On The Role of Volcanic Material in Submarine Landslide Initiation Processes

Doctoral Thesis

Submitted for the doctoral degree in natural sciences at the Faculty of Geosciences of Bremen University

Zur Erlangung des Doktorgrades der Naturwissenschaften im Fachbereich Geowissenschaften der Universität Bremen

by

Gauvain Wiemer

Bremen, Februar 2014
Abstract

Along active continental margins earthquake shaking was singled out as one of the most important triggering mechanisms for submarine landslides, which may trigger a tsunami wave and have tremendous impact on coastal communities and infrastructure. Seismic wave propagation through a slope-sediment body leads to cyclic loading in addition to static, gravitational loading of the sediment. It is known that seismic loading may lead to sudden strength loss in loosely deposited granular soils such as sands or silts due to liquefaction. Liquefaction of granular soils occurs when excess pore pressure induced by seismic shaking reaches the in situ effective overburden stress. Volcanic arcs provide a source of granular sediment along active margins through ejection of volcaniclastica. Volcanic ash is of sand to silt size, such that it is presumed to act preferentially as gliding plane in submarine landslides. However, volcanic ash differs significantly in its geotechnical properties compared to commonly studied sands and silts.

The aim of this dissertation is to tackle the question to what extent physical properties of volcanic sands and silts affect their response to gravitational and seismic loading in comparison to common sands and silts. In a multi-methodological, geotechnical approach several volcanic materials are characterized and set into contrast with common sand and silt. XRD and SEM analysis is applied in order to determine differences in mineralogy as well as particle shape and texture between both materials. Advanced laboratory shear experiments (direct shear and triaxial shear) were conducted with the purpose to investigate the shear behavior of volcanic material in contrast to common sands and silts. In generic laboratory studies it is shown that properties like angularity, roughness and particle strength (i.e. crushability) play a key role in sediment stability and majorly determine the shear behavior of volcanic ash at effective stress $< 0.5$ MPa. Under drained monotonic shear conditions, angularity and roughness dominate the shear behavior and lead to an increase in strength compared to common sands. Under undrained monotonic shear conditions frictional resistance is dominated by crushability whereby angularity of non-crushing soils still leads to an increase in strength. Undrained cyclic shear experiments revealed that
Abstract
cyclic shear stress may be too low in amplitude to produce prominent crushing which is why angularity and roughness dominate the shear behavior and lead to a significant increase in shear resistance under cyclic loading. If cyclic shaking exceeds a certain strength volcanic ash may eventually liquefy and undergo severe strength loss during failure.
When deposited in marine or lacustrine environments volcanic glass is subject to rapid weathering processes which may reduce frictional resistance due to the transformation into the mechanically weak smectitemineral. To address this problem a total of \( \sim 120 \) direct shear experiments on \( \sim 30 \) samples demonstrate that angularity and roughness of volcanic material maintain the high frictional resistance even at an elevated alteration stage (\( \sim 20\% \) clay). However, with completion of transformation into smectite volcanic ash may eventually transform from a high friction resistant granular sediment into a low friction resistant cohesive material (smectite).
Furthermore, a case study was conducted on a subaqueous landslide located in the earthquake prone area of South-Central Chile. Recovered sediment cores show volcanic fall-out ash a few mm from the basal shear plane of a landslide that occurred \( \sim 6 \) k years ago. In a highly detailed analysis that gathers seismic profile data with CPT data and advanced geotechnical laboratory shear experiments, it is shown that the sole presence of volcanic ash was not sufficient to result in liquefaction failure due to earthquake shaking. Other local and sediment physical preconditions were necessary to trigger slope failure above that exact fall-out tephra deposit. Fluid escape structures observed in seismic profiles give evidence for focused fluid flow and \textit{insitu} excess pore pressure, which is also observed in dissipation tests of Free-Fall Cone Penetration Testing data (CPT). The volcanic ash located at the base of the slide constitutes the most finely grained ash of the analyzed sediment sequence which makes it more susceptible to liquefaction as shown in laboratory experimentation. The ash itself was not involved in the slide mass, but served locally as effective stress reducer due to excess pore pressure build up within that granular sediment layer.
In essence, this dissertation attest relatively high mechanical strength to non-crushing angular volcanic ash subjected to low effective stresses. Crushability under undrained condition leads to
enhanced excess pore pressure and inherent reduction of effective stresses. Earthquake shaking of volcanic fall-out ash deposits may lead to liquefaction whereby the earthquake intensity necessary to do so tends to be higher than in common sands. Alteration with authigenic clay mineral formation and inherent mechanical weakening seems to be irrelevant for static slope failure initiation because the stage of alteration reached at depths concerned by slope failure is insufficient for a significant reduction in static shear strength.
Zusammenfassung


führen zu einer Erhöhung des Scherwiderstandes im Vergleich zu üblichen Sanden. Unter statisch-undränierten Scherbedingungen führt Kornbruch zur Scherwiderstandsverringerung durch den zusätzlichen Aufbau von Porenwasserüberdruck und einhergehender Verringerung der effektiven Spannung, wobei feste, angulare Partikel den Scherwiderstand wie im dränierten Fall maßgeblich erhöhen. Unter zyklisch-undränierten Scherbedingungen scheinen Scherspannungsamplituden zu gering zu sein um zu Kornbruch zu führen, was wiederum dazu führt, dass Angularität und Rauhigkeit maßgeblich für die Erhöhung des Scherwiderstandes verantwortlich sind. Wenn die Intensität seismischer Belastung groß genug ist können vulkanische Aschen liquifizieren und ihre Scherfestigkeit plötzlich verlieren. Wenn vulkanisches Glas subaquatisch abgelagert wird, dann ist es rascher authigenen Tonmineralbildung ausgesetzt, die durch die Transformation in das mechanisch schwache Mineral Smektit dazu führen könnte, dass die statische Scherfestigkeit verringert wird. Um dieses Frage anzugehen wurde anhand von insgesamt 120 direkten Scherversuchen an 30 Probenmaterialien gezeigt, dass Angularität und Rauhigkeit zum Erhalt von hoher Scherfestigkeit bei erhöhtem Tonmineralgehalt (~20%) führen. Jedoch könnte sich vulkanische Asche, durch vollständige Umwandlung in Smektit, von einem granularen, reibungsresistenten Material in ein kohäsives, schwaches Sediment transformieren.

Außerdem wurde im Rahmen einer Fallstudie eine subaquatische Hangrutschung untersucht, die sich im häufig von starken Erdbeben heimgesuchten Süd-Central Chile befindet. Sedimentkerne zeigen, dass sich eine vulkanische Aschelage nur wenige mm unterhalb der unteren Gleitfläche der Rutschung befindet, die sich vor ~6 k Jahren ereignet haben muss. In einer sehr detaillierten Studie, die seismische Profile mit CPT Daten und geotechnischen Scherversuchen vereint, wird gezeigt, dass die alleinige Präsenz einer Aschelage nicht ausreichend gewesen sein kann um im Falle eines starken Bebens zur Liquifizierung zu führen. Andere lokale und sediment-physikalische Vorraussetzungen mussten gegeben sein um in der Kombination zu einer Schwächzone zu führen, die kurz oberhalb genau jener Asche lag und nicht einer anderen innerhalb der Sedimentabfolge. 'Fluid escape' Strukturen, erkennbar aus seismischen Profilen liefern den Beweis für Fluidtransort sowie Porenwasserüberdruck, welcher sich ebenfalls aus CPT test ergibt. Die vulkanische Asche, die sich
Zusammenfassung

nahe der Gleitfläche befindet weist die feinste Körnung aller in der Sedimentabfolge vorgefundenen Aschen auf, und neigt daher eher zum Liquifizieren als andere im Kern beschriebenen Aschen, wie in Laborexperimenten gezeigt wird. Die Aschelage selbst war nicht Teil der mobilisierten Masse, aber trug lokal zur Verringerung der effektiven Spannung durch Porenwasserüberdruckaufbau bei.

Im Wesentlichen wird in dieser Dissertation vulkanischen Aschen mit festen Körnern eine hohe Scherfestigkeit bei geringer effektiver Spannung (< 0.5 MPa) zugeordnet. Kornbruch unter undränierter Scherbedingung führt verstärkt zum Aufbau von Porenwasserüberdruck, und verringert daher die Scherfestigkeit. Erdbeben können zur Liquifizierung von vulkanischen Ascheablagerungen führen; tendentiell muss das Beben dafür aber ein höhere Intensität aufweisen als zum Liquifizieren von gewöhnlichen Sanden nützlich ist. Alteration und authiger Tonmineralbildung in vulkanischen Aschen scheint irrelevant für die Initiierung subaquatischer Rutschungen, da der Alterierungsgrad, in von Rutschungen betroffenen Tiefen, nicht fortgeschritten genug ist, als dass die statische Scherfestigkeit signifikant reduziert würde.
Contents

1 Introduction ........................................ 11
   1.1 Motivation ..................................... 11
   1.2 Outline ...................................... 14

2 Submarine landslides and sediment failure processes .... 15

3 Sediment end-member shear behavior ............. 23
   3.1 Stress Terminology in Triaxial Testing .............. 23
   3.2 Granular sediment ................................ 26
   3.3 Cohesive sediment ................................ 38

4 Sediment shear strength determination ............ 45
   4.1 In-situ ........................................ 45
   4.2 On-site ......................................... 48
   4.3 Lab.-based ..................................... 50

5 Validation of the MARUM DTTD performance ... 59
   5.1 Static and Cyclic Shear Strength of Cohesive and Non-cohesive
       Sediments ........................................ 59

6 Laboratory based studies ........................ 73
   6.1 On the Mechanical Strength of Volcanic Material ...... 73
   6.2 Altered volcanic ash deposits as potential slope failure planes? 99

7 Case study ........................................ 117
   7.1 Subaqueous landslide in earthquake-prone South-Central Chile:
       The role of sediment composition and its behaviour under dy-
       namic loading conditions on slope failure initiation .... 117
   7.2 The influence of excess pore pressure, fluid flow and deposi-
       tional patterns on subaquatic slope stability: a detailed case
       study of Lake Villarrica (South-Central Chile) ......... 149
Chapter 1

Introduction

1.1 Motivation

Submarine landslides are possibly accompanied by dramatic scenarios such as tsunami waves, destruction of offshore and onshore infrastructure and loss of human lives [Masson et al., 2006]. One of the outstanding scientific questions within this research area concerns the role of volcanic material in submarine landslide initiation along active margins:

Volcanic arcs provide a source for granular sediment deposits (i.e. volcanic fall-out ash) along active continental margins [Fisher and Schmincke, 1984] which are frequently subjected to seismic shaking. The global coincidence of seismic and volcanic activity is illustrated in Figure 1.1. Relatively recently the hypothesis has been formulated that volcanic material may preferentially serve as basal shear plane for submarine landslides on active margins [Harders et al., 2010]. Volcanic material comprises sand and silt in terms of grain size which is why it is thought that it may follow the accepted geotechnical concepts of common sand. It has been shown that especially loosely deposited granular sediments such as sands and silts tend to sudden strength loss when stroke by seismic shaking [Castro, 1969]. This phenomenon is known as liquefaction and may lead ad hoc to the formation of a basal sliding plane for submarine slides [Sultan et al., 2004].

However, volcanic glass is known to alter rapidly in contact with water and forms authigenic clay minerals [Fisher and Schmincke, 1984]. Accordingly, volcanic ashes are commonly classified as residual soils. Figure 1.2 sketches the alteration of a volcanic glass shard in four different stages. It is shown that the formerly granular particle progressively alters to a clay dominated conglomerate. Thus, in terms of grain size, volcanic ashes may be deposited as non-cohesive, liquefiable sand and progressively alter to a
cohesive, clay. Clay dominated soils behave fundamentally different from granular soils under monotonic or cyclic loading conditions. The cohesive clayey materials are more resistant to seismic shaking [Seed et al., 1983], but tend to produce progressive failure even under static, gravitational loading conditions because of their overall intrinsic weakness (low shear strength and friction coefficient) [Leroueil and Hight, 2003].

This doctoral thesis tackles the question of whether or not volcanic ashes may play a major role in submarine landslide initiation on active continental margins. In a multi-methodological geotechnical approach the shear behavior of diverse fresh and altered volcanic materials was investigated. In generic laboratory studies several dozen direct shear and undrained monotonic and cyclic triaxial shear experiments were conducted at effective normal stress < 1 MPa. The objectives were to:

i) determine the shear behavior of fresh volcanic material and investigate to what extent existing geotechnical concepts on common sand and silt could be applied or validated

ii) quantify the effect of authigenic clay mineral formation in volcanic material on its shear behavior.

Furthermore, a detailed case study was conducted on a subaqueous landslide in a highly earthquake prone geological setting (South-Central...
1.1. MOTIVATION

Chile). Parts of the slide were initiated above a volcanic fall-out ash layer which makes the investigated slide to an ideal target for the third objective which was the:

iii) *in situ* investigation of volcanic material and its role in subaqueous sliding.

