Characterisation of Saturated Loose Sand Samples prepared by Fluidization

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By

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At the time I am writing this lines, many things come into my mind: experiences I lived and people I met for the last two years. As so, I would like to express my gratitude to those who have been walking with me this path and helped me to persevere until the very end:

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Ad maiorem Dei gloriam.

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Abstract .................................................................................................................................................. 9
Chapter 1: Introduction .......................................................................................................................... 11
  1.1. General Introduction ..................................................................................................................... 11
  1.2. Outline Research Project Report ................................................................................................. 13
  1.3. Scope of Research Work ............................................................................................................. 14
Chapter 2: Research Description ........................................................................................................... 16
  2.1. Problem Description ..................................................................................................................... 16
  2.2. Societal Relevance ....................................................................................................................... 17
  2.3. Research Questions ...................................................................................................................... 17
Chapter 3: Literature Review .................................................................................................................. 20
  3.1. Liquefaction .................................................................................................................................. 20
  3.2. Static Liquefaction ........................................................................................................................ 22
  3.3. Stress and Strain Measures: Introduction to Stress Invariants ..................................................... 23
  3.4. Critical State Theory .................................................................................................................... 24
  3.5. Stress-Dilatancy Behaviour .......................................................................................................... 27
  3.6. Static Liquefaction and Slope Instability ..................................................................................... 28
  3.7. Microstructure of Natural Sands .................................................................................................. 32
  3.8. Trends in Constitutive and Numerical Models ............................................................................. 37
  3.9. Element Testing Methods ........................................................................................................... 43
  3.10. Sample Preparation Methods .................................................................................................... 48
  3.11. Fluidization .................................................................................................................................. 50
  3.12. Closing Remarks on this Chapter ............................................................................................... 55
Chapter 4: Testing Programme ............................................................................................................... 57
  4.1. Testing Programme ....................................................................................................................... 57
  4.2. Overview of Sample Preparation and Control (Jager and Molenkamp, 2015a) ......................... 58
Chapter 5: Results and Discussion ........................................................................................................ 67
  5.1. Material Characterization: Grain Size Distribution ...................................................................... 67
  5.2. Material Characterization: Density ............................................................................................... 69
  5.3. Material Characterization: Grain Shape ....................................................................................... 71
  5.4. Standard Triaxial Compression Tests ......................................................................................... 72
  5.5. Fluidization Tests for Sample Preparation via Fluidization, Sedimentation and Excavation ....... 80
  5.6. Detailed Fluidization Assessment Using Permeability and Porosity ......................................... 88
  5.7. Error Analysis on Void Ratio Measurements .............................................................................. 91
Chapter 6: Conclusions and Recommendations .................................................................................... 93
  6.1. Conclusions .................................................................................................................................. 93
  6.2. Limitations .................................................................................................................................... 96
  6.3. Recommendations for Further Research ..................................................................................... 97
References .............................................................................................................................................. 99
Appendix A: Sand Fluidization Technique and Modified Triaxial Testing .............................................. 105
  A.1. Apparatus Parts Description ......................................................................................................... 105
  A.2. Preparation and Checking of Apparatus ..................................................................................... 109
  A.3. Material Characterization prior to Testing .................................................................................. 110
  A.4. Saturation of Filter Plate above Infiltration Cell ......................................................................... 110
  A.5. Placing the Membrane, Split-Mould and Fluidization Column .................................................. 111
  A.6. Protocol A: Sample Preparation by Fluidization, Column Sedimentation and Excavation ....... 111
      Removal of Split-Mould and Control of Sample surrounded by Cell Water ............................... 117
Abstract

The following thesis has three major purposes: (1) to explore the static liquefaction phenomenon in saturated sand as a relevant geotechnical hazard, (2) to show the conventional testing procedures for index characterisation and strength testing of saturated fine sands at low stress levels, and (3) to look into the fluidization mechanism and the ongoing research into its applications in saturated sand testing, done at Delft University of Technology.

The exploring on static liquefaction was performed by a literature study, which allowed to relate the macroscopic scale of this hazard in underwater slopes with the common practices performed in laboratory conditions. The literature found shows that a change in the framework of understanding saturated sand behaviour, accounting for anisotropy, may occur within the next years. In addition, the literature supports that improvements on the liquefaction prediction can only be done via: improvements of: the theoretical framework in which the sand behaviour is studied, the constitutive and numerical models used for prediction of the liquefaction mechanism and the testing and the physical modelling methods allowing for both sand characterisation and the determination of the constitutive model parameters.

Some tests performed to characterise a fine sand were selected to see how the method can affect the obtained results and how the conditions of the sample preparation and control can affect the liquefaction potential on a saturated sand sample. The results support that idea that the amount of fines on the sand, indicated generally by a sieving test, can affect the results of the index minimum and maximum void ratio, differentiating the properties of a sand with fines and a sand without fines. In addition, several variables such as the degree of saturation, confining stress levels and membrane penetration can contribute to the development of pore pressures and the development of liquefaction.

The literature study reveals that fluidization has been used on research on granular soils sedimentation, documented in the work by Allen (1984), and is not as new in earth sciences as it was thought to be. Based on the undrained triaxial testing procedure suggested by Jager and Molenkamp (2015a), where fluidization will be used for sample preparation, some sand batches were prepared, in which layers were removed to characterise the final product. While the results on void ratio distribution along the depth of the column are not sufficient to draw further conclusions, it was found that the discharge rate can affect the porosity distribution during and after fluidization. The study not only showcases two techniques that can allow a quantitative assessment of fluidization mechanism, but also that inclusion of this method in loose sample preparation could bring improvements into granular soils characterisation.

Keywords: liquefaction, fine sands, sample preparation, characterisation, laboratory testing, fluidization.
Introduction

1.1. General Introduction

Dredging is an excavation activity which is carried out not only on shallow seas but also on fresh water areas, with the purpose of gathering up bottom sediments and disposing of them at a different location. Some of the uses of dredging activities include the extraction of material for allowing underwater space for facilities like harbours and waterways, preparatory works for future civil engineering structures, maintenance of waterways and channels, land reclamation works, extraction of sand and gravels for construction materials, etc.

During sand dredging, depending whether the operation is for sand mining or for excavation, achieving steep slopes during material extraction is important since the sand volume during those operations can have an effect on the profitability of a project. However, the risk of failure from a water-saturated loose fine sand slope increases with the steepness of the slope angle. This is true for much smaller slope angles than the natural drained slope angle, but even a minor trigger may cause a significant, almost instantaneous undrained increase of the pore water pressure including undrained instability. This phenomenon is known as static liquefaction and may be followed by a flow slide involving a large sand volume (Robertson and Wride, 1998; Olsen and Stark, 2003; Mróz, Boukpeti and Drescher, 2003; Jager and Molenkamp, 2015a).

At the moment field observations involving underwater slopes which failed under static liquefaction are exaggerated in terms of precision and thus with unwarranted confidence in this aspect (Jefferies and Been, 2006); the aforementioned authors state these underwater slides lead to some of the lower estimated residual strengths and comment that some authors have apparently, and incorrectly, omitted the stabilizing effect of the external water pressure acting on the face of the failing slope (Duncan, Wright and Brandon, 2014). In addition, according to Jefferies and Been, much of the case history records has been gathered with geology rather than mechanics as the starting point. This could support Jager and Molenkamp’s idea that an experimental database of the triggering mechanism of static liquefaction of a submerged slope by means of testing is required.

For achieving this, experimentation is being carried out at the liquefaction tank in the Geo-Engineering section at TU Delft, in which a submerged slope of saturated loose fine sand is subjected to static liquefaction with a subsequent propagation of the deformation into a flow slide (Oever, Graafland, Jager and Molenkamp, 2014); this instability modes would be activated either by fast loading of a strip footing along the crest of the slope, by gradual tilting of the slope in the liquefaction tank or fast tilting of the liquefaction tank and then suddenly bringing it to a halt. The experiments within this tank are intended to obtain an experimental database of the initiation and development of liquefaction flow slides, being the first experimental facility to obtain high-quality data of one of the most important geotechnical hazards around the world.
While the tests in the liquefaction tank could be considered as large-scale tests, it is possible to conduct tests at a smaller, element scale, in which the intergranular stress conditions, range of density and microstructure are representative to those of the sand in the liquefaction tank. By means of element-scale, undrained laboratory tests such as triaxial testing and the hollow cylindrical apparatus, reproducing the stress conditions found in an underwater sand slope will help to characterise the mechanical properties of a loose saturated sand. In both the liquefaction tank and the element testing, the sample preparation will be done by means of fluidization, in which water will be flushed upward in such a way that the vertical intergranular stresses become zero, thus “liquidizing” the sand for some period of time. Eventually the fluidization mechanism is removed, resulting in sedimentation of the sand material in a loose state.

Both tests have different roles in the research programme, which should be distinguished more clearly. The role of the undrained triaxial tests is to gather undrained response data of samples for the parameter determination of constitutive models. The role of the tests in the liquefaction tank is to gather response data of slopes for evaluation of the quality level of the predictive models of submerged slope behaviour.

It is important to mention that the response of water-saturated loose fine sand samples to undrained testing will depend strongly on:

- The intergranular stress range (understanding intergranular stresses as those occurring at the contact between the sand particles)
- The sand’s bulk density (i.e. the mass of the sand particles divided by the total volume they occupy, including the particle volume, inter-particle void volume and internal pore volume).
- The sand’s microstructure (dependent on the grain shape and particle’s arrangement)

The local response of the sand and pore water of the slope in the liquefaction tank concerns “dynamic consolidation”, thus with local relative motion of the pore water with respect to the solid skeleton. In an undrained triaxial test, the boundary conditions are impervious, suggesting equal motion of water and the soil skeleton. Nevertheless, during slow undrained deformation of a saturated sample locally the pore water can still move relatively with respect to the sand skeleton as well, thus allowing a redistribution of the pore pressure in the sample towards a hydrostatic one.

For the first two variables, intergranular stress and sand bulk density, quantification is common practice, while topics about formulation and quantification of microstructure of natural fine sands are still being in research. Evaluation of the level of quality of a numerical model for simulations will not only help the prediction of underwater failures of slopes from hydraulic fills but also to obtain, on a long term, an opportunity to obtain better predictions of production yields for sand dredging without risking the stability of the underwater sand layer.
1.2. **Outline Research Project Report**

The main body of this report will be divided into six main chapters. Chapter 2 will be focused on describing the ongoing research project at Delft University of Technology regarding the study of static liquefaction and the corresponding triaxial testing for determining the characteristics of the soil behaviour involved in the occurrence of static liquefaction of the slope in the liquefaction tank. The main focus of this research concerns the corresponding triaxial sample preparation for enabling the same mean intergranular stress level, soil density and soil microstructure of the sand composing the submerged slope in the liquefaction tank.

Chapter 3 contains an extensive literature study based on the topics associated with the key questions. The literature review will not only cover in general aspects about static liquefaction and the stress-dilatancy relation behind this failure mechanism, but also indicate a difference with a similar phenomenon which has been in discussion among dredgers and geotechnical engineers during the last years, known as breaching. The review will take a smaller scope when briefly discussing about microstructure of natural sands and the development of constitutive and numerical models which attempt to capture and simulate sand behaviour and the liquefaction phenomenon. The study will focus on the characterisation and testing aspects of sands when studying their stress-dilatant behaviour and their susceptibility to liquefaction and the features behind the different sample preparation methods currently used. Finally, an introduction into fluidization as a mechanism which has similar effects into the sand as liquefaction will be given; the main focus will be given into how the particle size and the fluid velocity are related into the development of this process.

Chapter 4 will cover the experimental part of the project in two parts. The first part will mention the conventional characterisation methods that will be used on the sand being analysed, while the second part will introduce the sample preparation and control as proposed by Jager and Molenkamp (2015a, 2015b), including some observations from the fluidization process to be used. At the time of preparing this document undrained triaxial testing of the samples prepared by the new process was not possible, thus it will not be covered on this report. For more information the reader could refer to Jager and Molenkamp (2015a, 2015b) and to Paz Noriega (2015).

Chapter 5 will present the most relevant results of the testing process while providing a place for interpretation and discussion of the process. The complete set of results can be found in Paz Noriega (2015).

Finally, the report will present the conclusions drawn from the research project while providing recommendations for further research on the topic. This will be of relevance for the research to be done once this work has been presented.
1.3. **Scope of Research Work**

The scope of the research project will be divided into the following clusters:

- **Research of Background Literature on Liquefaction and Static Liquefaction** (Sections 3.1, 3.2, 3.6 and 3.7). This aims to provide an understanding of its mechanism and effects on the soil. Associated to liquefaction, breaching will be briefly discussed since it is a phenomenon that generally is present during liquefaction failure of underwater slopes and may cause a misinterpretation when analysing post-failure geology.

- **Research of Background Literature on Stress and Strain Measures, Critical State Theory and Stress-Dilatancy** (Sections 3.3, 3.4 and 3.5). This aims to provide some insight into the framework used for the last years in understanding sand behaviour and liquefaction.

- **Research of Background Literature on Trends of Constitutive and Numerical Models** (Section 3.8). This aims to bring some insight into the history of the development of models for sand liquefaction, including the recent development of the Anisotropic Critical State Theory.

- **Research of Background Literature on Sample Preparation and Element Testing Methods** (Sections 3.9 and 3.10). This aims to show the different aspects contributing to results differences during element testing (i.e. shear strength tests), including the effect of the sample preparation and testing selected.

- **Research of Background Literature on Fluidization** (Section 3.11). This will provide insight on the technique which will be used for preparing sand samples and the influence of diverse aspects such as fluid velocity and particle size on the triggering of this mechanism.

- **Review on the Preliminary Study Reports from Jager and Molenkamp (2015a, 2015b) for the proposed sample preparation, control and testing methods** (Section 4.2). This will give an idea on fluidization process applied for preparation of a sample that will be tested on a further stage.

- **Experimentation by Determination of Grain Size Distribution, Density and Grain Shape** (Sections 5.1, 5.2 and 5.3). By performing a series of index tests on the sand material which will be used in the liquefaction tank, the author will obtain an idea on the size and shape of the grains that can be found on the sand, its particle density and how the arrangement of particles at a defined volume space can result in a loose or dense packing (determined by the maximum and minimum void ratio). At this stage, the gathered data will be analysed and reflection will be given on the methods used.

- **Experimentation by Standard Shear Stress Tests** (Section 5.4). At this point drained and undrained testing on the material will be performed, in an attempt to recreate liquefaction behaviour. The standard triaxial test will be chosen as it is one of the most common laboratory tests used for characterising the strength of sands and because this is the test whose setup has been modified to perform undrained triaxial tests on a sand specimen prepared by fluidization. At this stage, the gathered data will be analysed and reflection will be given on the methods used, including any potential problems that may affect the results during the modified triaxial testing for samples prepared by fluidization.

- **Experimentation by Testing of the Fluidization Process** (Sections 5.5, 5.6 and 5.7). From the preliminary studies from Jager and Molenkamp, some of the components were fabricated as part of the new sample preparation and testing setups. In this phase the setup will be built, calibrated and tested by preparing two batches of sand by fluidization, in which measurement of pore pressure
differences along a section of a column will help to observe whether fluidization has been reached. Data and differences between both setups will be compared and analysis of the data gathered and the approach will be done. At this point, the testing phase was limited only to two batches, whose data will be analysed as much as possible.

- Elaboration of an Internal Laboratory Report which will be used to document the experimental part of this research. This report will be used as reference material for some of the sections along this document.

- Elaboration of a Research Project Report (i.e. this document), which will bring up together all the work done within the scope of work described before.
2

Research Description

In the following section, the research project description will be presented and the questions to be addressed on the project will be discussed below.

2.1. Problem Description

Saturated loose sand is highly sensitive to sample disturbance, which means that even a small change on the sample may result in a change in its microstructure and intergranular stress state when subjected to loading. This is the reason why, during sample preparation and testing special attention must be paid not only on the sample preparation technique and the results, but also to take into account the microstructure of the sample itself and the stress levels at which the material should be characterized. Jager and Molenkamp (2015a) propose a technique used in granular materials known as fluidization, in which a fluid is flushed upwards in such a way that the intergranular stresses are reduced to zero and a looser material state can be then reached after the fluidization mechanism is removed and the particles settle down, its details can be found on Chapter 3. This process of fluidization will be used to prepare a sand bed underneath the water, which will be then dredged into a slope. An instability of this slope would be triggered by two different methods: a rapid loading of a strip footing on the crest of the slope and by rapid halting of a tilting motion that would act on the tank.

On a smaller, element scale, static liquefaction will be reproduced by means of an undrained triaxial test. In this undrained triaxial tests the same fluidization process will be used, in order to achieve reproducibility of conditions in the triaxial samples, with respect to those on the slopes in the liquefaction tank. The aforementioned authors propose two variants of the sample preparation protocol, which will be described in detail on Chapter 4; one of them recreates the expected conditions at the liquefaction tank, while the other one may lead to less disturbance during sample preparation. Questions will arise at the moment of comparing undrained triaxial tests results, not only when looking if the results at different confining pressures fit into any existing conceptions and theories applicable to sands (like Critical State Theory), but also when comparing the tests results between the different sample preparation protocols. For the study of static liquefaction stress levels are low and thus may present a challenge during the testing phase, especially since the stress levels at the liquefaction tank must be reproduced in the triaxial tests; minor stress levels mean that the sample will not only be prone to liquefaction during testing, but also to densification due to disturbance on the sample during its handling.
2.2. Societal Relevance

The societal relevance of the selected research project can be detailed below:

- As it will be shared on Chapter 3, soil liquefaction is one of the most important geotechnical risks worldwide, since its reduction of effective shear strength, due to an increase of pore water pressure, makes the soil behave like a liquid. By significantly reducing its strength, the soil cannot bear any loading and eventually will collapse. This phenomenon has been related to human casualties and considerable financial losses (Jefferies and Been, 2006).
- The potential of soil liquefaction, due to an increase in pore water pressure, controls the construction process of an embankment or an underwater excavation, especially when referring to the steepness of the slopes. This has an effect on the planning and revenue of projects involving these type of works.
- Preparation of saturated loose sand samples for constitutive parameter determination is generally a challenge in element-scale testing, especially when considering low stress levels. The process of fluidization can allow for saturated sand samples characterised by a high porosity to be achieved, while reproducing the depositional nature of soil particles underwater. These will be of importance during the undrained triaxial tests.
- As explained in Section 1.1, the amount of data gathered from successive undrained triaxial tests at low intergranular stress levels will allow the determination of the parameters of a constitutive model under development at Delft University of Technology, which will then be used in predictive models of submerged slope behaviour, enabling the evaluation of their predictive quality level, by comparing with the results obtained from the tests in the liquefaction tank. This will allow not only to understand better the mechanism of static liquefaction but particularly to quantify the level of quality of predictions of the liquefaction of slopes in engineering practice.
- By developing predictive models which can produce more accurate results, construction procedures can be further improved which will optimize projects in terms of planning (i.e. the costs and times).

2.3. Research Questions

From the identified problem, the following research question came:

*How to experimentally characterise loose saturated sand by element-scale testing, under similar conditions as in reproducible slopes in the liquefaction tank?*

Soil characterisation consists of the description of the soil in terms of its properties and behaviour; what is to be characterised depends on the desired parameters and the behaviour to be studied, but generally in geotechnical engineering it refers to the physical properties of the material and the way it behaves under a series of conditions (i.e. the strength and stiffness of the material). A general overview of soil characterisation is presented as follows: define the type of material to be characterised, select the testing method in accordance to the problem being studied, obtain representative soil samples for this testing method, perform the material
testing, select a soil model that is suitable to describe the material’s behaviour and determine the representative magnitudes of its parameters given the obtained results.

Soil classification can be done within the earlier phases of a project, either by a desk study or a preliminary exploration. At this point, material classification can be limited to basic geotechnical tests such as grain size distribution and relative density, to mention some examples. However, at later stages of the project tests on soil samples, like undrained triaxial testing, become more important. Physical testing can be done by:

1. Physical models of the engineering problem(s) under study. For example, a submerged slope at gravitational conditions for extremely loose sands and/or under centrifugal conditions for denser sands.
2. Soil elements for soil characterisation. For example, undrained triaxial tests.

A constitutive model attempts to capture the characteristics of the relevant type of soil behaviour at the level of continuum mechanics in terms of the change of the deformation of the soil skeleton per unit of volume as function of the changes of the intergranular stress, the pore water stress, the void ratio and the fabric tensor quantifying the microstructure. In this mathematical constitutive model, several parameters occur for quantifying any specific soil behaviour, which also depends on the boundary conditions of the considered unit volume in the continuum, for example distinguishing between the limit cases of drained and undrained behaviour (Molenkamp, Jager and Mathijssen, 2014).

Defining the practical application of a constitutive model is important since different construction methods or engineering structures can produce different stresses and strain levels and rates, which can render some phenomena not applicable. Finally, depending on the type of loading the soil will experience (i.e. the stress and strain levels), the list of models that could be used can be narrowed. PLAXIS material model manuals (PLAXIS, 2011) show how some of the models have various options for different construction activities on sand, like the HS Small model. Discussing the different material models is not part of the scope of this work, but it gives the reader a general message: at the moment there is no perfect soil model and the geotechnical engineer or researcher must understand the advantages and limitations behind each model.

In short, the testing of representative soil samples gives experimental data which allows the derivation of magnitudes of the corresponding constitutive parameters of a certain model. The process of parameter determination for any constitutive model involves the minimization of the differences between the observed behaviour and the simulated behaviour by means of that specific model (i.e. fitting the model to the observed test data).

Derived from this main question and in accordance to the time and resources availability, the following key questions will be investigated during the entire project:

**What are the difficulties in preparing saturated sand samples which could be further used in undrained triaxial testing at low stress levels for analysing liquefaction?**

Since the main goal of the research on soil elements is to perform undrained triaxial tests using the procedure described by Jager and Molenkamp (2015a, 2015b), the testing phase will include some drained and
undrained triaxial tests performed according to the British Standard BS 1377: Part 8: 1990 (BSI, 1990b). From the literature study a sample preparation technique currently used will be chosen and tests will be done at different confining pressures. While the objective is to perform a successful test in which liquefaction can be witnessed, unsuccessful tests can provide the opportunity to get some insight into the process and the error sources which could also exist in the modified triaxial test protocol to be executed later.

**Which protocol of sample preparation and control is the best for saturated loose sand characterisation at low stress levels?**

As it will be described further in Chapter 4, a total of four variants on the sample preparation and control as suggested by Jager and Molenkamp (2015a) will be seen. This question will be answered in two ways: the first one will be based on the review of the preliminary study presented by Jager and Molenkamp, while the second will consist on experiences during the fluidization process and removal of the split-mould after sample preparation.

It is clear that testing at a later stage can provide a better answer on which sample preparation and protocol is better, taking into consideration the undrained triaxial loading procedure; for that point, comparison between the four variants will not be only from the practical point of view but also on the results: is there a trend in the results within each protocol and are there similarities between the results among the four protocols?

**What will be the effects of the fluidization on the density and void ratio distribution along the sediment columns for the two main protocols proposed?**

According to the preliminary study of Jager and Molenkamp, a discharge rate in the range 0.47 -0.64 L/min at the overflow is considered, based on the discharge rate to be used at the liquefaction tank; in principle, this discharge will allow as well the washout of fines. However, it is important to consider two things at the time of fluidization: (i) the permeability of the sand being fluidized and how the discharge can affect the grain size distribution of the particles being washed out and (ii) the mass of the sand being fluidized (i.e. the height of the sand column fluidized) and the grain size distribution profile along the column for a given fluidization discharge rate. Thus, it is important to expect some types of results during fluidization and after sedimentation, represented by the vertical distribution along the column for each of these states; these results are, namely: grain size distributions, pore pressure, void ratio, intergranular stresses and permeability.
In the following sections an extensive literature study is presented, based on the key questions identified previously. This will aim not only to provide an overview on the phenomenon of static liquefaction as a subject being currently studied, but also on other aspects that play a role in the development of this type of instability and which are also present in the characterisation of saturated loose sand samples.

3.1. Liquefaction

Jefferies and Been (2006) define soil liquefaction as a phenomenon in which a soil loses much of its strength and/or stiffness for a short period of time, in response to an applied stress, causing it to behave as a liquid. However, this period of time can be long enough for liquefaction to be the cause of many failures, the latter being characterized by either human casualties or considerable financial losses; examples of disasters triggered by this phenomenon include the building damages during the 1964 Niigata earthquake in Japan and the abandonment of the Nerlerk artificial island in Canada.


Figure 1, taken from Olson and Stark (2003), presents a pair of graphs showing the response of undrained, loose sandy soil under different loading paths. The plot on the left represents a $\tau$-$\gamma$ plot (shear stress vs. shear strain or deformation), while the one in the right a $\tau$-$\sigma$ plot (shear stress vs. normal or vertical effective stress). The concept of static or constant rate loading is characterized by a stress path going from Point A (initial stress level) to Point B (maximum shear strength the soil can mobilize under undrained conditions); when the shear stress in this element induced by the loading attempts to exceed Point B, the loose soil structure yields and collapses, meaning that liquefaction has been triggered, and the stress path moves from B to C (an undrained shear resistance in a post-liquefaction state).

The case of deformation-induced liquefaction (i.e. undrained creep behaviour) considers that Point A has been reached by undrained or drained loading and that its static shear stress is greater than its liquefied shear strength (Point C). In this case, the static shear stress resulting from the loading is large enough to initiate shear strain, creep or another deformation mechanism within the soil body (whether on an embankment or a foundation, for example). If the shear deformation is large enough and the element in Point A is undrained, then the path will move from Point A to Point D, which is located on the undrained yield strength envelope (refer to the right side plot in Figure 1), thus triggering liquefaction and then moving to a residual liquefied shear strength at Point C. This type of shear failure is different from others in terms of the material deformation (constant or cyclic loading rate vs. creep deformation under constant shear) and because a reduction of normal
(vertical) effective stresses is seen for a constant shear stress level up to the point when it reaches the undrained yield strength envelope.

Seismically-induced flow failures can be described if an initial condition in Point A’ is considered, as similar to those in Point A. Under action of a cyclic or seismic loading long enough to cause pore water pressure build-up, the element moves from Point A’ to Point E and liquefaction is triggered.

Figure 1: Schematic undrained response of saturated, loose sand. The yield strength envelope in the figure is for undrained conditions. Taken from Olson and Stark (2003).

When assessing the undrained yield shear strength and strength ratio, only the static loading-induced failures can be used confidently since the shear strength back-calculated from the prefailure geometry corresponds directly to the undrained yield strength envelope (Olson and Stark, 2003). For the cases of cyclic loading-induced and the deformation-induced (creep) failures, the values obtained from a limit-equilibrium analysis for shear stress and strength ratio of the prefailure geometry may be less than the undrained yield values and thus underestimating the real undrained material strength. In cases where the deformation-triggering liquefaction occurred simultaneously with loading (for example fill placement during construction of an embankment), prefailure shear stress and stress ratio conditions may correspond closely to the undrained yield strength and strength ratio.

The development and effects of liquefaction can vary with conditions and applications. Youd and Carter (2005) analysed the effects of liquefaction in the amplification of long-period spectral acceleration values due to the lengthening of the fundamental site period as the site softens, which can decrease the seismic performance of a structure once the ground undergoes liquefaction after an earthquake. In seabed buried structures, it was found that the excess pore pressure builds up in the vicinity of them, rather than in the far field, and the boundary conditions of the structures have an effect on the rate of pore pressure build-up (Dunn, Vun, Chan and Damgaard, 2006; Mutlu Sumer, Truelsen and Fredsøe, 2006). Taiebat, Shahir and Pak (2007) concluded that liquefaction can increase significantly the coefficient of permeability of a soil, but the rapid changes in the pattern of the excess pore pressure in the soil sample during testing showed that permeability is not a stationary parameter in the liquefaction process and it may either increase or decrease at different levels.

On subaqueous slope failures in fine sand, liquefaction can be present with another, less commonly recognized type of failure: breaching, involving the instability of the sand grains at the submerged slope surface. While liquefaction is associated to very loosely-packed sand exhibiting contractive behaviour, breaching is associated with very steep slopes composed of medium to densely-packed sand with dilatant
behaviour (Berg, Gelder and Mastbergen, 2002). An example of a breach failure can be found on Figure 2, where it can be seen that the failure starts at a steep subaqueous slope, which was temporarily stable under the influence of dilation-induced negative pore pressures and shows a retrogressive behaviour while sand grains fall down from the surface and mix with water to create an eroding, turbulent sand-water mixture (Schweckendiek, Ham, Groot, Gijt, Brassinga and Hudig, 2009). Breaching is present during suction dredging operations in presence of very steep slopes, but can also be triggered after liquefaction and be the dominant mechanism: in this sense breaching can be considered as liquefaction-driven and even mistaken as liquefaction. However, as Berg et al. (2002) suggest, even though both failures can produce the same post-event morphology, the sand transport mechanisms are different. A liquefaction slope failure produces a thick, excessively concentrated density flow that may dilute to a turbidity current along extended slopes, such as in a submarine canyon. A breaching failure, on the other hand, produces a relatively thin turbidity current. As a result of this, liquefaction-originated deposits will be dominated by features like floating clasts, slump folds, distorted bedding and shear planes, while those originated from breaching failures will be characterised by subhorizontal faint lamination as the main flow-related sedimentary structure.

