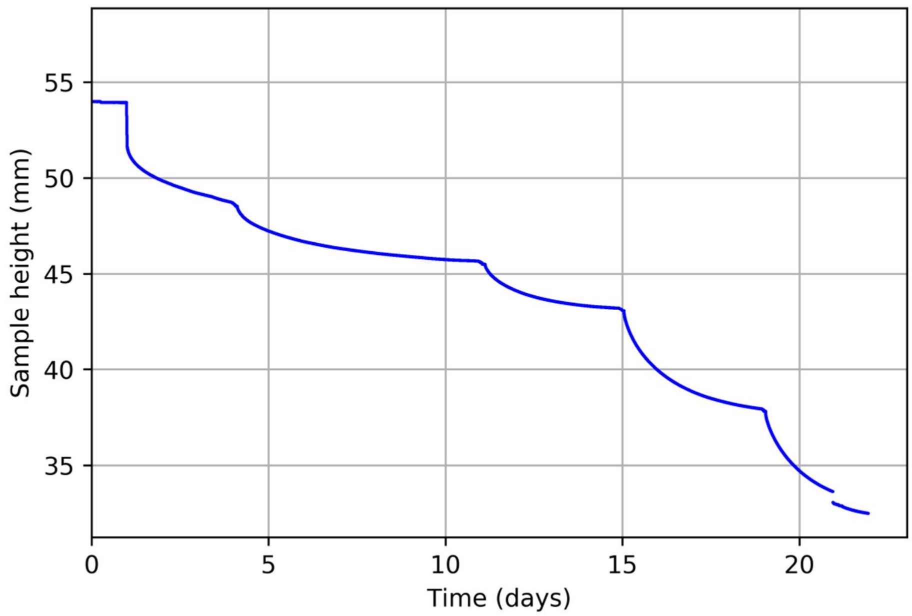


Figure 4.8 Loading steps for SIC on Delfzijl mud: (above) in total applied vertical stress and (below) the resulting changes in sample height

It is visible that hardly any settlement occurs at the loading step of 2 N. This is studied in more detail in section 4.3.4. The other loading steps show a typical consolidation process. Although all the loading steps of 5 N and higher lasted at least 3 days, the settlement at the end of these steps is still increasing. This is due to creep. Hardly any excess pore pressure is present at that stage, see section 4.3.2. At the end of the 100 N loading step the computer control failed, which gave some jumps in settlement, that are unexplainable. At least some swell of the sample occurred afterwards, see section 4.3.3.

4.3.2 Excess pore pressure

At the start of a loading increment the excess pore water pressure increases, although this is not always measureable, as the pore pressure dissipates during the time period the load is gradually increased. At higher loadings the pore pressures dissipates slower (as the permeability is lower). The permeability measurements at the end of the loading steps give suction peaks.

4.3.3 Sample height, water content and gas content

At the start of the SIC it was carefully checked at what lvdt reading the sample was loaded first. This is considered as the initial sample height, which is somewhat smaller than the height of 56 mm of the sample ring, i.e. 54 mm. The filter stone had compressed the sample slightly during the 3 days after mixing and before loading in the SIC with 2 N, at the time the sample was still not stiffened that much yet.
At the end of the SIC-test the sample was taken out and its height was measured with a ruler. Also its weight and water content were determined. Results are shown in Table 4.4.

Table 4.4 Height and water content of SIC sample

Stage	**Height [mm]**	**Water content (%)**
Start, i.e. just before 2N load	54	131.1
At end of 100N step	32.7	64.0 (*) (*) As calculated from settlement = water release only
After removal of sample from SIC-test set-up	46	90.6

It is visible that the sample swelled to a large extent after removal, i.e. 40% of height at the end of the final loading step. This swell could have occurred at the end of the experiment, when the loading piston control was hampered, and the sample, being still in the SIC, could take up water for about one day. The final gas content of the sample was 11.4% by volume (after the sample had swelled and taken from the SIC). In the SIC an absolute backpressure of 350 kPa was present, which means that the gas volume in the sample at the final load of 100 kN was about 3 times less (i.e. 4 %), as the gas pressure was about 3 times larger than atmospheric.