For that purpose about thirty MARUM Free Fall CPTU deployments were carried out in the study area. The deployments allowed for the determination of *in situ* effective stress conditions and *in situ* undrained shear strength of the sediment. Moreover, a detailed onshore laboratory based investigation on the geotechnical properties of the failure-involved sediment was undertaken to build on the *in situ* tests. Undrained cyclic triaxial shear experiments were conducted to the failure-plane volcanic ash to investigate seismic shaking as a potential triggering mechanism.

**Figure 1.2:** Schematic illustration of progressive alteration and authigenic clay mineral formation in volcanic glass (after [Brey and Schmincke, 1980])
1.2 Outline

Chapter 1 peruses the target of introducing the scientific question that was tackled in the course of this thesis, and how the work is structured into individual chapters.

Chapter 2 outlines basic background information on the initiation processes of submarine landslides. Furthermore, sediment failure processes are introduced with the sediments end-members sand and clay, i.e. granular and cohesive sediment. The phenomenon of liquefaction of granular soils and progressive failure of cohesive soils are illustrated from a geological descriptive perspective.

Chapter 3 reviews the phenomenon of liquefaction and progressive failure from a geotechnical point of view. The purpose is to integrate the shear behavior of volcanic ash in existing geotechnical concepts as far as possible.

Chapter 4 introduces the geotechnical devices that have been used to determine the shear strength and shear behavior of volcanic material both, in situ and in the laboratory. A focus is set on the MARUM Dynamic Triaxial Testing Device that builds a center piece of this thesis.

Chapter 5 contains a generic laboratory based study on the mechanical strength of the sediment end-members, sand and clay conducted with the MARUM Dynamic Triaxial Testing Device (DTTD). The data presented therein is the first published data obtained with this highly sophisticated device and served as validation study.

Chapter 6 presents the laboratory based studies on volcanic material.

Chapter 7 presents a case study where volcanic material is involved in a subaqueous landslide.

Chapter 8 serves as a summary recapitulating and accentuating the main aspects of the thesis. Discussions on each single aspect are included in each manuscript.

Chapter 9 points out the ongoing study on the shear behavior of diatomaceous ooze.

Finally Chapter 12 shows an Appendix including additional unpublished data generated in the courses of the dissertation.
Chapter 2

Submarine landslides and sediment failure processes

This chapter first introduces some facts on submarine landslides and their triggering mechanism. Failure mechanisms and shear behavior of marine sediments are subsequently tackled from a geological, more descriptive perspective.

Submarine landslides may occur on any submarine slope worldwide. They occur when downslope driving forces acting on the material composing the seafloor are greater than the forces acting to resist major deformations [Hampton et al.; Masson et al., 2006]. Quantifying the balance between resisting and driving forces is one of the major aims in submarine slope failure studies. The ratio of driving forces and resisting forces is called the Factor of Safety (FoS) (Fig. 2.1a). The FoS is a measure for the stability of slope sediment. As long as FoS > 1, the sediment can be considered as stable, whereas FoS < 1 indicates driving forces exceeding resisting forces and inherent sediment failure [Morgenstern and Price, 1965]. The monotonic driving force acting upon the slope sediment is gravity. Resisting forces are mainly determined by the sediment’s shear strength. The shear strength of soil is proportional to the effective stresses that the soil is subjected to. Effective stress is calculated from the difference of two parameters, total stress (σ) and pore pressure (u) [Terzaghi, 1925]

\[ \sigma' = \sigma - u \]  

(2.1)

Soils are mostly fully saturated in the subaqueous realm [Leroueil and Hight, 2003] and any change in total stress may lead to a change in pore pressure. If the change in total stress is faster than the possibility for drainage of excess pore pressure, shear strength of a soil ultimately decreases proportional to
CHAPTER 2. MARINE SLIDES AND FAILURE PROCESSES

Figure 2.1: a) definition of the Factor of Safety (FoS) and a list of factors influencing the factor of safety (after [Locat and Lee, 2002]), b) frequency distribution of slides as a function of triggering mechanisms (after [Hance, 2003])
the increase in pore pressure. Under completely undrained conditions in a fully saturated soil, any change in total stress is expressed in an equivalent amount of pore pressure without change in absolute volume [Castro, 1969]. Any process that either reduces resisting forces or increases the driving forces by building up excess pore pressure for instance, is a potential triggering mechanism for submarine landslides. Figure 2.1a provides a list of potential triggering mechanisms split in processes affecting the resisting forces and processes affecting the driving forces. It can be seen, that many of these processes reduce the shear strength of sediment as well as increase the driving forces simultaneously. Regarding the temporal aspect a distinction is made between short-term and long-term triggering mechanisms. Short-term triggering mechanism comprise: i) seismic loading, ii) storm-wave loading, iii) low tides, iv) rapid sedimentation, v) gas charging, vi) gas hydrate dissociation, vii) ground water seepage, viii) glacial loading, ix) erosion and x) diapirs. One of the most prominent short-term triggering mechanisms is seismic shaking [Biscontin and Pestana, 2006; Biscontin et al., 2004; Canals et al., 2004; Cochonat et al., 2002; Havenith, 2003; Keefer, 1984; Locat and Lee, 2002; Masson et al., 2006; Strozyk, 2009; Sultan et al., 2004; Urgeles et al., 2002; Wright and Rathje, 2003; Zakeri et al., 2010]. In fact more than 40% of submarine landslide events worldwide are attributed to seismic hazard [Hance, 2003] (Fig. 2.1b). Seismic shaking induces short duration, but heavy stress changes. It is considered to happen too fast to allow drainage in any soil which is why it leads to i) an increase in pore pressure with inherent reduction of shear resistance of soils and ii) an increase in driving forces due to additional shear stress related to the propagation of seismic shear waves [Biscontin et al., 2004; Kramer, 1996].

Figure 2.2: Illustration a granular soil (left) before earthquake shaking and (right) in a liquefied state
The material being subjected to shear stresses on submarine slopes is essentially equivalent to terrestrial soil, except that it is either fully saturated or partly saturated and otherwise gas-charged. It consists of clay, silt and sand or a mixture of these components [Leroueil and Hight, 2003]. Geotechnical investigations conducted during the past century showed that especially loosely deposited silts and sands can show abrupt strength loss in case of earthquake loading. Loosely deposited granular soils have the tendency to contract and quickly build up pore pressure during seismic shaking. Once the pore pressure reaches the total stress, frictional and contact forces between the particles are reduced to zero. The consequence is a liquid-like behavior of granular soils known as liquefaction (Fig. 2.2) [Castro, 1969]. A geotechnical perspective on the phenomenon of liquefaction is given in Chapter 3.

Although clayey soils get weakened by seismic shaking [Boulanger and Idriss, 2007], a great majority of clays will not liquefy during earthquakes [H Seed et al., 1983]. However, clayey soils don’t necessarily need an external trigger to show major deformation. Gravitational, monotonic loading may be sufficient to produce small shear bands initiated at a certain level in the slope and progressively extend and form a distinct shear plane via progressive failure; [Leroueil, 2001; Petley et al., 2005].

Figure 2.3 a-d schematically illustrates the progressive failure process in a slope composed of mechanically homogeneous, cohesive soil. Parallel to that, Figure 2.3 e and f schematically illustrate sudden failure of a slope composed of inhomogeneous material. In Figure 2.3 a-d, the displacement rate (\( \lambda \)) and the FoS evolution are shown at four stages. The extension of the shear surface is illustrated for each stage on a schematic slope. At first, the slope can be considered as stable with a FoS fluctuating at values > 1 (Fig. 2.3 a). The fluctuation is considered as being a product of natural pore pressure changes through time. At some point, somewhere within a weak spot of the slope sediment, microfractures may develop that progressively extend and start forming a distinct shear surface (Fig. 2.3 a and b). As this process progresses, FoS is reduces, the shear surface keeps growing, the displacement rate reaches a linear trend (Fig. 2.3 c) until finally the shear surface is fully developed and builds a concave upward and scoop-shaped failure plane (Fig. 2.3 c) [Petley et al., 2005]. This failure process may take a long time (years-thousands of years) on a geological scale and depends strongly on the sensitivity and plasticity of the clay.

Figure 2.3e and 2.3f are inspired by Figure 2.3a-d and show conceptually sudden slope failure due to liquefaction of granular material. Instead of the displacement rate (\( \lambda \)), the pore pressure (u) evolution is shown against time. Note that the time-axis here is non-linear. The duration of earthquake
Figure 2.3: a-d) Conceptual model illustrating the development of progressive failure in cohesive soil. For each stage a plot of factor of safety (FoS) against time is shown, with inset illustrations of (upper) state of development of shear surface and (lower) plot in $\lambda$-t space. (a) Natural pore-pressure variation. Microfractures are development at critical FoS. (b) Second threshold is achieved at lower FoS, at which the formation of a shear surface starts due to progressive extension of microfractures to a significant fracture. (c) Further development of shear surface leads to stress concentration and acceleration of landslide. Pore pressures become increasingly unimportant for movement of slide, as acceleration is controlled by rupture-surface propagation. Clear linear trend is traced in $\lambda$-t space. (d): Failure of landslide occurs when shear surface is fully developed. (taken from [Petley et al., 2005]). e) Pendant to Figure (2.3a) on a stratified slope containing a layer of granular, liquefaction susceptible material. (f) FoS decrease, pore pressure increase until liquefaction of the granular layer due to earthquake (EQ) shaking.

shaking (i.e. seconds to minutes) is stretched. Figure 2.3e shows the same starting conditions as shown in Figure 2.3a on a stratified slope containing a layer of granular material. The fluctuation in the FoS is related to natural fluctuation of the pore pressure. Figure 2.3b illustrates the reduction of the FoS to values < 1 due to earthquake shaking, inherent pore pressure increase and finally liquefaction.

Hance [2003] showed that the majority of submarine slides involve clayey soils, (Fig. 2.4) which is related to the fact that the majority of marine sediment consists of clayey hemipelagic sediment [Noorany, 1989]. The sum of all slope failures involving granular, liquefiable soil types, such as silts and sands, is even larger (Fig. 2.3). This combined with the fact that more than
40% of the slides are earthquake triggered points out the primary importance to understand the response of slope sediments to cyclic loading. However, not every earthquake leads to a submarine landslide and not every submarine landslide can be related to a short-term triggering mechanism. Out of the 534 sliding events that Hance [2003] reviewed, 30% could not be related to a clear triggering mechanism either because traces of triggers could not be found or because failure occurred under gravitational monotonic loading only. Hence, just as cyclic shear behavior, monotonic shear behavior of marine sediments is of utmost interest for slope stability analysis and risk assessment.

![Figure 2.4: Documented soil type are plotted against the number of slope failures. A total of 147 events are included. The sum of the events over the types of soils exceeds the total number of events because one event may involve several types of soils (modified after [Hance, 2003])](image)

The stratigraphic horizons where failure is initiated be it due to liquefaction of granular soil or progressive failure of cohesive soil is called **weak layers**. A weak layer has been defined as:

"A layer (or band) consisting of sediment or rock that has strength potentially or actually sufficiently lower than that of adjacent units (strength contrast) to provide a potential focus for the development of a surface of rupture. Such a layer or a band can follow stratigraphic horizons, but this is not a requirement." [Locat et al., 2014]
Figure 2.5 shows a sedimentological and geotechnical classification system for weak layers. *Inherited* and *induced* weak layers are differentiated in the sense pointed out in Figure 2.3. *Inherited* weak layers do not necessarily require an external stimulus to fail even under monotonic condition, whereas *induced* weak layers need an external stimulus such as an earthquake to reduce their strength. Induced weak layers tend to be mechanically strong under monotonic shear conditions (see section 3.3).

**Figure 2.5:** Sedimentological and geotechnical classification system for weak layers. Horizontal arrows point towards the primary effect of the process involved (modified after [Locat et al., 2014]).
Chapter 3

Sediment end-member shear behavior

The following chapter gives an overview on the shear behavior of the sediment end-members sand and clay in order to provide a basis of comparison with volcanic material. In the first subchapter the stress terminology applied in triaxial shear experimentation is introduced. Subsequently liquefaction of common non-cohesive, granular soils (i.e. sands and silts) will be regarded from a geotechnical perspective. Similarities and differences that volcanic ashes show in contrast should thereby be pointed out later. In the third subchapter the shear behavior of cohesive, clayey soils is addressed with the purpose to integrate the behavior of altered volcanic material in the existing geotechnical concepts on shear behavior of clays. Furthermore, most marine sediments consist of or contain clay phases [Noorany, 1989] and are consequently frequently involved in submarine landslide processes [Hance, 2003].