![Figure 2: Development of a Breach Failure in a Channel Bank. Taken from van den Berg et al. (2002).](image)

### 3.2. Static Liquefaction

Static liquefaction can be defined as a liquefaction instability, caused by monotonic loading, of a sand element whose deviator stress-axial strain curve obtained during strain-controlled loading displays a descending response (or apparent softening) (Robertson and Wride, 1998; Olsen and Stark, 2003; Mróz, Boukpeti and Drescher, 2003). Beyond the peak, the material can only sustain a decreasing load and failure occurs when the external load is greater than the decreasing material strength; this can be when the load remains constant and the material strength is still decreasing. This descending part of the undrained response is associated to the contractive behaviour of loosely packed sands, which is prevented by the almost-constant volume constraint in the undrained case and by consequent pore water pressure build-up. For medium-dense sands, the descending part is followed by a rising part, which levels off eventually when the material reaches critical state conditions.

The reason that static liquefaction is associated with monotonic conditions under loose sands does not necessarily mean it is only present in this situation. Jefferies and Been (2006) mention that even when the
loading is of cyclic nature, static liquefaction largely controls stability. If there is sufficient residual strength after cyclic loading, fatigue-like strains product of loading and unloading will be manifested which are generally unlikely to endanger anyone. However, static liquefaction has killed several hundred people on more than one occasion. Excess pore pressure can arise through undrained loading, whether static or cyclic, if the soils are loose enough; however, while straining can be evident for days before the failure, the transition to high excess pore pressures is normally very rapid. This can render static liquefaction a latent threat, as shown in the cases of Fort Peck and Nerlerk (Jefferies and Been, 2006).

The undrained behaviour of loose sands and its static liquefaction potential is frequently only of academic interest (Jefferies and Been, 2006). This is because in engineering practice soils that are sufficiently loose to fail under undrained monotonic shear are given a specific treatment in order to improve their density. The authors state, however, that knowing how dense a sand should be in order for avoiding static liquefaction is not simple given different site conditions. In addition, ground improvement has its limitations and is not an effective solution on silty soils or sandy soils with fine particles. In addition, even if in dredging operations ground improvement is a possibility, it is not applicable on sites where material is to be extracted.

From this point onwards it will be understood for the reader that the liquefaction type to be covered in this thesis will be the static loading type, also addressed as flow liquefaction by other authors (Robertson and Wride, 1998). In some of the following sections, attention will be given to the influence of the initial state of the sand on the potential development of static liquefaction, based primarily on the works of Jefferies and Been (2006), Li and Dafalias (2012) and Buscarnera and Whittle (2013).

3.3. Stress and Strain Measures: Introduction to Stress Invariants

In order to measure stress and strains, stress invariants are used, namely the mean stress (\( \bar{\sigma} \) or \( \sigma' \)) and deviator stress (\( q \)), defined as:

\[
\bar{\sigma} = \frac{(\sigma_1 + \sigma_2 + \sigma_3)}{3} \quad [1]
\]

\[
q = \sqrt{\frac{1}{2} (\sigma_1 - \sigma_2)^2 + \frac{1}{2} (\sigma_2 - \sigma_3)^2 + \frac{1}{2} (\sigma_3 - \sigma_1)^2} \quad [2]
\]

In this form, the subscript 1 is generally associated with the major principal stress loading direction, while subscripts 2 and 3 are in the plane perpendicular to the loading direction and generally associated with the confining pressure in a triaxial cell. In the formulas above, the bar superscript on the stresses represent effective stresses, which are the difference between the principal stresses (\( \sigma \)) and the pore water pressure (\( u \)). The Lode angle (\( \theta \)), a variable which relates the principal stresses to the deviator stress invariant, is defined in Jefferies and Green (2006) and Sitters (2006), in radians, as:

\[
\sin(3\theta) = -13.5 \frac{\sigma_1 \sigma_2 \sigma_3}{q^3} \quad [3]
\]

The strain invariants associated to the mean and deviator stresses, at constant Lode angle, the volumetric (\( \dot{\varepsilon}_v \)) and shear strain rates (\( \dot{\varepsilon}_q \)), respectively, are estimated by:

\[
\dot{\varepsilon}_v = \dot{\varepsilon}_1 + \dot{\varepsilon}_2 + \dot{\varepsilon}_3 \quad [4]
\]

\[
\dot{\varepsilon}_q = \frac{1}{3} \left( (\sin \theta + \sqrt{3} \cos \theta) \dot{\varepsilon}_1 - 2 \sin \theta \dot{\varepsilon}_2 + (\sin \theta - \sqrt{3} \cos \theta) \dot{\varepsilon}_3 \right) \quad [5]
\]
The strain definitions are associated with the convention that length reduction is a positive strain, so that a positive volumetric strain is associated with a void ratio reduction; this is common in the soil mechanics sign convention, where positive magnitudes are related to compression and downward displacements while being negative for extension and upward displacements. This is the opposite from the convention used in continuum mechanics, where positive magnitudes are related to extension and upward direction, and which is used on material constitutive modelling. The research done at the moment uses continuum mechanics sign convention, which will be the same used in this work for both stress and strain.

These stress and strain measures are interlinked work conjugate or interlinked in the following form:

\[ q \varepsilon_q + \bar{p} \varepsilon_v = \overline{\sigma}_1 \varepsilon_1 + \overline{\sigma}_2 \varepsilon_2 + \overline{\sigma}_3 \varepsilon_3 \]  

[6]

When looking at soil behaviour it is usually the relative amount of shear (deviator) stress to mean (isotropic) stress that matters, since soil behaviour is usually characterised by it; by plotting the effective stress data in an isotropic-deviator stresses graph it is possible to obtain “stress paths” which can describe the soil behaviour under certain loading conditions. Usually the stress ratio (\( \eta \)) is adopted:

\[ \eta = \frac{q}{\bar{p}} \]  

[7]

The strain measure matching the stress ratio is the dilation rate, defined as:

\[ D = \frac{\varepsilon_q}{\varepsilon_v} \]  

[8]

Triaxial testing has provided the basis for understanding soil behaviour, since generally it can reproduce axisymmetric stress conditions. Thus, Equations [2] and [5] generally used to describe shear stresses and volumetric strains can be reformulated to Equations [9] and [10] shown below, respectively, by making the stresses and strains on both the directions perpendicular to the major stress direction the same (\( \sigma_2 = \sigma_3 \) and \( \varepsilon_2 = \varepsilon_3 \)):

\[ q = \overline{\sigma}_1 - \overline{\sigma}_3 \]  

[9]

\[ \varepsilon_q = \frac{2}{3} (\varepsilon_1 - \varepsilon_3) \]  

[10]

### 3.4. Critical State Theory

Before addressing critical state theory, the concept of soil dilatancy must be introduced. When soils are sheared, they increase in volume if they are initially dense or contract if they are initially loose; this tendency of volume change under shearing is called dilatancy (Jefferies and Been, 2006). While Jefferies and Been use this term for any volume change, in general “dilatancy” is associated in geotechnical and material engineering to a volume increase, while “contractancy” or “contraction” is associated to volume decrease.

Dilation is usually presented in literature in two different definitions: the “absolute” concept presents it as the change in volumetric strain with respect to the initial condition, while the “rate” concept defines it as a ratio of rate of volume change with rate of shear strain. While the use of the rate definition of dilation has been considered of wider usage in the English-speaking world, the absolute definition has been used in The Netherlands, where a vast experience on liquefaction problems has been gathered.
Research into the critical state behaviour began with Casagrande in 1936, who by means of shear box tests found that loose sands contracted and dense sands dilated (expanded) until approximately the same void ratio was attained at large strains. Casagrande defined this demarking void ratio as the critical void ratio, which is affected by mean effective stress and becomes smaller as the stress level increases; this relationship is called the critical state locus (CSL). A visualization of Casagrande’s hypothesis on critical void ratio is shown in Figure 3. This served as a basis for critical state soil mechanics, a field in which the density of soils began to be seen as a state variable rather than a soil property. This state variable, denoted by \( \psi \), is defined as:

\[
\psi = e - e_c,
\]

where \( e \) represents the material void ratio at a given point and \( e_c \) the critical state void ratio.

Casagrande’s concept of critical state was essentially formalized by Roscoe, Schofield and Wroth (1958), defining it as the state at which a soil continues to deform at constant stress and constant void ratio, while showing no propensity to change from this constant void ratio condition. A different definition for critical state is given by Seed and Lee (1967) (Yamamuro and Lade, 1998), who define it as the combination of void ratio (after consolidation) and confining pressure that produces zero total volume changes at peak failure under drained conditions, while under undrained conditions it would be the combination of the void ratio after consolidation and the effective confining pressure at peak failure. This is presented in Figure 4, taken from Figure 1(a) of the paper by Yamamuro and Lade (1998).
Differences between both concepts can be understood from different perspectives. Seed and Lee consider the combination of maximum stress difference (noted as the difference between the axial load and the confining cell pressure at a triaxial cell) and no volumetric strain, which could be seen as a pre-failure condition in materials. On the other hand, recalling from Figure 3, maximum shear might not occur until reaching the ultimate strength. For this reason, and due to the fact that Casagrande’s concept is utilized commonly in liquefaction analysis techniques (Yamamuro and Lade, 1998; Li and Dafalias, 2012; Zhao and Guo, 2013), the original concept of critical state of Casagrande will be considered in this thesis.

Gonzalo Castro, one of Casagrande’s students, undertook a series of stress-controlled triaxial tests on loose sands in an attempt to reproduce field loading conditions Casagrande conjectured as stress-controlled (Jefferies and Been, 2006). This resulted in liquefaction failures that lead to a well-defined steady state at the end of the test; he termed this critical void ratio state as a steady state, and in general both terms are the same (Yamamuro and Lade, 1998). During steady state deformation the soil mass is continuously deforming at a constant volume, constant normal effective stress, normal shear stress and constant velocity. This velocity term, however, is never specified and could in principle be so small that could be neglected, making both definitions of critical state and steady state the same.

One important thing to be mentioned in this sense is that the initial theories regarding critical state assumed the condition of an isotropic material, in which properties such as void ratio and density are the same along a given soil layer. Several authors have currently worked on the influence of anisotropy on the critical state (Li and Dafalas, 2012; Zhao and Guo, 2013; Papadimitriou, Dafalas and Li, 2014) and its influence on static liquefaction (Buscarnera and Whittle, 2013), which will be addressed in another section. This will aim to not only consider realistic conditions of a sand body but also the real depositional origin of sands present either in the surface, subsurface or underwater, which may result in a realistic prediction of the liquefaction phenomenon.
3.5. Stress-Dilatancy Behaviour

Having set up the concepts regarding critical state, it is important to know that a basic yet possible framework for understanding soil, is its stress-dilatancy behaviour, or the fact that dense sands dilate and are markedly stronger than loose sands.

Something to be addressed first is that even for a given initial void ratio, the steady-state points in the effective stress failure line will not coincide on the same point under different stress paths (Yamamuro and Lade, 1998), due to the post-consolidation void ratio being decreased with increasing confining pressures. From Figure 5 it can be seen that at low stress levels as shown in the $p'-q$ diagram the material will show a stable behaviour before reaching the steady state, while for high effective stress levels instability happens. However, it must be noted from Kramer and Seed (1988) that the static liquefaction resistance increases with increasing density (i.e. a lower void ratio), which explains why major stress levels are needed for achieving liquefaction.

Figure 5: Steady State Diagram and Typical Steady State Line, with Schematic Undrained Stress Paths ending near Steady State Point. Taken from Yamamuro and Lade (1998).

It is known that soil behaviour represented by the stress ratio $\eta$ would follow a function of the form:

$$\eta = f(M_f, D^p)$$  \[12\]

In Equation [12], $M_f$ is an equivalent value, used in Critical State Theory, of the mobilized critical friction angle $\phi_f$, which varies a little with the strain and state and depends on whether the conditions are of compression, extension or plane strain. $D^p$ is different from the dilatancy or dilation rate defined in equation [8] since the superscript “$p$” represents has been introduced to indicate that the plastic component of the strain rates is relevant here. Therefore, it can be said that the behaviour of the soil will depend on: stress level, stress path and plastic strain development; this is supported by the conclusions of Gutierrez, Ishihara and Towhata (1991).

From axisymmetric stress and strain conditions (similar to the ones presented in a triaxial test), a simple form of the stress-strain equation relating the plastic volumetric strain increment rate $\dot{\varepsilon}_v^p$ (note that the superscript “$p$” was included to denote plastic behaviour) to the shear strain increment is presented below, being more or less similar to the relations found from Equations [4] (once it has been considered that the strains in $\varepsilon_2$ and $\varepsilon_3$ are the same), [8] and [10]:
Here it is noted that $\phi_f$ represent the friction angle setting the threshold for the zero-dilatancy rate and the transition from contractancy to dilatancy, defined by the critical state line on an isotropic-deviator stress space, while $\phi_m$ is the mobilized friction angle which varies during loading before reaching critical state. From Equations [12] and [13] it is observed that both positive and negative dilatancy rates can be obtained depending on the relative magnitudes of $\phi_f$ and $\phi_m$; thus, according to Wan and Guo (2004), a positive dilatancy rate can be obtained even though the current void ratio is looser than the critical. Furthermore, Equation [13] represent a family of energy dissipation curves during the course of dilation that correspond to different densities, stresses and fabric states.

3.6. Static Liquefaction and Slope Instability

It is important to understand the behaviour of a slope in terms of its failure mechanisms, especially in the case where static liquefaction occurs. When analysing slope failure it is common to assume a sliding path similar to the one shown in Figure 6. It is important to mention in advance that the size and shape of this sliding plane depend on factors like the dimensions and extension of the slope, the soil properties and its variation along the slope; however one thing can be identified from this sketch: the way that shearing mechanism is triggered varies along the sliding path. Let the effective stress with subscript 1 be the major principal stress representing the loading; on the upper part of the sliding path it can be considered that the action of a load on the upper part of the embankment results in a shear mode where the soil behaviour is dominated by compression. As it continues further in the subsoil, to the point where the tangent of the curved path is parallel to the horizontal plane, a simple shear mechanism is found where a rotation of stresses can be seen. By reaching to the toe of the slope a mode of shear with a prevailing extension mode is seen and the rotation of the major and minor principal stresses is more accentuated.

The shearing conditions described above and shown in Figure 6 could be reproduced on laboratory testing. In the case of the first and third mechanisms, they can be reproduced on a triaxial test on both compression and extension, respectively. The mechanism of simple shear can be tested on a direct simple shear. It must be noted, though, that the three tests give different shear values for a given strain level, showing the anisotropic nature of strength in soils; under rotation of principal stresses, as the loading direction becomes aligned to the horizontal, the sand sample fails at lower values of shear stress (Gutierrez et al., 1991).
The difference between drained and undrained soil conditions may play a role in slope instability. Lade (1992) concludes after analysis of submarine slope failures in Norway and instability of tailings slope that soils are stable as long as they remain under drained conditions (when the sand is perfectly stable at stress points where the normal to the yield surface is pointing in the outward direction of the hydrostatic axis), whereas instability may be obtained under undrained conditions in the region where the yield surface opens up in the outward direction of the effective isotropic stresses axis (allowing plastic strains to occur while the stresses are decreasing; here loading occurs inside the failure surface and instability may develop in the form of inability to sustain the current deviator stresses). A second condition of instability initiation is that the soil tends to contract during undrained shear. Something to recall from Lade’s work is that he indicates that the region of potential instability is located underneath the slope and that it is specific to the slope inclination and the soil parameters; additionally, the size of the region of potential instability is directly proportional to the slope height, which means that for smaller, less steep slopes, this region is diminished.

Construction process also has an influence on soil stability. In the case of saturated materials with low drainage, it is advised to perform a staged construction where excess pore water pressures dissipate over a certain period of time before placing the next layer, while consolidation increases the material shear resistance due to densification and increased confining stresses. For the cases of excavated slopes, it is important to know that without protection its stability will decrease over time after construction due to pore water pressure increase within the slope mass and swelling and weakening of the soils within the slope due to outward flow from the point of high pressure inside the slope to a point of low pressure outside of the slope (USACE, 2003); this might not be the case for sands if it is a material permeable enough that can allow water flow outside of the slope, unless there is a considerate content of silts in it. But according to the United States Army Corps of Engineers, the pore water pressure increase during construction can increase the amount of stages required for construction if no adequate drainage is provided in situ.

Modelling the static liquefaction potential on a slope has been previously done using different approaches, mainly due to historical and geographical reasons, thus varying their ways of predicting soil behaviour or formulating their stability criterion (Helbo, 1996). Three main groups can be identified and will be briefly covered, since covering the full details of these methods are beyond the scope of this work. First, the methods based on steady-state soil mechanics differentiate between the concepts of instability and failure,
and take the point from where typical stress states may cause instability as the criterion for liquefaction potential; in addition they use an ordinary stability analysis to determine the static liquefaction potential. Liquefaction evaluation procedures include those of Poulos, Castro and France (1985), Sladen, D’Hollander and Krahn (1985), Lade (1992) and Kramer and Seed (1988).

The second group is known as the Classical Dutch Approach, influenced mainly by the research programs conducted by Delft Geotechnics and Delft Hydraulics (both part nowadays of Deltares) and by the practical data gathered, mainly from liquefaction induced flow slides in the Dutch province of Zeeland. This approach can be considered as an attempt to find a correlation between pore pressure response and volume change tendency of dry soils, based in the research on the critical density of sand (Helbo, 1996). Lindenberg et al. (1981) tried to avoid the problems and difficulties with undrained triaxial testing by calculating the stress path development during undrained testing, based on the results of drained tests; first the strain measurements during drained triaxial tests are corrected for any effects caused by membrane penetration, and then a relation is derived between relative shear stress and normalized shear strain, and between isotropic stress and volume strain. The stress-strain behaviour is described by means of an elastoplastic model as explained by Stoutjesdijk and Groot (1994), assuming a plane strain condition; the same author defines a relation between stresses and strain rates by means of a “flexibility matrix” containing superposed elastic and plastic strains, whose eigenvalue \( \lambda \) is used as criterion of stability: if \( \lambda < 0 \) then it is said that the soil has liquefaction potential.

The third approach is the one developed at the University of Karlsruhe and describes the soil by means of a hypoplastic constitutive law and the formulation of a stability criterion for slopes. The use of a hypoplastic model does not only allow to model evolution of the stress rate as a function of the strain rate or model soil behaviour after failure, but also to incorporate the effect of changing void ratio (densification) and means stress level to the stress path—pycnotropy and barotropy factors, respectively. The stability criterion is based on the net work, the difference between the increase in internal energy and the work employed by external forces; if the net work is greater or equal to zero, then the system is considered as unstable (Helbo, 1996).

Having determined whether or not a slope has liquefaction potential, the next question is related to the propagation process of liquefaction in the soil. Lo, Mizanur Rahman and Bobei (2008) describe that liquefaction in the field is not element behaviour since the in situ stress states are not uniform within a slope. As liquefaction initiates within the slope, void ratio and pore water pressures will redistribute, leading to an increase in void ratio in the soil above the initially liquefied zone due to cracking as a result of a reduced effective stress. Figure 7 shows a sketch of this mechanism, also addressed as “Mechanism C” according to a report of the National Research Council in 1985 (Lo et al., 2008); as pore water pressure increases on the sand body in such a way that its water flow will go outward resulting not only in sand loosening due to internal erosion (possibly piping) but also cracking at the face of the slope. The liquefaction process has a retrogressive nature, which means that the liquefied portion of the slope grows into the sand body.
Attempts have been made in order to model retrogressive failure by breaching and by successive liquefaction mechanisms; some methods have been based on the research of pore pressures near moving underwater slopes and cutting forces in saturated sand (Helbo, 1996). In The Netherlands research was conducted in dense sands, within the framework of dredging (Meijer and Os, 1976 and Os and Leussen, 1987), showing that the assumption of linear elastic soil behaviour does not apply under rapid deformations and that dilatancy can play a role in the deformation resistance of soil, when considering dense sands. Os and Leussen (1987) considered the failure of densely packed sand, when forced to deform by a cutting blade, as a continuous creation of discrete sliding surfaces of the same shape, where the porosity suddenly increases; this proposition was supported by experiment results. Approximation of the process was schematized as a thin sliding zone moving through the soil.

A hypothesis was drawn from the liquefaction failures along the southern part of the Mississippi River in the United States. Torrey et al. (1988) came to the statement that the dilative behaviour of the material is suspected to be responsible for liquefaction-triggered failures, instead of the contractive behaviour. However, this could also represent the breach failure mechanism discussed earlier.

Picarelli, Olivares and Avolio (2008) mention that in order for soil liquefaction to occur in a slope, the following conditions must apply:

1. Complete soil saturation on the liquefied area: this was previously addressed and it is understood that saturation has an effect on the effective stresses path. In the case of slopes above water level, full saturation can be achieved by rainfall and reached before or at the beginning of liquefaction.
2. Susceptibility to liquefaction: index and state properties such as grain size, soil plasticity and density have an effect on the liquefaction potential of a soil.
3. A mechanism of slope deformation forcing rapid volumetric deformation capable to trigger positive excess pore pressure: this relates the mechanism of deformation associated to failure. Liquefaction can be caused by volumetric soil collapse induced by saturation, by progressive failure and by impact.
4. A mechanism of soil movement capable to sustain or to re-generate excess pore pressure: this condition seems to apply to the case of relatively steep slopes, because of the rapid increase of velocity of the soil mass after failure.

From the above it can be concluded that, based on the shearing mode, dominant on a great part of the sliding path, and on the dimensions of the zone of potential instability, it is just common to perform triaxial compression tests to evaluate static liquefaction of soils. However, as it will be indicated later, the
microstructure of the soil plays a role on the process of liquefaction, especially when considering that natural sands, and all soils in general, are a result from a depositional process that depends greatly on the geological processes which does not result in a completely homogeneous soil formation.

3.7. **Microstructure of Natural Sands**

The microstructure is, along with the density and stress state, one of the dominant factors that influence the behaviour of sands (Vranna and Tika, 2014). But the microstructure in itself is dependent mainly on the geological processes taking place, therefore it is important to take a look first on the geological processes involving the formation of these type of soils, especially the deposition process.

Soil grains generally come from rocks exposed to weathering and further transported to another location for sedimentation. While sediment transport and deposition methods include gravity, wind and ice, water is the most important since rivers move most of the sediment in land and the flow velocity is directly proportional to the potential of carrying a diverse amount and size variety of soil particles. Most of the sand material is transported by this way and end up at the sea, where it is generally sorted and stratified. Wind transport and deposition is characterized by depositing very well sorted sand and silt material, mostly in or near dry source areas, making this characteristic of desert regions. The other processes are generally restricted by climate, topography or geology itself, and usually carry coarser, poorly sorted material.

Different alluvial deposits exist which contain sand material: alluvium (sorted and bedded, vertically and laterally variable), floodplain (build-up of alluvium over time, resulting in mostly fine grained and horizontally bedded deposits) and meander scrolls (cross-bedded, crescentic lenses of sediment, mostly sand or gravel). Some deposits may be formed from meltdown of glaciers and transport of sediments, mostly being composed of sand and gravel with moderate sorting and bedding: these deposits are outwash, kames and eskers; glacial till will not be considered here since it is basically a mixture of all or any material size, thus not fitting exactly as an exclusive, sandy soil deposit. Semi-arid environments can produce aeolian sand deposits, the most commonly known being dunes (irregular in shape and with presence of loose and dense sands) and alluvial plains, the latter being formed from isolated rainstorms. Periglacial sediments like lowlands may include outwash gravels, alluvial and blown sand, transported from another site. On coastal environments characterized by wave action, beaches are formed by sand deposition where the upwash is greater than the backwash, due to the water soaking down into the sand, while coastal dunes are formed from beach sand blown inland. For more information, the reader can refer to Waltham’s (2009) book on Engineering Geology or to any other literature material covering sediment transport and deposition.

From the above it can be observed that different depositional processes and geological history affect the microstructure of sands, not forgetting that geological history also influence the stress levels in situ. Weathering, chemical deposition, environmental changes and other phenomena may form within the sands small amounts of material (like silica, hydrous iron oxides and carbonates) that form weak interparticle bonds (Vranna and Tika, 2014); even in the case of clean sands, the presence of loose and dense sand pockets result on a soil behaviour not dependent on the mean initial porosity but on the degree of heterogeneity between loose and dense sand pockets (Feda, 1994). Figure 8 shows a schematization of this. Comparison of literature from different sites regarding the formation of sands indicates that sands may possess different properties around the world, depending on the transport and deposition mechanism, parent rock, climate, etc. (Martins, Ferreira, Altamirano Flores, Bressani and Bica, 2005; Al-Ansary, Pöppelreiter, Al-Jabry and Iyengar, 2012; Chen, Xu, Chen, Wang, Long and Shen, 2014; Kalińska-Nartiša, Nartišs, Thiel, Buylaert and Murray, 2014;
Zhou, Wang and Zhao, 2015 and Ryashchenko, Akulova and Rubtsova, 2015); this ultimately affects the microstructure of sands.

The way the sand is bedded on the soil affect also its microstructure. Yoshimine and Koike (2005) tested reconstituted layered and uniform loose and dense sand samples and found that layering loose sands are more dilative and stiffer compared to its uniform counterparts, while dense-loose layered sands showed a slightly accentuated contractive behaviour; the range between minimum and maximum void ratio and the undrained strength is larger in layered structures than in uniform ones. The above can be explained on the basis of the particle packing in the grain size scale (fabric) and on the laminar structure of the deposit on a larger scale (structure); in essence, layering of a reconstituted sample may cause a shear band to take longer to develop along this type of samples since it needs to pass through “different” layers.

Liquefaction potential is affected also by the orientation of the bedding with respect to the major principal stress direction: if the bedding plane is perpendicular to the major principal stress direction then the sand will behave in a stiff manner with no liquefaction (similar to a dense sand, even if the material is loose), while when being both parallel to each other result in an extended excess pore water pressure build-up and, eventually, liquefaction (Wan and Guo, 2001 and Yu, Zeng, Li and Ming, 2013). Figure 9 shows the effect of bedding orientation on pore water pressure build-up, while Figure 10 presents the effect of layering on the stress paths, for a given confining pressure. Both figures represent the results of an earthquake geo-centrifuge test for models of layers with deposition angles of 0°, 45° and 90° in which acceleration-time history, excess pore water pressures and ground settlements were investigated; details on the test setup can be found on Yu et al. (2013).
**Figure 9:** Time Histories of Pore Water Pressure on Different Tests with Changing Deposition Angle. Taken from Yu et al. (2013).
As the material reaches failure, microstructure effects in the scale of grain size should diminish if the soil is largely deformed, due to destruction of the structure and the rearrangement of particles during large deformations (Yoshimine and Koike, 2005). This behaviour is more sudden in sands with well-rounded grain shape with void ratios greater than 0.95, acting as a brittle material (Konrad, 1990); under shearing, the loads are suddenly transferred to the water, resulting in a sharp increase in the excess pore water pressure. At the shearing bands (see Figure 11), the material density tends towards a constant value regardless of its initial value; during strain localization, volumetric and shear strains are locally increased at the shear bands, being a locus of significant density changes (Dersues, Chambon, Mokni and Mazerolle, 1996).
The study of microstructure of sands in situ has been a challenge due to potential disturbance during the processes; some methods like freezing or resin impregnation combined with digital scanning are effective on this, but some of these methods are expensive enough to make people move into the use of cheaper methods that pose the risk of disturbing significantly or even destroying the microstructure. Furthermore, reproducing the microstructure of natural sands on both laboratory tests and material models has been a challenge for the last years. The tests taken to study the undisturbed microstructure of a sandy soil are beyond the scope of this work but, as it will be discussed in the following sections, the current approaches on sample preparation, testing and computer simulations attempt to reproduce, to the best of its capabilities, the in situ conditions; this eventually allows not only to understand the real behaviour of soils or the phenomenon of liquefaction under real conditions, but also to move to more accurate modelling and prediction of this engineering problem.

The influence of microstructure of sands in soil behaviour has not been considered in the traditional theories of stress-dilatancy based on energy principles. Wan and Guo (2001) remarks that microstructure has an effect on the stress-dilatancy behaviour through the volume change in the drained condition and the accumulation of excess pore water pressure in the undrained condition, and advocated that continuum variables like stress and strain are related to the microscopic counterparts of particle contact force and relative displacement, respectively (Wan and Guo, 2004). In the next section the integration of fabric and structure on
modern theories and models of soils will be covered, especially the development of a critical state theory that takes into account the effects of anisotropy of the sand.

3.8. Trends in Constitutive and Numerical Models

By the end of the 20th Century several constitutive models for sand have been already developed, the basis of this were models dating as far back as 1864 with Tresca’s differentiation on elastic and plastic components of the total strains, and going through the introduction of yield surfaces; however, only some of them were models that accounted for void ratio and therefore being based on critical state theory (Jefferies and Been, 2006). All these models are part of either a descriptive approach, idealized approach, or a combination of both. Descriptive models are intrinsically curve-fitting and related to test data, their accuracy in representing a particular situation is too often offset by a lack of understanding or insight into the physical processes behind it. On the other hand, idealized models start from postulated mechanisms and then move forward to the behaviour prediction; even though they are based on a consistent and usually known physics, there is a potential for reduced accuracy.