4.3.4 Detailed study of behaviour of sample at low stresses

Figure 4.9 shows loading step 2 N (imposed load) and part of the subsequent step of 5N with the resulting settlement. It is clearly visible that the sample hardly settles at an imposed load 2N and, in addition, the force increases up to 3.5 N. This effect is due to the fact that the SIC only regulates the force when it drops below 2 N, while the sample volume stays constant when the force increases above the imposed 2 N. When normal consolidation occurs the latter will not happen. At the end of the SIC-test, the load cell was checked, and its calibration was still OK, thus no thermal drift occurred. By vibration of the sample it was possible to lower the force again, which suggests that the soil fabric was destroyed. Also wall friction might have been influenced by doing this. To test this effect further, the sample was manually loaded at a constant displacement speed after 87500 seconds up to about 88500 seconds (see Figure 4.10 for more detail), until the force drops to 5 N. At 5 N this value was imposed, and the SIC was able to regulate again, and normal consolidation seemed to happen now, see also Figure 4.8. Possible wall friction effects seem to be less dominant now.

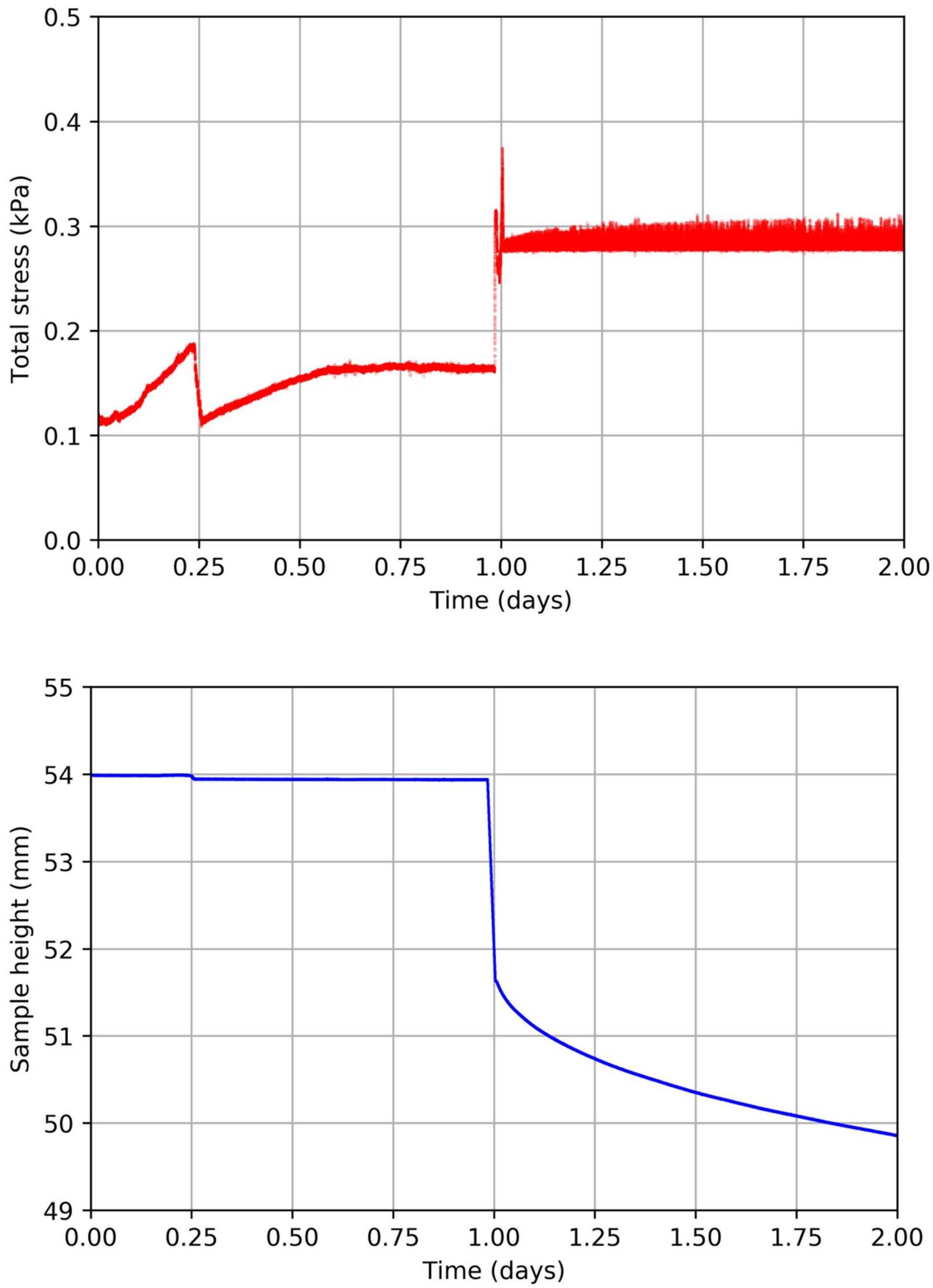


Figure 4.9 First loading step and part of second loading step for SIC on Delfzijl mud: (above) in total applied vertical stress and (below) the change in sample height