3.1 Stress Terminology in Triaxial Testing.

Figure 3.1a schematically illustrates a cylindrical sample consolidated under isotropic stress conditions. Isotropic stress conditions are given when the major principle stress ($\sigma_1$) equals the minor principle stress ($\sigma_3$). The sample surrounding stress is then also called confining stress ($\sigma_c$). In case drainage of a fully saturated sample is prevented during consolidation or shearing, pore-water pressure ($u$) is induced that reduces the effective stress conditions (see equation 2.1). Major and minor principle stress decrease by the amount of pore pressure increase and are then called major and minor effective principal stress, respectively (Fig. 3.1a). The state of saturation of a sample is defined
by Skempton’s B-value (B) [Skempton, 1954] which is the ratio of pore water pressure increase ($\Delta u$) and an increase in confining pressure ($\Delta \sigma_c$) under undrained conditions:

$$B = \frac{\Delta u}{\Delta \sigma_c}$$  \hspace{1cm} (3.1)

During undrained shearing pore water pressure may increase. The ratio of $u$ and $\sigma_c$ is then defined as pore pressure ratio ($ppr$):

$$ppr = \frac{u}{\sigma_c}$$  \hspace{1cm} (3.2)

Figure 3.1b schematically illustrates a sample with monotonic or cyclic stress acting upon it including the stress terminology. In triaxial testing the effective normal stress ($\sigma_n'$) and shear stress ($\sigma_s$) acting on a plane inclined by $\theta$ with $\sigma_3'$ are defined as follows:

$$\sigma_n' = \frac{\sigma_1' + \sigma_3'}{2} + \frac{\sigma_1' - \sigma_3'}{2} \times \cos(2 \theta)$$  \hspace{1cm} (3.3)

$$\sigma_s = \frac{\sigma_1 - \sigma_3}{2} \times \sin(2 \theta)$$  \hspace{1cm} (3.4)

Under isotropic conditions $\sigma_s = 0$ and $\sigma_n' = 1/2(\sigma_1' + \sigma_3')$. As soon as major and minor principal stresses differ, shear stress is induced. It can be deduced from equation 3.3 that the maximum normal stress acts on a plane forming an angle of $\theta = 0^\circ$ with $\sigma_3'$ and the maximum shear stress acts on a plane forming an angle of $\theta = 45^\circ$ with $\sigma_3'$ (Fig. 3.1b). As shear strength is the parameter to be determined the plane of maximum shear stress is crucial in triaxial testing. By setting $\theta = 45^\circ$ the equations 3.3 and 3.4 simplify to:

$$\sigma_n' = \frac{\sigma_1' + \sigma_3'}{2}$$  \hspace{1cm} (3.5)

$$\sigma_s = \frac{\sigma_1 - \sigma_3}{2} = \frac{q}{2}$$  \hspace{1cm} (3.6)

$\sigma_n'$, corresponds to the distance from the origin to the centre of a Mohr-Coulomb circle in the Mohr-Coulomb diagram. $\sigma_s$ corresponds to the radius of the Mohr-Coulomb circle (Fig. 3.1b inlet ii). However, triaxial tests are mostly analyzed in the so called $p'$-$q$ space, where $p'$ is the effective mean stress and $q$ is the deviator stress (Fig. 3.1) defined as:

$$p' = \frac{\sigma_1' + \sigma_3'}{2}$$  \hspace{1cm} (3.7)
3.1. STRESS TERMINOLOGY IN TRIAXIAL TESTING.

Figure 3.1: Schematic illustration of a cylindrical sample used in triaxial testing experiments a) under isotropic stress conditions and b) under monotonic/ cyclic loading conditions.
\[ q = \sigma_1 - \sigma_3 \]  

Hence, \( \sigma_s \) equals \( q/2 \) and \( p' \) equals \( \sigma'_n \) on the plane forming an angle of \( \theta = 45^\circ \) with \( \sigma_3 \). Monotonic strain is induced by variation of \( \sigma_1 \) at constant \( \sigma_3 \). Cyclic stress is induced by cyclic variation (mostly sinusoidal) of \( \sigma_1 \) with full stress reversal and rotation of the principle stress axes by \( \pi/2 \) (Fig. 3.1 b). I.e. during cyclic loading \( \sigma_3 \) may be \( > \sigma_1 \) and thus become the major principle stress. A sample consolidated to anisotropic stress conditions experiences initial shear stress \( (q_0/2 \neq 0) \) as \( \sigma_1 \neq \sigma_3 \). Anisotropic stress conditions are closer to natural stress conditions than isotropic stress conditions because \( \sigma_3 < \sigma_1 \) in the simplest case of a planar natural settings without additional tectonic stresses. The ratio of \( \sigma_3 \) and \( \sigma_1 \) in a natural setting is the coefficient of lateral earth pressure (\( K_0 \)). \( K_0 \) depends on the internal friction angle (\( \phi' \)) of the soil and is defined as [Jaky, 1944]:

\[ K_0 = 1 - \sin(\phi') = \frac{\sigma'_3}{\sigma'_1} \]  

A portion of soil would follow the stress path schematically shown in Figure 3.1b, inlet iii in the \( p' \)-q space with increasing burial depth. Most of the triaxial shear experiments, monotonic or cyclic, are conducted under initial isotropic stress conditions with the initial deviatoric stress \( q_0 = 0 \) [Kramer, 1996].

### 3.2 Granular sediment

Fundamentals of the process of liquefaction of granular soils are summarized in the following subchapters. First, shear behavior and definitions under monotonic loading conditions are summarized and secondly cyclic loading conditions are implemented:

#### 3.2.1 Drained monotonic shear behavior

With the monotonic shear behavior of granular soils the preconditions for liquefaction shall be illustrated. Monotonic shear behavior is highly dependent on the state of compaction of the sediment and the stress conditions acting upon it. Loosely deposited granular soils tend to compact during shear whereas compacted granular soils tend to dilate during shearing. Figure 3.2 shows a schematic example of a drained monotonic triaxial shear experiment on a dense (D) and a loose (L) sample both sheared at equivalent confining stress (\( \sigma_3 \)). Deviatoric stress (\( q \)), relative volume change (\( \Delta V/V_0 \)) and void
3.2. GRANULAR SEDIMENT

ratio (e) are plotted against strain (ε). Void ratio is defined as:

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

(3.10)

Where, \( V_v \) is the volume of voids and \( V_s \) is the volume of solids. The relation between \( e \), \( q \) and \( \Delta V/V_0 \) is illustrated in Figure 3.2b: At high strains (\( \epsilon > 20\% \)) both samples (D and L) reach the same constant void ratio at equal deviator stress (\( q \)). Both parameters at that state are called the critical void ratio (\( e_{\text{crit}} \)) and critical deviatoric stress (\( q_{\text{crit}} \)), respectively.

Figure 3.2: Schematic illustration of two monotonic drained shear experiments on a loose (L) and dense (D) sample at identical effective mean stress (\( p'_0 \)) (after Wood, 1990)
CHAPTER 3. SEDIMENT END-MEMBER SHEAR BEHAVIOR

At equal $\sigma_3$, $e_{\text{crit}}$ and $q_{\text{crit}}$ are independent of the initial state of density. The loose sample shows an initially high void ratio that is reduced during shearing (Fig. 3.2c). Consequently a reduction of the total volume can be observed (Fig. 3.2b), i.e the loose sample shows a contractive behavior or compaction. The dense sample shows an initially lower void ratio and gets only initially compacted until $q$ passes $q_{\text{crit}}$ for the first time, i.e before the peak. Subsequently dilation takes place and results in a net volume increase with increasing strain. The dense sample shows a clear peak in shear resistance and a subsequent decrease of $q$ towards $q_{\text{crit}}$. The peak shear resistance of the loose sample corresponds to $q_{\text{crit}}$. The state of density of a granular soil has major influence on its peak shear resistance [Muir Wood, 1990].

Relative density

The relative density ($I_d$) is a measure for the state of density of a granular soil independent of grain-size distribution, grain shape or grain density. As the state of compaction of a soil is known to majorly influence its shear resistance, relative density is always explicitly mentioned for any laboratory shear experiment. The relative density is given in % and defined as:

$$I_d = \frac{(e_{\text{max}} - e)}{(e_{\text{max}} - e_{\text{min}})} \times 100$$

(3.11)

Where, $e_{\text{max}}$ and $e_{\text{min}}$ are the void ratio in the loosest and densest state, respectively. $e$ corresponds to the void ratio at the beginning of the shear experiment. The standard determination of $e_{\text{max}}$ and $e_{\text{min}}$ is shown in the DIN [Deutsches Institut für Normung, 1996]. Table 3.1 shows the designation of different ranges of relative density.

<table>
<thead>
<tr>
<th>Relative density</th>
<th>$I_d$ %</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very loose</td>
<td>&lt;20</td>
</tr>
<tr>
<td>Loose</td>
<td>20 - 40</td>
</tr>
<tr>
<td>Medium dense</td>
<td>40 - 60</td>
</tr>
<tr>
<td>Dense</td>
<td>60-80</td>
</tr>
<tr>
<td>Very dense</td>
<td>&gt; 80</td>
</tr>
</tbody>
</table>

Table 3.1: Designation of relative densities (after [Mitchell and Soga, 2005])
3.2. GRANULAR SEDIMENT

Critical Void Ratio Line (CVR - Line)

Whether or not a sample will show contractive or dilatational behavior depends upon the initial void ratio and the confining pressure. Casagrande [1936] performed a series of drained, strain-controlled triaxial shear experiments on initially dense and loose samples at different confining pressures ($\sigma_3$). It was shown that the critical void ratio ($e_{\text{crit}}$) is i) independent of initial void ratio ($e$) and ii) decreases with increasing confining stress. Casagrande [1936] found that $e_{\text{crit}}$ is uniquely dependent upon $\sigma_3$ and called the line that shows $e_{\text{crit}}$ as a function of $\sigma_3$ the critical void ratio line (CVR-line) (Fig. 3.3). The CVR line separates samples with contractive behavior ($e > e_{\text{crit}}$) from samples with dilatational behavior ($e < e_{\text{crit}}$).

![Figure 3.3: Schematic illustration of the critical state void ratio as a function of log ($\sigma_3'$). Samples with an initial void ratio higher than the CVR line tend to contract during shear. Samples that plot below the CVR line tend to dilate (after Casagrande, 1936).](image)

3.2.2 Undrained monotonic shear behavior

About 30 years after Casagrande [1936] had shown the existence of the CVR-line a student of his performed equivalent triaxial tests under undrained monotonic condition. Castro [1969] proved what Casagrande had postulated after he had found the CVR line.

The hypothesis was that a fully saturated sample sheared under undrained conditions would be unable to be subject of volume changes. A loose sample would build up positive pore pressure due to the tendency to contract, whereas a dense sample would show negative pore pressure due to the tendency to dilate. Figure 3.4 shows schematic examples of undrained monotonic strain-controlled anisotropic shear experiments performed on samples with different initial void ratios or densities (very loose (VL), loose (L), and dense (D)). The deviatoric stress ($q$) and pore pressure ($u$) are plotted against strain ($\epsilon$) (Fig. 3.4a and b, respectively).
Figure 3.4: Schematic illustration of three undrained monotonic triaxial shear experiments on a very loose (VL), a loose (L) and a dense (D) sample. a) deviatoric stress ($q$) plotted against strain ($\epsilon$) b) pore pressure (u) plotted against strain c) effective stress paths. SS: Steady state of deformation, QSS: Quasi steady state, PT: Phase transformation point, SSP: steady state point, SSL Steady state line.

Furthermore the effective stress paths are shown in the $p'$-$q$ space (Fig. 3.4c). It is illustrated, that the very loose sample (VL) reaches a peak in undrained shear strength ($q/2$) at low strain and subsequently 'collapses' and flows toward a low steady shear strength in a strain softening behavior. Pore pressure increases until a steady plateau is reached. The mean effective normal stress ($p'$) and the deviatoric stress ($q$) reach a steady minimum (at large strain). At that point constant strain accumulation takes place at constant $p'$, $q$ (and hence $u$). The sample has reached the steady state of deformation in the steady state point (SSP). This type of collapse today is called flow liquefaction and only occurs if the static shear stress ($q_{0}$) is larger than the steady state shear strength ($s_{u,s} = q_{steady-state}/2$). The loose sample (L) initially shows a similar behavior to the very loose sample with a peak in deviatoric stress and increasing pore pressure. However, the strain softening behavior is limited in strain. At large strain pore pressure decreases
and deviatoric stress inherently starts increasing in a strain hardening behavior. The effective stress path at that point shows an 'elbow' at the minimum in the p'-q space. Subsequently, p' and q start increasing. The 'elbow' designates the phase transformation point (PTP) [Ishihara et al., 1975] and determines the point at which the sample changes from a contractive behavior to a dilatational behavior. This type of shear behavior is called limited liquefaction. The limited state of constant p', q and u is called quasi steady state (QSS) and the phase transformation point may be called the quasi-steady-state point. The dense sample (D) shows a constant increase in shear stress. The pore pressure only increases slightly in the initial shearing phase and constantly decreases subsequently due to a constant dilatational behavior [Castro, 1975; Castro and Poulos, 1977; Kramer, 1996; Mohamad and Dobry, 1986; Poulos et al., 1985; Vaid and Sivathayalan, 2000].