There have been, generally, two key aspects that have an effect on the way modelling is approached: fabric nature and strain level. In the past, most models considered an isotropic nature of the soil since they assume a condition where the properties are the same in the entire domain; models consider isotropy since it was desired to understand how isotropic material behaved, before going into detail with anisotropy. Nevertheless, anisotropy can be approximated in an isotropic model provided there is supporting evidence in doing so. In geomechanics it has been seen that small strain theory has been useful in the context of elasticity, but that soil behaviour large strains may be required to reach failure. Jefferies and Been indicated that, even though properties determined in calibrating small strain definitions are commonly considered as sufficiently representative for its subsequent use in large strain analysis, there is a present problem with the accuracy between how strain can be determined from laboratory testing and how accurate these measurements, when calibrated, are at large strain conditions.

There are two soil behaviours apparent to the average observer: plasticity and density dependence, both under constant mean stress; the first one represents the irrecoverable nature of deviator deformations imposed on the soil, while the second one represents the irrecoverable nature of isotropic deformation and so density and its change. Since a micromechanical modelling approach, considering grain realignments and movements, result in complicated and generally unusable models, plasticity theory based used on engineering metals can be used in soil mechanics with the assistance of thermodynamics.

The following section will describe some modern trends in constitutive and numerical modelling of critical state and liquefaction on sands, on the basis of constitutive modelling, numerical simulations and testing which proved the validity of some models. Covering all the details of these models is beyond the scope of this work, but by having a quick overview of the current trends will contribute to the understanding of the current research approaches and the relation of this work with the ongoing modelling research at the Geo-Engineering section at Delft University of Technology.

Pradel and Lade (1990) investigated elastoplastic isotropic materials with non-associated flow. Drucker’s stability postulate and Hill’s maximum work principle, within the theory of plasticity, require the second increment of plastic work $d^2W^p = d\sigma_{ij} \cdot d\epsilon_{ij}^p$ to be positive in order to guarantee mechanical stability.
of the frictional materials and ensure associated plastic flow. The following formulations for the yield and plastic potential functions, respectively denoted as $f$ and $g$, for a given stress state located on the yield surface were considered; on these equations, $\mu$ and $\eta$ are the local slopes of the yield surface and plastic potential functions, respectively, in the $p'$-$q$ plane at Point $P$, and $C$ and $D$ are constants (refer to Figure 12 for a visual representation of these functions):

$$f = q - \mu \cdot p' - C$$  \hspace{1cm} [14]

$$g = q - \eta \cdot p' - D$$  \hspace{1cm} [15]

![Figure 12: Yield Function and Plastic Potential for Frictional Materials with Non-Associated Flow. Taken from Pradel and Lade (1990).](image)

Experimental studies suggest that stability and instability can be expressed in terms of the slope of the yield surface, the slope of the plastic potential surface, plastic-hardening modulus and the elastic compressibility of the soil skeleton and pore fluid. Thus, by means of drained tests it was demonstrated that the condition $d^2W^p < 0$ does not imply mechanical instability. With undrained conditions and full saturation, instability occurs in a region where the yield surface opens in the outward direction of the isotropic axis and the material tends to contract. The degree of saturation was found to play an important role in the stability of soils. This was supported by Gutierrez et al. (1991): the direction of principal plastic strain increment in sand during principal stress rotation is very much dependent on the stress increment direction, a response which violates the postulate of flow uniqueness in plasticity theory.

Jefferies presented in 1993 a constitutive model for sand following the fundamentals of critical state theory: existence of a critical state locus or line (CSL) and CSL as an ultimate condition of all deformation processes in soil during monotonic deformation, the latter being a basis of an incremental hardening rule with the state parameter $\psi$ as a rate variable. This model, referred as Nor-Sand, captures the influence of void ratio and confining stress on the constitutive behaviour of sand and possesses the following attributes: associated plastic flow and dilation, achievement of critical state with shear straining independent of initial conditions, inclusion of initial void ratio and mean stresses through the use of $\psi$, constringent of work-hardening to
maximum values to replicate the measured behaviour of sands, work-hardening to maximum strength even during dilation and work-softening during post-peak regime. More details of this model can be found on Jefferies (1993) and Jefferies and Been (2006).

For simulations, effects of microstructure are included by using a representative volume element with boundaries subjected to uniform strain fields. Thornton (2000) expresses the average stress tensor as

$$\sigma_{ij} = \frac{2}{V} \sum_{\text{contacts}} R N_i n_j + \frac{2}{V} \sum_{\text{contacts}} R T_i t_j; \ n_i t_j = 0,$$  

where $V$ denotes the volume within the element, $R n_i$ the radius vector to the contact point between grains, $N n_i$ and $T t_i$ the normal and tangential contact forces for the contact orientation, defined by the unit normal vector to the contact plane $n_i$, and $t_i$ defines the unit vector parallel to the contact plane. Rewriting the previous equation using a statistical average notation leads to

$$\sigma_{ij} = \sigma_{kk} \left[ \frac{\langle R N_i n_i \rangle}{\langle RN \rangle} + \frac{\langle R T_i t_i \rangle}{\langle RN \rangle} \right] = \sigma_{ij}^N + \sigma_{ij}^T,$$  

where $\sigma_{ij}^N$ and $\sigma_{ij}^T$ represent the normal and tangential contact forces part of the stress tensor and

$$\sigma_{kk} = \frac{2C}{V} \langle RN \rangle,$$  

where $C$ represent the number of contacts within the element of volume $V$. Alternatively, the fabric stress tensor may be defined as:

$$\sigma_{ij}^f = \sigma_{kk} \phi_{ij},$$  

where the structural anisotropy tensor, known as the ‘fabric tensor’, is defined by

$$\phi_{ij} = \langle n_i n_j \rangle$$  

On another work, Wan and Guo (2001) explored the effect of microstructure on sand undrained behaviour. They computed mean normal and tangential forces and mean relative normal and tangential displacements for an ensemble of particles, by relating the contact force ($f_c$) in a volume ($V$) with the Cauchy stress tensor ($\sigma$) and the inverse of the fabric tensor ($F$), which describes the geometrical particles arrangement, and with the “real” stress tensor ($\sigma^*$), obtained by factoring the fabric into the Cauchy stress tensor, and a branch vector connecting two particle centroids ($\Gamma$). This relation is defined as

$$F = \frac{1}{V} \sum_{\text{contacts}} I^c \otimes I^c; \ \sigma^* = (\sigma: F^{-1}); \ f^c = \sigma^* \cdot I^c$$  

$$\varepsilon^* = \varepsilon; \Delta u^c = \varepsilon^* \cdot I^c$$  

As shown in Equations [21] and [22], this branch vector also helped to relate the strains and fabric tensors to the contact displacements ($\Delta u^c$). The evolution of fabric under action of stresses or strains was considered. From their simulations they found that the particle arrangement had an influence on the stress-dilatancy property of sand during deformation, the fabric changes being more pronounced in undrained conditions than during drained conditions. Their work had its limitations, since the model predictions regarding microstructural issues was based purely on deviator stress changes, thus not being able to predict fabric changes caused by isotropic stresses applied to an initially anisotropic sand.
Imam, Morgenstern, Robertson and Chan (2005) presented a critical-state constitutive model for sands over a wide range of void ratios and consolidation pressures in a triaxial plane; this was an attempt to model properly aspects of loose sand behaviour that affect their susceptibility to static liquefaction. The elements of their proposed model consisted of five elements: a capped yield surface dependent on void ratio, consolidation stresses, inherent and stress-induced anisotropy; a Mohr-Coulomb failure criterion; a flow rule derived from stress-dilatancy relationships combined with a variable friction angle at zero dilatancy; hardening laws allowing changes in the size and shape of the yield surface; and isotropic elasticity. The model showed to predict in good agreement the behaviour of sand, provided that at higher confining pressure values some parameters required to be modified in order to improve its accuracy; in addition, the authors mentioned that more data from previous tests and case histories was required in order to assess in detail the accuracy of the model, especially when considering its use on field situations.

Taiebat et al. (2007) performed a study on pore water variation on a fully coupled soil-water interaction by using two different plasticity models: one considered an advanced critical state two-surface plasticity model developed by Manzari and Dafalias (1997) and a simple elastoplastic densification model developed by Zienkiewicz, Chang and Hinton (1978). The model of Manzari and Dafalias accounted for non-associated flow rule and yield surface updates based by kinematic and isotropic hardening, while the plastic potential and failure surfaces were fully determined by the value of the state parameter $\psi$. The densification model simulated densification of the solid phase as the essential cause of pore pressure build-up; it defined densification quantity as a function of history of shear strain and stress ratio. Results showed that the densification model can predict liquefaction phenomenon close to ground surface, but its accuracy decreases with increasing depth; this model performs poorly when predicting horizontal and vertical displacements during liquefaction. The critical state two-surface plasticity model, on the other hand, captured most of the important features of the soil-water interaction under cyclic loads, showing its superiority for liquefaction simulation.

Kim, Hwang, Ko and Kim (2009) tested saturated sand samples in order to study the liquefaction behaviour of level saturated sandy soils. Results showed that the excess pore pressure in the liquefied sand was dissipated by the combined process of sand grains solidification and consolidation of the solidified layer; from this a nonlinear model for the solidified layer thickness versus time was developed, as well as a new dissipation model which combined this nonlinear solidification model with Scott’s modified dissipation theory for improvement of time history of excess pore pressure and also a method for evaluating is input parameters. However, their model had its limitations with respect to its applicability on field site.

In 2012, Li and Dafalias presented a theory that combined critical state with the inherent anisotropic nature of soils: the Anisotropic Critical State Theory or ACST. This theory introduces a fabric anisotropy variable (FAV) “A” that, along with the stress ratio and void ratio, defines the necessary and sufficient conditions for a critical state to occur; the evolution of FAV depends on the changing loading direction and the evolution of the fabric tensor. This theory extends the e-p plane (void ratio vs. mean stress) by including a third axis (representing FAV) and introducing a Dilatancy State Line (DSL) that converges with the critical state line once critical state has been reached; before this convergence is reached, the DSL specifies the dilative or contractive state of soil. The introduction of DSL extends the state parameter $\psi$ into considering anisotropy; Figures 13 and 14 depict the role of the DSL on this ACST. This theory proves the uniqueness of the CSL on the basis of Gibbs stability condition of equilibrium (entropy of a system in equilibrium is the maximum among all the neighbouring states with the same initial energy; the internal energy of an equilibrium system is the minimum among all the neighbouring states having the same entropy).
The implementation of the ACST was presented by Papadimitriou, Dafalias and Li (2014), adopting the SANISAND (Simple ANIsotropic SAND) constitutive model platform. The SANISAND platform is based on the work of Manzari and Dafalias (1997) and Li and Dafalias (2000) on the dependency of the boundary and dilatancy model surfaces on the state parameter $\psi$. By simulating sand experiments and comparing them with real tests results, it was found that this new model was capable of characterizing the anisotropic behaviour of sands.

Buscarnera and Whittle (2013) proposed a modelling strategy inspired by the concept of latent instability, an instability condition caused by a change in drainage conditions, which provides conceptual tools.
that can be used to perform laboratory experiments and assess the predictive capabilities of constitutive models, while providing mathematical support for distinguishing liquefaction triggering from critical state conditions.

Wensrich (2014) reviewed the physical nature of contact moments within granular assemblies and developed a new approach for homogenization of stress within granular material, using the concept of contact eccentricity. By this it is possible to calculate an expression for bulk stress that is both symmetric for material in equilibrium and fully consistent with the usual definition of bulk stress as an ensemble average of material stress over a representative volume element, not considering the particle twisting. Assessment of this was done using Discrete Element Methods.

Jager and Molenkamp (2014) extended the microscale approach by applying the conditions of both conservation of linear and angular momentum of all individual particles; the characteristics of the resulting volume-averaged global stress measure were investigated for applications in continuum mechanics and its results elaborated for a plane case of a simple-shearing flow of a liquefied loose sand down a slope. They address that the modelling and analysis of static liquefaction and post-liquefaction flow by means of continuum mechanical methods require the availability of an appropriate stress measure, which could take account of all relevant types of interparticle motion, thus not only the interparticle sliding and related marginal interparticle rolling as occurring during deformation of dense granular materials, but also including the effects of particle rolling and rotation as expected during the flow of loose fluid-saturated granular materials.

Teunissen and Kruse (2014) performed static liquefaction simulations in layered loose and dense sand considering a known layering and using a simple Mohr-Coulomb type of model. Simulations captured the liquefaction potential of loose sand, in comparison of medium dense and dense sands, and concluded that the dimensions of the spatial variations in density and permeability had a significant influence on the pore pressure dissipation upon deformation and the occurrence of liquefaction of the total soil mass.

Fabric quantification is important in numerical modelling since it does not only considers the nature of isotropic or anisotropic structure of sands, but because it considers the evolution of the fabric under loading until the ultimate state where there are no changes in the fabric (Wan and Guo, 2001) and which particle orientation is mostly related to the applied stress. This is captured in a fabric tensor, describing geometrical arrangements of grain particles, which relates stresses and strains; this is especially for the ACST, in which the fabric tensor is defined on the basis of the statistical attributes of relevant microstructural entities.

Li and Dafalias (2012) showed the correlation of the material response at the continuum level with the fabric evolution. They resumed the numerical results on 2D random assemblies using DEM and showed:

- Upon shearing, all specimens become anisotropic and have a fabric tending to be coincident with the loading direction. When fabric is more anisotropic and oriented towards the loading direction, the response tends to be more dilative (or less contractive) under otherwise identical conditions.
- The development of fabric anisotropy has a direct effect on the isotropic part of fabric—the density.
- The shear-induced tendency of becoming more dilative will generally lead to a transition from an initially contractive response to a dilative response at late stages. In the case of very loose soils, the initial contraction may be so severe that the shear-induced tendency is not sufficient to attain a phase transformation (from contractive to dilative material response) before failure.

Zhao and Guo (2013) performed simulations on a 3D DEM model, based on the work of Li and Dafalias. They found that the critical state in granular material can be uniquely characterized, by associating it with a fabric structure compatible with the critical stress state. After several simulations under drained and
undrained triaxial compression conditions, it was found that the fabric reached at critical state is highly anisotropic and not unique, but dependent on the specific loading path. It is strongly related to the critical stress state and cannot be separated as an individual (thus additional) or unique reference for the soil state.

From the above, it can be seen that there is a current trend in bringing the inherent anisotropy of soils into the critical state models developed during the last century. The study of this has been done in both simulation of previous experimental tests, computational simulations of previous models and study at microscale level of the grain behaviour under loading and its relation to liquefaction. The author expects that, in the future, liquefaction models of slopes used in engineering practice will account for soil variability and will reproduce, within an acceptable accuracy level, the constitutive soil behaviour at scales ranging from microscopic scale to macroscopic.

3.9. **Element Testing Methods**

Recalling from Figure 6, the conventional testing done for liquefaction analysis on laboratory conditions include triaxial compression and extension and simple shear of soil samples; description of the different testing procedures will not be covered here since they can be found on the different national, regional and international norms. However, a brief description of the use of these tests in assessing static liquefaction on loose sands will be given.

Figure 15 shows the results of undrained triaxial compression tests in loose sand, on both $\varepsilon$-$q$ and $p'$-$q$ planes. Based on the results, it can be seen that, as the sample tends be more and more loose (increasing value of $\psi$), the peak value of deviator stresses is reached at a lower level for a constant confining pressure (tests G602, G605 and G609); at the same time, with increasing initial void ratio there is an increase in the tendency and that the undrained residual stress is lower than the peak strength, which is an indicator of the liquefaction potential. A second thing to identify is that increasing confining pressure results in an increase of the liquefaction resistance, which can be seen on the length of the stress path before reaching its residual strength level (test G602 is an exception to the rule since it seems that the sample is closer to a dense state). Finally, it can be seen that, as the sand state becomes denser, the behaviour tends to be more dilative than contractive; stress path from test G602 present a more dilative behaviour when compared with, for example, stress path from a looser state (e.g. test G609).
From Figure 16, similar behaviour is evident in terms of peak strength relation with void ratio when performing undrained triaxial extension tests, but in this case the peak strength is lower than in compression. In addition, although the effective stress paths have similar shape, the difference in peak strength flattens extension paths compared to those in compression.
The simple shear test is attractive as being a possible better analogue of conditions in the ground (Jefferies and Been, 2006). However, there have been fewer publications on static liquefaction studies using this test, especially when considering tests on sands with initial states looser than the critical state. Figure 17 presents the results between the three tests on silt, as a way to compare the results of these tests; the dilative behaviour is accounted for by the nature of the grain (smaller grain size, smaller void ratio, and dense material, as indicated in Figure 16 for a constant effective mean stress). It can be seen that the behaviour between triaxial shear and simple shear behaviour is similar, although the excess pore pressure is much greater initially in simple shear and with less intense subsequent dilation. This difference in results could be explained when analysing cyclic tests results: while triaxial tests do not in general duplicate in situ stress conditions, there are problems with calibration of constitutive models to the measured cyclic simple shear results since there is no measurement of horizontal stress. In addition, principal stress rotation takes place in the simple shear tests, the lack of horizontal stress measurement leads to large ambiguities in what the test data actually mean; thus, testing using the hollow cylinder method allows to investigate the effects of changes in principal stress direction.

Even though the above paragraph concludes that the hollow cylinder test is ideal for testing (Jefferies and Been, 2006), building and carrying out tests in these devices is complex and is usually limited to research; thus triaxial and, to a lesser extent, simple shear tests are carried out in commercial soil testing laboratories. This could have an effect on the quality of the data gathered on projects, which could have biasing effect when
using those projects as case histories for studying liquefaction, since the data from simple shear and triaxial tests is not as accurate as the one gathered by the hollow cylinder test. Comparison of the level of accuracy of simple shear and triaxial with respect to the hollow cylinder test is beyond the scope of this project.

![Figure 17: Comparison of Silt in Simple Shear, Triaxial Compression and Triaxial Extension at Constant Confining Stress. Taken from Jefferies and Been (2006).](image)

There are fundamental factors during testing that affect liquefaction susceptibility of sands. Vaid and Sivathayalan (2000) grouped them into four categories:

1. Ability of Testing Device, Loading Method and Data-Acquisition System to measure and record True Element Behaviour.
2. Effect of Initial State Variables-Void Ratio, Effective Stress State, Sand Fabric and Strain History prior to undrained loading.
3. Influence of Effective Stress Path during Loading.
4. Effect of any deviation from the Undrained Deformation Assumption. Such deviations are inevitable as a result of flow and volume changes, caused by the spatial variation of excess pore pressure generated during the duration of loading or their subsequent dissipation after the cessation of loading.
The influence of sample preparation method exerts an influence on the sand structure and its initial density, in the same way the depositional process of sands in the field have an effect on its structure, as seen in Figure 18. Water pluviation (WP), air pluviation (AP) and moist tamping (MT) are some of the most common sample preparation methods for loose sands. MT fabric results in a strain softening response, the strength ultimately reaching the steady state. AP fabric is strain softening to a lesser extent, demonstrating a quasi-steady state type of response. WP fabric does not strain soften but behaves in a strain-hardening (dilative) manner. The sand fabric from MT appears to be potentially collapsible and hence prone to liquefaction; therefore, modelling water-deposited sands using MT may unjustifiably label them as being liquefiable, given its discrepancy with the depositional process to be simulated. In that case WP is considered as a better alternative for this, since it mimics the deposition of fluvial and hydraulic-fill sands. Details on sample preparation methods for laboratory testing will be provided in another section.

![Figure 18: Effect of Disturbed Sample Preparation Technique on the Undrained Shear Response of a Sand. Taken from Vaid and Sivathayalan (2000).](image)

According to the British Standard on the shear strength tests (BS1377:Part8:1990), saturation and consolidation of the soil specimens go before the actual loading; the norm suggests two methods for achieving full saturation: application of water pressure (back pressure) to the specimen and increase simultaneously the cell pressure in order to maintain a small positive effective stress, or by increasing the cell pressure only; however, the norm warns that the applied effective stresses should not be so high as to excessively pre-stress or overconsolidate the specimen. This last statement was confirmed by Xia and Hu (1991): its effect on the liquefaction resistance is directly related to the level of the applied back pressure; this means that when using high back pressures the sand showed higher liquefaction resistance, the latter being overestimated as a result. These authors suggested that, during liquefaction tests, the back-pressure technique for increasing the degree of saturation of a sand tested for liquefaction should not be used.

Regarding the modelling of in situ stress states, Vaid and Sivathayalan (2000) agreed, based on data collected from the Canadian Liquefaction experiment (CANLEX), that in situ sands are unlikely to exist in states looser than those achieved by the loosest WP deposition; looser states could be achieved by MT, but their relevance in characterization of in situ sands seems questionable. WP is capable to substitute the specimens obtained from undisturbed saturated loose sand samples (which can be expensive), since it closely
replicates the fabric of water-deposited in situ sands with reconstituted equivalents, for confident material characterisation.

With regard to the effects of loading and data-recording techniques, Vaid and Sivathayalan (2000) conclude that the true strain softening static response using stress-controlled loading can be measured with confidence only by using an inertial system (in which a non-constant strain rate is applied by applying a dead mass, whose acceleration depends on the sample deformation); when loading is strain-controlled, inclusion of a high frequency response data acquisition system is required. The use of a non-inertial load controlled loading system may affect seriously the real strain softening behaviour of sand; this influence will depend on the degree of interaction between the apparatus and the specimen during testing and the frequency response of the data-recording device.

Finally, the loading method can affect the liquefaction behaviour; while in monotonic loading a constant loading is considered, in cyclic loading stress reversal is involved. In cyclic loading, strain softening also can result in liquefaction-causing deformations, provided three conditions are simultaneously satisfied (Vaid and Sivathayalan, 2000): the sand is strain softening in static loading, the maximum shear amplitude in either triaxial compression or extension pulse exceeds the static shear strength in steady or quasi-steady state in that mode, and a sufficient number of load cycles are involved. Incremental loading and step-loading during tests result in differences in the deformation rates; step-loading do not yield an accurate value of the peak deviator stresses and can also influence the pore water pressure response (Konrad, 1990).

### 3.10. Sample Preparation Methods

The previous sections showed the relation of the structure of a sand on its behaviour during loading and to what extent the specimen preparation methods affect the results during testing. In this section, some of these techniques used for sample preparation, especially for triaxial testing, will be introduced; a quick overview will be given to the “undisturbed” sampling techniques and then more detail will be given to the disturbed sample techniques.

From the literature reviewed, two methods for in situ “undisturbed” sample retrieval were identified: polymer impregnation (Kuo and Frost, 1996 and Sutterer, Frost and Chameau, 1996) and ground freezing (Konrad and Pouliot, 1997 and Ghionna and Porcino, 2006). Polymer impregnation allows for smaller sample size to be retrieved from the field and can allow for uniformity evaluation of the microstructure; however, these type of techniques are sensitive to microscopic and macroscopic variations in porosity, while even potentially damaging the soil fabric due to the temperature required for heating the soil to polymer impregnation. These polymer methods can be slightly more expensive than conventional sampling and cheaper than ground freezing. The success of ground freezing depends on factors like grain size distribution, particle shape, amount of fines and their mineralogy, and depends considerably on the ability to expel all excess pore water pressure as freezing progresses into the soil mass; it also needs to consider thawing effects on the soil structure.

In cases where “undisturbed” samples cannot be retrieved, reconstituted specimens can be prepared; Figure 19 shows some of the methods used for preparing clean sand specimens. The reconstitution method should reproduce, as much as possible, the microstructure and depositional nature of the ground conditions. In the following paragraphs some of the methods described by Yamamuro and Wood (2004) and Jefferies and Been (2006) will be presented.
Moist tamping is the easiest method of preparation to achieve a full range of densities. The principle is that the specimen is prepared at a moisture content of about 5%, resulting in capillary forces allowing bulking of the sand to a density that will not be achievable with wet or dry samples. The effective stress induced in the sample by the capillary forces also helps to keep the sample shape once the split mould is removed. As evidenced earlier, this method results in a specimen fabric or structure that is different to the one obtained in nature; because of that, pluviation methods are preferable.

Wet pluviation is a useful specimen preparation technique when samples without any pre-consolidation due to capillary tension or with a fabric different from moist tamping is required. It is difficult to control, however, the ultimate void ratio of a pluviated specimen. Jefferies and Been (2006) mention two things that should be considered in wet pluviation: preparation of a sample using this method may result in some fines loss from the sand, roughly 50%; this should be accounted for in density and dry weight calculations, and it is advisable to check the final fines content of the sample after the test. The second thing is related to the deposition conditions: even though this method is the most preferred to reproduce conditions in hydraulic fills and river deposits, the calm laboratory conditions are unlikely to be similar to the underwater deposition in rivers and sea beds where strong currents are usually active at the time of deposition.

The slurry deposition method was developed mainly to overcome the problem of particle segregation in poorly graded or silty sand samples. In this case the silt or clay fines are separated from the sand and then mixed with water and boiled to de-air the mixtures. This preparation method requires some operation of the triaxial cell in order to regulate the void ratio of the sand.

Dry pluviation is a commonly used and reliable method to achieve a uniform density in clean sands. By close control on the deposition rate and the drop height of the sand, a range of densities can be achieved with the technique. It needs more sophisticated equipment than moist tamping or wet pluviation, therefore many variants of the method exist. The principle behind this method is that the correct dry weight of sand is contained in a hopper of the same diameter as the specimen mould, which will be placed directly above it. Sand is then allowed to pluviate through a diffuser (e.g. a coarse mesh sieve), into the mould. The drop height is controlled by ensuring the diffuser is at constant height above the specimen surface. Pluviation rate is then controlled by the size and number of holes in the bottom of the hopper. Its application is limited since it cannot be applied for sands with plastic fines, it is also difficult to prepare very loose specimens with this technique, and unmeasurable volume changes may occur during specimen saturation.
3.11. Fluidization

Allen (1984) presented the term “liquidization” to describe the mechanisms that cause a change of state in cohesionless sand and coarse silt under natural circumstances, in such a way that the materials lose their strength and become like viscous liquids. When the liquidizing mechanism disappears, redeposition of sediments with escape of pore fluids occur. In the previous chapter, one of those mechanisms was extensively described: liquefaction, in which loading on the soil mass under undrained conditions triggers a pore pressure increase and a reduction of shear strength to almost zero. However, there is a second mechanism described by Allen which involves an induced fluid flow through the particles by means of an external source of fluid; this mechanism is known as “fluidization”.

While fluidization is of interest for chemical engineers for substances separation and mixing (Alavi and Caussat, 2005; Formisani, Girimonte and Longo, 2008), in sedimentology this mechanism is known to be involved in geological processes such as mobility of pyroclastic flows (Allen, 1984) and may result in distinctive structures, grain size sorting and density segregation effects within a sediment layer (Papanicolaou and Maxwell, 2006). Two main types of fluidization can be described, according to the origin of the monophase fluid and whether or not the fluidized mass is in translational motion. Figure 20 presents these types. The first type, known as stationary fluidization, is characterized by the granular mass being static in the horizontal direction and only presenting a vertical movement of individual particles relative to each other; according to the regime of fluidization, which will be discussed later, the granular mass thickness can increase with a decrease of particles concentration. Translational fluidization involves horizontal motion of the fluidized mass; while flow-fluidization is not known to occur in nature (although possible to occur), bulk self-fluidization and grain self-fluidization are known to describe debris transport in avalanches and magma fragments flows, respectively. In this section only stationary fluidization will be covered.

![Figure 20: Main classes of fluidization. Taken from Allen (1984).](image)

Figure 21 presents some regimes of stationary fluidization which are dependent on the properties of the granular soil and fluid involved in the process and on the geometrical boundary conditions; the figure presented by Allen will be useful since it considers the fluid to arise in a vertical tube and the granular bed supported on a porous plate, similar to what will be done for the sample preparation process; while the figures consider a granular material with an uniform grain size, which is not entirely similar to the conditions for sands, they will allow for describing the process. Just as in the previous figure, V will represent the superficial
velocity of the granular bed and $V_{mf}$ the superficial velocity needed to effect fluidization. The fixed-bed regime is characterized for $V < V_{mf}$; the particles will remain supported at grain contacts but the apparent immersed bed weight is less than the true weight.

When $V = V_{mf}$, incipient fluidization occurs and the bed becomes for the first time fluid-like. A slight expansion of the bed will have occurred in order for fluidization to occur and the particles immersed weight is just balanced by the total fluid drag. But there is a fractional volumetric grain concentration, $C_{mf}$, which permits incipient fluidization with no bed expansion: this is closely comparable with the concentration in loosest possible random packing. As the value of the superficial velocity increases and exceeds that required for fluidization, harmonic bed expansion will occur with a decrease in grain concentration along the vertical tube; naturally the concentration will eventually become higher at the top of the tube. This last regime is known as particulate fluidization and is normal in fluidization processes where liquids are used.