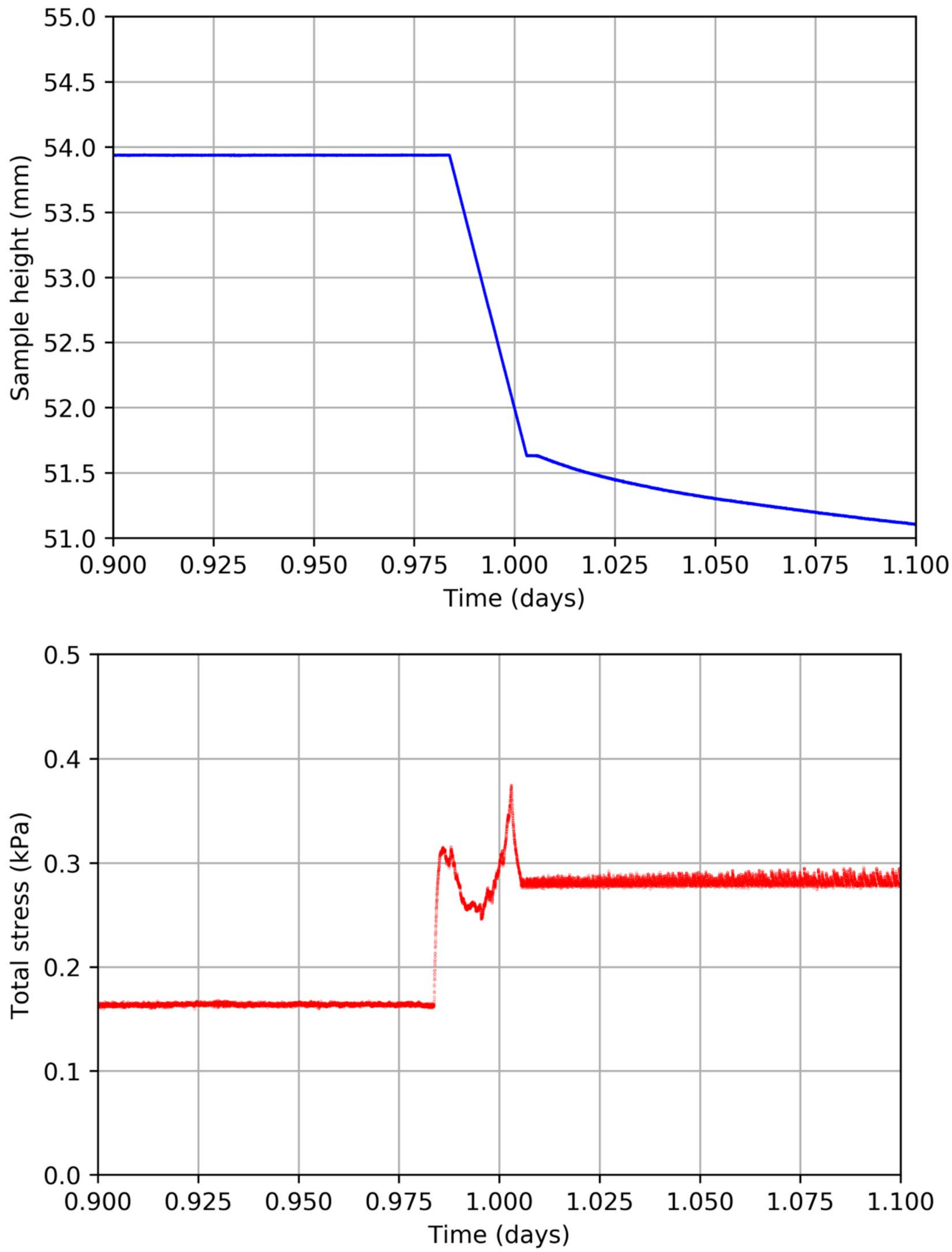


Figure 4.10 Detail of SIC on Delfzijl mud: (above) loading at constant displacement speed at the end of the first loading step; part of the second loading step is also shown; (below) the resulting change in applied total vertical stress

From Figure 4.10 it is visible that the force shows two peaks during the loading with constant displacement speed. This might be due to progressive failure of the soil fabric. Possibly wall friction had an influence in this. Finally, when the force dropped to 5 N the normal SIC procedure was resumed again with consolidation at a regulated force of 5 N.

5 Discussion

5.1 Introduction

The question arises if the effective stress and permeability values related to void ratio are representative for Delfzijl harbour mud, with added sand (46%), giving a sand content 47%.
In addition, new information on the field situation says that no additional sand will be added to the Delfzijl mud now. *How to get consolidation parameters for this mud with 1% sand content?*
In order to answer these questions, a comparison to similar mud can be made. Deltares has a large database of properties for Dutch mud from distinct locations, also Delfzijl Harbour, including effective stress – void ratio and permeability – void ratio relations.
A complete geotechnical classification was made for the Delfzijl mud that was supplied in Bucket 3 (originating form Harbour section 3) and this was also done after the addition of sand, the latter as used in the SIC, see Table 5.1. This mud seems to be quite similar to Slufter mud, see Table 5.1. For Slufter mud we have 3 samples (Slufter M9, M10 and M11) with