3.2.2.1 Steady State Line (SSL-line)

Castro [1969] plotted the effective mean normal stress (p') at steady state against steady state void ratio (e\textsubscript{ss}) analogue to the CVR-line. In the same study he referred to the resulting curve as the steady state line (SSL). As shown in (Fig. 3.5a), the SSL is a three-dimensional curve in the e-p'-q-space. The CVR-line essentially corresponds to the SSL projected in the e-p' space at constant q (Fig. 3.5b). However, the CVR line and SSL are not equal. The difference has been attributed to a 'flow structure' in which the grains orient themselves so the least amount of energy is lost by frictional resistance during flow [Kramer, 1996]. Figure 3.5c shows the steady state line in the e-q\textsubscript{steady-state}/2 space which separates a soil susceptible to flow liquefaction (above SSL) from, a soil non-susceptible to flow liquefaction (below SSL). The position of the SSL depends on the properties of the soil (grain size distribution, grain shape, angularity, roughness). Soils with rounded particles often show a flat SSL whereas angular particles tend toward a steeper SSL. Well graded soils show a higher vertical position of the SSL [Kramer, 1996].
3.2.2.2 State Parameter ($\psi$)

As illustrated in Figure 3.5 a granular soil with a specific void ratio may be non-susceptible to liquefaction under low mean effective normal stress conditions, but susceptible to liquefaction at high mean effective normal stress. [Been and Jeffersies, 1985] introduced the state parameter ($\psi$) which indirectly implements the mean effective normal stress acting upon a soil. The state parameter designates the distance of a sample from the SSL in terms of void ratio (Fig. 3.6). The state parameter is defined as:

$$\psi = e - e_{\text{steady-state}} \quad (3.12)$$

In case $\psi$ is positive, a soil is located above the SSL and is susceptible to liquefaction. If $\psi < 0$ it plots below the SSL and is non-susceptible to liquef-
3.2. GRANULAR SEDIMENT

faction. Soils with identical state parameter exhibited similar shear behavior independent of the absolute void ratio [Kramer, 1996].

Figure 3.6: schematic illustration of the state parameter (after [Been and Jefferies, 1985])

3.2.2.3 Flow Liquefaction Surface

The flow liquefaction surface describes the conditions at which flow liquefaction is initiated. Figure 3.7 shows five undrained shear tests of which the samples void ratios are equal. The tests show different initial isotropic $p'_0$. Three out of five tests (C, D, E) plot above the SSL and two others (A, B) plot below the SSL in the $e$-$p'$ space. The sample C, D and E show a peak in shear stress and subsequently ‘flow’ towards the steady state of deformation (steady state point) just as the very loose sample shown in Figure 3.4. The peak in shear stress is the point of liquefaction initiation. The points of liquefaction initiation of the samples that plot above the SSL lie on a straight line, the flow liquefaction surface (FLS). Note that in undrained loading dilatational or contractive behavior is not related to an actual change in volume, but expresses in negative or positive pore water pressure and an inherent increase or decrease in effective stress conditions.

3.2.3 Correspondence between cyclic and monotonic undrained shear behavior

No matter if monotonic or cyclic loading conditions are applied, flow liquefaction only occurs in case $q_0 > s_{u,s}$ (or $q_{steady-state}/2$). Figure 3.7 schematically shows a monotonic and cyclic shear experiment on two identical samples under identical initial stress condition and $q_0 > q_{steady-state}$. Initially both
samples are in a drained equilibrium (point A). The first sample is loaded monotonically. As soon as the effective stress path hits the FLS (point B and peak in q) the sample starts flowing toward the steady state of deformation (point C). The second sample is loaded cyclically; build up positive pore pressure which results in an effective stress path moving cyclically to the left. Again, as soon as the effective stress path hits the FLS (point B’), no additional shear stress is needed to make the sample flow towards the steady state of deformation (point C).

### 3.2.3.1 Flow Liquefaction Zone (FLZ) and Cyclic Mobility Zone (CMZ)

The zone to the right of the FLS and above the steady state shear strength determines the *flow liquefaction zone* (FLZ). Flow liquefaction only occurs if the static shear stress is greater than the steady state shear strength. Thus flow liquefaction only occurs if the stress condition acting on a soil plot in the FLZ in Figure 3.9. In case a sample is subjected to stress conditions that plot...
3.2. GRANULAR SEDIMENT

Figure 3.8: Schematic illustration of the correspondence between monotonic and cyclic undrained loading. (after [Kramer, 1996])

below the steady state shear strength it is located in the cyclic mobility zone (CMZ) (grey shaded area in Fig. 3.9). The CMZ designates the area where cyclic mobility may occur. Cyclic mobility describes failure in undrained cyclic shear experiments due to pore pressure build up, but without flow liquefaction. The development of cyclic mobility is illustrated in Figure 3.9a and b. Figure 3.9 shows three cyclic loading experiments with three different initial stress conditions: a) \( q_0 + q_{cyc} < s_{u,s} \) and \( q_0 - q_{cyc} > 0 \), i.e. no stress reversal, b) \( q_0 + q_{cyc} < s_{u,s} \) and \( q_0 - q_{cyc} < 0 \), i.e. with stress reversal and c) \( q_0 + q_{cyc} < s_{u,s} \), and \( q_0 - q_{cyc} > 0 \), i.e no major principle stress reversal occurs. In the first case (Fig. 3.9a) the effective stress is cyclically reduced until the effective stress path hits the drained failure line. From that point on additional loading results in an up and down movement of the effective stress path along the drained failure envelope. The second example (Fig. 3.9c) includes stress reversal, i.e. compressional- and extensional loading take place during the experiment. Mohamad and Dobry [1986] experimentally show that stress reversal leads to an enhanced pore pressure increase during the unloading phase which results in a quicker reduction of effective stresses. Again the effective stress path moves to the left until it hits the drained failure envelope of compression and extension. Figure 3.9c illustrates the occurrence of flow liquefaction under cyclic loading condition as shown before in Figure 3.8. Here the effective stress path moves toward the undrained steady state shear strength after hitting the FLS. Additional loading leads to phase transformation in cyclic loading. Phase transformation expresses in negative pore pressure and an increase in effective stress. Table 3 summarizes the conditions under which flow liquefaction may occur or not [Mohamad and Dobry, 1986; Kramer, 1996].

As shown in this sub-chapter, layers of granular saturated sediment may
## Table 3.2: Summary of factors affecting the behavior of saturated sand in anisotropic cyclic triaxial tests (taken from [Mohamad and Dobry, 1986]).

<table>
<thead>
<tr>
<th>Type of specimen (1)</th>
<th>No shear stress reversal $(\tau_\sigma &lt; \tau_s)$ (2)</th>
<th>Shear stress reversal $(\tau_\sigma &gt; \tau_s)$ (3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Contractive $(\theta_s &gt;&gt; \theta_{390})$</td>
<td>Pore pressure buildup limited by strength envelope ( \hat{p} = \delta_3 = 0 ) condition not possible Flow deformations can occur only if ( \tau_p = \tau_\sigma + \tau_\omega &gt; S_{us} ) If ( \tau_\sigma &gt; S_{us} ), increasing ( \tau_s ) always decreases cyclic strength</td>
<td>Includes isotropic case $(\tau_\sigma = 0$; $K_s = 1)$ Pore pressure buildup limited by strength envelope ( \hat{p} = \delta_3 = 0 ) possible Flow deformations can occur only if ( \tau_p = \tau_\sigma + \tau_\omega &gt; S_{us} ) Rate of pore pressure buildup controlled by larger cyclic strains in extension-reversal branch If ( \tau_\sigma &lt; S_{us} ), increasing ( \tau_s ) always increases cyclic strength due to reduction of extension-reversal branch</td>
</tr>
<tr>
<td>Partially-contractive $(\theta_s &gt; \theta_{390})$</td>
<td>Pore pressure buildup limited by strength envelope ( \hat{p} = \delta_3 = 0 ) condition not possible Limited flow deformations can occur only if ( \tau_p = \tau_\sigma + \tau_\omega &gt; S_{us} ) Effect of ( \tau_\sigma ) on liquefaction resistance similar to that for Contractive specimen at small specified accumulated strain</td>
<td>Includes isotropic case $(\tau_\sigma = 0$; $K_s = 1)$ Pore pressure buildup limited by strength envelope ( \hat{p} = \delta_3 = 0 ) possible Limited flow deformations can occur only if ( \tau_p = \tau_\sigma + \tau_\omega &gt; S_{us} ) Rate of pore pressure buildup controlled by larger cyclic strains in extension-reversal branch Effect of ( \tau_\sigma ) on liquefaction resistance similar to that for Contractive specimen at small specified accumulated strain; similar to that for Dilative specimen at larger specified accumulated strain</td>
</tr>
<tr>
<td>Dilative $(\theta_s &lt; \theta_{390})$</td>
<td>Pore pressure buildup limited by strength envelope ( \hat{p} = \delta_3 = 0 ) condition not possible Flow deformations do not occur</td>
<td>Includes isotropic case $(\tau_\sigma = 0$; $K_s = 1)$ Pore pressure buildup limited by strength envelope ( \hat{p} = \delta_3 = 0 ) possible Flow deformations do not occur Rate of pore pressure buildup controlled by larger cyclic strains in extension-reversal branch Increasing ( \tau_s ) always increases cyclic strength due to reduction of extension-reversal branch</td>
</tr>
</tbody>
</table>
act as *induced weak layers* (Fig2.5), triggered by earthquake shaking. However, in case the earthquake magnitude is not sufficiently high to trigger cyclic mobility or flow liquefaction a phenomenon called *cyclic strengthening* may occur.

### 3.2.3.2 Seismic strengthening

Cyclic strengthening describes the post-earthquake shear strength increase of a granular soil in case of non-appearance of failure [Tokimatsu and Seed, 1987]. Figure 3.10 conceptually illustrates the strengthening as a result of densification of a granular soil in the e-log (p’) space. Assuming a granular soil with an initial state parameter ($\psi$) > 0 (point A) gets subjected to a low magnitude earthquake that induces pore pressure insufficiently high to initiate failure. The soil would move to point B due to excess pore pressure build-up. Post earthquake drainage can be accompanied by void reduction.
[Tokimatsu and Seed, 1987] and lead to a higher density of the soil at more or less the same mean effective normal stress \( (p') \) (point C) as before the earthquake. Consecutive repetition of this mechanism (point C and D) can sandify the soil to a state of \( (\psi) < 0 \) (point E), where flow liquefaction is impossible.

**Figure 3.10:** Schematic illustration of the seismic strengthening effect in the e-log \( (p') \) space. (according to [Tokimatsu and Seed, 1987])

### 3.3 Cohesive sediment

The following subchapter lists major differences and factors of influence on the shear strength and shear behavior of cohesive soils in comparison to granular soils. It will be shown that cohesive, clayey sediments tend to be inherently weak and may form the *inherently weak layers* on submarine slopes. It is generally recognized that liquefaction as failure mechanism in clay dominated soils is rather unlikely [Yasuhara et al., 1992] although highly sensitive clays (quick clays) may show equivalently drastic strength loss when remolded by seismic shaking [Boulanger and Idriss, 2007]. Generally speaking the liquefaction susceptibility decreases with increasing plasticity, i.e. clay content [Bray and Sancio, 2006]. Andrews and Martin [2000] show that standard classification test on cohesive soils, i.e the determination of grain size and liquidity, provide basic information to Figure whether or not a soil is generally liquefaction susceptible or not. It has been found that most liqueable soils have a liquid limit < 32 and less than 10% clay content (particle < 2 \( \mu \)m). Here, the focus is set on non-liquefiable clayey soils (liquid limit > 32 and more than 10% clay) that are characterized by high plasticity and low internal friction resistance [Leroueil and Hight, 2003]. The failure process predominantly observed in these soils is of progressive nature, i.e relatively
small changes in stress conditions may lead to creep and progressive failure [Leroueil, 2001].

As drained and undrained monotonic shear experiments were already presented in the $p'$-$q$ space and furthermore, implication on void ratio or pore pressure were illuminated, the following subchapters summarize both, the monotonic drained and undrained behavior of clayey soils in a single composite Figure.