![Figure 21: Stationary Fluidization Regimes. Taken from Allen (1984).](image)

Aggregative fluidization regimes, also shown in Figure 21, are normal from gas-solids systems, but Allen (1984) warns that gas-solid systems may fluidize in a particular way and liquid-solid systems that can present aggregative fluidization. He concludes that aggregative fluidization is favoured by a large solid density, a low fluid viscosity and large solids diameter with no intergranular cohesion. The reasons for particulate behaviour in some systems and aggregative behaviour in others could be connected to: instability of fluid beds to small disturbances which affects the particle concentration and the properties of bubbles and their stability; the specifics of these reasons will not be discussed in this work but should be considered at a further stage of research since it may result in a material with a fluidization-induced anisotropy (due to gravity and vertical upward pore fluid flow). Here the author refers to “fluidization-induced” anisotropy to any non-uniformities within the sample produced by fluid pockets or channeling, as shown in the figure above.
As described earlier, fluidization process is accompanied by a change in packing, reflected by an increase in the granular bed height and a decrease in grain concentration. Figure 22 is taken from Allen (1984), in which the upward flow of fluid through an initially dense granular, cohesionless bed of height \( h \) supported on a porous plate in a vertical tube with pressure tappings is considered. As the superficial velocity increases, the pressure drop across the bed will increase linearly until it reaches certain height, defined by point B on the right graph, and then drop to C; this sharp change between B and C comes from the unlocking of the densely packed grains. It can be said that at point C incipient fluidization regime has been reached and the pressure drop will remain constant even during particulate fluidization, provided that no grains are swept from the tube. When the superficial velocity reduces to zero, the pressure drop will decrease at a different path (ECD) as when it increased initially; when raising again the superficial velocity the pressure drop will increase in a path DCE. It is clear that hysteresis is present when fluidizing for the first time and then stopping the fluidizing agent flow; however it will not be present when re-fluidizing the granular bed again.

![Figure 22: Pressure Drop as a Function of Superficial Velocity during Granular Bed Fluidization. Taken from Allen (1984).](image)

This hysteresis can be noted on Figure 23, based on a plot presented by McCabe, Smith and Harriott (2001), where the paths represent the variation of the granular bed height with increasing superficial velocity. From points A to B the bed height is still constant but starting from B the bed height increases with increasing velocity. As the velocity is reduced to zero, the bed height follows the path ECD and the bed has a different height; this could be explained by the unlocking of the packed grains and might be the loosest state possible.
The minimum superficial velocity for fluidization depends on the density and shape of the grain particle, initial packing state of the granular material and density and dynamic viscosity of the fluidization agent; the reader can find an extensive elaboration on fluidization on McCabe et al. (2001), where an equation relates the pressure drop on a packed granular bed with the fluid flow, known as the Ergun equation and defined as

$$\frac{\Delta p}{L} = \frac{150\bar{v}\eta (1-n)^2}{\Phi_s^2 D_p^2 n^3} + \frac{1.75\rho_f \bar{v}^2(1-n)}{\Phi_s D_p n^2}; \quad \Phi_s = \frac{6/D_p}{s_p/V_p}, \quad [23]$$

where $\bar{v}$ is the superficial velocity of the fluid, $\eta$ its dynamic viscosity, $\rho_f$ its density, $n$ is the porosity of the granular material, $\Phi_s$ is the sphericity of the grains, defined by the particle diameter $D_p$, its surface area $s_p$ and volume $V_p$. This form of the Ergun equation can be compared with some of the principles of fluid mechanics:

- The first term of Equation [23] is known as the Kozeny-Carman equation, which can be applied to flow through granular beds at Reynolds numbers up to about 1. The Kozeny-Carman equation indicates that, for a given system, the flow (i.e. $\bar{v}$) will be directly proportional to the pressure drop ($\Delta p/L$) and inversely proportional to the fluid’s viscosity. This can be associated to Darcy’s law for liquid flow through porous media.

- The second term of Equation [23] is known as the Burke-Plummer equation, which is an empirical correlation for pressure drops at high Reynolds numbers (>1000) and represents the kinetic energy loss component during flow. Adding this term to the Kozeny-Carman equation is similar to the Darcy-Forchheimer law, where an additional term is included to Darcy’s law in order to include the inertial effects at high flow rates (McCabe et al., 2001; Jambhekar, 2011).

- When the fluid is at rest, the pressure drop is equal to zero and the pore fluid pressure distribution along the granular bed can be described by a hydrostatic condition.

For estimating the minimum velocity needed for reaching incipient fluidization, a form of the Ergun equation (Wen and Yu, 1966a, 1966b) for friction factor in a packed bed, as function of the Reynolds number, is taken from Lowe (1976) and defined by
\[
V_{mf} = -\frac{150k_2\eta}{3.5k_1D\rho_f} + \left(\frac{150k_2\eta}{3.5k_1D\rho_f}\right)^2 + \frac{1}{\frac{1.75k_1\rho_f}{\eta gD}} \right)^{1/2},
\]  

[24]

where \(\rho_p\) and \(\rho_f\) represent the densities of the granular particle and the fluidizing agent, respectively, \(\eta\) the dynamic viscosity of the fluidizing agent, \(g\) the gravitational acceleration, \(D\) the granular particle diameter, and \(k_1\) and \(k_2\) the numerical constants dependent on particle sphericity and particle concentration at incipient fluidization, respectively. Wen and Yu suggest values for \(k_1\) and \(k_2\) of 14 and 11, respectively, for general use; this is adequate for an initial estimation and can later be refined for materials with shapes different from spheres.

Elutriation is the process where particles are separated based on their shape, size and density by flushing a fluid in the direction opposite to sedimentation; in essence it is when the superficial velocity is high enough to become greater than the particle free falling velocity and thus being transported by the fluidizing agent. This velocity, denoted here by \(W_e\), comes from Rubey’s general settling law (Rubey, 1933) and is defined by the following set of equations

\[
W_e = \frac{8a(\rho_p-\rho_f)g}{3c_D\rho_f} \quad [25]
\]

\[
C_D = \frac{24}{Re} + 2, \quad [26]
\]

where the variables \(a\) and \(C_D\) are introduced and represent the particle radius and a non-dimensional drag coefficient, respectively. This drag coefficient can be determined empirically or calculated theoretically; in this case the theoretical expression proposed by Rubey’s (1933) is shown which is dependent on the Reynolds number. Reynolds number is the ratio between inertial and viscous forces and helps to predict similar flow patterns in different fluid flow situations; in this case flow through a packed bed should be considered in determining Reynolds number, but Dwivedi and Upadhyay (1977) state that different equations for determining this value have been used to correlate data for different types of packed and fluidized beds.

The relevance of the minimum fluidization and elutriation velocities is presented in Figure 24, for the case of solid particles with densities similar to that of quartz. A granular bed with a varying grain size in its particles means that for fluidizing the entire granular bed the minimum velocity for fluidization for the coarsest grains should be reached; the risk of this is that the particles of another size might undergo elutriation and thus being washed out of the fluidization column. This could also mean that particle segregation will occur, resulting in a layered bed after fluidization; in particulate fluidization systems segregation will be negligible if the \(V/V_{mf}\) is close to unity and if the settling velocities of the grains differ by less than about two times (Allen, 1984).
3.12. Closing Remarks on this Chapter

This chapter showed the relevance of the static liquefaction mechanism in geotechnical and dredging engineering. From the literature studied, it can be said that static liquefaction:

- Involves a monotonic load increment, either displacement-controlled or load-controlled.
- Applies for saturated, loose soils in undrained boundary conditions or where the drainage rate is low enough to become negligible.
- Is triggered by an increase in excess pore water pressure, due to contraction prevention (i.e. a constant volume is kept), and a reduction of strength and stiffness of the soil. This strength reduction can be noted in an undrained triaxial test when the residual deviator stress is significantly lower than the peak value (in some cases to a point where strength is nearly or gone at all).
• Excess pore water pressure and reduction of strength and stiffness result in: fabric collapse and no contact between grains, particle weight transfer to the pore water, and a material state change to a two-phase “fluid” deforming even under the slightest force applied.

It was possible to observe that the way a saturated sample is prepared and tested can have an influence on the results, which can have an effect on the study of sand behaviour and even on the formulation of constitutive models at the continuum scale.

The first point supporting the above statement is the fact that most liquefaction case histories are based on field data gathered after liquefaction occurred and on the geological conditions on site, not on the underlying mechanics. This calls for a need on a large scale experiments that allows, up to some point, control of the boundary conditions and keep track of the behaviour of an underwater slope at low stress levels before, during and after liquefaction has been triggered.

A second point is related on the method a sample is prepared. For liquefaction studies attention must be paid not only on how a sample preparation process reproduces the natural sediment transport and deposition processes, but also on how full sample saturation is achieved. In drained and undrained triaxial testing different techniques can be used for achieving full sample saturation; however, increasing cell pressure with back pressure may limit the possibility of achieving liquefaction during undrained testing. While flushing CO$_2$ is widely accepted as practice, the sample preparation technique can affect the initial degree of saturation. Here it is important to see that new sample preparation techniques might be developed for very loose, saturated sands that will be subjected to shearing.

A third point is on the different tests that can be used to test a sand under shearing. While it is clear that the hollow cylinder test can reproduce the rotation of the principal stresses, important on the research on liquefaction, at this point it is not commercially used for characterisation and usually the triaxial test is chosen. The fact that triaxial tests are generally used on projects and their results being used on case histories only means that there is still room for research on testing procedures that can accurately characterise sand behaviour when subjected to liquefaction and yet be cost effective.

A fourth point is on the development of contemporary constitutive models at a continuum scale, which are capable to describe sand behaviour on both small and large strain scales. The introduction of the Anisotropic Critical State Theory will provide a new framework on future constitutive models for sands at a continuum scale and numerical models at field-scale which will account for heterogeneity and/or randomness of the sand particles. This will result in attempts to predict the behaviour of sands in conditions similar to what is found on the field, even if the micromechanics will be fairly captured in the first numerical models developed.

Finally, while there is extensive works in fluidization in granular materials, it is not common to hear about this technique being used in geotechnical engineering, specifically in material characterisation. This technique has the potential to reproduce granular samples in its loosest state possible, which can help to develop laboratory protocols which can not only guarantee loose materials and reduce significant variability among different techniques, but also contribute to research in liquefaction phenomena.
Testing Programme

In this chapter, the laboratory testing programme will be described, which aims to characterise the material to be used both in the liquefaction tank and in the element tests. This will include a reflection on the sample preparation method by fluidization (Jager and Molenkamp, 2015a), which will give an idea of the disturbance on the sample during preparation. More information on the tests may be found in Paz Noriega (2015).

4.1. Testing Programme

The different phases of the intended testing programme will be listed below:

1. Material Characterization to identify sand properties. Includes wet and dry sieving, minimum and maximum void ratio, relative density and grain size and angularity identification. Geba sand will be used for this purposes, which is an industrial high silica sand used generally as a component for mortar, plasters and polymer concrete, among others (SIBELCO, 2014). The reason for using this sand is because of its uniformity in terms of grain size, giving some easiness in physical modelling of soils assuming uniform initial conditions, and because this sand will be the one used for the liquefaction tank tests.

2. Standard Drained and Undrained Triaxial Compression Tests. This will allow not only to perform an early estimation of the Geba sand properties but also on the process itself, while providing a benchmark for the triaxial test using a modified setup proposed by Jager and Molenkamp (2015a, 2015b). Samples were prepared using wet pluviation method since, as shown on the literature review, is the closest to resemble the depositional process of underwater sand banks.

3. Sample Preparation by Fluidization. The objective behind preparing the samples using the two different protocols is to observe the variations of the void ratio along the fluidized column and the development of the intergranular stresses before, during and after fluidization (applicable only to the first main protocol type). The effects on the mould removal under different cell fluid conditions can only be seen once the combined load cap and piston arrangement on the modified setup are placed on top of the sample, in order not only to limit the preloading but also to apply vacuum and avoid the sample to fall down.

4. Sample Saturation Control and Undrained Triaxial Testing. At this point it is intended to apply the sample saturation control proposed by Jager and Molenkamp (2015b) accounting for the flexibility of the pore water in determining the bulk flexibility ratio of the solid skeleton and the pore water with air bubbles; the intended method aims at minimization of the potential densification of very
loose sand samples due to the induced intergranular changes needed for the determination and achievement of a saturated sample. Using the modified triaxial test setup, undrained tests will be run at low stress levels; their results would be compared to the results obtained during phase 2. Additionally, the tests would be performed at different stress levels to further analyse its behaviour. The hypothesis at this stage is that liquefaction can be triggered, provided the procedure in phases 3 and 4 have been properly performed.

Due to force majeure, at the time this report was delivered testing was only done up to the early stages of phase 3. However the testing performed gave many insight on the details that are sometimes ignored during testing and on the potential fluidization has in characterisation of sands.

4.2. Overview of Sample Preparation and Control (Jager and Molenkamp, 2015a)

The reproduction of conditions at the liquefaction tank at an element level with approximately uniform conditions, in order to determine material parameters for static liquefaction modelling, was the basis for Jager and Molenkamp (2015a) preliminary study on sample preparation and control before performing an undrained triaxial testing. From this, two main protocols are proposed with different boundary conditions for sample control during preparation.

The proposed protocols for sample preparation are mentioned below, assuming that the bottom of the future sample is located at 1.00 meter beneath the upper part of the underwater embankment in the liquefaction tank; details on the process can be found on Appendix A of this document:

1. A sand column will be fluidized upward to a height of 1.00 m in different flushes so that fines are washed away and the sediment layer after consolidation is of about 0.75 m. For obtaining a final sample height of 0.20 m on the sample, the upper 0.55 m of the sediment column will be removed in several layers (at the preliminary study it was assumed in 5 layers). Figure 25 shows the intended procedure.

2. A sand column will be only fluidized upward to a height of about 0.267 m in different flushes so that fines are washed away and the sample height after sedimentation is somewhere between 0.20 and 0.267 m. This difference between the final sample height and the height after sedimentation will serve to apply a 1-D compression in the sample in order for the intergranular stresses at the bottom of the sample to arrive at the same values as those if the sample were to be prepared according to the first protocol (these intergranular stresses at the bottom of the samples are mentioned in the report as “control points”). Figure 26 shows the procedure intended to be applied.
For both protocols the final sample is inside a latex membrane surrounded by a split-mould designed by Jager and Molenkamp (2015a), built at TU Delft and shown in Figure 27; note that the split-mould can be removed even if the outer-cell wall is placed and the cylindrical gap between the mould and the cell wall is filled with water. This split mould differs from those used generally in triaxial tests sample preparation, shown in Figure 28: the advantage of this new split-mould is that only horizontal movement from the parts is required. As shown in Figure 27, rotation of the upper cogs on both sides of the mould allows for a force transmission along the cog lines which result in the screwing and unscrewing of the bottom bolts that unites both halves; this reduces the amount of vibrations due to mould removal and limiting the densification due to sample control, something that is common in conventional split-moulds.
At the time when the split-mould is removed, the lateral boundary control of the sample can be ensured by either application of water pressure from the bottom part of the cell or by atmospheric air pressure. While the first one limits the sample disturbance, the second boundary allows for a simpler test procedure. In total, four sample preparation and control variants are presented by Jager and Molenkamp (2015a) which produce different degrees of sample disturbance, represented by the intergranular stresses, which will be described in the figures below, where $\sigma_{zz}$ and $\sigma_{xx}$ represent the vertical and horizontal intergranular stresses, respectively; while the figures are based on defined property values defined by the preliminary study, this should illustrate the reader on the effects of the selected protocol on the sample state before the triaxial test.

Figure 29 shows the state of the sample according to the first protocol after sedimentation, excavation and sample control by either lateral hydrostatic pressure or atmospheric pressure. It can be seen that after the
sedimentation has occurred the intergranular stresses on the future sample are dependent on the depth and on the load from the sand layer above. Removal of the upper layer causes a reduction on the vertical intergranular stresses whose profile is still dependent on the depth of the sample; on the other hand, horizontal intergranular stresses due to unloading develop in such a way that are significantly greater than the vertical stresses, causing a passive failure that affects the upper third of the sample. The top cap with the loading piston is lowered to the top of the sample until an increase in the sensors reading is seen and the split-mould is then removed. When removing the split-mould into the cell-water support the horizontal intergranular stresses increase along the sample in such a way that its value is constant along the sample and equal to the horizontal intergranular stress at the bottom of the sample after sedimentation; in the case when the sample is exposed to lateral atmospheric pressure the horizontal intergranular stresses increase but its values are inversely proportional to the sample depth: they are higher at the top of the sample than at the bottom, in which the value is the same as the horizontal intergranular stresses after sedimentation. In both cases the vertical intergranular stresses return to the profile right after sedimentation by control of the drainage on top of the sample and the vertical load cap.

Figure 30 resume the stress paths in terms of isotropic and deviator intergranular stresses on the top, passive transition point and bottom of sample during the described process. It can be seen that, while the stress path at the bottom of the sample is consistent to an unloading and reloading to the conditions after sedimentation, at the transition and top points of the sample the reloading path are less steep than those during unloading, indicating that the deformations in the passive zone between the top and the passive transition point of the sample are irreversible on both scenarios. However, the cell water acts as a support which reduces the

![Figure 29: Vertical and Horizontal Intergranular Stresses Development on Sample for Fluidization, Sedimentation, Excavation and Sample Control by Cell-Water/Cell-Air. Taken from Jager and Molenkamp (2015a).](image-url)
severity of the irreversible deformations in this upper third of the sample, in comparison to the condition where the sample is exposed to lateral air pressure.

Figure 30: Intergranular Stress Paths as Function of Isotropic and Deviator Stresses on Sample Fluidization, Sedimentation, Excavation and Sample Control by Cell-Water/Cell-Air. Taken from Jager and Molenkamp (2015a).

Figure 31 shows the state of the sample according to the second protocol after sedimentation, excavation and sample control by either lateral hydrostatic pressure or atmospheric pressure. It is clear that the intergranular stress levels reached during sedimentation is lower than the values achieved on the first protocol, which is explained by the absence of the upper sand layer. A 1-D compression is applied so that the sample reaches a final height of 0.20 m while achieving the horizontal and vertical intergranular stresses profiles along the sample which would be the same as those achieved after sedimentation under the first protocol. Under any of the sample control options the horizontal and vertical intergranular stresses profiles will be the same as those under the first protocol.
By observing the intergranular stress paths in Figure 32 after the 1-D compression it is clear that, while the bottom of the sample shows no variations under any of the sample control options, the top of the sample will involve a downward direction vector during the removal of the split-mould, which is characteristic of induced contraction. This effect will result in irreversible deformations, which are stronger when the sample becomes supported by atmospheric pressure.
Table 1 compares each of the four variants for the sample preparation protocol, based on the figures shown before and on the analysis of the sample preparation and control protocols which can be found on Appendix A.
Table 1: Variations of Fluidization Sample Preparation and Control Protocols

<table>
<thead>
<tr>
<th>SAMPLE PREPARATION AND CONTROL PROTOCOL</th>
<th>HIGHLIGHTS</th>
</tr>
</thead>
</table>
| **Fluidization, Sedimentation, Excavation and Lateral Hydrostatic Pressure at Mould Removal** | • Final sample disturbance begins during removal of upper sediment layers.  
• Upper 1/3 of sample is disturbed due to passive failure caused by horizontal intergranular stresses during unloading, which will not participate in irreversible deformation and volumetric contraction.  
• Depth of passive layer on final sample depends on the size of the removed upper layer.  
• Disturbing effect of excavation and sample control at the bottom of the sample remains minimum.  
• Lateral water pressure regulates additional sample disturbance at the passive zone. |
| **Fluidization, Sedimentation, Excavation and Lateral Atmospheric Pressure at Mould Removal** | • Final sample disturbance begins during removal of upper sediment layers.  
• Upper 1/3 of sample is significantly disturbed due to passive failure caused by horizontal intergranular stresses during unloading, which will not participate in irreversible deformation and volumetric contraction.  
• Depth of passive layer on final sample depends on the size of the removed upper layer.  
• Disturbing effect of excavation and sample control at the bottom of the sample remains minimum.  
• Atmospheric pressure not enough to limit additional sample disturbance at passive zone. Intergranular isotropic and deviator stresses at the top and passive transition point located at a farther location from those at the bottom of the sample, when compared to the sample stress conditions under lateral hydrostatic pressure. |
| **Fluidization, Sedimentation, 1-D Compression and Lateral Hydrostatic Pressure at Mould Removal** | • Assuming stress conditions after 1-D compression as the starting point, sample disturbance starts at split-mould removal.  
• Upper 1/2 of sample is disturbed due to contractive intergranular stress path after 1-D compression and transition from mould to lateral water pressure. Upper half of sample will not participate in irreversible deformation and volumetric contraction.  
• Magnitude of 1-D Compression to be experimentally determined, in order to reach at the "control point" intergranular stresses at bottom of the sample (similar to the stress conditions of the protocol above).  
• No disturbing effects during sample control at the bottom of the sample.  
• Lateral water pressure regulates sample disturbance during split-mould removal. |
| **Fluidization, Sedimentation, 1-D Compression and Lateral Atmospheric Pressure at Mould Removal** | • Assuming stress conditions after 1-D compression as the starting point, sample disturbance starts at split-mould removal.  
• Upper 1/2 of sample is significantly disturbed due to contractive intergranular stress path after 1-D compression and transition from |
mould to lateral atmospheric pressure. Upper part of sample will not participate in irreversible deformation and volumetric contraction

- Magnitude of 1-D Compression to be experimentally determined, in order to reach at the "control point" intergranular stresses at bottom of the sample (similar to the stress conditions of the protocol above).
- No disturbing effects during sample control at the bottom of the sample.
- Atmospheric pressure not enough to limit additional sample disturbance at passive zone. Intergranular isotropic and deviator stresses at the top and passive transition point located at a farther location from those at the bottom of the sample, when compared to the sample stress conditions under lateral hydrostatic pressure.

It can be seen that, while the first two variants are characterized by long stress paths at the top and passive transition points of the samples, the disturbance is limited only to 1/3 of the final sample height. In contrast, the last two variants present shorter stress paths at the top of the sample but the disturbance during sample preparation extends to half of the sample. In addition, the use of a cell-water confinement during the removal of the split-mould seems to reduce the disturbance on the sample. Just on the basis of the sample preparation and control before the triaxial test, the first variant may be the better option for the current study. However, the third and fourth variants presented in Table 1 may be quicker and cheaper, while inducing smaller unloading at the top of the sample due to removal of the mould. In any case, equipment calibration and human errors during the preparation should be accounted for. In addition, at this stage of the research project no undrained triaxial tests have been performed, thus not allowing a benchmark for comparison for tests at low stress levels in terms of repeatability and when compared to normal triaxial testing results.

Nevertheless, it can already be concluded that for the considered extremely small intergranular stress range both protocols involve far less disturbance than other classical counterparts without cell-water, thus only with cell-air.
Results and Discussion

In this chapter the relevant results obtained from the research protocol described in Chapter 4 will be presented. A detailed description on the test procedures and results obtained can be found in Paz Noriega (2015).

5.1. Material Characterization: Grain Size Distribution

Wet and dry sieving tests were performed according to the standard BS 1377: Part 2: 1990 (now EN 1997-2: 2007), in which grain size distribution curves for the sand range were obtained. Figure 33 and 34, taken from Paz Noriega, show the grain size distribution curves for the dry and wet sieving tests, respectively. For the curves shown in Figure 34, additional sieves were included in order to obtain more plotting points for the curves; thus the results from the previous samples were not included.

![Grain Size Distribution Geba Sand using Dry Sieving. Taken from Paz Noriega (2015).](image-url)
From both curves the grading characteristics results were obtained, shown in Table 2; first, the maximum grain size of the smallest 10%, 30%, 50% and 60% of the sample were estimated from the grading curves and represented by $D_{10}$, $D_{30}$, $D_{50}$ and $D_{60}$, respectively. $D_{10}$, or the effective size, can be used to estimate the hydraulic conductivity and permeability of a soil on a first basis, as it will be shown on another section when using the Hazen (1911) formula. $D_{50}$ is known as the average particle size and, while it only indicates the grain diameter for which half of the sample weight is smaller and the other half is larger, it can also be used to estimate the hydraulic conductivity and permeability of a soil by using a variation of the Kozeny-Carman equation (Bear, 1972; Fitts, 2013), which will be shown on another section. As a second step, the coefficients of uniformity and curvature of the grain size distribution curve, denoted by $C_u$ and $C_c$, respectively, are estimated by using the following equations:

$$C_u = \frac{D_{60}}{D_{10}}$$  \[27\]

$$C_c = \frac{(D_{30})^2}{D_{60}D_{10}}$$  \[28\]

It is evident that the differences in results obey not only to the inclusion of additional sieves, but also on the fact that the percentage of material passing the sieves becomes greater, as seen at the decrease of the average particle size $D_{50}$. This new value of $D_{50}$ suggests that the particles still to be retained will be fine, even suggesting presence of silt and clay-size particles. From both tests, it can be concluded that the sand is uniformly graded in terms of its grain size distribution and well graded within its grain size range; this indicates that particle segregation during the sample preparation process by fluidization will be relatively low.
**Table 2: Grading Characteristics Results between Dry and Wet Sieving Tests. Taken from Paz Noriega (2015).**

<table>
<thead>
<tr>
<th>GRADING CHARACTERISTICS</th>
<th>DRY SIEVING</th>
<th>WET SIEVING</th>
</tr>
</thead>
<tbody>
<tr>
<td>$D_{10}$ ($\mu m$)</td>
<td>75.13</td>
<td>88.22</td>
</tr>
<tr>
<td>$D_{30}$ ($\mu m$)</td>
<td>106.45</td>
<td>113.23</td>
</tr>
<tr>
<td>$D_{50}$ ($\mu m$)</td>
<td>138.07</td>
<td>130.29</td>
</tr>
<tr>
<td>$D_{60}$ ($\mu m$)</td>
<td>139.22</td>
<td>139.22</td>
</tr>
<tr>
<td>Cu (-)</td>
<td>1.85</td>
<td>1.58</td>
</tr>
<tr>
<td>Cc (-)</td>
<td>1.08</td>
<td>1.04</td>
</tr>
</tbody>
</table>

Two things are important to note from the results above:

1. The inclusion of intermediate sieves at a given range allows for more data points which can be used to plot the grain size distribution curve using calculation software like Microsoft Excel. While this can be useful for soils in which a certain grain size range can be estimated, such as the fine sand used in this project, for soils with a major grain size range this approach can become unpractical.

2. For fine sands, or sands in which fines can be expected, the wet sieving test must be preferred over the dry sieving test. The use of a deflocculant agent on the wet sieving test breaks some bonds formed between sand and silty/clayey particles which form particle conglomerates of greater size. While in the case studied the fine sand showed an average percentage of silty/clayey particles of less than 2%, which didn’t affect significantly the values of the coefficients of uniformity and curvature, for soils with a major percentage of fines this issue becomes more relevant.

### 5.2. Material Characterization: Density

Three types of density tests were performed. The first one consisted in determining the average particle density and the specific gravity of the sand particles by means of a gas pycnometer machine. The value obtained for the specific gravity was of 2.6557, which is consistent to the estimate value for quartz ($G_s=2.65$) (Das, 2001).

The second and third tests were index tests to determine the minimum and maximum void ratios for the Geba sand, using the Japanese Geotechnical Society Method (JGS, 1996), described in Appendix B of this document. The tests were performed twice for the sand with fines and the sand without fines; in this case the sand with fines is that in which the dry sieving method was used, while the sand without fines is that obtained after the wet sieving, in which the fines are washed away. On one of the two test series mentioned, the sand was deposited while moving the deposition medium (i.e. a funnel) both vertically and laterally, the results being compared with those obtained when performing the JGS Method. Different results for the minimum and maximum void ratio were obtained, as shown in Table 3.
Table 3: Minimum and Maximum Void Ratio Test Results for Geba Sand with Fines and without Fines. Values taken from Paz Noriega (2015).

<table>
<thead>
<tr>
<th>GEBA SAND</th>
<th>Vertical and Lateral Displacement during Deposition</th>
<th>Vertical Displacement Only (JGS Method)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( e_{\text{min}} ) (-) ( e_{\text{max}} ) (-) ( e_{\text{max}} - e_{\text{min}} )</td>
<td>( e_{\text{min}} ) (-) ( e_{\text{max}} ) (-) ( e_{\text{max}} - e_{\text{min}} )</td>
</tr>
<tr>
<td>With Fines</td>
<td>0.66 0.91 0.25</td>
<td>0.70 1.03 0.33</td>
</tr>
<tr>
<td>Without Fines</td>
<td>0.66 0.82 0.16</td>
<td>0.67 0.95 0.28</td>
</tr>
</tbody>
</table>

The results in Table 3 show:

1. It is possible to obtain a lower value of void ratio when the deposition funnel is displaced laterally and vertically, instead of only raising the funnel as prescribed by the JGS Method. While in the JGS Method the kinetic energy from the falling sand is applied only at the central axis of the cylindrical mould, resulting in a denser material at the centre of the specimen, the other approach allows for a more-even densification of the specimen.

2. Taking into reference the results of the minimum and maximum void ratios tests based on the JGS Method, there is a significant reduction in the maximum void ratio, while a smaller reduction within the minimum void ratio (the values between the sand with fines and that without fines are close to each other, in comparison to those for the maximum void ratio). This reduction of the maximum void ratio after removing the fines during wet sieving may seem contradictory at first hand, since the washout would result in an increase in void space for a constant sand volume. However, the use of a deflocculant agent during wet sieving eliminates any particle conglomerates formed by the fines, resulting in finer sand particles which can be rearranged into a tight packing; in addition, the removal of fines will represent an available void space which can be occupied by sand particles, resulting in a denser material.