the only difference in composition being the sand content. This information can be used to validate the effect of adding the sand to the Delfzijl mud. Slufter 1994 has a high Liquid Limit and a large fraction smaller than 2 µm, which is also the case for the Delfzijl mud from section 3, before the sand was added.

Table 5.1 Geotechnical properties for Delfzijl and Slufter mud. Last line is for the SIC sample

CO nr	mud type	name/	Atterberg limits			grains size distribution			activity	organics	CaCO3	ρ_s
		description	LL	PL	PI	< 2 µm	< 16 µm	> 63 µm	I_P/<2 µm	[H_2O_2]		[10^3kg/m^3]
			[%]	[%]	[%]	[%]	[%]	[%]	[ ]	[%]	[%]	
374700	Slufter	Slufter M9	123	39	84	39.4	75.6	8.8	2.13	6.9	25.8	2.523
374700	Slufter	Slufter M11	57	18	39	19.4	40.2	51.1	2.01	3.6	12.8	2.586
374700	Slufter	Slufter M10	87	22	65	28.8	57.2	26.5	2.26	7.2	14.4	2.559
347080	Slufter	S1994	145	40	106	57.5	79.6	10.9	1.84	8.9	25.5	2.47
348400	Delfzijl	Delfzijl A	60	26	34	27.5	32.6	55.8	1.24	2.1		2.570
348400	Delfzijl	Delfzijl B	68	28	40	42.9	72.6	11.3	0.93	2.4		2.504
	Delfzijl mud	section 3	148.7	54	94.7	61.4	94.9	1	1.54	11.2		2.54
	Delfzijl mud	from SIC	98.1	37.1	61	33	51	47	1.85	6.6	9.8	2.593
			Note:	Atterberg limits of SIC sample refers to 36% sand						Note: ignition loss for Delfzijl mud		
			Rest of properties SIC sample with 47% sand									

In addition, we could ask:
What time dependent effects, partly driven by salt content gradients, might affect the results from the SIC? I.e. these effects affect consolidation, especially at low stress levels.
What will happen in the field, and what are the differences with respect to the SIC testing-procedure, including sample preparation?