**Figure 3.11**: Schematic illustration of two drained and two undrained monotonic shear experiments on clayey samples. Samples B and D are overconsolidated. Samples A and C are only slightly overconsolidated. a) evolution in the $p'$-$q$ space, b) stress-strain evolution, c) void ratio evolution relative to the normal consolidation line NCL and the critical state line CSL, d) void ratio evolution of all samples and pore pressure evolution of the undrained schematic experiments (modified after [Leroueil and Highet, 2003])
3.3.1 Drained monotonic shear behavior

From a purely schematic point of view, overconsolidated clays subjected to drained monotonic shear forces essentially behave like dense sand and normally consolidated clays behave like loose sands. Figure 3.11 schematically illustrates two drained strain controlled shear experiment on a clayey soil (samples A and B). Figure 3.11a presents the stress paths of both samples at low (B) and high (A) initial mean effective stress (p') in the p'-q space. In Figure 3.11b the stress-strain evolution is represented. Both sample's positions in the e-p' space are shown relative to the normal consolidation line (NCL) and the critical state line (CSL) in Figure 3.11c. It can be seen that the shear strength of sample A evolves towards a plateau without forming a peak. Furthermore sample A is located on the right of the CSL in the e-p' space and close to the NCL, i.e. the sample is only slightly overconsolidated. As long as q does not exceed the elastic domain (Fig. 3.11b), the void ratio (e) doesn't change significantly. However, once the deformation reaches the plastic, irreversible domain the void ratio decreases toward the critical state void ratio (Fig. 3.11c and d). Sample B on the contrary is located far left from the NCL and CSL in the e-p' space and is thus highly overconsolidated. Consequently, a peak in shear stress (q) can be observed followed by a decrease toward critical state conditions (Fig.3.11b). The void ratio initially decreases slightly, but then increases continuously toward the critical state void ratio similarly to a dense sand (compare (Fig. 3.2 and Fig. 3.11c and d). Sample B behave in a more brittle manner compared to the plastic deformation behavior of sample A. The limit state curve indicated in Figure 3.11a delimits the area of elastic yield in the p-q space. The point M located on the p' axis designates the limit of normal consolidation of that specific sample material under isotropic conditions [Leroueil et al., 2003].

Despite the schematic similarity of the shear behavior of dense sands vs. overconsolidated clays and loose sands vs normally or slight overconsolidated clay, both materials are characterized by striking differences in absolute values:

i) The absolute strength can be more than 50% less in clayey soils than in sands. The decrease of the coefficient of friction with increasing plasticity/clay content is generally accepted [Leroueil and Hight, 2003; Lupini et al., 1981] and is presented in Figure 3.12.

ii) High elasticity and plasticity of clays lead to ductility also expressed in peak and critical strengths that are only reached at substantial strains (in comparison to sands)[Leroueil and Hight, 2003].

iii) The lowest shear resistance, i.e. the residual shear strength is reached after large strains, beyond the critical state [Leroueil and Hight, 2003].
3.3. COHESIVE SEDIMENT

3.3.2 Undrained monotonic shear behavior

Monotonic undrained loading of cohesive soils is also schematically presented in Figure 3.11 (samples C and D), parallel to the drained examples (samples A and B). Again, an example of a slightly (C) and highly (D) overconsolidated sample are given in the different stress, strain and void ratio spaces. Additionally, Figure 3.11d schematically illustrates the pore pressure evolution. At first (point C and D) the pore pressure increases with the deviator stress (Fig. 3.11d and b) until the samples reach the limit of the elastic domain (point C1 and D1) where a sudden shift in the deviator stress gradient occurs to maintain the constant shear rate. From that point on the deviator stress curves may show slight strain hardening, while other show slight strain softening. Here slight strain hardening is presented in both sample, i.e. the deviator stress increases slightly. The weakly overconsolidated sample (D) shows an increase in pore pressure and inherent reduction of the mean effective stress (p’) until steady state or critical state conditions are reached (Fig. 3.11a and d). The overconsolidated sample presents a peak in pore pressure (u) and a subsequently decreasing trend resulting in an increase in the mean effective stress toward steady state conditions (Fig. 3.11a, b and c). The point D1 may be compared to the phase transformation point. The deviator stress stays comparatively steady, i.e. only slight strain hardening can be observed and the reduction in pore pressure results in only an increase in p’.

The void ratio under undrained conditions is obviously constant (Fig.3.11).

![Figure 3.12: Internal friction angle as a function of the plasticity index (taken from Terzaghi, 1996)](image-url)
3.3.3 Progressive failure

However, the failure mechanism in clayey soils is usually not dominated by excessive pore pressure build up (close to 100 % $p'_0$) and inherent dramatic reduction of the effective stress conditions as it is in granular soils. The shear strength and shear behavior in clayey soils is rather driven by mineralogy, inherent plasticity and stress history. The limit of elastic to plastic behavior is critical to clayey soils because progressive deformation (creep) may occur without further stress increase (example C and D in Fig. 3.11). Progressive failure in strain-softening soils such as example B in Figure 3.11 may lead to sudden slope failure as illustrated in Figure 3.13: If the peak shear strength in a homogeneous slope sediment body is reached such as at point A in Figure 3.13a, the shearing resistance at that point might drop to the residual state (point A Fig. 3.13b). Shear stresses that are no longer opposed to by the former resistance of the sediment at point A (Fig. 3.13a) are then redistributed to the surrounding sediment (point B and C). This redistribution may cause the peak shear resistance of the surrounding soil to be reached (point B in Figure 3.13b). The slope becomes progressively unstable as the shear stress redistribution processes continues and the failure zone grows [Kramer, 1996]. The whole process of progressive failure is ac-

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{figure3.13.png}
\caption{Schematic illustration of progressive failure development in a slope composed of homogeneous, strain-softening material. a) exceeding of the peak shear stress at any point A leads to a reduction of the shear resistance to the residual state, b) redistribution of the shear stresses from the failure zone (point A) to the surrounding sediment may progressively lead to failure. (after [Kramer, 1996])}
\end{figure}
celerated and amplified with increasing sensitivity of the soil. Sensitivity is defined as the ratio of peak and residual shear resistance which is a measure for the factor between the shear resistance at point B and A in Figure 3.13b. The larger the difference between peak and residual shear strength, the more sensitive is the soil. Hence, if point A in Figure 3.13b is reached by a highly sensitive soil (quick clay) the amount of shear stress to be redistributed on the surrounding sediment (point B and C) is proportionally larger. Thus it is more likely that the peak shear strength is also reached in the surrounding sediment.

### 3.3.4 Undrained cyclic shear behavior

Undrained cyclic loading experiments on clays follow the exact same principles as those presented on sand (section 2.4.1.3) with the difference that clays don’t show flow liquefaction and the corresponding flow liquefaction surface (FLS) in the p’-q space. However, cyclic or seismic loading leads to cyclic pore pressure increase and inherent reduction of effective stresses. Failure occurs when the effective stress path hits the failure line or the steady state/critical state line (Fig. 3.14). The period of seismic shaking is usually too short to produce significant pore pressure or even failure in clayey soils [Yasuhara et al., 1992]. However, repeated loading without intercalated drainage phases may lead to successive weakening of clayey soils due to successive strain accumulation and effective stress reduction. If drainage of excess pore water pressure is possible, cyclic strengthening is observed [Moses, 2003; Yasuhara et al., 1992], similarly to the effect illustrated in sands. In case failure is not reached during cyclic loading, the void ratio reduces during the draining phase and leads to an overall strengthening of the material [Yasuhara et al., 1992]. Overconsolidated or cemented clays however, get absolutely weakened by cyclic loading [Moses, 2003].

![Figure 3.14: Schematic illustration of cyclic loading on clayey soils a) without stress reversal and b) with stress reversal](image-url)
Chapter 4

Sediment shear strength determination

The following chapter introduces the geotechnical apparatuses that have been used to determine the undrained and drained shear behavior of investigated materials.

4.1 In-situ

Cone Penetration Testing (CPT) is a common in situ onshore and offshore measuring technique to determine key physical parameters of the sediment [Lunne et al., 1997]. The MARUM- Shallow-Water-Free-Fall-CPTU (SWFF-CPTU) [Stegmann et al., 2006] (Fig. 4.1) has been deployed in the course of this thesis to determine the in situ undrained shear strength ($s_u$) of subaqueous sediment. The centerpieces of the instrument are i) an industrial 15 cm$^2$ piezocone (dynamic-CPTU cone) and ii) an aluminum pressure-tight housing containing a microprocessor (Avisaro AG), logging unit, including a standard secure digital memory card (SD), power supply (battery package), tiltmeter, accelerometer and data communication unit. The maximum water depth at which this SWFF-CPTU can be operated is 600 m. The instrument is modular and can be adapted in length from 1.5 m - 8.5 m depending on the encountered geological setting and expected penetration depth. The SWFF-CPTU is lowered through the water column either with the help of a winch or in free-fall until it is decelerated by the penetration of the sediment column. Analog to the length, weights may be added to reach higher penetration. The total weight can vary from minimum 45 kg (manually operational) to maximum 200 kg (winch needed) in its longest and heaviest configuration. The piezocone located at the tip of the CPT measures cone
resistance \( (q_c) \), sleeve friction \( (f_s) \), pore pressure \( (u) \) and inclination. Further, deceleration and inclination sensors are placed in the housing of the instrument. Data recording takes place at 1 kHz. Figure 4.2 illustrates the processing steps needed to obtain the \textit{in situ} undrained shear strength from the recorded dynamic parameters deceleration, cone resistance \( (q_c) \) and pore pressure \( (u) \) obtained during penetration of the sea floor. Post-processing includes double integration of the deceleration data and implementation of the inclination which allows determining penetration depth accurately with a precision of 2% of the maximum penetration depth. At that point, cone resistance, sleeve friction and pore pressure along the profile are dynamic parameters, i.e. they are velocity dependent. In order to obtain quasi-static parameters comparable to static CPTU tests these parameters need to be corrected for strain-rate [Lunne et al., 1997; Stegmann et al., 2006; Steiner, 2013]. The \textit{in situ} undrained shear strength \( (s_u) \) is a function of the strain-rate and pore pressure corrected cone resistance \( (q_{t,\text{quasi-stat}}) \), the effective overburden stress \( (\sigma'_{v,0}) \) and an empirically determined cone penetration resistance factor \( (N_{k,t}) \) which is related to \( q_{t,\text{quasi-stat}} \) [Lunne et al., 1997]. \( s_u \) can be determined by applying the following geotechnical solution:

\[
 s_{u,\text{quasi-stat}} = \frac{q_{t,\text{quasi-stat}} - \sigma'_{v,0}}{N_{k,t}} \tag{4.1}
\]
4.1. IN-SITU

Once cone resistance has been corrected for pore pressure and strain rate cone resistance data are comparable to static industrial CPTU tests. The vertical resolution of a CPTU profile is \( \approx 1 \text{cm} \) at penetration velocities \(< 10 \text{ m/s} \) [Steiner, 2013]. A detailed description of the MARUM SW FF- CPT equipment including technical details, data processing and post-processing is given in [Steiner, 2013].

**Figure 4.2:** Schematic illustration of the processing steps needed to obtain the in situ undrained shear strength \((s_u)\) from deceleration and pore pressure corrected cone resistance \((q_{t,dyn})\). By integration of the deceleration data over time, penetration velocity \((v_{dyn})\) is obtained. A second integration over time results in the penetration depth over time. The cone resistance data \((q_c)\) is corrected for pore pressure \((u)\) and becomes \(q_{t,dyn}\). \(q_{t,\text{quasi-stat}}\) is the strain-rate corrected \(q_{t,dyn}\) and is comparable to cone resistance from static pushed CPTU data. \(s_u,\text{quasi-stat}\) is the in situ undrained shear strength obtained by resolving equation 4.1 (modified after Steiner [2013]).
4.2 On-site

Assuming a sediment core is available for sedimentological and geotechnical description, the standard on-board or on-site procedure includes fall cone penetrometer and vane shear tests. Both methods are applicable to cohesive sediments:

4.2.1 Fall Cone Penetrometer

Fall cone testing is a fast and simple way to get first estimation of the undrained shear strength ($s_u$) of the sediment recovered. Fall cone testing should be done right after core splitting to make sure that the sediment still has its natural water content (w %). Here, a Wykeham-Farrance cone penetrometer WF 21600 (Fig. 4.3 a) was used for the measurements. The procedure should follow the British Standards Institutions BS-1377 [British-Standard-Institute, 1977]: A metal cone is brought to a point exactly on the split core face. A manual displacement transducer is then used to measure the distance prior to and after release of the cone (Fig. 4.3b). The penetration process should take 5 s. The undrained shear strength can be determined directly via five parameters: The penetration depth ($d$) combined with the cone mass ($m = 80.51$ g), the cone geometry ($30^\circ$ tip angle), gravitational acceleration ($g$) and a cone factor ($k$) determined by [Wood, 1985]. With these parameters the undrained shear strength can be calculated by using the following empirical equation [Hansbo, 1957]:

$$s_u = \frac{k \times m \times g}{d^2}$$

(4.2)

4.2.2 Vane shear testing

In addition to the cone penetrometer a vane shear apparatus by Wykeham-Farrance (Fig. 4.3c) is used for additional information on sediment undrained shear strength and particularly residual shear strength, strain behavior and sensitivity. For the measurements, a bladed vane is inserted into the split undisturbed core faces. Torque is transmitted to the vane via a spring that is rotated at a constant rate of $8^\circ$/min (Fig. 4.3d). Shearing occurs along the vertical and horizontal edges of the vane. The torque and degree of rotation of the vane in the sediment are measured simultaneously which can be translated into shear stress and strain. The undrained shear strength, $s_u$ depends on the torque $T$, the vane constant $K$, the maximum torque angle at failure and the spring constant ($B$) that relates the deflection angle to the torque (Blum, 1997). The vane constant, $K$ is a function of the vane size.
and geometry. The undrained shear strength can then be calculated using the following equation:

\[ s_u = \frac{T}{K} \]  

(4.3)

Sensitivity is given by the ratio of peak undrained shear strength and re-molded shear strength, both at natural water content. Details on vane shear testing are given in [Richards, 1988].