3. Range of values between minimum and maximum void ratios obtained from the JGS Method is consistent to what is found in literature for similar sands (Youd, 1973; Cubrinovski and Ishihara, 2002).
5.3. **Material Characterization: Grain Shape**

A sample of sand was analysed through an electronic microscope, in order to determine the particle’s angularity, general shape and surface characteristics; an image can be found on Figure 35.

![Figure 35: Geba sand microscope image. White line in upper left corner represents a length of 200 µm. Taken from Paz Noriega (2015).](image)

From the image shown, and from the data obtained by the electronic microscope (which can be found in the original work), it can be seen that the particles possess a general angular roundness, with shapes ranging from elongated to cubic. This is consistent to what can be found on a sand mined from a quarry and not to sands being transported by water, whose round shape is given by the currents.
5.4. **Standard Triaxial Compression Tests**

A series of standard drained and undrained triaxial compression tests were performed according to the British Standard BS 1377: Part 8: 1990, with Geba sand samples prepared by wet pluviation; this sample preparation method was selected in accordance to the findings gathered from literature. The results of these tests at low stress levels would be compared to those which will be obtained with the modified triaxial testing setup proposed by Jager and Molenkamp, and the magnitude of these stress levels in the element tests will be similar to those occurring in the slopes to be prepared at the liquefaction tank. A list of relevant parameters to be measured and controlled is shown on Table 4.

**Table 4: Variations of Fluidization Sample Preparation and Control Protocols**

<table>
<thead>
<tr>
<th>PARAMETER/MEASURE</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>$D_0$</td>
<td>Sample Diameter before Saturation Step</td>
</tr>
<tr>
<td>$H_0$</td>
<td>Sample Height before Saturation Step</td>
</tr>
<tr>
<td>$V_0$</td>
<td>Sample Volume before Saturation Step</td>
</tr>
<tr>
<td>$\rho$</td>
<td>Dry Density of Sand in Sample</td>
</tr>
<tr>
<td>B-Value</td>
<td>Skempton’s (1954) Pore Pressure Parameter B</td>
</tr>
<tr>
<td>$A_c$</td>
<td>Sample Cross-Sectional Area after Consolidation</td>
</tr>
<tr>
<td>$H_c$</td>
<td>Sample Height after Consolidation</td>
</tr>
<tr>
<td>$V_c$</td>
<td>Sample Volume after Consolidation</td>
</tr>
<tr>
<td>$e$</td>
<td>Void Ratio</td>
</tr>
<tr>
<td>R.D.</td>
<td>Relative Density of Sample; R.D.=$(e_{\text{max}}-e)/(e_{\text{max}}-e_{\text{min}})$</td>
</tr>
<tr>
<td>$\sigma_{\text{mb}}$</td>
<td>Stress Correction Value due to Membrane Effects (BS 1377: Part 8: 1990)</td>
</tr>
<tr>
<td>$\sigma_3$</td>
<td>Cell Pressure</td>
</tr>
<tr>
<td>$P_b$</td>
<td>Back Pressure</td>
</tr>
<tr>
<td>$\sigma_3'$</td>
<td>Effective Consolidation Pressure (Effective Confining Pressure before Triaxial Loading); $\sigma_3'=\sigma_3- P_b$</td>
</tr>
</tbody>
</table>

Three samples were prepared for performing drained triaxial compression tests, whose values can be seen in Table 5. The value of the specific gravity of the grains will be the one obtained on the pycnometer test (see Section 5.2).
Table 5: Input parameters for Drained Triaxial Compression Test. Taken from Paz Noriega (2015).

<table>
<thead>
<tr>
<th>PARAMETERS/MEASURES</th>
<th>NTC-01</th>
<th>NTC-02</th>
<th>NTC-03</th>
</tr>
</thead>
<tbody>
<tr>
<td>D₀ (mm)</td>
<td>49.0</td>
<td>49.2</td>
<td>48.9</td>
</tr>
<tr>
<td>H₀ (mm)</td>
<td>109.8</td>
<td>110.5</td>
<td>108.3</td>
</tr>
<tr>
<td>V₀ (mm³)</td>
<td>207054.36</td>
<td>210078.87</td>
<td>203393.03</td>
</tr>
<tr>
<td>ρ (kg/m³)</td>
<td>1412.33</td>
<td>1444.27</td>
<td>1424.58</td>
</tr>
<tr>
<td>B-Value (-)</td>
<td>0.97</td>
<td>0.95</td>
<td>0.91</td>
</tr>
<tr>
<td>Aₑ (mm²)</td>
<td>1880.94</td>
<td>1898.81</td>
<td>1872.08</td>
</tr>
<tr>
<td>Hₑ (mm)</td>
<td>109.66</td>
<td>110.43</td>
<td>108.13</td>
</tr>
<tr>
<td>Vₑ (mm³)</td>
<td>206264.36</td>
<td>209688.87</td>
<td>202423.03</td>
</tr>
<tr>
<td>e (-)</td>
<td>0.87</td>
<td>0.84</td>
<td>0.86</td>
</tr>
<tr>
<td>R.D. (-)</td>
<td>0.48</td>
<td>0.58</td>
<td>0.52</td>
</tr>
<tr>
<td>σₘₙₘ (kPa)</td>
<td>1.36</td>
<td>1.35</td>
<td>1.36</td>
</tr>
<tr>
<td>σ₃ (kPa)</td>
<td>465</td>
<td>370</td>
<td>440</td>
</tr>
<tr>
<td>Pₘ (kPa)</td>
<td>415</td>
<td>345</td>
<td>360</td>
</tr>
<tr>
<td>σ₃’ (kPa)</td>
<td>50</td>
<td>25</td>
<td>80</td>
</tr>
</tbody>
</table>

As found in literature (Skempton, 1954) and defined on testing standards (BSI, 1990b), a minimum value of 0.95 is generally accepted for sands, meaning that at least 95% of the cell pressure will be transmitted to the pores. From the samples listed above, the B-Values for the samples to be tested at confining pressures of 50 and 25 kPa are the only ones “acceptable” for a “saturated” sample; even though the drained compression tests were done on all three samples, only the results for “saturated” samples will be shown (a view of the complete set of results is presented by Paz Noriega (2015)). From Figures 36 and 37, it can be seen that for drained tests at low stress levels:

1. The peak deviator stresses are relatively low, when compared to higher confining pressures, indicating that the peak deviator stresses is proportional to the magnitude of the confining pressure applied.

2. The magnitude of the volumetric strains in the sample, at a certain axial strain, is inversely proportional to the confining pressure. This means that samples at low stress levels are prone to significant volumetric changes due to a low confinement. For the case of the test at a confining pressure of 25 kPa, as seen in Figure 40, it can be seen that the sample is highly dilative with respect to that at a confining pressure of 50 kPa.
Figure 36: Axial Strain vs. Corrected Effective Deviator Stresses for Drained Triaxial Testing. Adapted from Paz Noriega (2015).

Figure 37: Axial Strain vs. Volumetric Strains for Drained Triaxial Testing. Adapted from Paz Noriega (2015).

Figure 38 shows the Mohr circles (Terzaghi, 1943) for the samples at different confining pressures, describing the relation between effective normal and shear stresses (understanding normal stresses as those acting in the principal stress directions, defined by the axisymmetric geometry of the sample); in this case the maximum deviator stress failure criterion is considered. The calculated angles of internal friction for confining pressures of 25 kPa and 50 kPa are 42.86° and 40.81°, respectively; there are two ways to interpret these results:

- Assuming a linear Mohr-Coulomb failure criterion, drawing a line tangent to both circles would intercept the vertical axis at a point different from the origin; in this case, the value of the intercept
is estimated to be between 5 and 10 kPa. This suggests that the tested material has some degree of cohesion, provided by the fines present on the sample at the time of testing.

- Considering that the percentage of fines in the sand is approximately less than 2% and that the contribution of the fines on the cohesive strength can be neglected, it can be assumed that the change in friction angle with confining stresses is explained by a curvilinear failure criterion that starts at the origin and is tangent to all circles.

The only way to conclude what applies in reality is to perform at least a third drained test, at a lower confining pressure, in order to capture the Mohr-Coulomb failure criterion line for the sand under study.

![Mohr Circles for Maximum Effective Stress Levels for Drained Triaxial Testing. Adapted from Paz Noriega (2015).](image)

For the undrained triaxial compression tests four different samples were prepared, as shown in Table 6. From the data presented, samples NTC-06 and NTC-07 are suitable for the analysis given that their B-Value is higher or equal to 0.95. For NTC-04 and NTC-05 the B-Value is lower than 0.90, meaning that the samples were partially saturated and thus not suitable for a saturated material description. Therefore only the results for NTC-06 and NTC-07 for undrained triaxial compression (confining pressures of 25 kPa and 100 kPa, respectively) will be shown in this section.
Table 6: Input parameters for Undrained Triaxial Compression Test. Taken from Paz Noriega (2015).

<table>
<thead>
<tr>
<th>PARAMETERS/MEASURES</th>
<th>NTC-04</th>
<th>NTC-05</th>
<th>NTC-06</th>
<th>NTC-07</th>
</tr>
</thead>
<tbody>
<tr>
<td>$D_0$ (mm)</td>
<td>49.2</td>
<td>47.3</td>
<td>49.2</td>
<td>49.3</td>
</tr>
<tr>
<td>$H_0$ (mm)</td>
<td>111.5</td>
<td>109.5</td>
<td>111</td>
<td>105.8</td>
</tr>
<tr>
<td>$V_0$ (mm³)</td>
<td>211980.03</td>
<td>192409.40</td>
<td>211029.45</td>
<td>201961.87</td>
</tr>
<tr>
<td>$\rho$ (kg/m³)</td>
<td>1454.15</td>
<td>1530.17</td>
<td>1456.57</td>
<td>1486.82</td>
</tr>
<tr>
<td>B-Value (-)</td>
<td>0.89</td>
<td>0.86</td>
<td>0.95</td>
<td>0.96</td>
</tr>
<tr>
<td>$A_c$ (mm²)</td>
<td>1897.04</td>
<td>1754.61</td>
<td>1900.33</td>
<td>1902.29</td>
</tr>
<tr>
<td>$H_c$ (mm)</td>
<td>111.38</td>
<td>109.42</td>
<td>110.98</td>
<td>105.62</td>
</tr>
<tr>
<td>$V_c$ (mm³)</td>
<td>221290.03</td>
<td>191989.40</td>
<td>210319.45</td>
<td>200911.87</td>
</tr>
<tr>
<td>$\varepsilon$ (-)</td>
<td>0.91</td>
<td>0.73</td>
<td>0.83</td>
<td>0.78</td>
</tr>
<tr>
<td>R. D. (-)</td>
<td>0.36</td>
<td>0.91</td>
<td>0.61</td>
<td>0.76</td>
</tr>
<tr>
<td>$\sigma_{mb}$ (kPa)</td>
<td>1.35</td>
<td>1.41</td>
<td>1.35</td>
<td>1.35</td>
</tr>
<tr>
<td>$\sigma_3$ (kPa)</td>
<td>445</td>
<td>445</td>
<td>470</td>
<td>445</td>
</tr>
<tr>
<td>$P_b$ (kPa)</td>
<td>395</td>
<td>345</td>
<td>445</td>
<td>345</td>
</tr>
<tr>
<td>$\sigma_3'$ (kPa)</td>
<td>50</td>
<td>100</td>
<td>25</td>
<td>100</td>
</tr>
</tbody>
</table>

Figure 39 shows, once again, the relation between confinement pressure and peak deviator stresses, even though the difference is not as marked as compared to the drained case shown in Figure 36: the reason behind this is difference can be attributed not only to the confining pressure but also to the density of the sample; it can be seen that there is a reduction of the peak deviator stresses due to excess pore pressure generation, shown in Figure 40, which seems to result in a residual deviator stress lower than the peak deviator stress.

![Figure 39: Axial Strain vs. Corrected Effective Deviator Stresses for Undrained Triaxial Testing. Adapted from Paz Noriega (2015).](image-url)
As Figure 40 suggests, the magnitude of the peak excess pore pressure generated during loading is inversely proportional to the confining pressure applied, which supports the idea behind liquefaction in which lower confining pressures and undrained boundary conditions increase the pore water pressure and decreases the effective stresses in the soil skeleton. Whether or not liquefaction is achieved can be observed with the stress paths, which will be done on another figure.

![Figure 40: Axial Strain vs. Excess Pore Pressures for Undrained Triaxial Testing. Adapted from Paz Noriega (2015).](image)

Figure 41 shows the Mohr circles for the effective stresses, in which the failure criterion of maximum effective deviator stresses was chosen per test, where the mobilized internal friction at the chosen failure criteria can be found in Table 7; the table shows not only the internal friction angle at peak effective deviator stresses, but also the residual value at the end of the test. The results from the undrained condition differ with those from the drained condition tests in the sense that there is not much of a difference between the circles at confining pressures of 25 kPa and of 100 kPa, even when accounting differences in sample densities. There is, however, an evident relation between the density and the peak deviator stress for undrained triaxial testing: a denser material results in an increase in peak deviator stresses (Paz Noriega, 2015). At the end of the testing, the formation of a shear band is seen, as shown in Figure 42.

The reasons of variations between the peak and residual friction angle, as shown in Figure 7, is related to the reference stress level where the parameter is taken; the reduction on the friction angle is caused by the increase of the pore pressures, resulting in the reduction of the effective deviator stress. The difference in the values for friction angles on both drained and undrained tests, and the unrealistic value of cohesion when drawing a tangent line between the two circles in Figure 41, show that the undrained test is not reliable for estimating the shear strength parameters using effective stresses, something that Verruijt (2012) admits; Karstunen (2012) notes that the drained loading analysis is appropriate for soils with high permeability, which would be the case when comparing the permeability of sands with respect to silts and clays. On the other hand, Verruijt notes that the undrained analysis is useful in engineering practice, in situations where very little or no drainage will occur; by plotting the Mohr circles in total stresses for an undrained analysis it is possible to estimate the undrained shear strength.
Figure 41: Mohr Circles for Maximum Effective Deviator Stress Levels for Undrained Triaxial Testing. Adapted from Paz Noriega (2015).

Table 7: Mobilized Effective Peak and Residual Friction Angle at Different Confining Stress Levels. Adapted from Paz Noriega (2015).

<table>
<thead>
<tr>
<th>FRICTION ANGLE</th>
<th>25 kPa</th>
<th>100 kPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak Friction Angle</td>
<td>35.70°</td>
<td>34.58°</td>
</tr>
<tr>
<td>Residual Friction Angle</td>
<td>31.23°</td>
<td>29.84°</td>
</tr>
</tbody>
</table>

Figure 42: Shear Band after Undrained Triaxial Compression Test.
Figures 43 shows the stress paths as a function of effective isotropic and deviator stress invariants, where it can be seen that the samples present a rather contractive behaviour at the beginning but then become dilative. As such, no stress paths get close to the origin which would have evidenced a liquefaction state of the samples.

However, the resulting curves in Figures 40 and 43 and the picture in Figure 42 demonstrate the undrained hardening until the undrained peak strengths are reached, and the development of shear bands during post-peak softening. From literature it is known that often the dilation-induced pore water suction, evidenced by the difference between the total and effective stress paths, experiences cavitation, which allows the subsequent sample behaviour to become “like drained”, allowing the generation of dilative shear bands, like occurring in drained compression tests.

There are several reasons affecting the development of liquefaction during the performed undrained triaxial tests; some of them include:

1. The degree of saturation, related to the Skempton’s B-Value, is not high enough to completely transfer any load increase to the pore water, thus affecting the generation of instability in the sample. (Skempton, 1954; Lindenberg and Koning, 1981; Paz Noriega, 2015). This will be relevant during the experimental study of the sample saturation process recommended by Jager and Molenkamp (2015b).
2. The sample preparation process during the testing may have produced air bubbles when pluviating the sand into the water, thus affecting the final degree of saturation of the sample. This will be relevant when thinking about how the fluidization method saturates the sample during the process.
3. Selected method for increasing the degree of saturation, related to the Skempton’s B-Value. The isotropic increase of cell water and back pressures subjected the samples at high stress levels, contributing to the undrained resistance of the sample. For standard testing other methods can
apply, where possible, such as saturation by carbon dioxide injection. This will be relevant during the experimental study of the sample saturation process recommended by Jager and Molenkamp.

4. Effects of membrane penetration on the sample behaviour (Molenkamp and Luger, 1981; Baldi and Nova, 1984; Noor, Nyuin and Derahan, 2012; Raghunandan, Sharma and Pradhan, 2015). The rubber membrane deforms into the pores between the grains due to membrane loading by the cell pressure, pore pressure or via rigid platens; the local deformation of the membrane under the aforementioned forces causes a global displacement of the membrane with respect to the granular particles at the circumference of the sample, being dependant on the mean grain size of the soil ($D_{50}$), the effective lateral stress and the sample geometry. Some reports on drained loading and unloading tests, and on undrained triaxial compression tests, of medium loose, fine sand, membrane penetration effects can increase the deviator stress levels to 20%-40% and to 50%, respectively (Molenkamp and Luger, 1981; Kiekbusch, 1987). This will become relevant for the modified triaxial test preparation protocol involving fluidization.

5. Densification during sample preparation and handling. During sample preparation and handling, while the vacuum via the back pressure line is not applied, any vibrations on the sample cause its densification, requiring for more sand material to be placed (resulting in an increase of mass over a given volume).

5.5. Fluidization Tests for Sample Preparation via Fluidization, Sedimentation and Excavation

Two batches of sand were fluidized in order to test the current sample preparation process setup. The setup for fluidization can be seen in Figure 44, where the sand is flushed upwards at a discharge rate that can reduce the vertical effective stresses to zero and liquidize the sand column. The manometers measure any pressure variations along the column during the fluidization process; these readings will allow to calculate the hydraulic gradient on the fluidizing sand column and compare it with the critical hydraulic gradient, defined by Verruijt (2012) as

$$i_{cr} = \frac{\rho_{sat} - \rho_w}{\rho_w} \quad \text{[29]}$$

$$\rho_{sat} = (1 - n_{fluidized}) \rho_s + n_{fluidized} \rho_w. \quad \text{[30]}$$

where $\rho_{sat}$ and $\rho_w$ represent the densities of the saturated sand column during fluidization and water, respectively. The principle behind this gradient is based on the fluid head during upward flow, which becomes high enough to reduce the effective stresses to zero; this becomes a critical situation since no forces are transmitted between the particles. Subsequent increase of the upward flow will result in the soil mass being lifted. The saturated density is dependent on the porosity of the fluidized column, which it is assumed at this point to be uniform along the column. Even though visual inspection of the bed expansion is a qualitative assessment of whether fluidization occurred, based on the previous statements, if the hydraulic gradient between two manometers is greater or equal than the critical gradient, then it is said that fluidization has been reached.

At this point it will be assumed that the porosity along the column is constant, which will allow its estimation based on the amount of sand in the column and its height during fluidization; this will help to evaluate fluidization as a first estimate. On another section, a detailed analysis based on equilibrium of vertical
stresses will be presented, which will account for the relation between the hydraulic gradient due to upward flow and the vertical stresses between the sand grains.

For the first batch, a total mass of 8.23 kg was used in order to reach a sediment layer of about 75 cm, which was later fluidized at a discharge rate of 0.47 L/min. Table 8 shows how the critical gradient is estimated, based on the following steps:

• After the granular bed expansion has stopped, its total height is measured and represented by the variable \( H_{\text{fluidized}} \).
• With \( H_{\text{fluidized}} \) known, the total volume of the fluidized granular bed, \( V_{\text{fluidized}} \), is calculated, using the internal diameter of the fluidization column as part of the calculations.
• The volume of sand present in the column, \( V_s \), is estimated based on its weight and the density of the particles, previously estimated on another section.
• The void ratio and the porosity is estimated by using the following relations
  \[
  e_{\text{fluidized}} = \frac{V_{\text{fluidized}} - V_s}{V_{\text{fluidized}}} \quad \text{and} \quad n_{\text{fluidized}} = \frac{e_{\text{fluidized}}}{1 + e_{\text{fluidized}}} \tag{31}
  \]
• Equations [29] and [30] are used to estimate the critical hydraulic gradient by Verruijt (2012).
The manometer readings were registered and tabulated as shown in Table 9; each manometer was assigned a number, starting from the lowermost manometer in the fluidization column. The difference in readings between two manometers was divided by the vertical distance between them (in this case, 15 cm), making sure consistent units were used. From the gradient values, denoted by $i$, it can be seen that the entire column has been fluidized.

### Table 8: Determination of Critical Hydraulic Gradient during Fluidization of Batch #1. Taken from Paz Noriega (2015).

<table>
<thead>
<tr>
<th>PARAMETERS/MEASURES</th>
<th>BATCH #1</th>
</tr>
</thead>
<tbody>
<tr>
<td>$H_{\text{fluidized}}$ (cm)</td>
<td>104.50</td>
</tr>
<tr>
<td>$V_{\text{fluidized}}$ (cm$^3$)</td>
<td>8207.41</td>
</tr>
<tr>
<td>$V_s$ (cm$^3$)</td>
<td>3098.99</td>
</tr>
<tr>
<td>$\epsilon_{\text{fluidized}}$ (-)</td>
<td>1.65</td>
</tr>
<tr>
<td>$n_{\text{fluidized}}$ (-)</td>
<td>0.62</td>
</tr>
<tr>
<td>$\rho_{\text{sat}}$ (kg/m$^3$)</td>
<td>1629.17</td>
</tr>
<tr>
<td>$i_{cr}$ (-)</td>
<td>0.63</td>
</tr>
</tbody>
</table>

After sedimentation it was seen that Batch #1 had a total thickness of about 77.2 cm, resulting in an overall void ratio and porosity of 0.96 and 0.49, respectively. Five layers were attempted to be removed in equal parts, so that the total sample height would be of 20 cm, required for the further steps of triaxial testing as proposed by Jager and Molenkamp. For each layer it was proceeded to estimate the void ratio and vertical intergranular stresses at midpoint of each layer, as shown in Table 10; since there was no further testing on the remaining sample, it was decided to split it into two different layers and also estimate their correspondent void ratios. The vertical intergranular stresses at the midpoint of a layer of thickness $t$ can be estimated as follows

$$
\sigma_{zz}^* = (1 - n)\rho_s g \frac{t}{2} + \sum \sigma_{zz}^* \text{ from Upper Layers},
$$

[32]

where the second term of the equation applies for all layers underneath the first one, and is equal to zero for the uppermost layer.

The information will be plotted with that from the second batch later on this section. A picture of the sand suction system used, based on fluid siphoning, is shown on Figure 45; the tip of the suction tube is placed
as close as possible to the surface of the sand column in order to allow suction of the sand into another recipient (in this case a permeameter) without loading the column when in contact with the tube. The removal is done in such a way that the surface is kept as uniform as possible, which can take a long period of time, depending on the suction rate; it is important to note that the suction rate depends on the difference on the elevation head between the water levels at the fluidization column and that from the permeameter.

![Figure 45: Sand-Water Siphoning Setup](image)

**Table 10:** Void Ratio, Porosity and Vertical Intergranular Stresses at Midpoints of Removed Layers on Batch #1. Taken from Paz Noriega (2015).

<table>
<thead>
<tr>
<th>LAYER</th>
<th>z (cm)</th>
<th>t (cm)</th>
<th>e (-)</th>
<th>n (-)</th>
<th>$\sigma_{zz}$ (Pa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5.30</td>
<td>10.6</td>
<td>1.00</td>
<td>0.50</td>
<td>430.27</td>
</tr>
<tr>
<td>2</td>
<td>15.85</td>
<td>10.5</td>
<td>0.96</td>
<td>0.49</td>
<td>1295.73</td>
</tr>
<tr>
<td>3</td>
<td>26.10</td>
<td>10.0</td>
<td>0.86</td>
<td>0.46</td>
<td>2166.92</td>
</tr>
<tr>
<td>4</td>
<td>36.20</td>
<td>10.2</td>
<td>0.88</td>
<td>0.47</td>
<td>3043.85</td>
</tr>
<tr>
<td>5</td>
<td>46.45</td>
<td>10.3</td>
<td>1.16</td>
<td>0.54</td>
<td>3872.14</td>
</tr>
<tr>
<td>6</td>
<td>58.00</td>
<td>12.8</td>
<td>1.42</td>
<td>0.59</td>
<td>4689.34</td>
</tr>
<tr>
<td>7</td>
<td>70.80</td>
<td>12.8</td>
<td>0.98</td>
<td>0.49</td>
<td>5645.09</td>
</tr>
</tbody>
</table>

For the second batch, a total mass of 7.79 kg was used in which fluidization rates were increased to observe the variations in the critical and measured hydraulic gradients. From the data gathered on Table 11, it is seen that increasing rate of discharge results in an expansion of the fluidized bed and the decrease of the critical gradient as a result of an increased porosity.
Table 11: Determination of Critical Hydraulic Gradient during Fluidization of Batch #2 at Different Discharge Rates. Taken from Paz Noriega (2015).

<table>
<thead>
<tr>
<th>PARAMETERS/MEASURES</th>
<th>Q ≈ 0.52 L/min</th>
<th>Q ≈ 0.56 L/min</th>
<th>Q ≈ 0.59 L/min</th>
<th>Q ≈ 0.63 L/min</th>
</tr>
</thead>
<tbody>
<tr>
<td>H&lt;sub&gt;fluidized&lt;/sub&gt; (cm)</td>
<td>102.50</td>
<td>103.80</td>
<td>104.50</td>
<td>105.00</td>
</tr>
<tr>
<td>V&lt;sub&gt;fluidized&lt;/sub&gt; (cm&lt;sup&gt;3&lt;/sup&gt;)</td>
<td>8050.33</td>
<td>8152.43</td>
<td>8207.41</td>
<td>8246.68</td>
</tr>
<tr>
<td>V&lt;sub&gt;s&lt;/sub&gt; (cm&lt;sup&gt;3&lt;/sup&gt;)</td>
<td>2933.31</td>
<td>2933.31</td>
<td>2933.31</td>
<td>2933.31</td>
</tr>
<tr>
<td>ε&lt;sub&gt;fluidized&lt;/sub&gt; (-)</td>
<td>1.74</td>
<td>1.78</td>
<td>1.80</td>
<td>1.81</td>
</tr>
<tr>
<td>n&lt;sub&gt;fluidized&lt;/sub&gt; (-)</td>
<td>0.64</td>
<td>0.64</td>
<td>0.64</td>
<td>0.64</td>
</tr>
<tr>
<td>ρ&lt;sub&gt;sat&lt;/sub&gt; (kg/m&lt;sup&gt;3&lt;/sup&gt;)</td>
<td>1604.33</td>
<td>1596.05</td>
<td>1591.08</td>
<td>1589.43</td>
</tr>
<tr>
<td>i&lt;sub&gt;cr&lt;/sub&gt; (-)</td>
<td>0.60</td>
<td>0.60</td>
<td>0.59</td>
<td>0.59</td>
</tr>
</tbody>
</table>

At each discharge rate the changes in the manometers were registered and the hydraulic gradient between them calculated, in a similar fashion as for the previous batch. Table 12 shows how increasing discharge results in a pressure increase along the column, but at some point these pressure changes start to reduce (consistent to what is shown in Figure 23); by comparing the critical gradient values from Table 11, it is seen that the sand column was fluidized on all discharge rates.

Table 12: Hydraulic Gradient between Manometers during Fluidization of Batch #2 at Different Discharge Rates. Taken from Paz Noriega (2015)

<table>
<thead>
<tr>
<th>PARAMETERS/MEASURES</th>
<th>Q ≈ 0.52 L/min</th>
<th>Q ≈ 0.56 L/min</th>
<th>Q ≈ 0.59 L/min</th>
<th>Q ≈ 0.63 L/min</th>
</tr>
</thead>
<tbody>
<tr>
<td>h&lt;sub&gt;1&lt;/sub&gt; (cm)</td>
<td>95.4</td>
<td>95.9</td>
<td>95.5</td>
<td>95.3</td>
</tr>
<tr>
<td>i&lt;sub&gt;1-2&lt;/sub&gt; (-)</td>
<td>0.59</td>
<td>0.61</td>
<td>0.61</td>
<td>0.62</td>
</tr>
<tr>
<td>h&lt;sub&gt;2&lt;/sub&gt; (cm)</td>
<td>86.5</td>
<td>86.7</td>
<td>86.4</td>
<td>86.0</td>
</tr>
<tr>
<td>i&lt;sub&gt;2-3&lt;/sub&gt; (-)</td>
<td>0.70</td>
<td>0.61</td>
<td>0.61</td>
<td>0.61</td>
</tr>
<tr>
<td>h&lt;sub&gt;3&lt;/sub&gt; (cm)</td>
<td>76.0</td>
<td>77.5</td>
<td>77.2</td>
<td>76.8</td>
</tr>
<tr>
<td>i&lt;sub&gt;3-4&lt;/sub&gt; (-)</td>
<td>0.67</td>
<td>0.61</td>
<td>0.60</td>
<td>0.59</td>
</tr>
<tr>
<td>h&lt;sub&gt;4&lt;/sub&gt; (cm)</td>
<td>66.0</td>
<td>68.4</td>
<td>68.2</td>
<td>68.0</td>
</tr>
</tbody>
</table>

After fluidization and sedimentation, the measured sediment column height was of about 72 cm, resulting in void ratio and porosity values of about 0.93 and 0.48, respectively. Using an improved setup for the sand suction, the sand was removed in layers of equal thickness until a final sample height of 20 cm was obtained. The results for void ratio, porosity and vertical intergranular stresses can be found on Table 13.
Table 13: Void Ratio, Porosity and Vertical Intergranular Stresses at Midpoints of Removed Layers on Batch #2. Taken from Paz Noriega (2015).