All these insights can be used to simulate scenarios in the field test sections, see Section 5.4.
To do this we need up to date parameters for Delfzijl mud, where Harbour section 3 was selected as representative for the mud that will be deposited in the test fields. In Section 5.2 the consolidation parameters are estimated by comparison to the other mud type from Delfzijl Harbour and the Slufter, as given in Table 5.1.

5.2 Comparison to similar mud from Deltares database

The Deltares mud database consists of geotechnical classification properties, effective stress-void ratio relations and permeability-void ratio relations as defined in FSCongas (see section 3.2.1).
From this database Delfzijl mud and Slufter mud with varying sand content were taken, see Table 5.1. Estimates were made for the Delfzijl mud as supplied in Bucket 3 (from Harbour

section 3) with 1% sand content. Figure 5.1 gives a comparison of the available parameter relations for Delfzijl mud. The pink lines are the estimated effective stress – void ratio and permeability – void ratio relations for the Delfzijl mud (section 3) with 1% sand content.

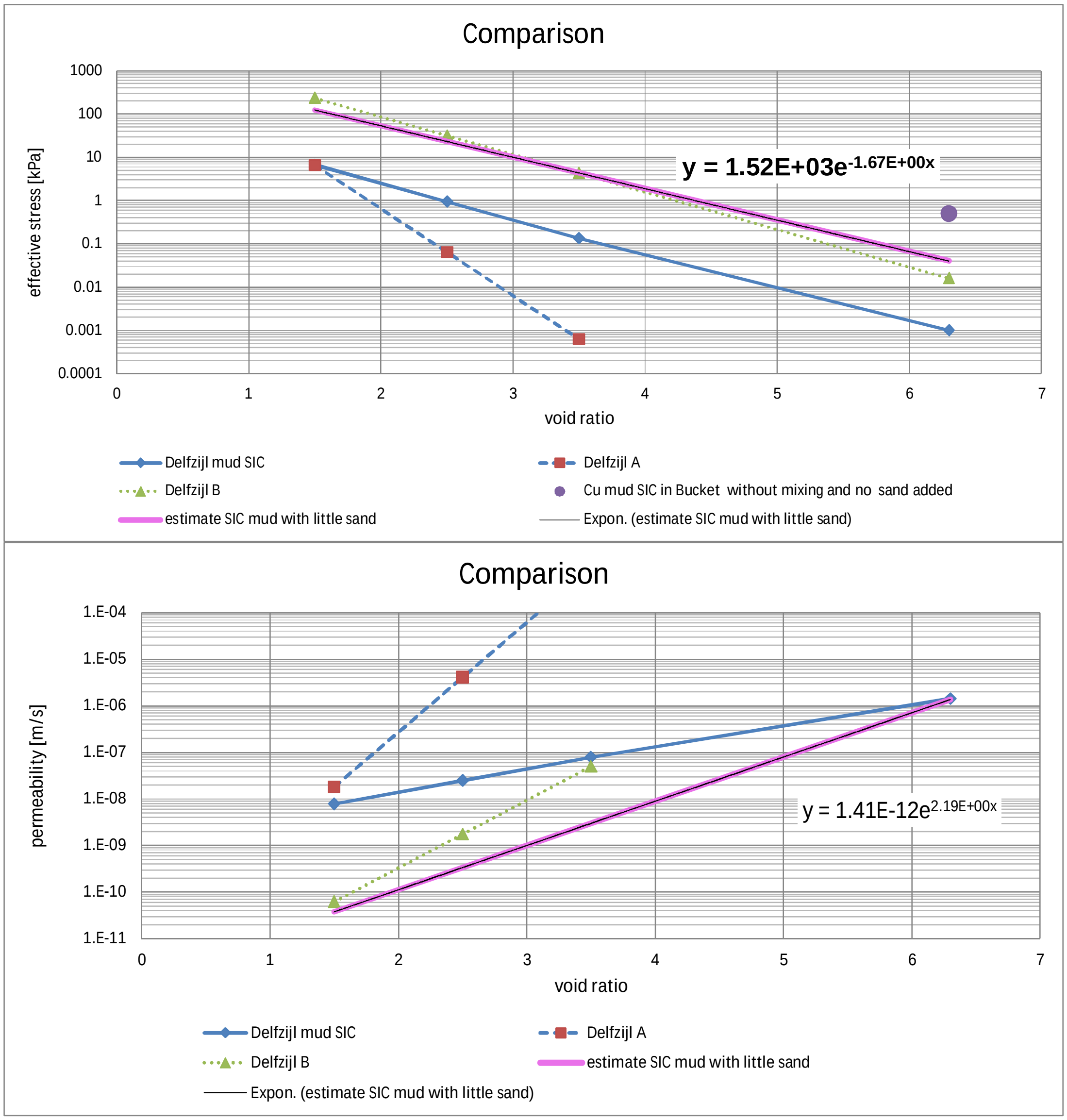
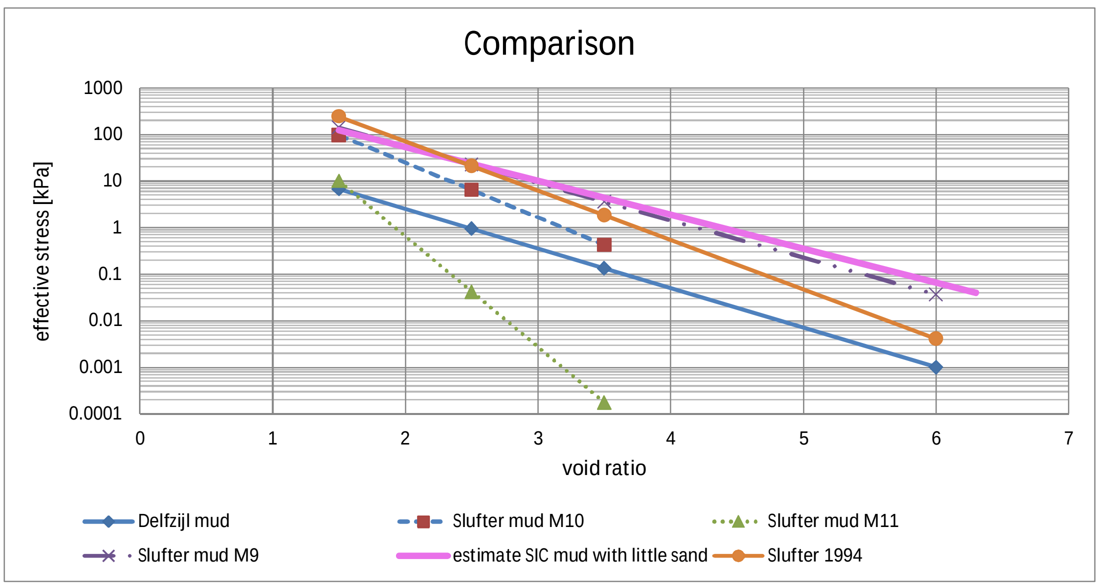


Figure 5.1 *The effective stress – void ratio (above) and permeability – void ratio (below) relations for the Delfzijl mud type in Table 5.1. The 'Delfzijl mud SIC' indicates the SIC sample (sand content 47%). The pink lines are the estimates for Delfzijl mud from Harbour section 3 (sand content 1%)*