**Figure 4.3:** a) Image of a fall cone penetrometer, b) Schematic illustration of fall cone penetrometer test, c) Image of a fall vane shear testing device, d) Schematic illustration of fall vane shear test
4.3  Lab.-based

4.3.1  Direct Shear Testing

The monotonic drained shear strength ($\tau'$) of natural and generic samples has been determined with a state of the art direct shear testing device of the type Giesa RS5 (Fig. 4.4). Circular samples with an area of 25 cm$^2$ and 2.5 cm height were placed in a direct shear box and consolidated to desired effective normal stresses ($\sigma'_n$) via a vertical ram. Two force sensors are attached to the vertical ram, one with a maximum load capacity of 1 kN and another with a maximum capacity of 10 kN. The accuracy of the 1 kN and 10 kN sensors are $\pm 2.5$ N and $\pm 1.25$ N, respectively.

![Image of a Giesa RS5 direct shear device](http://www.giesa.de)

The Giesa RS 5 is capable of displacement- or force-controlled shear tests. Using a direct shear device implies shear along a pre-defined horizontal shear plane that is orthogonal to the vertical effective normal stress. The shear-box consists of two stacked metal plates that move horizontally relative to
each other in one direction. Figure 4.5 a shows a schematic illustration of a sheared sample in a direct shear cell including a brief description of the cell components. The effective horizontal shear stress ($\tau'$) is recorded via a force sensor with a maximum capacity of 5 kN and an accuracy of $\pm 1.5$ N. The vertical and horizontal displacements are recorded during shear. The vertical displacement ($\delta h$) gives information about volume changes of the sample induced by the shearing process. The shear rate depends upon the plasticity index ($I_p$) of the material. The more plastic it is the lower is the adequate shear rate. In generally permeability decreases with increasing plasticity. Effective stress conditions are only known if constant drainage and zero pore pressure can be assumed. For that purpose the shear rate is decreased in highly plastic, impermeable materials such as clays. Most samples follow the Mohr Coulomb constitutive failure law [Handin, 1969] that describes a linear relation between effective normal stress and shear strength of the material:

$$\tau' = c' + \sigma'_n \times \tan(\phi')$$

(4.4)

Where $c'$ is the cohesion, i.e. the strength at $\sigma'_n = 0$ or the intercept of the failure line with the y-axes, $\phi'$ is the angle of internal friction. Figure 4.5b schematically shows the results of three direct shear tests at different effective normal stresses, both in the stress-strain ($\tau-\epsilon$) and effective normal stress - shear stress ($\sigma'_n-\tau$) space. The linear fit of these three points corresponds to the Mohr-Coulomb failure line with the cohesion and internal friction given as mentioned above. The standard testing procedure is given in the DIN 18137 [Deutsches Institut für Normung, 2002].
4.3.2 The Dynamic Triaxial Testing Device (DTTD)

The Dynamic Triaxial Testing Device (DTTD) of the marine geotechnics working group at MARUM has been introduced by Kreiter et al. [2010]. This apparatus is capable of drained and undrained, monotonic and cyclic shear experiments each, force or strain controlled following the DIN 18137 [Deutsches Institut für Normung, 2002]. The test equipment is composed of

![Image of the Marum Dynamic Triaxial Testing Device (DTTD) and its components surrounding the triaxial cell (taken from [Kreiter et al., 2010b])

Figure 4.6: left) Image of the Marum Dynamic Triaxial Testing Device (DTTD) right) schematic illustration of the triaxial load frame and its components surrounding the triaxial cell (taken from [Kreiter et al., 2010b])

a servo-driven hydraulic cylinder with a free configurable real time controller, a hydraulic power unit, displacement-, load- and pore pressure transducers, a load frame, a pneumatically controlled confining and backpressure unit, and a control station with a user interface to initiate the testing scenarios (Fig. 4.6) [Kreiter et al., 2010]. The displacement is currently measured via (i) an internal inductive displacement sensor within the hydraulic cylinder (ii) an external laser optical triangulation system with a measuring range of 100 mm, a dynamic resolution of 0.003 mm at 2.5 kHz sampling frequency and since 5/2013 (iii) an external triangulation system with a measuring range of 20 mm, a dynamic resolution of 0.001 mm at 2.5 kHz. I.e. the external laser sensors are accurate and fast enough to track the strain of
4.3. LAB.-BASED

stiff materials under dynamic loads of up to 50 Hz. Load information is acquired via i) a triaxial cell external high-precision force transducers with a range of 2.5 kN and 0.001% full scale accuracy and ii) since 09/2013 an additional cell-internal force sensor with a range of 1.5 kN and 0.001% full scale accuracy has been implemented. Figure 4.7 shows a technical sketch of a triaxial cell used in the course of this thesis. This cell is placed in the load frame of the DTTD after sample assessment. Cylindrical samples with a surface area of 10 cm$^2$ and a height of 7-10 cm can be tested with this system. The maximum compatible absolute pressure is 800 kPa. The sample can be drained at its upper and lower end. Back pressure (BP) can be induced via the pneumatically driven back pressure unit. A rubber membrane separates the sample from the surrounding fluid that is set under pressure and constitutes the minor principal stress ($\sigma_3$).

Figure 4.7: Schematic illustration of a triaxial cell and its components. $\sigma_1$: major principle stress, $\sigma_3$: minor principle stress. (Design and development of this triaxial cell has been done by in the working group of Prof. Tobias Mörz at MARUM, Centre for Marine Environmental Sciences, University of Bremen. Cell drawing by Marc Huhndorf)
4.3.3 Earthquake shaking simulation in the laboratory

In case of seismic shaking, cyclic shear forces add to the static gravitational forces that constantly act on the slope sediment and may lead to failure (Fig. 4.8) [Biscontin and Pestana, 2006]. The dynamic properties of slope sediments determine their resistivity to earthquake shaking. Cyclic loading experiments can be conducted in the DTTD to simulate earthquake shaking and analyze the behavior of sediment under cyclic loading conditions. Earthquake shaking generally induces a complex three-dimensional motion of a sediment body that is not easily reproducible one-to-one and as well mathematically quantifiable in laboratory testing. If this motion is recorded on a seismometer, the horizontal acceleration component which represents the shear-wave component leading to shear failure of the soil can be extracted and expressed in terms of shear stress variation through time. (Fig. 4.9a and b). The shear-stress signal i) is an irregular time history and ii) depends on the site-to-source distance, magnitude and duration of the earthquake [Kramer, 1996]. Seed and Idriss [1971] resorted to the Palmgren-Miner (P-M) cumulative damage hypothesis which applies to high cyclic fatigue behavior of metal [Green and Terri, 2005; Miner, 1945; Palmgren, 1924] and developed a simplified procedures to convert an irregular time history of shear stresses to an equivalent damaging number of uniform stress cycles. The amplitude of uniform cyclic loading is taken at 65% of the maximum shear stress amplitude of any arbitrary signal. In comparison of field and laboratory data, this value (65%) has been found to represent the arbitrary loading signal with a reasonable degree of accuracy. Since, the uniform stress amplitude has mostly been estimated as:

\[ \tau_{\text{cyc}} = 0.65\sigma_{\text{max}} = q_{\text{cyc}} \frac{q_{\text{cyc}}}{2} \]  

\[ (4.5) \]
\[ \tau_{\text{max}} = \frac{a_{\text{max}}}{g} \times \sigma' \times \tau_{\text{cyc}} \]  

(4.6)

Where \( \tau_{\text{cyc}} \) is the amplitude of uniform cyclic shear stress, \( \tau_{\text{max}} \) is the peak shear stress of the arbitrary loading signal, \( a_{\text{max}} \) is the maximum horizontal acceleration during earthquake shaking, \( g \) is the gravitational acceleration, \( \sigma' \) is the effective overburden stress, \( q \) is the deviatoric stress and \( r_d \) is a reduction factor that takes flexibility of the soil into account (\( r_d = 1 \) corresponds to a rigid body behavior) (Fig. 4.10).

**Figure 4.9:** Schematic illustration of a) a horizontal acceleration time history b) the corresponding horizontal shear stress signal and c) the equivalently damaging uniform cyclic loading signal. \( a_{\text{max}} = \) maximum horizontal acceleration, \( \tau_{\text{max}} = \) maximum shear stress, \( \tau_{\text{cyc}} = q_{\text{cyc}}/2 = \) cyclic shear stress amplitude.

**Figure 4.10:** Schematic for determining maximum shear stress, \( \tau_{\text{max}} \), and the stress reduction coefficient, \( r_d \). \( \gamma' \) = effective unit weight, \( h = \) depth, \( a_{\text{max}} = \) horizontal acceleration, \( \tau_{\text{max},r} = \) maximum shear stress in a rigid body, \( \tau_{\text{max},d} = \) maximum shear stress in a ductile body, \( r_d = \) reduction coefficient (modified after Idriss and Boulanger, 2006).
In case a soil is tested for a specifically known earthquake signal, \( r_d \) can be defined precisely. It is a function of depth, moment magnitude, maximum acceleration and shear wave velocity [Cetin and Seed, 2004]. Else, if a soil is tested preventatively for its resistance to cyclic loading, \( r_d \) can be estimated from Figure 4.11. In addition to the equivalently damaging loading amplitude, the equivalent number of uniform stress cycles also needs to be estimated. Seed and Idriss [1971] analyzed several large magnitude earthquakes for their number of peaks with \( \tau > 0.65 \tau_{\text{max}} \) and showed that the number of equivalently damaging uniform cycles increase with the earthquake magnitude (Fig. 4.12). Liu et al. [2001] analyzed 1664 recording from 150 worldwide earthquakes with a magnitude range of \( m = 4.7-7.6 \) and a distance of 0-200 km for their equivalent number of uniform cyclic loadings at \( \tau > 0.65 \tau_{\text{max}} \). Figure 4.13 shows the resultant plot of the uniform number of equivalent cycles (N) as a function of the site-to-source distance and earthquake magnitude. It can be seen that (N) increases i) with earthquake magnitude and ii) with distance to the epicenter. N is always slightly higher in soil compared to rock sites. In laboratory triaxial testing sediment samples are usually subjected to uniform cyclic shear stress. The resistivity of a soil is often illustrated in ‘Cyclic Stress Ratio’ (CSR) vs ‘Number of Cycles at Failure’ plot. The CSR is defined as the uniform cyclic stress amplitude (\( \tau_{\text{cyc}} \)) normalized to the confining pressure \( \sigma'_{\text{3}} \) or initial mean effective normal

**Figure 4.11:** Stress reduction coefficient as a function of earthquake magnitude (M) and depth.
stress \((p'_0)\) which are equivalent under isotropic conditions:

\[
CSR = \frac{\tau_{cyc}}{\sigma'_3} = \frac{q_{cyc}}{2 \times p'_0}
\]

(4.7)

In order to determine the number of cycles at failure, failure criteria need to be defined. Failure in cyclic loading experiments is usually defined either via a critical pore-pressure ratio or a critical strain accumulation. In isotropic undrained cyclic triaxial loading experiments the occurrence of failure is equivalent to the occurrence of liquefaction and/or the occurrence of 5% double amplitude axial strain \(\epsilon_a\). Liquefaction is defined by a pore pressure ratio \((\text{ppr})\) of 100%, i.e. \(u = p'_0\). Liquefaction typically occurs in non-cohesive soils such as sands and silts (see also section 4.1). Samples loaded under anisotropic cyclic loading may be considered as failed after 2.5% axial strain as \(\text{ppr} = 100\%\) is seldom reached for anisotropically consolidated conditions. These failure criterions are not mandatory, but necessary to specify. The cyclic strength data can be normalized to the simulated lateral earth pressure coefficient \((K_0)\) by plotting the number of cycles to failure against the Modified Stress Ratio \((\text{MSR})\), defined as [Garga and McKay, 1984]:

\[
MSR = \frac{\tau_{cyc}}{\sigma'_{1,0}} = \frac{q_{cyc}}{2 \times \sigma_{1,0}}
\]

(4.8)

Where \(\sigma'_{1,0}\) is the initial effective principal stress.