<table>
<thead>
<tr>
<th>LAYER</th>
<th>z (cm)</th>
<th>t (cm)</th>
<th>e (-)</th>
<th>n (-)</th>
<th>σ^*zz (Pa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5.20</td>
<td>10.4</td>
<td>1.26</td>
<td>0.56</td>
<td>373.04</td>
</tr>
<tr>
<td>2</td>
<td>15.60</td>
<td>10.4</td>
<td>1.00</td>
<td>0.50</td>
<td>1169.09</td>
</tr>
<tr>
<td>3</td>
<td>26.00</td>
<td>10.4</td>
<td>0.82</td>
<td>0.45</td>
<td>2056.86</td>
</tr>
<tr>
<td>4</td>
<td>36.40</td>
<td>10.4</td>
<td>0.97</td>
<td>0.49</td>
<td>2949.90</td>
</tr>
<tr>
<td>5</td>
<td>46.80</td>
<td>10.4</td>
<td>1.19</td>
<td>0.54</td>
<td>3763.44</td>
</tr>
<tr>
<td>6</td>
<td>57.00</td>
<td>10.0</td>
<td>1.20</td>
<td>0.55</td>
<td>4517.42</td>
</tr>
<tr>
<td>7</td>
<td>67.00</td>
<td>10.0</td>
<td>0.54</td>
<td>0.35</td>
<td>5413.43</td>
</tr>
</tbody>
</table>

The results for the void ratio against depth were plotted together, as shown in Figure 46. Some things can be identified from the plots:

1. The first layer is generally the loosest of all since it receives no surcharge other than the one from the water on top of it. At the same time, this layer is generally characterized by fine sand grains which have been expanded along the fluidized sand column and reached the uppermost parts of the column.

2. At the second and third layer it can be seen that the material becomes denser, obeying to both the surcharge coming from the upper layers and the water and to the presence of coarser grains whose concentration only expanded to some part of the fluidized bed. In fact the expansion of a granular bed, for a given fluidization velocity, is inversely related to the grain size; this means that granular beds with fine grains will expand the most, while those with coarse grains will not expand much and would require a higher fluidization velocity to expand further.

3. After the third layer there is an increase in the void ratio, which could be explained by the unloading from the upper layers’ excavation. Coarser material can be expected to be found here, due to the phenomenon explained on the previous numeral. However, comparison of the level of the top of a sand layer after removal of an upper layer and its level immediately before starting the excavation does not show any significant difference.

4. It can be found that at the upper half of the sample the material is at its loosest state, most likely due to the removal of the upper layers and also due to the expected predominance of coarse grains over fine ones. At the lowermost layer of the column the sample is, once again, denser, which can be explained by a decrease in its void ratio due to water being displaced by more sand grains during deposition and densification due to the loading from the upper layers.

5. Most of the void ratio values along the depth were greater or equal than the maximum void ratio determined via the JGS method. It must be recalled that the conditions of sample preparation on both cases are different in terms of how the sand is deposited; while on the JGS the sand particles are falling through an “air” medium, during the fluidization process they fall through water, which can reduce its falling velocity and thus resulting in a material with a looser structure.
Another thing to notice is based on the data estimated for the hydraulic conductivity measurements; as shown in Figure 47, at a rate of about 0.56 L/min the values of the hydraulic gradient between the three manometers reach the same value, which could suggest that the porosity of the material within a length of 0.45 m would be the same. Due to the absence of a manometer at the bottom of the column, by linear extrapolation of the points it could be even said that the fluidized material even has a uniform porosity on its lowermost part. However further assessment on whether the discharge rate may play a role on the grain size distribution and porosity along the column should be done.

Finally, when a sieving test is performed on the sand removed at each layer a trend is found with increasing depth. In this case a dry sieving test was done for this purposes, since the main interest was to see to which degree the grain size distribution varies with depth, as shown in Figure 48. In addition, Table 14 show the estimated values for $D_{10}$, $D_{30}$, $D_{50}$, $D_{60}$, $C_u$ and $C_c$. 
Figure 48: Grain Size Distribution for each Layer on Batch #2 (Dry Sieving). Taken from Paz Noriega (2015).

Table 14: Values for $D_{10}$, $D_{30}$, $D_{50}$, $C_u$ and $C_c$ for each Layer on Batch #2. Taken from Paz Noriega (2015).

<table>
<thead>
<tr>
<th>LAYER</th>
<th>$D_{10}$ (µm)</th>
<th>$D_{30}$ (µm)</th>
<th>$D_{50}$ (µm)</th>
<th>$D_{60}$ (µm)</th>
<th>$C_u$ (-)</th>
<th>$C_c$ (-)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>86.55</td>
<td>110.14</td>
<td>116.70</td>
<td>117.84</td>
<td>1.36</td>
<td>1.19</td>
</tr>
<tr>
<td>2</td>
<td>97.15</td>
<td>112.28</td>
<td>121.30</td>
<td>126.07</td>
<td>1.30</td>
<td>1.03</td>
</tr>
<tr>
<td>3</td>
<td>106.99</td>
<td>116.70</td>
<td>123.66</td>
<td>133.59</td>
<td>1.25</td>
<td>0.95</td>
</tr>
<tr>
<td>4</td>
<td>108.03</td>
<td>116.70</td>
<td>128.53</td>
<td>136.20</td>
<td>1.26</td>
<td>0.93</td>
</tr>
<tr>
<td>5</td>
<td>108.03</td>
<td>120.13</td>
<td>133.59</td>
<td>137.52</td>
<td>1.27</td>
<td>0.97</td>
</tr>
<tr>
<td>6</td>
<td>106.99</td>
<td>120.13</td>
<td>133.59</td>
<td>138.85</td>
<td>1.30</td>
<td>0.97</td>
</tr>
<tr>
<td>7</td>
<td>112.28</td>
<td>124.86</td>
<td>144.32</td>
<td>155.87</td>
<td>1.39</td>
<td>0.89</td>
</tr>
</tbody>
</table>

It is possible to conclude, based on the results on the grain size distribution, the following points:

1. There seems to be a trend in a reducing percentage of particles passing through a sieve with increasing depth. This can be explained by the major presence of coarser sand grains at deeper locations of the sediment column formed after fluidization, and supported when comparing the values for $D_{10}$, $D_{30}$, $D_{50}$ and $D_{60}$.

2. An idea on the hydraulic conductivity of the removed layers can be estimated on the information gathered in Table 14 and with the use of empirical formulas. One of them is the Hazen equation (Hazen, 1911), which correlates the hydraulic conductivity $K$ with $D_{10}$, expressed as

$$K = C (D_{10})^2$$  \[33\]

where $C$ is a constant that varies from 40 to 150 (cm · sec)$^{-1}$ for sands. Another empirical expression, which uses the value for $D_{50}$, and accounting for the spread of grain sizes is based on the Kozeny-Carman equation (modified from Bear, 1972), expressed as

$$K = \left(\frac{\rho_w g}{\mu} \left(\frac{n}{1-n}\right)^2 \left(\frac{D_{50}}{180}\right)^{2}\right)$$  \[34\]

By visual inspection of the values of $D_{50}$ and Equation [33], it would be safe to assume that the hydraulic conductivity of the layer would depend only on the size of the grain diameter that represents 10% of the sample mass; in other words, that there will be an increase in the hydraulic conductivity and permeability, which will be described in detail on the next section. On the other
hand, the Kozeny-Carman equation takes into account not only the median grain diameter of the sample but also its porosity by means of the term \( \frac{n^4}{(1-n)^2} \); this is important, especially since the pore size in a poorly sorted material will tend to be smaller than in a well sorted material with the same value for \( D_{50} \). The results on Table 15 show that, while the median particle size increases with depth, the hydraulic conductivity value changes in the same pattern as the porosity value for each layer, showing how the inclusion of the porosity can affect the estimation of the hydraulic conductivity; this is evident when comparing the values of \( K \) for both Layers 5 and 6, which have the same value for \( D_{50} \).

Table 15: Values for \( n, D_{50}, \) and Hydraulic Conductivity \( K \) using Kozeny-Carman (Equation [34]) for each Layer on Batch #2. Taken from Paz Noriega (2015).

<table>
<thead>
<tr>
<th>LAYER</th>
<th>( n (-) )</th>
<th>( D_{50} (\mu m) )</th>
<th>( K (m/s) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.56</td>
<td>116.70</td>
<td>6.73 x 10^{-4}</td>
</tr>
<tr>
<td>2</td>
<td>0.50</td>
<td>121.30</td>
<td>4.01 x 10^{-4}</td>
</tr>
<tr>
<td>3</td>
<td>0.45</td>
<td>123.66</td>
<td>2.51 x 10^{-4}</td>
</tr>
<tr>
<td>4</td>
<td>0.49</td>
<td>128.53</td>
<td>4.07 x 10^{-4}</td>
</tr>
<tr>
<td>5</td>
<td>0.54</td>
<td>133.59</td>
<td>7.24 x 10^{-4}</td>
</tr>
<tr>
<td>6</td>
<td>0.55</td>
<td>133.59</td>
<td>7.99 x 10^{-4}</td>
</tr>
<tr>
<td>7</td>
<td>0.35</td>
<td>144.32</td>
<td>1.15 x 10^{-4}</td>
</tr>
</tbody>
</table>

5.6. Detailed Fluidization Assessment Using Permeability and Porosity

The results obtained on the distribution of the void ratio along the column for both batches arises the question of whether or not fluidization really occurred. While evidence such as the pressure drop variation with increase in applied discharge rate and increase on granular bed thickness may seem to be enough criteria for supporting the activation of fluidization, the next question would be whether or not the intergranular stresses have decreased to zero. For this a detailed assessment will be done on Batch #2, using the data for a discharge rate of about 0.63 L/min, considering the following assumptions:

1. On any stationary fluidization regime bed expansion will be uniform (McCabe et al., 2001). This means that each layer will be expanded in the same fashion. Localization of each layer boundary to its “location” at fluidized state will be done by multiplying its elevation with respect to the base of the column by a factor defined by \((1+H_s/H_f)\), where \( H_s \) and \( H_f \) represent the total column height at both its sediment and fluidized state, respectively.

2. All pressure readings taken from the manometers will be linearly interpolated to the boundaries of each layer, which will help estimate both the porosity and the permeability of the layer at its fluidized state.

3. Equation [24] from Jager and Molenkamp (2015a), supported by Equation 7.56 from McCabe et al. (2001), is rewritten for porosity so that

\[
n = 1 - \frac{\rho_w \Delta h}{\Delta z (\rho_s - \rho_w)} \quad [35]
\]

This formulation allows to estimate the porosity from the interpolated values described on the previous numeral.

4. The permeability of the layer will be calculated using the following equation

\[
\kappa_{zz} = K_{zz} \frac{\mu}{\rho_w g} = \frac{Q}{\Delta z} \left( \frac{\mu}{\rho_w g} \right) \frac{\Delta z}{\Delta h}, \quad [36]
\]
where $\kappa$ is the sand’s permeability, $K$ its hydraulic conductivity and $\mu$ the dynamic viscosity of water. $Q$ is known as the volume per unit of time which is controlled by the peristaltic pump, and $A$ the cross-sectional area of the fluidization column.

5. The vertical equilibrium of the saturated soil during fluidization (i.e. the vertical total stress along the column) is defined by the following relation (Jager and Molenkamp, 2015):

$$\frac{\partial \sigma_{zz}}{\partial z} = \frac{\partial (\sigma_{zz} + p_{w})}{\partial z} = \left( (1 - n) \rho_{s} + n \rho_{w} \right) g$$

[37]

Knowing the total vertical stress along each layer will serve as a first step for calculating the vertical intergranular stresses and determine whether or not fluidization occurred; in this case the total vertical stresses at the mid-height of each layer will be calculated.

6. By rewriting Equation [21] from Jager and Molenkamp as a function of the discharge applied by the peristaltic pump and the cross-sectional area of the fluidization column, the pore pressure can be represented by:

$$\frac{\partial p_{w}}{\partial z} = \rho_{w} g + \mu \kappa_{zz}^{-1} \left( \frac{Q}{A} \right)$$

[38]

At the top of the fluidized column the only pore pressure acting is the one coming from the water above.

7. If, for a given point in the extended column, the values estimated using Equation [37] are more or less the same with those from Equation [38], it will be said that fluidization has been reached since there are no vertical intergranular stresses.

The relation between the sediment layer height and the fluidized bed column was estimated to be of about 0.4583, based on what was described on the first of the step series listed above, meaning that the location of each layer boundary is done by multiplying its measure at the sediment layer by 1.4583. Figure 49 shows the elevation with respect to the bottom of the column of the layer boundaries on both the fluidized and sediment state; the measurements at the right side of the fluidized column correspond to the ones taken at the manometers and the ones interpolated at the interest locations.
Based on the information above, calculations on each layer at their “fluidized” state were done and presented on Table 16. The calculations show that the vertical intergranular stresses are lower than zero, likely due to the calculation process; since the pore pressure are more or less equal to the total stress, it can be said that there are no vertical intergranular stresses and that, therefore, fluidization has been reached.

Figure 49: Boundaries between Layers at Sediment State and at Fluidization State.
Table 16: Calculations for Porosity, Hydraulic Conductivity, Permeability, Pore Water and Vertical Intergranular Stresses for Batch #2 at Fluidized State. Taken from Paz Noriega (2015).

<table>
<thead>
<tr>
<th>LAYER</th>
<th>( n_t ) (-)</th>
<th>( K ) (m/s)</th>
<th>( \kappa_f ) (m(^2))</th>
<th>( \sigma_{zz} ) (Pa)</th>
<th>( p_w ) (Pa)</th>
<th>( \sigma_{zz}^* ) (Pa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.65</td>
<td>2.28 x 10(^{-5})</td>
<td>2.32 x 10(^{-10})</td>
<td>1622.03</td>
<td>1622.08</td>
<td>-0.05</td>
</tr>
<tr>
<td>2</td>
<td>0.65</td>
<td>2.30 x 10(^{-3})</td>
<td>2.34 x 10(^{-10})</td>
<td>3979.34</td>
<td>3979.43</td>
<td>-0.09</td>
</tr>
<tr>
<td>3</td>
<td>0.63</td>
<td>2.20 x 10(^{-3})</td>
<td>2.24 x 10(^{-10})</td>
<td>6352.41</td>
<td>6352.47</td>
<td>-0.06</td>
</tr>
<tr>
<td>4</td>
<td>0.67</td>
<td>2.42 x 10(^{-5})</td>
<td>2.46 x 10(^{-10})</td>
<td>8704.28</td>
<td>8704.40</td>
<td>-0.12</td>
</tr>
<tr>
<td>5</td>
<td>0.59</td>
<td>1.95 x 10(^{-3})</td>
<td>1.99 x 10(^{-10})</td>
<td>11113.08</td>
<td>11113.25</td>
<td>-0.17</td>
</tr>
<tr>
<td>6</td>
<td>0.63</td>
<td>2.16 x 10(^{-3})</td>
<td>2.20 x 10(^{-10})</td>
<td>13525.76</td>
<td>13526.04</td>
<td>-0.28</td>
</tr>
<tr>
<td>7</td>
<td>0.63</td>
<td>2.16 x 10(^{-3})</td>
<td>2.20 x 10(^{-10})</td>
<td>15843.83</td>
<td>15844.14</td>
<td>-0.31</td>
</tr>
</tbody>
</table>

5.7. Error Analysis on Void Ratio Measurements

Analysis of the tests performed indicates that the important variable to assess the fluidization process and the resulting product is the void ratio, whose estimation depends mainly on three measurements: the density of the particles, the mass of the removed sand layer and the thickness of the sand layer for which the void ratio will be estimated. An error analysis using partial differentiation was done to see which variable was the most sensitive to error. For this the equation of the void ratio is rewritten into those variables and the partial derivative with respect to each variable represents the magnitude of error corresponding to the change on the value of each variable.

In this scenario, the values of the particles’ density, the thickness of the removed layer and mass are assumed as 2.6557 g/cm\(^3\), 10 cm and 1080 g, respectively. The void ratio was estimated for this case as 0.9313. The expressions for estimating the error magnitude for each variable are shown below

\[
|\Delta e|_t \leq \frac{\pi}{4} D^2 \frac{\rho_s}{M_s} \cdot |\Delta t| \]  \[39\]
\[
|\Delta e|_{\rho_s} \leq \frac{\pi}{4} D^2 \frac{t}{M_s} \cdot |\Delta \rho_s| \]  \[40\]
\[
|\Delta e|_{M_s} \leq \frac{\pi}{4} D^2 \frac{t}{\rho_s} \cdot |\Delta M_s| \]  \[41\]

For determining the value of variation of each measurement (i.e. the variation percentage), it is important to analyse how each measurement is made and its accuracy. The measurement of the removed thickness was done by comparing the elevation of the sand column before and during the removal of the sand layer, until the difference of the new measurement, with respect to that at the beginning of the layer removal, is equal to the desired removed thickness. Since measurements were performed by reading from a measuring tape at the transparent fluidization column, it can be expected that the value of the removed layer will depend to the visual acuteness of the test operator; furthermore, reading variations can occur at both the reading of the top and bottom of the layer. For this reason it was considered to use a variation of 0.30 cm with respect to the total removed layer, based on what was previously described. This results in a total percentage of variation of 3%, which will be used on this analysis.

When considering the variation percentage on the particle’s density and mass, it is important to consider the calibration and accuracy of the devices. A maximum deviation percentage was set for the gas pycnometer device of 0.005% as part of the configuration before sand testing; even when the recorded deviation was of +/- 0.0016%, for error estimation the maximum percentage will be considered. For the case
of the measurement of the dry mass of the removed layer, it was found that the electronic weighting scale used had a deviation of 0.01 g for measurements up to 800 g, and up to 0.02 g for measurements between 800 g and 1600 g; this represents a variation of 0.00125% with respect to the measured mass.

The results of each error magnitude and percentage with respect to the measured value are shown in Table 17, which show that the measurement of the layer thickness is the most significant source of error in the estimation of the void ratio. It is important to notice that the thickness of the removed sand layer depends greatly on how the process is done, including aspects like: distance from the suction tube tip to the surface of the column, the suction rate, the handling process of the tube during and after the layer has been removed and even ergonomics. Furthermore, failure to control the error sources on any of the layers will only result in the propagation of the error along the column.

**Table 17: Error Percentage Estimation for the Void Ratio Calculation. Taken from Paz Noriega (2015).**

<table>
<thead>
<tr>
<th>Variable</th>
<th>∆e (g)</th>
<th>Error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>t</td>
<td>0.0579</td>
<td>6.2171</td>
</tr>
<tr>
<td>ρ_s</td>
<td>9.66 \times 10^{-5}</td>
<td>0.0104</td>
</tr>
<tr>
<td>M_s</td>
<td>2.41 \times 10^{-5}</td>
<td>0.0026</td>
</tr>
</tbody>
</table>

The error percentage was included on the plot in Figure 46 for the data points on Batch #2 and compare it with the results in Batch #1, as shown in Figure 50. While in some cases the measurements taken are within the error margin, there are no evident reasons behind the oscillatory variation found along the column. Additional reasons such as the discharge rate and the volume of water could play a role on the variations along the column; however, more data is required in order to understand this.

**Figure 50: Void Ratio with Error Percentage**
Conclusions and Recommendations

In the following chapter, the conclusions drawn from the research project will be presented, based on the questions raised along the project. At the same time, recommendations for further research will be given which may not only improve the quality of the information from this work but also on the process in itself.

6.1. Conclusions

Both the concepts of liquefaction and fluidization fall under one common category, addressed by Allen (1984) as “liquidization”. In terms of the characteristics of this mechanisms, both share: high pore water pressures and behaviour of the soil mass as a two-phase liquid. Three differences can be identified from both phenomena:

- While in liquefaction the pore pressure increase is due to external loading of a saturated soil domain, in fluidization it is caused by the increase in the flow rate of the fluid entering the saturated soil domain.
- The hydraulic boundary conditions for fluidization correspond to drained conditions, meaning that water is allowed to flow through the pores. In liquefaction, the boundary conditions still correspond to drained conditions; however, the material behaviour is undrained because the pore water has no time to move with respect to the grain particles skeleton due to the short duration of this phenomenon.
- While for fluidization the intergranular stresses are equal to zero, in liquefaction a post-peak deviator intergranular strength remains, being significantly smaller than its peak strength; in extremely loose cases, liquefaction may result in a residual deviator intergranular strength approaching zero.

Static liquefaction is a relevant phenomenon in geotechnical and dredging engineering, applicable to saturated, loose soils. In underwater slopes, this is characterised by: drained, slow-consolidating changes to reach undrained instability with or without a trigger, post-stability reduction of the undrained resistance due to softening, and flow slide. The development of this mechanism induced by the undrained instability in a body of soil depends on several factors, namely: boundary conditions on the pore water flow in the domain, source of instability (with or without trigger), mechanical properties and heterogeneity of the body of soil.

Reproducible laboratory tests on static liquefaction, both on field models and soil elements, may allow to gather enough data to understand and quantify the behaviour of saturated soils for such phenomenon. The objective of undrained triaxial tests is to gather enough data to obtain the parameters for constitutive models,
while the role of reproducible field-model tests is to obtain response data to assess the quality level of the potentially relevant predictive models. Naturally, variations between both types of tests are not only limited to the size and shape of the soil volumes studied but also on the way the instability mechanisms are activated; while on an undrained triaxial test liquefaction is produced due to application of a constant deviator strain rate, in the liquefaction tank this instability may be produced either by loading or by rotation.

When characterising fine sands, the method to be used is important, especially when there are silts and clays present. For grain size distributions it is suggested to use the wet sieving method over the dry sieving, since there is a lower chance that the silt and clay particles create bonds among some sand particles, thus affecting the resulting distribution curve. The effects of fines (silts and clays) on the density of a sand can be seen by comparing the range between the maximum and minimum void ratio; while removal of fines does result in an increase on the void space, it also means that there is more space for sand particles to occupy this space, resulting in a range lower than that for a sand with fines. In addition, while index tests are useful for estimating the minimum and maximum void ratio of a sand, it is important to know that several factors can affect the values obtained, such as the procedure used or the ability of the test operator.

While liquefaction is a mechanism that has been extensively studied in soil mechanics and geotechnical engineering, the amount of literature available on fluidization of granular soils is significantly less; while it is a technique widely used in chemical engineering, fluidization is relevant for geotechnical engineering since it is related with the way particles are initially settled in environments where a transport fluid is involved (generally water). Fluidization reduces the intergranular stresses of a sand to zero, liquidizing it and allowing the formation of a looser material after the fluidization mechanism is removed. This potential of producing samples at a looser state, supported by the data from the previous chapter, may not only contribute to the study of liquefaction but also on determining the minimum void ratio of a granular material.

Two quantitative approaches have been identified to assess the development of fluidization in a sand column. The first one is based on the critical hydraulic gradient, defined by Equation [30], which can be compared with the hydraulic gradient between two elevation points during the inflow of water into the column; when this gradient is greater than that calculated via Equation [30], then it is said that fluidization has been reached. A second, more detailed, approach is based on the formulations of Jager and Molenkamp (2015a), and described in Section 5.6; this approach allows to calculate the total vertical stresses and the pore water pressure profiles, based on the hydraulic gradient. While the first approach is more like a “rule of thumb” which can be used as a preliminary assessment, the second approach allows to calculate whether or not there are still intergranular stresses.

In Section 2.3 the key questions of this research project were drawn and will be answered below:

What are the difficulties in preparing saturated sand samples which could be further used in undrained triaxial testing at low stress levels for analysing liquefaction in the liquefaction tank?

The main challenges could be resumed in five points:

1. Selection of a sample preparation method that can produce a microstructure similar to that found in the liquefaction tank.
2. Full saturation of sand samples, either during placement of the sand or by means of an auxiliary method like CO\textsubscript{2} injection or reversible loading-unloading-reloading.
3. Estimation and control of the membrane penetration effects on the liquefaction resistance of the sample.
4. Densification of the sample during its preparation and handling.
5. Calibration of the sensors and equipment used during testing.

The first challenge is being currently solved by using fluidization as a method for preparing a sample which not only reproduces the sedimentation process of sand in the liquefaction tank, but also produces a loose material which might present a high degree of saturation (this later statement needs to be proved on the subsequent phases of the main research project).

The degree of saturation of a sample has a relation with the development of liquefaction during undrained triaxial testing. In the cases where the B-Value is lower than 0.95, auxiliary means such as a CO₂-injection or increasing the cell pressure and back pressure are generally used. However, these methods present its risks (Paz Noriega, 2015). The method proposed by Jager and Molenkamp (2015b) could become an alternative for bulk density measurement and sample saturation; however more testing on this is needed.

Leaving out the effects of the membrane penetration on the development of deviator stresses on the sample can be done by two ways: estimating the contribution of the membrane to the development of deviator stresses, or limiting the membrane penetration. While approaches, such as using a membrane several times thicker than the mean grain size of the soil, exist for the standard triaxial testing procedures, for the sample preparation and testing procedures proposed by Jager and Molenkamp these effects could be further studied.

The last two points have to do with the way the testing in itself is done. While the accuracy and precision of the sensors and equipment is important for the measurements, the human error must be reduced by carefully performing the sample preparation and testing.

Which protocol of sample preparation and control is the best for saturated loose sand characterisation at low stress levels?

From the intergranular stress paths shown in Section 4.2 it may be seen that, at the stage of sample preparation, the protocol consisting on fluidization, sedimentation, excavation and mould removal with surrounding cell water presents the most limited sample disturbance from all four variants presented, since the disturbance was only limited to 1/3 of the sample’s height and the surrounding cell-water helps as a containment during the split-mould removal, along with the vacuum applied on the back pressure. On the other hand, the unloading process during layer removal takes the final sample to a stress state different from the one following sedimentation while causing a passive failure due to unloading defined by 1/3 of the sample’s height.

Removing the split-mould in cell-air (atmospheric pressure) increases the degree of disturbance on this upper third of the sample. The protocols of fluidization, sedimentation, 1-dimensional compression and split-mould removal on cell-water or cell-air present results in a major sample disturbance, which is extended to ½ of its height. On the other hand, the 1-D compression proposed aims to take the final sample to a stress state similar to the one found at mid-height of the slope at the liquefaction tank; the possibilities and limitations of this approach will be further estimated by means of experimentation. This conclusion does not take into account the effect of the human error from the person performing the sample preparation and testing.
What will be the effects of the fluidization on the density and void ratio distribution along the sediment columns for the two main protocols proposed?

Given a saturated granular bed with an initial height and porosity, the effects of fluidization include an increase in the overall porosity, an increase in the bed height and a decrease of its saturated density. Whether or not fluidization has been reached can be assessed numerically by two means: as a first approach, by estimating its critical gradient and compare it with the hydraulic gradient reached at a specific discharge rate and variation on the pressure head, measured by means of manometers or an equivalent measuring device; another, more elaborate method, is by estimating the porosity and permeability parameters using measurable data like the pressure change and the specific discharge, and assuming an uniform bed expansion, and from there calculate the total stress distribution and the pore pressure along the column. By using the elaborate approach it can be proved whether or not the vertical intergranular stresses are zero.

By comparing the changes in void ratio along the depth of each batch, it was found that the density distribution along the column (evaluated by means of the void ratio) follows an oscillatory trend, which could be caused by both the applied discharge rate during fluidization and by the nature of sands given their graded grain-size distribution (i.e. potential particle segregation). A third aspect that could play a role in this would be the wall effects during fluidization and sedimentation, which is more difficult to assess. A way to determine whether or not the nature of this “layering” can be expected is to compare the void ratio along the depth of the fluidized slope in the liquefaction tank with a sand column prepared at the same discharge rate.

6.2. Limitations

Several limitations on the performed work and the methods used can be identified:

- The selection of the best sample preparation and control protocol for the fluidization method only accounts the degree of sample disturbance, and not the overall performance of the sample during testing. Only when conducting a significant amount of undrained triaxial tests for samples prepared with the proposed protocols, and compare their results with those available on literature, under similar stress levels and with different sample preparation techniques, it will be possible to support or not the conclusion issued before in this matter.

- When the sand column is completely fluidized, it is only possible to estimate its properties along the column under assumptions of the manometer pressure profile along the column; in addition, at this point it is not possible to estimate the grain size distribution by experimental means.

- The estimation of the hydraulic conductivity of the removed layers of the sand column after sedimentation was done using empirical relations. A better approach would have been the execution of permeability tests.

- As Jager and Molenkamp (2015a) report, the total fluidized granular bed size at the liquefaction tank is of 2 metres, while on the undrained triaxial test a fluidization column of 1 metre will be used, assuming that the final sample obtained at the column is similar to that if it were located at mid-height of the granular bed in the liquefaction tank. However, the grain size distribution at the lowermost section of the fluidization column will not be the same as that obtained at mid-height of the sand at the liquefaction tank, since the elevation of the water infiltration source will be different.
• The analysis performed in Section 5.5 through 5.7 is based only on one adequate data set. While the void ratio profile obtained from Batch #1 only serve to support the trend of this same variable on Batch #2, at least two additional data sets would be needed to extend the analysis of the work done. However, at the moment of the elaboration of this work it was not possible to perform additional batches.