The starting point to determine the estimated pink lines were the parameters for Delfzijl B mud, that has a sand content of 11.3%. The effective stress line was slightly rotated towards a higher effective stress at higher void ratio, in order to approach the rather high Cu-value (purple bullet) for the mud (from Harbour section 3) that had settled in Bucket 3 as delivered. The permeability relation for Delfzijl B crosses the line from the SIC, tending to higher values at higher void ratio than for the SIC-sample. This is not logical, because the SIC-sample had

a much higher sand content and should have a higher permeability. See also the discussion of Figure 5.2. At low void ratio's the permeability relation for Delfzijl B mud (11.3% sand content) is more in line with the permeabilities for Delfzijl A and the SIC (both having a high sand content around 50%), as there should be at least one order of magnitude difference due to the large difference in sand content. This is visible in Figure 5.2 for the Slufter mud.
The permeability for the 1% sand content mud was estimated by drawing a line from the 'Delfzijl B permeability' at void ratio of 1.5 (lowest void ratio value in SIC) to the permeability from the SIC at void ratio 6.3 (void ratio as supplied in Bucket 3). Possibly, the permeability for the 1% sand content at higher void ratio is lower than for the SIC sample, see also the discussion of Figure 5.2.
Figure 5.2 gives a comparison of the parameter relations from the SIC on Delfzijl mud and all the parameter relations for Slufter mud, see Table 5.1. The estimated lines for the low (1%) sand content Delfzijl mud are added (in pink). In Figure 5.2 the order in magnitude in effective stress and permeability (in the given range of void ratio) for the Slufter mud corresponds to the order in magnitude in sand content. The estimated pink lines are close to Slufter M9 mud with 8.8% sand content. The Liquid limit and < 2 µm fraction of the Slufter 1994 mud are more close to that of the Delfzijl mud from Harbour section 3 than those for Slufter M9 mud. Both properties affect the parameter relations significantly. For the Slufter 1994 mud the permeability at higher void ratio is significantly lower than for the estimated pink line. To address the effect this might have in the field, also the parameters for Slufter 1994 were used in the modelling.



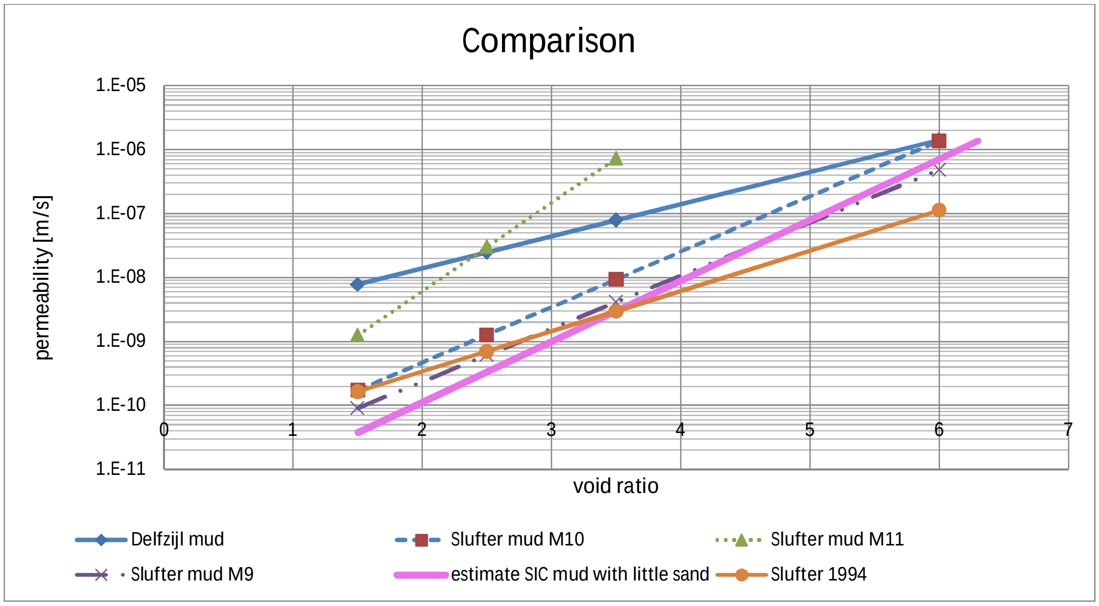


Figure 5.2 The effective stress – void ratio (above) and permeability – void ratio (below) relations for the Slufter mud type in Table 5.1. The 'Delfzijl mud' indicates the SIC sample (sand content 47%). The pink lines are the estimates for Delfzijl mud from Harbour section 3 (sand content 1%)

Table 5.2 gives the FSCongas parameter values for the estimated curves (Delfzijl mud with little sand) and the Slufter 1994 curves. These values will be used for the modelling of the test sections.