Figure 4.12: Number of equivalently damaging uniform stress cycles as a function of earthquake magnitude \((M)\) (taken from [Idriss and Boulanger, 2006]).
Figure 4.13: Number of equivalently damaging uniform stress cycles as a function of earthquake magnitude (M) and site-to-source distance. (taken from [Liu et al., 2001])
Chapter 5

Validation of the MARUM DTTD performance

5.1 Static and Cyclic Shear Strength of Cohesive and Non-cohesive Sediments

MARUM - Research Center for Marine Environmental Science and Faculty of Geosciences, University of Bremen, Leobener Str., 28359 Bremen, Germany.

Published in Submarine Mass Movements and Their Consequences, 5th International Symposium

5.1.1 Abstract

Submarine slope failures are common along tectonically and seismically active margins and may have devastating impact on onshore and offshore infrastructure as well as coastal communities. Soils show a variable response to periodic loading compared to static loading – making dynamic and cyclic loading experiments compulsory for submarine slope stability and mass-movement initiation studies. Here we present results from (i) a generic study investigating the shear strength of water-saturated sediment upon static vs. cyclic loading, and from (ii) a comparison to natural samples. We used a direct shear apparatus and the MARUM Dynamic Triaxial Testing Device to compare cyclic to static shear strengths of reconstituted samples with different clay-to-quartz (sandy silt) ratios. With this experimental set-up we aim to identify the ratio of cohesive to granular material at which either liquefaction potential or
the coefficient of friction dominates submarine slope stability. Results indicate that the cyclic shear strengths of material mixtures with less than 15% clay content are significantly lower than their static shear strengths. Mixtures with a clay content exceeding 23% show similar cyclic and static shear strengths. Ongoing studies build on the knowledge gained from the generic end-member tests and integrate natural samples from the Nankai Trough accretionary wedge (Japan).

**Keywords:** Slope failure, shear strength, cyclic vs. static loading, Nankai Trough

### 5.1.2 Introduction

Submarine landslides may occur along the shelves and slopes throughout the world’s oceans and transport sediments and/or rock blocks from upslope, high potential energy positions, downslope by converting potential to kinematic energy. A part of the energy may be transmitted to the ocean surface causing tsunamis (Feeley, 2007). Economically speaking, the understanding of submarine mass movements and their consequences becomes crucial due to their destructiveness. The cost and efforts that evolve from the damage submarine mass movements can have on pipelines and telecommunication lines through rupture, is tremendous (Locat Lee; 2002). Scientists and geo-engineers have been working on the understanding of submarine mass movements for several decades aiming for reliable prediction and mitigation of both, the initiation and the consequences. Among the numerous short-term triggers for initiation (see review by Hampton et al., 1996), earthquake shaking is a prominent one.

Given that most sediments show a different response to seismic loading than to static loading, the comparison of cyclic and static loading experiments are compulsory for submarine slope stability and mass-movement initiation studies. Seismic activity can lead to alternating stress amplitudes, which usually result in a decrease in strength (see Sultan et al., 2004). Numerous cyclic triaxial tests show that granular soils often fail in conjunction with liquefaction (e.g. Ishihara et al., 2004; Singh, 1995, Kreiter et al., 2010). Liquefaction occurs when frictional forces acting upon the grain skeleton are annihilated due to excess pore pressure increase.

Failure of clay-dominated soils may either be caused by cyclic softening or by static shear. The presence of weak mineral phases, such as clays, causes low internal friction and might lead to gravitational slope destabilization due to gravitational forces (Ferentinos et al., 1981; Boulanger Idriss, 2006; Brown et al., 2003; Huhn et al., 2006).
However, non-ambiguous allocation of slope failure either to the presence of weak mineral phases or to granular sediment deposits that may liquefy during cyclic loading is difficult. Here we aim at addressing this challenge by comparing the shear strength of artificial sample materials with different clay content upon static and cyclic loading conditions. The artificial samples are alike different types of hemipelagic sediments occurring along continental margins. In an applied case study, we then expose natural slope-sediment samples from the Nankai Trough accretionary wedge (Japan) to cyclic loading. Earthquake shaking is common along this subduction margin (e.g. Ando, 1975) so that knowledge concerning the sediment's strength to cyclic stress is indispensable.

5.1.3 Methods

Research approach A soil’s response upon loading depends highly on the amount and type of clay minerals present (Bray Sancio, 2006). Brown et al. (2003) show within a series of ring shear tests that the coefficient of residual friction $r$ of granular, non-cohesive material (quartz) decreases from about 0.6 to 0.1 when mixed in ratios from 0% to 100% with cohesive, weak mineral phases (e.g. smectite, illite). On the other hand, the cyclic shear strength of silt-clay mixtures with PI-values exceeding 5%-10% increases with increasing plasticity index (Prakash Puri, 1999). The plasticity index is the difference between the water content at the liquid limit and the plastic limit. Based on these observations we conceive a threshold value of cohesive to non-cohesive material ratio at which the static shear strength of a soil might drop below the cyclic shear strength. In a first order approximation we compare cyclic and static shear strength of reconstituted sandy silt–clay mixtures in a bandwidth of 0% to 60% clay mineral content. The static peak shear strength determined in a direct shear box is compared to the shear strength at failure during cyclic triaxial loading at almost identical initial boundary conditions.

5.1.3.1 Sample description

For the generic study, all samples were reconstituted from two end member materials in different ratios. An industrially produced silica powder (Microsil M4) served as granular, non-cohesive end member. SEM-photos of the crushed granular end member indicate high angularity and low sphericity. The clay size fraction consists of non-cohesive quartz particles. Natural illite and montmorillonite were mixed in a 50/50 ratio and served as cohesive, plas-
tic end member. Seven different mixtures with clay mineral contents ranging from 0% to 60% have been saturated with sea water and tested. Figure 5.1 shows the grain size distribution for all reconstituted material mixtures and the natural samples of the Nankai Trough. Note that sieve analysis has only been done on the S_{10} sample material. The clay mineral content of the clayey end member that results from XRD analysis equals the amount of clay size particles (<2 μm) within an error of ±5%. All silt-size particles of the clayey end member are therefore regarded as non-cohesive.

Figure 5.1: Cumulative grain size distribution curves for all artificial samples (continuous lines) determined after Bouyoucos-Casagrande method and natural samples (dotted lines) determined by laser diffraction. Indices of reconstituted sample materials indicate the clay mineral content.

Table 1 lists the index properties water content (wc), bulk density, B-value, medium grain size (D_{50}), grading (G_0 = \sqrt{D_{75}/D_{25}}) and the coefficient of uniformity (U = D_{60}/D_{10}) of the artificial and natural samples. Atterberg limits are listed for artificial samples with clay mineral contents of 14% or more.

The three natural samples from the Nankai Trough originate from undisturbed whole round core samples recovered from the hemipelagic slope apron in the mid-slope at the NanTroSEIZE Sites C0004 and C0008, from depths down to 41 mbsf. See Kinoshita et al., 2009, or also Strasser et al., (this volume) for more information about the Nankai Trough study area, site locations and detailed description of related data and results. Sample materials
5.1. COHESIVE VS. NON-COHESIVE SEDIMENT

consist mainly of clayey silt with little fine sand fractions. The principal constituents are different clay minerals, quartz, plagioclase and calcite. The grain size distribution of the natural samples are nearly congruent and have a grain size fraction < 2 \( \mu \)m (clay size fraction) that lies between the artificial sample \( S_{14} \) and \( S_{31} \). In terms of grading the natural samples are comparable to the sample material \( S_{14} \). The uniformity coefficient of the natural samples is similar to that of \( S_0 \).

5.1.3.2 Testing Procedure

The direct shear apparatus GIESA RS2 that complies with DIN (18137) was used to measure the static peak shear strength of each reconstituted sample. The direct shear box consists of two stacked rings that hold a sample of 2.5 cm height and 40 cm\(^2\) surface. The contact between the two rings is at approximately the mid-height of the sample. The sample is placed between two porous plates to allow free drainage and then loaded vertically with constant stress until the consolidation is finished. The lower ring is then pulled laterally with a rate of 0.002 mm/min (PI>40), 0.008 mm/min (25<PI<40) or 0.04 mm/min (PI<25) for a path of at least 8.5 mm until residual shear strength is reached. Shear strength of each sample material was tested at normal stresses of 150 kPa, 300 kPa and 450 kPa.

The shear strength under cyclic loading conditions was tested using the MARUM Dynamic Triaxial Testing Device (DTTD). The capacity of the DTTD is to induce cyclic stresses upon a specimen over a computer control unit that allows the user to apply up to 15 kN dynamic load at frequencies of up to 50 Hz and displacements of \( \pm 0.5 \) mm (see details in Kreiter et al., 2010).

Cylindrical specimens with a basal area of 0.001 m\(^2\) and a minimum height of 68.11 mm were prepared. The vacuum saturation method (i.e. applying low-pressure of - 1.0 bar to the samples with subsequent flooding with de-aired water) was applied to improve saturation for artificial samples. Saturations of 95% were reached for six of seven samples (B-value in Tab.3.1). Due to their fragility, vacuum saturation was not carried out for the Nankai samples and, therefore, lower saturation ratios of 60% -85% were reached.

Undrained, symmetric, cyclic loading was carried out at a loading frequency of 1.0 Hz at (i) 100 kPa initial effective confining pressure for artificial samples and (ii) in-situ confining pressure for natural Nankai samples.

For artificial samples, maximum loading period was 2.5 minutes resulting in 150 cycles. Unlike the generic end member samples, the natural samples were tested independently of their numbers of cycles to failure.
5.1.3.3 Data acquisition and analysis

The results of the direct shear tests were used to deduce the parameters of the Mohr-Coulomb constitutive law:

$$\tau'_f = \sigma'_n \times \tan(\phi') + c'$$

(5.1)

where $\phi'$ is the effective friction, $c'$ the effective cohesion, $\sigma'_n$ the effective normal stress and $\tau'_f$ the shear stress, both at failure. The shear strength at 100 kPa was calculated using the appropriate Mohr-Coulomb parameters for all the materials.

In cyclic loading experiments, the cyclic stress ratio (CSR) and the cyclic resistance ratio (CRR) are often used to quantify the cyclic load and the resistance of the sediment to that specific cyclic load (Kramer, 1996). The CSR is defined as:

$$CSR = \frac{q}{2 \times \sigma'_3}$$

(5.2)

$$q = \sigma'_{1,\text{dyn}} - \sigma'_3$$

(5.3)

where $q$ is the cyclic deviator stress, $\sigma'_{1,\text{dyn}}$ is the effective dynamic vertical stress and $\sigma'_3$ is the effective minor principal stress at the start of the cyclic loading. The CRR is the CSR for a given number of cycles at failure. The CRR is a measure for the capacity of a sediment layer to resist failure (Sultan et al., 2004). The ratio between CRR and CSR is the factor of cyclic safety.

The maximum cyclic shear stress and mean normal stress are defined as:

$$\tau'_{\text{dyn}} = \frac{q}{2}$$

(5.4)

$$\sigma'_{n,\text{dyn}} = \frac{\sigma'_{1,\text{dyn}} + 2\sigma'_3}{3}$$

(5.5)

where $\tau'_{\text{dyn}}$ and $\sigma'_{n,\text{dyn}}$ are the effective cyclic shear and mean normal stress, respectively. Hence, the CSR can also be understood as ratio between cyclic shear stress and the effective minimum principal stress.

The recorded parameters during cyclic loading experiments include the axial strain, vertical load $\sigma_1$, cell pressure $\sigma_2 = \sigma_3$ and pore pressure $u$. The axial strain is defined as the vertical displacement of the piston. Vertical load was monitored by a force sensor located outside the triaxial cell. Pore pressure was measured with a differential pressure transducer connected to the pore pressure port. In order to improve the data handling and to reduce the noise the mean of 10 values of the data (sampled at 1000 Hz) was used, resulting in 100 data points per second.

Artificial samples were considered as failed when either:
5.1. **COHESIVE VS. NON-COHESIVE SEDIMENT**

1. Pore pressure reached 90% of the effective confining pressure within 150 cycles, i.e. when liquefaction is initiated (Kramer, 1996), or when

2. Stiffness drops below 0.5 kPa, i.e. the required pore pressure for liquefaction initiation has not been reached within 150 cycles.