• While the sand removal during the physical modelling of the slope in the liquefaction tank is done by a machine, the sand removal process at the fluidization column is executed by the test operator using manual tools. The non-ergonomic nature of this process, at its current state, may affect the overall process of sample removal, which can only be countered at this stage by removing the sand in thinner layers (i.e. in 5 cm, instead of 10 cm). The relevance of the removed sample to the measurements and estimation of the void ratio along the column is supported by the error analysis performed in Section 5.7.

6.3. **Recommendations for Further Research**

• When performing the tests for the minimum void ratio using the JGS Method, it was found that the way the sand is deposited inside the mould has an effect on the magnitude of the void ratio. It was found that by moving the deposition funnel laterally, to place the sand in the uniform way inside the mould, the minimum void ratio could be reduced to values smaller than those found by only raising the funnel vertically while doing the test. This approach should be further tested on different sands and, should the changes be significant, propose a variation of this index test.

• At this point, while testing the setup, it was only possible to fluidize two different batches of sand, excavate its upper layers and determine its void ratio distribution along the removed layers (i.e. comparing void ratio and vertical intergranular stress). It is recommended to prepare at least three additional batches and compare its results with those obtained on these first two batches; with enough data, it would be possible to relate the void ratio with the vertical intergranular stress after virgin compression during sedimentation and subsequent unloading during removal of the upper layers, by means of the following expression (Jager and Molenkamp, 2015a):

\[
e = e_0 - A \left( \frac{\sigma_{zz}}{p_a} \right)^a
\]  

[42]

where \(e_0\) is the void ratio at zero virgin compression, \(a\) and \(A\) are dimensionless scalar parameters and \(p_a\) the mean atmospheric pressure as a standard reference value which makes the term inside the parenthesis dimensionless.

• A study relating the discharge rate and the grain size distribution and porosity along the column should be performed. This would confirm on whether or not the discharge rate needs to be set to a given value so that the porosity would be uniform along the column. This is important, since it could show up to what extent the parameters needed for Equation [42] change their values with a change in the discharge rate. In addition, for each removed layer a permeability test needs to be performed in order to accurately measure the hydraulic conductivity and permeability of the sand layer and compare it with values estimated by using the Hazen (Equation [33]) or Kozeny-Carman (Equation [34]) formulas. However, further study into fluidization could be improved by using a
completely transparent column with manometers located at its lowest elevations; this can accurately indicate the pressure change profile along the total elevation, and not on a single section.

- Membrane penetration effects for this research programme should be further researched, since it is dependent on the sand type and testing equipment. One way to measure membrane penetration, in standard triaxial testing, is by preparing a cylindrical sand sample around a steel insert, with a diameter of about 6 mm less than the full sample (Jefferies and Been, 2006). The total volume of this thin (3 mm thick) cylindrical sample of sand is small, making the volume changes due to unloading and reloading small compared to membrane penetration. This sample is then "consolidated" in the triaxial cell, but the measured volume change will be due entirely to membrane penetration effects. For the case of a fluidized sample the suitability of this method should be tested by observing whether the steel insert will move during fluidization. It also needs to be taken into account that membrane penetration effects is still under study, which means that other approaches may come.

- The setup used for preparing the first two batches is too elevated for the work performed, especially when considering the height of the fluidization column and the need of using an elevated platform when performing the sand suction; this is not ergonomic and could affect the way the test is performed, therefore affecting its quality. It is suggested, therefore, that the current elevation of the setup is lowered by, at least, 50 cm; any modifications on the drainage lines should be considered as well. In any case, careful sample preparation, control and testing contribute in reducing human error during testing.

- At this stage, mouth suction was used to fill in the hose used for the sand siphoning. It is suggested to use or design a tool that allows to start siphoning in a more hygienic way.

- In order to further assess which method of sample preparation, proposed by Jager and Molenkamp, is to not only take into account the disturbance during sample preparation and control, but also to consider the results during the subsequent undrained triaxial testing at low intergranular stress levels.

- An interesting project that may come from the study of stationary fluidization may be the development of a model that can allow to predict the particle concentration profile of a certain sand during and after fluidization, taking into consideration its grain size distribution.

- The technique of fluidization could be used to develop a new procedure for index testing for maximum void ratio for sands, in which a fluid (either water or air) could be passed through a known mass of sand contained inside a vessel. Understanding of stationary fluidization is important for developing these technique and an equipment design that would allow containment of particles within a chamber would be needed. The applicability of this method could be measured in terms of results repeatability and the costs associated to production and incorporation to the commercial soil testing market.

- Jager and Molenkamp (2015b) propose an alternative method for the measurement of the bulk density and increasing the degree of saturation of the sample. This method uses a reversible deviator unloading, in contrast to the conventional irreversible isotropic loading applied on triaxial tests, documented on the test manuals. While this could result in a saturation and measurement process with minor effects on the sample, testing will be required to prove whether this new method can be used.
References


SIBELCO (2014). Geba. Technical Data Sheet. SIBELCO.


Sand Fluidization Technique and Modified Triaxial Testing

In this appendix, a description on the modified triaxial test based on the fluidization sample preparation technique is presented, based on the experiences learnt during the handling of the built equipment. This could be used as a guideline for subsequent triaxial tests using the equipment available at the laboratory. Note that this process applies for cohesionless soils.

A.1. Apparatus Parts Description

A description on each of the tools and parts to build the setup for the modified triaxial testing will be given in this section. Figure A1 shows the infiltration cell with the membrane ring and filter plate. The infiltration cell is installed on the pedestal connected to the cell water, pore pressure and back pressure lines; the filter plate is placed on top of the infiltration cell and subjected to a saturation process using the suction cap shown in Figure A2.
After saturation the latex membrane is placed and fixed to the infiltration cell by means of a membrane ring, shown in Figure A1 as well. By fixating tightly the membrane ring a part of the filter plate will be emerging from the centre of the ring. After this step, the split-mould shown in Figure A3 is placed and the membrane is closely placed on the mould.
Having fixed the membrane to the mould, the next step is to place the fluidization column and overflow setup. Figure A4 shows the fluidization column and the pressure panel that will be used to measure the pressure differences during fluidization, this setup is used only for Protocol A, to be discussed in another section.

Figure A5 shows the overflow to be placed on top of the fluidization column, which receives the water coming from the fluidization process and drains it out of the setup. Additionally, a water inlet will help to compensate the water being siphoned during the sand suction process; a description of this mechanism is described in Paz Noriega (2015). In the case where Protocol B is executed, the overflow is placed on top of the split-mould.
In the case where Protocol A applies, suction will be applied by the tool shown in Figure A6, where the tube is lowered as the sand is being sucked out. The uniform removal of the sand is ensured by moving the tube laterally by means of the guide placed on top of the overflow.

After the sample preparation, the setup of the triaxial equipment is shown in Figure A7, taken from Jager and Molenkamp (2015a). The following sections will deal with the preparations and protocols for preparing the sample.
A.2. Preparation and Checking of Apparatus

1. Apparatus used for triaxial tests shall be subjected to rigorous inspection and check testing before use. These checks shall be carried out on the cell pressure, back pressure and pore pressure systems at the stated frequency and shall be of two kinds: ‘complete’ checks and ‘routine’ checks.

2. ‘Complete’ checks will be defined as those stated by the British Standard BS 1377:Part 8:1990, and shall be carried out: (a) when any item of new equipment is introduced into the system; (b) if an integral part of a system has been removed, stripped down, overhauled or repaired; (c) at intervals not exceeding 3 months.

3. ‘Routine’ checks will be defined as those stated by the British Standard BS 1377: Part 8:1990 and must be carried out immediately before starting a test.

4. The porous discs to be used on both ends of the specimen and the ones used for the standpipes for the fluidization column must be inspected to ensure that water drains freely through them. Before use, it is important to boil the discs for at least 30 minutes in distilled water and then keep them under de-aired water in a beaker or in a recipient until required. For the filter plate going directly under specimen and above the infiltration cell, this process might help to ensure full saturation but a separate process is required as well.

5. The split-mould’s cogs and screws must be properly lubricated by means of a silicone grease, similar to the one to be used for the test. This grease consists of a high-viscous silicon oil filled with spherical Teflon particles with a mean diameter of 10 µm.
6. The peristaltic pump which will be used to perform the fluidization process needs to be calibrated; this is done by measuring the discharge rate of water coming from the overflow by different voltage input. This will allow to control the discharge rate for different type of sands, since different grain sizes require different minimum fluidization velocities.

A.3. Material Characterization prior to Testing

1. For estimating the requirements for the fluidization process it is important to know the amount of sand that will be used on the test. This is the reason why the information on the void ratio, porosity and the dry weight of the sand is needed. At the time of initial estimations the specific density quartz was used.

2. For obtaining the information on void ratio, porosity and dry weight of the sand several procedures can apply. At the time of the project the author used the Japanese Geotechnical Society Method (JGS, 1996) for minimum and maximum dry densities and void ratios, while a gas expansion pycnometer was employed for estimating the specific gravity ($G_s$). However the procedures for performing this characterization might differ on other projects, from those employed on this project; it is not within the scope of this section to discuss about these procedures.

A.4. Saturation of Filter Plate above Infiltration Cell

1. Place the filter plate on top of the infiltration cell; make sure that it is free of soil particles and that it is flat, not tilted.

2. Place the cylindrical cap on top of the infiltration cell in such a way that the filter plate is covered and a provisional chamber is created between the cap and the plate. Lubricant must be placed at the contact areas between the infiltration cell and the cylindrical cap, in order to ensure removal of the cap at a later stage. Be careful that the lubricant is not into contact with the water that will come out of the infiltration cell, as it may contaminate the filter plate.

3. Open the back pressure line and allow de-aired water to flow into the infiltration cell and through the filter plate. Water height above the plate should be equivalent to at least its thickness. Make sure that no leakage of water outside of the cap happens.

4. Close the back pressure line and place a vacuum line on top of the cylindrical cap, in order to extract all air inside the infiltration cell and within the filter plate. Make sure the vacuum pump system connected to the cylindrical cap is running and open the vacuum line valve; it is important that the pressure applied is of at least -1 bar, which will allow the air pockets to come up to the water surface and de-air the water inside the infiltration cell while ensuring full saturation of the filter plate. This vacuum must be kept until no air bubbles come from the filter plate. Although in principle the water being used for this test is de-aired and the filter plate has been previously immersed on de-aired water before this, it is advised to leave the vacuum operate overnight to ensure full filter plate saturation.

5. Close the vacuum line on the cylindrical cap. Remove the cylindrical cap by rotating it while pulling it up. Once the cap is removed water will come out from this provisional chamber.
A.5. Placing the Membrane, Split-Mould and Fluidization Column

1. Make sure the pedestal is placed on top of the piston and that it is aligned with the frame which will hold the fluidization column. Place the latex rubber membrane in such a way that it surrounds the infiltration cell and has contact with its walls.
2. Place the metallic support ring on top of the infiltration cell in such a way that it pushes the membrane to the infiltration cell, avoiding leakage through the membrane. This would be an equivalent to the O-rings used in conventional triaxial testing, but this metallic support ring will allow the support of the split mould.
3. Apply a thin layer (~50 µm) of grease for lateral lubrication inside the split-mould. This grease consists of a high-viscous silicon oil filled with spherical Teflon particles with a mean diameter of 10 µm.
4. Place the split-mould in such a way that it surrounds the membrane. To connect both sides of the mould it is important to rotate the topmost cogs on both sides in an anticlockwise direction in such a way that the bolts located at the bottom of the system rotate clockwise, thus locking effectively the split mould.
5. Apply a small vacuum on the space between the membrane and the mould in such a way that the membrane is tightly in contact with the split mould.

The following two sections will describe the different protocols that were studied, as proposed by Jager and Molenkamp (2015a).

A.6. Protocol A: Sample Preparation by Fluidization, Column Sedimentation and Excavation

1. Align the fluidization column and the upper overflow recipient and place them on top of the split mould, in such a way that the central axis of this set up coincides with the future loading axis.
2. Estimate the amount of sand to be used for a fluidized column height approximately 1 m, which will be estimated to reduce to about 0.75 m after sedimentation. These heights are estimated on the assumption that the liquefaction tank is of 2 meters and that the sample will be extracted at mid-height of the fluidized layer. The following expressions will be used

\[
W_{s\text{and}} = h_{\text{sediment}} A_0 \frac{\gamma_s}{1+e} \quad [A.1]
\]

\[
\gamma_{\text{dry}} = (1-n)\gamma_s \quad [A.2]
\]

\[
\gamma_{\text{sat}} = (1-n)\gamma_s + n\gamma_{\text{water}} \quad [A.3]
\]

in which \( h_{\text{sediment}} \) is the height of the sediment, \( A_0 \) is the sample’s initial area, \( \gamma_s \) the dry volumetric weight of the sand (or quartz as an initial estimate), \( e \) is the void ratio after sedimentation, \( n \) the porosity after sedimentation, \( \gamma_{\text{dry}} \) the dry volumetric weight after sedimentation, \( \gamma_{\text{sat}} \) its correspondent saturated volumetric weight and \( \gamma_{\text{water}} \) the volumetric weight of water.
3. From the amount of sand estimated using Equation [A.1], recalculate the values for porosity and void ratio—\( n_{\text{fluidized}} \) and \( e_{\text{fluidized}} \), respectively. Use Equation [A.1] while keeping \( A_0 \) and \( \gamma_s \) as constants and change the height of sediment (0.75 m) to height of column (1 m).
4. Estimate the amount of water in the fluidized column of approximately 1 m and cross-sectional area of \( A_0 \) using the following expression:

\[
M_{\text{water}} = n_{\text{fluidized}} \rho_{\text{water}} h_{\text{column}} A_0 \quad [A.4]
\]
5. Estimate the time for flushing the whole mass of pore water, $M_{water}$, with a discharge velocity $v_{fluidized}$ of $10^{-3}$ m/s. This velocity is based on the fluidizing discharge estimated for the liquefaction tank (Jager and Molenkamp, 2015a). The expression to be used is as follows:

$$t = \frac{n_{fluidized}h_{column}}{v_{fluidized}} \quad [A.5]$$

The estimated time accounts for only one flush. It is possible to calculate the amount of time required for flushing the sample with de-aired water 10 times by just multiplying the value obtained in [A.5] by ten. This volume is necessary in case additional flushing is required for washing out fines or ensuring major saturation.

6. Estimated the pore water stress at the bottom of the column during fluidization, using the following expression:

$$p_{water, fluidized} = \left( (1 - n_{fluidized})\rho_s + n_{fluidized}\rho_{water} \right) gh_{column} \quad [A.6]$$

This profile will be compared during fluidization by means of the standpipes at the fluidization column.

7. If distribution of porosity is to be calculated, knowing the vertical distribution of the pore water stress along the column, the following expression can be used; in this case $z$ represents the vertical upward direction:

$$-\frac{\partial p_w}{\partial z} + \left( (1 - n_{fluidized})\rho_s + n_{fluidized}\rho_{water} \right) g = 0 \quad [A.7]$$

Vertical equilibrium of the stationary fluidizing pore fluid flow is represented by the following expression

$$-(1 - n_{fluidized})(\rho_s - \rho_{water})g + \mu/k_{zz}^{-1}q_z = 0 \quad [A.8]$$

The absolute value of the first term of Equation [A.8] represents the buoyant weight per unit of the bulk volume of the saturated soil ($\gamma_{buoy}$), $q_z$ is the rate of vertical discharge of pore water, being constant over the height of the column and equal to $nv_z$, where $v_z$ is the vertical velocity of the pore water. Note that at the bottom of the column this velocity will be equal to the fluidization velocity $v_{fluidized}$. The second term of Equation [A.8] allows the calculation of the vertical intrinsic permeability measure, represented by $\mu/k_{zz}^{-1}$.

8. Having the fluidization column, split-mould and overflow prepared, open the back pressure valve and let de-aired water to fill to a height of 1 m. This is in order to allow sand to be poured into the column according to the mass estimated on [A.1] and to avoid capillary pressure to limit saturation. The sand could be poured first and then fluidize the mass of water estimated on [A.4]; but then more flushing would be required in order to break any bonds between sand particles.

9. Close the back pressure line and set the peristaltic pump which will be used for fluidization so that its $v_{fluidized}$ is equal to $10^{-3}$ m/s at the base of the column. Preliminary calculations of maximum discharge $Q$ of pore water through the sample for fluidization were of 0.47 l/min (Jager and Molenkamp, 2015a). Open again the back pressure line and allow water to flow upwards during fluidization, while observing the water pressure readings on the measurement equipment and fines being washed out of the column, as well as the amount of water being flushed into the column. Note that more flushing might be needed to ensure full fluidization and major sample saturation.

10. To start the sedimentation process, turn off the pump and close the back pressure valve and allow the sand particles to settle. Estimate the hydrostatic pore water stress distribution by calculating its value at the bottom of the column using the following expression:

$$p_{water, hydrostatic} = \rho_{water}gh_{column} \quad [A.9]$$
11. After leaving the sediments to settle for a period of at least 1 hour, measure the height of the sand column after sedimentation, which should be around 0.75 m with a homogeneous porosity of 0.56 (see Jager and Molenkamp, 2015a). Estimate the stress state in the sand column using the following set of equations:

\[ \sigma_{zz} = \gamma_{buoy}z \]  \hspace{1cm} [A.10]

\[ \sigma_{xx} = K_0 \sigma_{zz} \]  \hspace{1cm} [A.11]

\[ K_0 \approx 1 - \sin \varphi_{crit} \approx 0.5 \]  \hspace{1cm} [A.12]

The subscripts \(zz\) and \(xx\) denote the vertical and horizontal directions intergranular stresses, respectively, \(K_0\) the coefficient of intergranular stresses at rest and \(\varphi_{crit}\) the critical state friction angle.

For the present project, the preliminary estimate of \(\varphi_{crit}\) is of about 30° (Jager and Molenkamp, 2015a).

12. Calculate the isotropic and deviator stress measures \(p^*\) and \(q^*\) after sedimentation and the stress ratio \(R_0\) using the following set of equations:

\[ p^* = \frac{\sigma_{zz} + 2\sigma_{xx}}{3} \]  \hspace{1cm} [A.13]

\[ q^* = \sigma_{xx} - \sigma_{zz} \]  \hspace{1cm} [A.14]

\[ R_0 = \frac{q^*}{p^*} \]  \hspace{1cm} [A.15]

13. Following sedimentation and consolidation, the upper part of the settled sand column will be removed by application of a sand-water suction tool. The upper part will be divided into 5 sections and removal will be done in a layered process, so that a final loose sample height of 0.20 m is obtained. Each layer will be of approximately 0.11 m (this is assuming a sediment column of 0.75 m and a final sample height of 0.20 m). Each removed sand-water mixture layer will be collected in a container for estimating its dry weight and grain size distribution; additional characterization of each layer may be done provided there is enough material. The flow rate of sand-water suction must be kept constant over the removed height of the sand column by maintaining a constant hydraulic head between the water level at the top of the transparent fluidization column and the water level in the container receiving the outflow of the sand-water suction tube. Water supply for sand-water suction is done by a lateral supply opening just below the overflow at the top of the fluidization column.

14. Estimate the induced intergranular stresses at the top and bottom of the sample for each layer removal using the following set of equations:

\[ \sigma_{xx,k}\text{top} = (5 - k)\gamma_{buoy}\Delta h_{layer} \]  \hspace{1cm} [A.16]

\[ \sigma_{xx,k}\text{top} = (5K_0 - kK_e)\gamma_{buoy}\Delta h_{layer} \]  \hspace{1cm} [A.17]

\[ \sigma_{zz,k}\text{bottom} = (5 - k)\gamma_{buoy}\Delta h_{layer} + \gamma_{buoy}\Delta h_{layer} \]  \hspace{1cm} [A.18]

\[ \sigma_{xx,k}\text{bottom} = (5K_0 - kK_e)\gamma_{buoy}\Delta h_{layer} + K_0\gamma_{buoy}h_{sample} \]  \hspace{1cm} [A.19]

where \(K_e\) is the equivalent elastic reduction factor, estimated to of the approximate order of magnitude \(0.1 \leq K_e \leq 0.3\), and \(k\) represents the layer which is removed (considering five layers). During each unloading phase the stress changes occur along the remaining sand column. However, at some point below the top of the remaining sand column the stress changes makes the horizontal intergranular stresses to become significantly larger than the remaining vertical stress, thus reaching its passive failure limit defined as

\[ \frac{\sigma_{xx}}{\sigma_{zz}} \leq K_p = \frac{1 + \sin \varphi_{crit}}{1 - \sin \varphi_{crit}} \]  \hspace{1cm} [A.20]
where $K_p$ represents the dimensionless coefficient of lateral passive pressure of loose sands which limits the intergranular stress ratio. At the preliminary study it was estimated to be around 3; this value might change, however.

The location of this “passive transition depth”, estimated with respect to the top of the remaining sample, is calculated by

$$\Delta z_{(k+1)} = \frac{(k_0-K_p)}{(k_p-K_0)} k \Delta h_{\text{layer}}$$  \[A.21\]

At $k=5$, the location of the “passive transition depth” (i.e. $\Delta z_{(6)}$) will be at the remaining sand sample; therefore it is important to know the induced intergranular stresses with each layer removal at the final sample’s “passive transition depth” by means of the following expressions.

$$\sigma_{zz,k|\text{trans}} = (5 - k) \gamma_{\text{buoy}} \Delta h_{\text{layer}} + \gamma_{\text{buoy}} \Delta z_{(6)}$$  \[A.22\]

$$\sigma_{xx,k|\text{trans}} = (5K_0 - kK_p) \gamma_{\text{buoy}} \Delta h_{\text{layer}} + K_0 \gamma_{\text{buoy}} \Delta z_{(6)}$$  \[A.23\]

15. Estimate the density distribution along the sand column using the following expression for virgin compression during sedimentation and subsequent unloading, by relating the void ratio as a function of the vertical intergranular stress, namely

$$e = e_0 - A \left( \frac{\sigma_{zz}}{p_a} \right)^a$$  \[A.24\]

where $e_0$ is the void ratio at zero virgin compression, $a$ and $A$ are dimensionless scalar parameters and $p_a$ is the mean atmospheric pressure as standard reference value, making the ratio of $\sigma_{zz}/p_a$ dimensionless. By measuring the void ratios $e$ of the removed column layers together with the estimated mean vertical intergranular stresses $\sigma_{zz}$ at mid-height of these column layers after sedimentation may enable to calculate the three dimensionless parameters $e_0$, $a$ and $A$. Fitting can be done, in principle, by numerical minimization by means of least squares. However, for easy fitting the natural logarithm of Equation [A.24] would make the parameter determination rather simple, if $e_0$ could be estimated directly. At the current research stage it is unclear of the wall effects on the particle distribution during fluidization and subsequent sedimentation.

16. Having removed the sand by water-suction from the column above the level of the top of the sample, drain the remaining water column through the draining hole in the lower part of the fluidization tube. Then proceed to remove the fluidization tube from the top of the split-mould.

17. Carefully mount the lower cell wall on the pedestal around the sample without causing any vibration.

18. Ensure that the loading frame is composed by mounting the upper horizontal cross-beam between the vertical spindles, where its vertical position can be adjusted manually.

19. Fix the top cap and the load cell, for measuring the axial load inside the triaxial cell, to the lower end of the long piston.

20. Pass the upper end of the long piston through the bearing in the top plate of the upper triaxial cell; mount them together to the horizontal beam of the loading frame in a way allowing the alignment of the top cap at the lower end of the vertical piston with respect to the central vertical axis of the sample. Make sure the top plate of the upper triaxial cell is also connected to the beam of the loading frame. At this point a gap with a height of approximately 0.216 m between the upper edge of the lower cell wall and the lower edge of the upper cell wall is obtained, which will provide access to the inner side of the lower cell wall for the further removal of the split-mould.

21. Ensure that the top cap and the tube for supplying the back pressure are already saturated with de-aired water. At this state of the experimental setup the top cap is located just above the top of the sample, its horizontal position aligned with respect to the vertical axis of the split-mould by moving the upper
support of the piston with respect to the cross-beam. When aligned properly, the vertical piston is rigidly clamped to the cross-beam.

22. Proceed to raise very slowly the vertically movable loading platform at the bottom of the loading frame, carrying the sample in the split-mould, while monitoring the load on top of the sample as measured by the load cell until a marginal increase can be observed, confirming contact between the top cap on the lower end of the piston and the top of the sample. The top cap should not touch the top of the split-mould while landing on top of the sample. It is expected that both the accuracy of the applied load cell and the lowest possible controlled raising rate of the pedestal may affect the marginal vertical effective stress at which “contact” is experienced.

23. Being the top cap in contact with the sample and the membrane still surrounding the top of the split-mould, proceed to move the top part of the membrane to the top cap and attach it by means of two O-rings. In principle, the annular space between the lower cell wall and the mould could be filled with de-aired water, which at that stage only supporting the mould with its hydrostatic pressure. However, during and after the later removal of the mould the significance of the presence of this cell water may increase, becoming a matter for further consideration.

24. Control of the sample in the mould will be done by the load cell and the back-pressure control via the top cap, controlling their respective quantities $\sigma_{v,lc}$ and $p_b$. For “free drainage” boundary condition, $p_b$ must be equal to atmospheric air pressure $p_a$. In case $p_a$ is used as reference pressure for water pressures, which is common in saturated soil mechanics, then $p_a$ is equal to 0. First the following pore water stress $p_w_{top}$ and vertical and horizontal intergranular stresses $\sigma_{zz, top}$ and $\sigma_{xx, top}$ are induced:

$$p_{w, top} = p_b$$  \[A.25\]

$$\sigma_{zz, top} = \sigma_{v,lc} - p_b$$  \[A.26\]

$$\sigma_{xx, top} = -p_b$$  \[A.27\]

In equation [A.27] the lateral support by cell water and mould are both considered to be zero. The correspondent isotropic and deviator intergranular stress measures at the top of the sample are

$$p^*_{top} = \frac{\sigma_{v,lc}}{3} - p_b$$  \[A.28\]

$$q^*_{top} = \sigma_{v,lc}$$  \[A.29\]

Pore water and vertical intergranular stresses at the bottom of the sample can be expressed by

$$p_{w, bottom} = p_b + \gamma_{water}\bar{h}_{sample}$$  \[A.30\]

$$\sigma_{zz, bottom} = \sigma_{v,lc} + \gamma_{buoy}\bar{h}_{sample} - p_b$$  \[A.31\]

It should be noted that Equation [A.31] is independent of the horizontal boundary condition of the sample, being it either due to the presence of the split-mould or after its later removal. The corresponding horizontal intergranular stress at the bottom of the sample will depend on the horizontal external support of the membrane. When the lubricated membrane is still laterally supported by the split-mould

$$\sigma_{xx, bottom, mould} = \sigma_{xx, bottom, mould} - \gamma_{water}\bar{h}_{sample} - p_b$$  \[A.32\]

For this condition, allowing practically zero lateral deformation, the horizontal intergranular stress $\sigma_{xx, bottom}$ will develop according to the constitutive response to the increase of $\sigma_{zz, bottom}$ according to Equation [A.31] and the condition of zero lateral strain. Such local reloading is practically the reverse of the previous unloading during the removal of the upper column layers. As a first approximation, for a small reloading increment the soil response will be approximately reversible as expressed by

$$\Delta\sigma_{xx} \approx K_e \Delta\sigma_{zz}$$  \[A.33\]
while for a large increase of the vertical intergranular stress, the irreversible $K_0$ reloading direction, in accordance with Equation [A.11] and described by

$$\Delta \sigma^*_x = K_0 \Delta \sigma^*_z$$  \hspace{1cm} [A.34]

will be approached.

As a very approximate transition point from reversible to irreversible behaviour the maximum intergranular stress point for the bottom of the sample, following sedimentation and consolidation according to Equations [A.10] and [A.11] could be considered, earlier indicated as the ‘control point’. The corresponding intergranular stress state in terms of isotropic and deviator intergranular stress measures are given by Equations [A.13] and [A.14]; this intergranular stress control point can be reached by deriving the corresponding expressions for the control variables $\sigma_{v,lc}$ and $p_b$, in which Equations [A.31] and [A.32] may incorporate the values of $\gamma_{buoy,h_{sample}}$, $\gamma_{water,h_{sample}}$, and the values of Equations [A.10] and [A.11] at the bottom of the sand column (and future sample) after sedimentation and consolidation, leading to

$$\sigma_{v,lc} - p_b \approx \gamma_{buoy}(z - h_{sample}); z = 0.75 \text{ m}$$  \hspace{1cm} [A.35]

$$\sigma_{xx}|_{bottom,mould} \approx \gamma_{buoy}zK_0 + \gamma_{water}h_{sample} + p_b$$  \hspace{1cm} [A.36]

Equations [A.35] and [A.36] do not enable both control variables to be calculated, due to the displacement-controlled boundary condition for the lateral boundary of the sample as enforced by the split-mould, which acts on the membrane with total lateral stress $\sigma_{xx}|_{bottom,mould}$. Furthermore for the upper sample surface maintaining “free drainage”, the back pressure remains constant. Nevertheless it is interesting to observe that Equation [A.35] shows that the height of the removed sand column determines the difference between the vertical total stress as measured by the load cell and applied to the top of the sample ($\sigma_{v,lc}$) and the back-pressure ($p_b$). In addition, the isotropic and deviator stress measures at the bottom of the sample before the removal of the split-mould can be expressed by

$$p^*|_{bottom} = \sigma_{v,lc} + (\gamma_{buoy} + \gamma_{water})h_{sample} - \sigma_{xx}|_{bottom,mould}$$  \hspace{1cm} [A.37]

$$q^*|_{bottom} = \sigma_{v,lc} + (\gamma_{buoy} + \gamma_{water})h_{sample} - \sigma_{xx}|_{bottom,mould}$$  \hspace{1cm} [A.38]

By substituting Equations [A.35] and [A.36] into [A.37] and [A.38] it is found that the maximum intergranular stress point is identical to those determined in Equations [A.13] and [A.14] for the sand column after sedimentation and consolidation.