Table5.2 FSCONGAS consolidation parameters

Sample	**m1**	**m2**	**n1**	**n2**
Estimated Delfzijl mud	7.325	-1.674	-27.286	2.188
Slufter 1994 mud	9.16	-2.441	-24.71	1.451

5.3 Discussion of time dependent effects

5.3.1 Observations

At a low load (2 N) the SIC sample had a tendency to swell, which gave an increase in force (up to 3 N at the end of the first '2 N' loading step) at constant volume. In addition, the sample swelled a lot (40%) after removal of the last 100 N load, the sample still being in the SIC for about one day. By vibration of the sample it was possible to lower the 3 N load back to 2 N. This is thought to be due to a gradual collapse of the soil structure, even at constant soil volume. Also the wall friction might have played a role in this. In case of subsequent loading the sample at a constant displacement speed, the force showed two peaks, indicating progressive failure of the soil fabric. Again wall friction might have played a role in this too.
A limited amount of gas (about 4%) was present after 25 days of testing at about 22 °C (at a backpressure of 3.5 Bar). At atmospheric pressure the gas content was 11% after unloading and swelling of the sample. This means that there was a significant gas production over a period of 22 days at a temperature of 21 ± 0.5 °C.

In Figure 4.8 it is visible that the settling rate is still significant at the end of the respective loading steps, whereas by then the remaining excess pore pressure is very small. This effect is thought to be due to creep, and is normal for clays.

5.3.2 Typical time dependent effects

Here a couple of possible time dependent processes affecting consolidation at low stress level are given, with reference to literature.

Thixotropy and Creep

Two processes were found to have a major effect at low stress levels, i.e. thixotropic hardening and creep. Mitchell (Mitchell 2005) says:

"Thixotropy is defined as an isothermal, reversible, time-dependent process occurring under conditions of constant composition and volume, whereby a material stiffens while at rest and softens or liquefies upon remolding".

This effect was found to be largest in a soil structure with high water and active clay content, as the soil particles have more freedom to rearrange, react and form bonds. Due to this effect the sample gradually stiffens and subsequent loading steps might give less compression. If a loading step at a low stress level (in the SIC) lasts longer (for example 1 week instead of 1 day), than the settlement in the subsequent loading step will be less, due to this stiffening. Strength evolution in soft mud layers and its interaction with consolidation in columns was studied by for example (Merckelbach 2000). This effect plays a role in the settling behaviour in the SIC at low stress levels.

Sills (Sills 1995) has shown that creep can be a mayor effect at low effective stress levels, i.e. the volume effect (related to compressibility) induced by creep is one order of magnitude larger than at higher stress levels (say 50 kPa). Creep can be considered as a time dependent adjustment of soil structure at constant effective stress, which gives a reduction in pore volume and an increase in strength. After a longer period (months to years) creep becomes significant. It acts partly simultaneously with consolidation, especially when the latter takes long (months to years).

Osmotic pressure and water adsorption influences on compression and swelling

Swelling might be enhanced by gradients in salt concentration between the surrounding water and the SIC sample. This could counteract the consolidation. In the SIC the salt concentration in the sample is expected to be almost the same as in the surrounding water, as this water was expelled water from the mud, taken from the Buckets.

Gas production, accumulation and escape

Gas production resulting in the occurrence of large gas voids retards the consolidation of soft mud, especially when the main driving force is the 'self weight' of the mud, such as in a column test. Some of the gas might escape and this causes preferential drainage paths, which will give an acceleration of consolidation. See (Wichman 1999a and 1999b) and (Wichman et al. 2000). In the SIC-test gas production played a minor role in retarding consolidation.

Literature

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