25. Proceed to remove the split-mould by controlling the lateral boundary conditions in the sample while controlling the pore water stress. At the current state two methods could be used: removal of sample while being in contact with atmospheric air pressure and removal of sample while laterally supported by hydrostatic cell pressure. While the first one is simple, the second one might play the beneficial role of reducing the sample disturbance induced by the sample set-up procedure. Derivations of stress conditions estimated by Jager and Molenkamp (2015a) on each alternative will be presented in order to be accounted during the testing; however practical experience will determine whether the second procedure is not as beneficial as expected.
Removal of Split-Mould and Control of Sample surrounded by Cell Water

1. Proceed to fill the lower cell wall with water to the level of the top of the sample. Set the readings of the pore water pressure $p_{w \text{bot}}$ (taken via the bottom of the sample), the cell water pressure $p_c$ and the back-pressure $p_b$ (via the top of the sample) to zero.

2. Close the valves for pore water tube and the back-pressure, while monitoring the pore water pressure sensor reading $p_{w \text{bot}}$ and the back-pressure sensor reading $p_b$. Then removal of the split-mould can begin.

3. To facilitate the split-mould removal, first pour some water from above in between the membrane and the split-mould and give some time for this water to infiltrate and distribute over the contact surface. The process should be carefully performed in order to avoid any sample disturbance.

4. Proceed to remove both sides of the split mould slowly and carefully, allowing for cell water to enter and spread tangentially over the contact surface between the membrane and the mould, changing the lateral boundary condition from the radial displacement-controlled support by the mould to the radial pressure-controlled cell water pressure on the membrane. In other words, the previously unknown lateral total pressure distribution over the sample height applied by the mould will be changed to the hydrostatic cell water pressure distribution in the cell.

5. Estimate the stress changes produced by the change into the hydrostatic water pressure support provided by the cell water (i.e. $\sigma_{xx}|_{\text{bottom\_cell}} = \gamma_{\text{water}} h_{\text{sample}}$). Thus, for the boundary condition of horizontal equilibrium for the bottom of the sample, Equation [A.32] becomes

$$\sigma_{xx}|_{\text{bottom\_cell}} = -p_b$$ \hspace{1cm} [A.39]

while the intergranular stress $\sigma^*|_{\text{bottom}}$ estimated on Equation [A.31] remains the same. Proceed to estimate the required values for the control variables ($\sigma_{v\_lc}$ and $p_b$) by substituting the intergranular stresses of the maximum intergranular stress control point defined in Equations [A.10] and [A.11] and the buoyant weight (i.e. $\gamma_{\text{buoy}} h_{\text{sample}}$) in Equations [A.31] and [A.39]. These values for the pressure by load cell and the negative back-pressure (suction by pore water) are small but measurable and can be substituted into Equations [A.35] and [A.36] to estimate the internal horizontal pressure on the split-mould just before its removal; this internal horizontal pressure is counteracted by the external hydrostatic pressure on the mould provided by the cell water. Also by knowing the values of the control variables and the lateral hydrostatic cell pressure, the correspondent intergranular stress measures at the other relevant levels in the sample can be calculated, using Equations [A.26], [A.27], [A.28] and [A.29] for the top of the sample. For the passive transition depth $\Delta z|_{(6)}$ (derived from Equation [A.21] for $k=5$), [A.31] and [A.32] are rewritten so that $h_{\text{sample}}$ is replaced by $\Delta z|_{(6)}$ and the intergranular stresses and stresses measures are estimated by

$$\sigma^*_{zz}\mid_{\text{trans}} = \sigma_{v\_lc} - p_b + \gamma_{\text{buoy}} \Delta z|_{(6)}$$ \hspace{1cm} [A.40]

$$\sigma^*_{xx}\mid_{\text{trans}} = -p_b$$ \hspace{1cm} [A.41]

$$p^*|_{\text{trans}} = \frac{\sigma_{v\_lc} + \gamma_{\text{buoy}} \Delta z|_{(6)}}{3} - p_b$$ \hspace{1cm} [A.42]

$$q^*|_{\text{trans}} = \sigma_{v\_lc} + \gamma_{\text{buoy}} \Delta z|_{(6)}$$ \hspace{1cm} [A.43]

Profiles on the stress paths at the top, passive transition and bottom of the sample can be built from the previous steps, from the unloading until the split-mould removal. For this protocol and considering removal of split-mould surrounded by cell water, it can be concluded that the induced stress paths at the upper passive part of the sample due to unloading of the upper layers and the reloading due to the control variables may cause significant irreversible deformation, including contraction, which affects
the behaviour of the sample in subsequent undrained behaviour. During subsequent undrained deviator behaviour this upper part of the sample will not fully participate in irreversible deformation, including the volumetric contraction, which is at the basis of static liquefaction. In the remaining lower part of the sample the induced intergranular stress paths by initial unloading and subsequent reloading will be far less disturbing than in the upper part.

**Removal of Split-Mould and Control of Sample surrounded by Cell Air**

1. Set the readings of the pore water pressure $p^w_{bot}$ (taken via the bottom of the sample), the cell water pressure $p_c$ and the back-pressure $p_b$ (via the top of the sample) to zero.
2. Close the valves for pore water tube and the back-pressure, while monitoring the pore water pressure sensor reading $p^w_{bot}$ and the back-pressure sensor reading $p_b$. Then removal of the split-mould can begin.
3. To facilitate the split-mould removal, first pour some water from above in between the membrane and the split-mould and give some time for this water to infiltrate and distribute over the contact surface. The process should be carefully performed in order to avoid any sample disturbance.
4. Proceed to remove both sides of the split mould slowly and carefully. At this point the membrane will only be supported by the atmospheric air pressure (i.e. $\sigma_{xx}\mid_{\text{bottom cell}} = 0$).
5. Estimate the stress changes produced by the change into the atmospheric air pressure support. The expressions for the intergranular stresses at the bottom of the sample in this case will be expressed by Equation [A.31] and by

$$
\sigma_{xx}\mid_{\text{bottom cell}} = -p_b - \gamma_{\text{water}} h_{\text{sample}}
$$

[A.44]

Proceed to estimate the required values for the control variables ($\sigma_{v,lc}$ and $p_b$) by substituting the intergranular stresses of the maximum intergranular stress control point defined in Equations [A.10] and [A.11] and the buoyant weight (i.e. $\gamma_{\text{buoy}} h_{\text{sample}}$) in Equations [A.31] and [A.44]. These values for the pressure by load cell and the back-pressure are measurable and can be substituted into Equations [A.35] and [A.36] to estimate the internal horizontal pressure on the split-mould just before its removal. By knowing the values of the control variables and the lateral atmospheric pressure, the correspondent intergranular stress measures at the other relevant levels in the sample can be calculated, using Equations [A.26], [A.27], [A.28] and [A.29] for the top of the sample. For the passive transition depth $\Delta z\mid_6$, (derived from Equation [A.21] for $k=5$), [A.31] and [A.44] are rewritten so that $h_{\text{sample}}$ is replaced by $\Delta z\mid_6$ and the intergranular stresses and stresses measures are estimated by Equation [A.40] and

$$
\sigma_{xx}\mid_{\text{trans}} = -p_b - \gamma_{\text{water}} \Delta z\mid_6
$$

[A.45]

$$
p^*_{\text{trans}} = \frac{\sigma_{v,lc} + (\gamma_{\text{buoy}} - 2\gamma_{\text{water}}) \Delta z\mid_6}{3} - p_b
$$

[A.46]

$$
q^*_{\text{trans}} = \sigma_{v,lc} + (\gamma_{\text{buoy}} + \gamma_{\text{water}}) \Delta z\mid_6
$$

[A.47]

Profiles on the stress paths at the top, passive transition and bottom of the sample can be built from the previous steps, from the unloading until the split-mould removal. For this protocol and considering removal of split-mould surrounded by atmospheric air pressure, it can be concluded that the induced stress paths at the upper passive part of the sample due to unloading of the upper layers and the reloading due to the control variables may cause significant irreversible deformation, including contraction, which affects the behaviour of the sample in subsequent undrained behaviour. During subsequent undrained deviator behaviour this upper part of the sample will not participate appropriately.
in irreversible deformation, including the volumetric contraction, which is at the basis of static liquefaction.

**A.7. Protocol B: Sample Preparation by Fluidization, Column Sedimentation and 1-Dimensional Compression**

1. Align the upper overflow recipient and place it on top of the split mould, in such a way that the central axis of this set up coincides with the future loading axis.

2. Estimate the amount of sand to be used for a fluidized column height approximately 0.267 m, which will be estimated to reduce to about 0.20 m after sedimentation. This heights are estimated on the assumption that the liquefaction tank is of 2 meters and that the sample will be extracted at mid-height of the fluidized layer. Equations [A.1] to and including [A.3] will serve for this purpose.

3. From the amount of sand estimated using Equation [A.1], recalculate the values for porosity and void ratio—$n_{\text{fluidized}}$ and $e_{\text{fluidized}}$, respectively. Use Equation [A.1] while keeping $A_0$ and $\gamma_s$ as constants and change the height of sediment (0.20 m) to height of column (0.267 m).

4. Estimate the amount of water in the fluidized column of approximately 0.267 m and cross-sectional area of $A_0$ using Equation [A.4].

$$M_{\text{water}} = n_{\text{water}} h_{\text{sediment}} A_0 \quad \text{[A.4]}$$

5. Estimate the time for flushing the whole mass of pore water, $M_{\text{water}}$, with a discharge velocity $v_{\text{fluidized}}$ of $10^{-3}$ m/s, by using Equation [A.5]. This velocity is based on the fluidizing discharge estimated for the liquefaction tank (Jager and Molenkamp, 2015a). The estimated time accounts for only one flush. It is possible to calculate the amount of time required for flushing the sample with de-aired water 10 times by just multiplying the value obtained in [A.5] by ten. This volume is necessary in case additional flushing is required for washing out fines or ensuring major saturation.

6. Estimated the pore water stress at the bottom of the column during fluidization, using Equation [A.6]. At this set up measurements with standpipes is not possible.

7. If distribution of porosity is to be calculated, knowing the vertical distribution of the pore water stress along the column, Equation [A.7] can be used, in which $z$ represents the vertical upward direction. Vertical equilibrium of the stationary fluidizing pore fluid flow is represented by Equation [A.8].

8. Having the split-mould and overflow prepared, open the back pressure valve and let de-aired water to fill the mould. This is in order to allow sand to be poured into the column according to the mass estimated on [A.1] and to avoid capillary pressure to limit saturation. The sand could be poured first and then fluidize the mass of water estimated on [A.4]; but then more flushing would be required in order to break any bonds between sand particles.

9. Close the back pressure line and set the peristaltic pump which will be used for fluidization so that its $v_{\text{fluidized}}$ is equal to $10^{-3}$ m/s at the base of the column. Preliminary calculations of maximum discharge $Q$ of pore water through the sample for fluidization were of 0.47 l/min (Jager and Molenkamp, 2015a). Open again the back pressure line and allow water to flow upwards during fluidization, while fines are being washed out of the mould, as well as the amount of water being flushed into the mould. Note that more flushing might be needed to ensure full fluidization and major sample saturation.
10. To start the sedimentation process, turn off the pump and close the back pressure valve and allow the sand particles to settle. Estimate the hydrostatic pore water stress distribution by calculating its value at the bottom of the column using Equation [A.9].

11. Remove the overflow and measure the height of the sand column after sedimentation, which should be around 0.20 m with a homogeneous porosity of 0.56 (see Jager and Molenkamp, 2015a). Estimate the stress state in the sample using Equations [A.10], [A.11] and [A.12]. Note that for applying this equations, $z$ is equal to the sample height. For the present project, the preliminary estimate of $\phi_{\text{crit}}$ is of about 30° (Jager and Molenkamp, 2015a).

12. Calculate the isotropic and deviator stress measures $p^*$ and $q^*$ after sedimentation and the stress ratio $R_0$ using Equations [A.13], [A.14] and [A.15].

13. Carefully mount the lower cell wall on the pedestal around the sample without causing any vibration.

14. Ensure that the loading frame is composed by mounting the upper horizontal cross-beam between the vertical spindles, where its vertical position can be adjusted manually.

15. Fix the top cap and the load cell, for measuring the axial load inside the triaxial cell, to the lower end of the long piston.

16. Pass the upper end of the long piston through the bearing in the top plate of the upper triaxial cell; mount them together to the horizontal beam of the loading frame in a way allowing the alignment of the top cap at the lower end of the vertical piston with respect to the central vertical axis of the sample. Make sure the top plate of the upper triaxial cell is also connected to the beam of the loading frame. At this point a gap with a height of approximately 0.216 m between the upper edge of the lower cell wall and the lower edge of the upper cell wall is obtained, which will provide access to the inner side of the lower cell wall for the further removal of the split-mould.

17. Ensure that the top cap and the tube for supplying the back pressure are already saturated with de-aired water. At this state of the experimental setup the top cap is located just above the top of the sample, containing a layer of hydrostatic water and the fresh sediment composing the extremely loose future sand sample. The horizontal position of the top cap is aligned with respect to the vertical axis of the split-mould by moving the upper support of the piston with respect to the cross-beam. When aligned properly, the vertical piston is rigidly clamped to the cross-beam.

18. Proceed to raise very slowly the vertically movable loading platform at the bottom of the loading frame, carrying the sample in the split-mould, while monitoring the load on top of the sample as measured by the load cell until a marginal increase can be observed, confirming contact between the top cap on the lower end of the piston and the top of the sample. The top cap should enter the top of the split-mould without experiencing any additional resistance until landing on the top of the sample; this free movement can be facilitated by a top cap whose external diameter is marginally smaller (around 1 mm) than the diameter of the sample, defined by the internal diameter of the split-mould with internal lubrication and membrane.

19. Being the top cap in contact with the sample and the membrane still surrounding the top of the split-mould, proceed to move the top part of the membrane to the top cap and attach it by means of two O-rings. In principle, the annular space between the lower cell wall and the mould could be filled with de-aired water, which at that stage only supporting the mould with its hydrostatic pressure. However, during and after the later removal of the mould the significance of the presence of this cell water may increase, becoming a matter for further consideration.
20. Control of the sample in the mould will be done by the load cell and the back-pressure control via the top cap, controlling their respective quantities $\sigma_{v,lc}$ and $p_b$. For “free drainage” boundary condition, $p_b$ must be equal to atmospheric air pressure $p_a$. In case $p_a$ is used as reference pressure for water pressures, which is common in saturated soil mechanics, then $p_a$ is equal to 0. First the following pore water stress $p_w|_{\text{top}}$ and vertical and horizontal intergranular stresses $\sigma_{zz}|_{\text{top}}$ and $\sigma_{xx}|_{\text{top}}$ are induced; Equations [A.25], [A.26] and [A.27] help define the intergranular stresses, while Equations [A.28] and [A.29] calculate the isotropic and deviator stress measures. The intergranular stress state at the bottom of the sample, defined by Equations [A.10] and [A.11], will be the same for those defined for Protocol A. For reaching this stress state application of 1-D compression will be required. It is estimated that the fresh sediment height will be somewhere between the height during fluidization and the final sample height. Therefore, there will be enough space for the sample to be compressed to its final height before testing and yet reaching these desired stress states. The magnitude of the 1-D compression will have to be determined experimentally.

21. Proceed to remove the split-mould by controlling the lateral boundary conditions in the sample while controlling the pore water stress. At the current state two methods could be used: removal of sample while being in contact with atmospheric air pressure and removal of sample while laterally supported by hydrostatic cell pressure. While the first one is simple, the second one might play the beneficial role of reducing the sample disturbance induced by the sample set-up procedure. Derivations of stress conditions estimated by Jager and Molenkamp (2015a) on each alternative will be presented in order to be accounted during the testing; however practical experience will determine whether the second procedure is not as beneficial as expected.

**Removal of Split-Mould and Control of Sample surrounded by Cell Water**

1. Proceed to fill the lower cell wall with water to the level of the top of the sample. Set the readings of the pore water pressure $p^w_{\text{bot}}$ (taken via the bottom of the sample), the cell water pressure $p_c$ and the back-pressure $p_b$ (via the top of the sample) to zero.

2. Close the valves for pore water tube and the back-pressure, while monitoring the pore water pressure sensor reading $p^w_{\text{bot}}$ and the back-pressure sensor reading $p_b$. Then removal of the split-mould can begin.

3. To facilitate the split-mould removal, first pour some water from above in between the membrane and the split-moult and give some time for this water to infiltrate and distribute over the contact surface. The process should be carefully performed in order to avoid any sample disturbance.

4. Proceed to remove both sides of the split mould slowly and carefully, allowing for cell water to enter and spread tangentially over the contact surface between the membrane and the mould, changing the lateral boundary condition from the radial displacement-controlled support by the mould to the radial pressure-controlled cell water pressure on the membrane. In other words, the previously unknown lateral total pressure distribution over the sample height applied by the mould will be changed to the hydrostatic cell water pressure distribution in the cell.

5. Estimate the stress changes produced by the change into the hydrostatic water pressure support provided by the cell water (i.e. $\sigma_{xx}|_{\text{bottom\_cell}} = \gamma_{\text{water}}h_{\text{sample}}$). Thus, for the boundary condition of horizontal equilibrium for the bottom of the sample, Equation [A.32] becomes Equation [A.39], while the intergranular stress $\sigma_{zz}|_{\text{bottom}}$ estimated on Equation [A.31] remains the same. Proceed to estimate
the required values for the control variables (\(\sigma_{v,lc}\) and \(p_b\)) by substituting the intergranular stresses of the maximum intergranular stress control point defined in Equations [A.10] and [A.11] and the buoyant weight (i.e. \(\gamma_{buoy}h_{sample}\)) in Equations [A.31] and [A.39]. These values for the pressure by load cell and the negative back-pressure (suction by pore water) are small but measurable and can be substituted into Equations [A.35] and [A.36] to estimate the internal horizontal pressure on the split-mould just before its removal; this internal horizontal pressure is counteracted by the external hydrostatic pressure on the mould provided by the cell water. Additionally the values for the control variables can be substituted in Equations [A.26] and [A.27] for estimating the vertical and horizontal intergranular stresses and the correspondent isotropic and deviator stress measures at the top of the sample. Profiles on the stress paths at the top and bottom of the sample can be built from the previous steps, from the 1-D compression until the split-mould removal.

For this protocol and considering removal of split-mould surrounded by cell water, the removal of the mould and the application of the hydrostatic cell water reduced the induced intergranular stress path to zero, as the resulting intergranular stress point coincides with the intergranular stress point reached during 1-D compression, thus representing minimum disturbance effects at the bottom of the sample. However, at the top of the sample the mobilized friction decreases during mould removal and the induced deformation may be more irreversible and contractive; therefore at the upper half of the sample the irreversible deformations will adversely affect the behaviour of the sample in subsequent undrained behaviour. This implies that during subsequent undrained deviator behaviour this upper part of the sample will not fully participate in irreversible deformation, including the volumetric contraction, which is at the basis of static liquefaction. In the lower half the induced intergranular stress paths will remain marginal and this half of the sample can be expected to remain looser than the more contracted upper half.

**Removal of Split-Mould and Control of Sample surrounded by Cell Air**

1. Set the readings of the pore water pressure \(p_{w_{bot}}\) (taken via the bottom of the sample), the cell water pressure \(p_c\) and the back-pressure \(p_b\) (via the top of the sample) to zero.
2. Close the valves for pore water tube and the back-pressure, while monitoring the pore water pressure sensor reading \(p_{w_{bot}}\) and the back-pressure sensor reading \(p_b\). Then removal of the split-mould can begin.
3. To facilitate the split-mould removal, first pour some water from above in between the membrane and the split-moult and give some time for this water to infiltrate and distribute over the contact surface. The process should be carefully performed in order to avoid any sample disturbance.
4. Proceed to remove both sides of the split mould slowly and carefully. At this point the membrane will only be supported by the atmospheric air pressure (i.e. \(\sigma_{xx}|_{bottom\_cell} = 0\)).
6. Estimate the stress changes produced by the change into the atmospheric air pressure support. The expressions for the intergranular stresses at the bottom of the sample in this case will be expressed by Equations [A.31] and [A.44]. Proceed to estimate the required values for the control variables (\(\sigma_{v,lc}\) and \(p_b\)) by substituting the intergranular stresses of the maximum intergranular stress control point defined in Equations [A.10] and [A.11] and the buoyant weight (i.e. \(\gamma_{buoy}h_{sample}\)) in Equations [A.31] and [A.44]. These values for the pressure by load cell and the back-pressure are measurable and can be substituted into Equations [A.35] and [A.36] to estimate the internal horizontal pressure on the split-mould.
mould just before its removal. For the top of the sample, the values for the control variables can be used in Equations [A.26] and [A.27] to find the vertical and horizontal intergranular stresses, which can be used to estimate the correspondent isotropic and deviator stress measures. Profiles on the stress paths at the top and bottom of the sample can be built from the previous steps, from the 1-D compression until the split-mould removal.
For this protocol and considering removal of split-mould surrounded by atmospheric air pressure, the removal of the mould in air reduced the induced intergranular stress path to zero, as the resulting intergranular stress point coincides with the intergranular stress point reached during 1-D compression, thus representing minimum disturbance effects at the bottom of the sample. However, at the top of the sample the mobilized friction decreases during mould removal and the induced deformation may be more irreversible and severely contractive; therefore at the upper half of the sample the irreversible deformations will deteriorate the sample in subsequent undrained behaviour. This implies that during subsequent undrained deviator behaviour this upper part of the sample will not participate appropriately in irreversible deformation, including the volumetric contraction, which is at the basis of static liquefaction.

A.8. **Initiation of Displacement-Controlled Undrained Triaxial Tests at Small Intergranular Stresses**

1. Proceed to lower the combined upper triaxial cell with the top plate, manually along the piston, towards the lower cell wall. After achieving a proper landing of the upper cell wall on the lower cell wall the top plate of the triaxial apparatus is connected firmly to the pedestal by means of 6 tie bars.
2. Proceed to fill in the triaxial cell with de-aired water, ensuring that all the air is displaced through the bleed plug. Fill the cell as quickly as possible but without allowing turbulence which could aerate the water. A layer of castor oil may be introduced on top of the water to act as a piston lubricant and to reduce leakage past the piston.
3. Keep the air bleed plug open until the cell is ready to be pressurized in order to maintain the pressure at atmospheric.
4. After closing the air bleed plug, apply the first cell pressure increment as soon as possible, as required by the saturation procedure.
5. The cell pressure, the back-pressure and the total axial load can be changed for the following two aims: (i) To increase the degree of saturation of the pore water by increasing the back-pressure to about 300 kPa while keeping the intergranular stress constant, and (ii) to arrive at any specified intergranular stress state during drained deformation and control for starting the subsequent undrained triaxial test. This is similar to the saturation process as indicated by the British Standard BS 1377: Part 8: 1990. However, for minimizing the effect of further sample disturbance it is recommended to start the undrained triaxial test in compression and/or extension at the current achieved intergranular stress distribution in the sample, which depends on the intergranular stress according to the control point at the bottom of the sample; the intergranular stress state at the top of the sample will depend on the chosen protocol and sample preparation control described in sections A.6 and A.7 of this appendix.
6. Having achieved a degree of saturation at the sample characterized by a B value of at least 0.95 and the sample reached a state of effective stresses required for carrying out the test, the undrained triaxial
test can be done according to British Standard BS 1377: Part 8: 1990 (or its equivalent). Thus, no description on the remaining steps will be given.

In conventional triaxial testing it is not common to quantify the stress paths along the sample during the process of preparation and sample control. However, for maximizing the insight into the accuracy of the measured constitutive behaviour during testing it is recommended to determine the intergranular stress paths for both the top and bottom of the sample, being the mean intergranular stress path the average of those at the top and bottom.

In addition, due to the large magnitude of the permeability of sand the pore pressure generation is due to the average volume change of the sample, while the corresponding deviator deformation is usually only measured as an average.

Finally, in conventional triaxial testing saturation is achieved with an increase in the cell pressure and the back-pressure. However, for loose samples prone to densification, a possibly novel method proposed by de Jager and Molenkamp (2015) has been proposed which accounts for the flexibility of the pore water and its effect on undrained pore pressure generation. At this stage no physical testing has been made on this method, although a numerical model worked by the authors showed to be suitable for estimating the degree of saturation. This might change the saturation process usually performed during triaxial testing, but more research on this is needed.
Appendix B

Determination of Maximum and Minimum Void Ratios using the Japanese Geotechnical Society (JGS) Method

In this section the procedure for determining the minimum and maximum void ratios by the JGS procedures will be described, as described by the Japanese Geotechnical Society (1996) and accounting the observations of Muszynski (2006). This method was selected not only because of its simplicity but also for its suitability for sands with poor grading (i.e. uniform grain size) and a loose to medium packing, its results being comparable to those from conventional methods.

B.1. Measurement and Determination of Void Ratio and Bulk Density Limits

In general, there are different procedures in which the minimum and maximum void ratio can be measured, commonly done according to the ASTM and EN standards. For determining the maximum densities methods such as the vibrating table (ASTM D4253-00) and the modified Proctor test (ASTM D1557-00, EN 13286-2) are used, while for minimum density the norms ASTM D4254 and EN 1097-3 are chosen.

While there are many available methods, Muszynski (2006) suggests the use of simplified methods based on conventional ones which use smaller specimen sizes and requires less time to obtain densities. The advantage of using this methods is that it requires less time and material to perform the tests, which allows for a major amount of tests to be performed within a set time and with the same principles as those on the
conventional tests. This simplified methods can be applied especially for dry materials characterized by a uniform grading and a loose to medium packing.

**B.2. Tools and Initial Setup**

The JGS Method involves the determination of the limit densities using a mould with an inner diameter of 6 cm and an internal depth of 4 cm, which results in a volume of 113.1 cm³. The mould is made of stainless steel with a thickness of 8 mm and it includes a collar for pouring material above the height of the mould.

Before beginning the test it is important to measure the internal space of the mould and to weight it. At the same time it is important to define the dry sand’s specific gravity, either by estimating it based on its mineralogy or by performing a pycnometer test. For this project a gas pycnometer machine was used.

**B.3. Minimum Void Ratio Procedure**

1. Assemble the mould by placing the top collar on top of the mould.
2. Use a funnel to pour a sand layer at a height slightly greater than 1/5 the inner height of the mould (not taking into account the collar). During the operation, segregation is to be avoided.
3. After completing each layer, tap the midpoint of the mould with a 70 g, 20 cm long dowel, while rotating the mould 90° for every 10 taps. Each layer is compacted by approximately 100 taps.
4. Repeat Steps 2 and 3 until the mould is completely filled with sand.
5. After completing the last layer, remove the collar and strike the top off. Fill any small divots that might be formed during levelling of the specimen.
6. Remove any excess sand from the outside of the mould and proceed to determine the mass of the specimen by subtracting the mould mass from the total mass of the sand and mould after compaction.
7. With the mass and the volume determine the density correspondent to the minimum void ratio: the maximum dry density.
8. Estimate the minimum void ratio using the following equation

\[
e = \frac{G_s \rho_{water}}{\rho_{dry}} - 1
\]  

[B.1]

in which \(\rho_{dry}\) is the dry density of the sand, \(G_s\) its specific gravity and \(e\) its void ratio.
9. Repeat the procedure for several other trials to obtain an average or another measure of statistical representativeness.

**B.3. Maximum Void Ratio Procedure**

1. Place the mould on top of the work surface without the collar attached.
2. Use a funnel with an opening of approximately 12 mm to gently place the oven dry sand in the mould, maintaining a minimal height of fall between the tip of the funnel and the conical sand pile by slowly raising the funnel during the procedure.
3. The funnel is pulled straight up during the procedure, allowing the sand to overfill the mould by a small amount. Segregation is to be avoided during the procedure.
4. When the mould is filled with sand, strike the top off with a straight edge. This must be done gently in order to avoid the application of vibration or compaction to the specimen which may result in densification.

5. Remove any excess sand from the outside of the mould and proceed to determine the mass of the specimen by subtracting the mould mass from the total mass of the sand and mould after the sand placement.

6. With the mass and the volume determine the density correspondent to the maximum void ratio: the minimum dry density.

7. Estimate the minimum void ratio using the following Equation [B.1], which is also valid for this case.

8. Repeat the procedure for several other trials to obtain an average or another measure of statistical representativeness.