### 5.1.4 Results and Discussion

#### 5.1.4.1 Exemplary cyclic test results

Figure 5.2 shows an exemplary cyclic test record of artificial sample S14 and natural sample C0004C-3H-02. The stiffness of the artificial sample drops below 0.5 kPa after 64 cycles at a CSR of 0.26. The double amplitude axial strain of 5% is reached after 70 cycles. The natural sample C0004C-3H-02 reaches that failure criterion after 9 significant cycles at a CSR of 0.49. Both plots show the evolution of excess pore pressure and axial strain induced by symmetric, cyclic loading, expressed in CSR with the number of loading cycles.

![Figure 5.2: Plot A and B show the evolution of excess pore pressure, axial strain and stiffness degradation induced by symmetric, cyclic loading, expressed in CSR with the number of loading cycles. A) sample material S20 and B) natural sample C0004C-3H-02](image)
5.1.4.2 Generic Study

Figure 5.3 shows the peak static shear strength and the cyclic shear strength as derived from DTTD and direct shear tests, respectively, versus the clay mineral content of the individual artificial test samples. Sample materials with clay mineral contents ranging between 0% and 15% show higher resistivity to static shear stresses than to cyclic shear stresses. The static and cyclic shear strength of the granular end member differ the most, although it must be noted that the static shear strength of the granular end member plotted here may not be totally correct. We expect relatively high static shear strength due to high angularity that leads to interlocking of particles, but a peak shear stress exceeding the normal stress is rather unlikely. The difference between static and cyclic shear strength decreases with increasing clay mineral content. Material with clay mineral content above 15% show similar static and cyclic shear strengths.

The only constraint of the cyclic test procedure on artificial samples is to induce failure within 150 loading cycles; at whichever cyclic shear stress. The exact number of cycles to failure must be regarded to make a prediction of a potentially lower or higher cyclic shear stress bringing the specimen to failure. Three tests have been conducted on the granular end member (S0, S01 and S02). These tests show exemplarily the effect of varying cyclic shear stress on the number of cycles to failure. The grey corridor in the background of Figure 5.3 shows a conceivable bandwidth of cyclic shear stresses leading to failure within the constraints of the cyclic testing procedure. We assume an increase of the number of cycles to failure with decreasing cyclic shear stress. The lower border of the grey band has been interpreted with the help of experiments that did not fulfill the constraint of failure within 150 cycles. The consideration of higher shear stresses in conjunction with lower numbers of cycles to failure leads to the interpretation that the peak static shear stress required for failure is similar or even below the cyclic shear stress at clay mineral contents above 23%.

5.1.4.3 Case study

Grain size distribution of the natural Nankai samples show a low clay content, even though their sand content is significantly lower than the sand content of the artificial samples (see Fig. 5.1). Considering the silt fraction to behave as granular material they may be comparable to artificial samples with low clay content. Therefore we tentatively follow the conclusion from the generic study that material with low clay content is less stable under cyclic loading.
5.1. COHESIVE VS. NON-COHESIVE SEDIMENT

Figure 5.3: Evolution of maximum shear stress ($q/2$) at failure under undrained cyclic loading [red graph] and the peak, drained shear strength ($\tau_f$) [blue graph] under static conditions in dependence of the clay mineral content. The standard error of the XRD analysis (±5% clay mineral content) is plotted. The legend lists the sample names ($S_0$ to $S_{60}$). The indices represent the clay mineral content. The CSR and the number of cycles to failure (indices of CSR) are listed for each specimen.

than under static loading, and suggest that cyclic loading tests are an important factor for assessing slope stability in the Nankai Trough setting. Figure 5.4 shows the variation of CRR as a function of the cycles to cyclic softening failure. To evaluate the failure potential of the slope sediments covering the Nankai Trough accretionary wedge, we fitted a logarithmic trend line to the data points. Initial results from natural Nankai slope sediment samples show 5% double-amplitude strain at a CSR of 0.49 after 3 loading cycles (C0008A-3H-10), CSR of 0.49 after 9 loading cycles (C0004C-3H-02) and CSR of 0.31 after 795 loading cycles (C0008A-5H-06), respectively. It has to be noted that the sample C0008A-5H-06 experienced multiple undrained dynamic loading steps until failure, as the original applied load was too low to trigger failure.

To conceptually show the potential of evaluating seismic slope stability using this data set, we examine critical stability conditions for a hypothetical earthquake of magnitude 7.5 in 10 – 60 km epicentral distance. According to
Figure 5.4: Potential failure diagram: cyclic resistance ratio as a function of the numbers of cycles to failure of natural samples from slope sediments overlying the Nankai Trough accretionary prism. The grey band illustrates the example of an earthquake of magnitude 7.5 at 10 – 60 km epicentral distance with 19 – 23 significant cycles according to the empirical regression equations given by Liu et al. (2001).

empirical regression equations given by Liu et al. (2001), such an earthquake may have 19 – 23 significant cycles. For such a scenario, our data indicate material failure at a CSR of 0.44 for 19 significant loading cycles and CSR of 0.43 for 23 significant loading cycles. Such results can then be evaluated to quantitatively assess critical ground motion scenarios on the slope stability conditions in the study area using the factor of safety method. However, this result is only based upon three cyclic triaxial tests of representative material and more tests are required to verify the failure criteria and account for the natural variability of the sediments.

5.1.5 Conclusion

The comparison of the cyclic shear strength with the peak stress during direct shear experiments reveals that sediments with a clay mineral content ranging from 23% - 60% show a similar likelihood to failure under static and
cyclic conditions. This enforces that slope destabilization can occur by the presence of weak mineral phases alone. There is no imperative need for an external, short-term stimulus to trigger soil failure, although further studies need to be carried out on samples with more than 60% clay content in order to validate this statement more broadly.

Initial results from natural slope sediment samples of the Nankai Trough show a general reduction of strength with an increasing cyclic stress ratio and an increasing number of cycles. It defines a preliminary empirical relationship, which – when further improved with additional tests to account for natural variability of samples – may contribute to a comprehensive submarine slope stability analysis of the Nankai Trough accretionary prism.

5.1.6 Acknowledgement

This research used samples and data provided by the Integrated Ocean Drilling Program (IODP). This study was funded through DFG-Research Center / Cluster of Excellence “The Ocean in the Earth System”. Reviewer Dr. G. Biscontin and Dr. K. Lesny are kindly acknowledged. Mathias Lange is thanked for outstanding technical assistance with the DTTD and Direct Shear devices. Alois Steiner is thanked for constructive ideas and discussions.

5.1.7 References


CHAPTER 5. VALIDATION OF DTTD


DIN Deutsche Institut für Normung e. V. (1990): DIN 18137-Bestimmung der Scherfestigkeit, Berlin, Beuth Verlag

Feeley. K. (2007), Triggering Mechanisms of Submarine Landslides, Research Report, Department of Civil and Environmental Engineering, Northeastern University Boston, Massachusetts 02115


Huhn, K., I. Kock, and A. Kopf (2006), Comparitive numerical and analogue shear box experiments and their implications for the mechanics along the failure plane of landslides, Norwegian Journal of Geology, 86 (3): 209-220


5.1. **COHESIVE VS. NON-COHESIVE SEDIMENT**


CHAPTER 5. VALIDATION OF DTTD
Chapter 6

Laboratory based studies

6.1 On the Mechanical Strength of Volcanic Material

G. Wiemer$^1$ and A. Kopf$^4$

$^1$ MARUM - Center for Marine Environmental Sciences, Bremen, Germany. gwiemer@marum.de and akopf@marum.de

Submitted to Géotechnique

6.1.1 Abstract

The largest, often tsunamigenic submarine slides are associated with volcanioclastic materials (e.g. Hawaii, Canary Islands). Volcanic fall out ashes may potentially liquefy and act as preferential glide planes for submarine slides along active margins. However, particle angularity and surface roughness of volcanic material are believed to significantly influence its shear behavior and hence do not unambiguously support this contention. This manuscript presents drained direct and undrained monotonic and cyclic triaxial shear experiments ($\sigma'_{n} < 1\text{MPa}$) on volcanic silts/sands and well-rounded quartz sand for reference. Results attest the key role of particle geometry in sediment stability. Decreased failure susceptibility of volcanic material compared to smoothed, rounded and hard-grained quartz sand was attributed to roughness, angularity, and low crushability at low effective confining stresses in drained direct and undrained cyclic shear experiments. However, major weakening due to crushability and inherent excess pore pressure build-up due to particle internal water expulsion was observed in undrained monotonic shear experiments on soft-grained and porous volcanic material. In contrast
hard-grained volcanic particles are stable whereas equivalent experiments on quartz sand show limited liquefaction. We hence suspect that unaltered ash layers may not serve as glide or failure planes in the majority of earthquake triggered submarine slides.

**Keywords:** Slope failure, volcanic ash, shear strength, liquefaction susceptibility, roughness, and angularity

**Notations**

- **B-Value**  
  Skempton's pore pressure parameter \((\Delta u/\Delta \sigma_c)\)
- **CSR**  
  cyclic stress ratio
- **\(e_{\text{max}}, e_{\text{min}}\)**  
  maximum and minimum void ratio
- **\(I_{d,0}\)**  
  initial density index
- **\(I_{d,c}\)**  
  densities index at critical state
- **\(I_{d,s}\)**  
  densities index after consolidation
- **\(K_0\)**  
  coefficient of lateral earth pressure
- **\(p'\)**  
  triaxial effective normal stress
- **\(p'_0\)**  
  initial triaxial effective normal stress
- **PPR**  
  pore pressure ratio \((\Delta u/p'_0)\)
- **\(q_0\)**  
  initial deviator stress
- **\(q_{\text{cyc}}\)**  
  cyclic deviator stress
- **\(r_c\)**  
  reversal coefficient [Galandarzadeh and Ahmadi, 2012]
- **\(\Delta u\)**  
  pore pressure
- **\(\epsilon_v\)**  
  vertical strain
- **\(\mu_c\)**  
  critical state friction coefficient
- **\(\mu_p\)**  
  coefficient of peak friction
- **\(\sigma'_{1}\)**  
  effective stress along the first principal axis
- **\(\sigma'_{1,0}\)**  
  initial effective stress along the first principal axis
- **\(\sigma'_{1,\text{max}}\)**  
  maximum effective stress along the first principal axis
- **\(\sigma'_{3}\)**  
  effective stress along the minor principal axis
- **\(\sigma'_{3,0}\)**  
  initial effective stress along the minor principal axis
- **\(\sigma'_n\)**  
  effective normal stress
- **\(\tau'_{c}\)**  
  critical state shear stress
- **\(\tau'_{p}\)**  
  peak shear stress
- **\(\Phi'\)**  
  angle of internal friction
6.1.2 Introduction

Submarine slope failures represent one of the most frequent, dynamic and devastating sedimentary processes along active margins impacting on onshore and offshore infrastructure and coastal communities [Hampton et al., 1996]. Landslides range greatly in their size, from small, frequently occurring failures in active environments such as coastal zones and canyons, to failures that involve hundreds of km$^3$ of sediment but occur much more infrequently. Those megaslides (involving >1000 km$^2$ in area and several 100 km$^3$ of sediment) are often associated with glaciated margins (e.g. along the North Atlantic margin [Maslin et al. 2004; Lee 2009]) and to volcanic flank collapses (Nuuanu slide off Hawaii [e.g. Morgan et al. 2003]; Canary Islands [e.g. Masson et al. 2002], or recent IODP drilling [Expedition 340 Scientists, 2012]). Among the short term triggering mechanisms for slope destabilization, earthquake shaking is abundant and effective [Sultan et al., 2004]. Naturally, seismic activity goes hand in hand with the phenomenon of liquefaction in loosely deposited granular material, such as sands and silts [Wright and Rathje, 2003]. Volcanic ashes by definition comprise sand and silt in terms of grain-size [Fisher and Schmincke, 1984], which is why the submarine landslide community suspects those layers to be highly liquefaction susceptible and as a consequence, to act as preferential glide planes for earthquake triggered submarine slides [Haders et al., 2010; Sassa et al., 2012]. However, the mechanical properties of volcanic ash are poorly investigated though they may play a key role in submarine landslide initiation and may be of further interest for the geotechnical community due to properties that deviate from more common sands. Volcanic ash can be fragile and easily crushable [Pender et al., 2006], but also highly angular and rough [Riley et al., 2003]. This manuscript investigates to what extent these two factors influence the shear behavior compared to more common quartz sands. In essence, the working hypothesis of our study is that volcanic ash sands are more resistant to static and cyclic shear stresses than common quartz sands due to their outstanding roughness and angularity. Moreover, we hypothesize that the strengthening effect is amplified under initial anisotropic stress conditions in laboratory testing, which are closer to natural conditions than isotropic stress conditions [Erguvanil, 1980].

6.1.2.1 Material and intrinsic characteristics

The tested materials comprise:

(a) two terrestrial to limnic deposited pumice sands (PS and PS II) from the Eifel region, Germany
(b) terrestrial deposited volcanic ashes (ChiT), sampled on the flank of Villarrica volcano, south-central Chile

(c) marine pumice sand deposits from the northern Cretan margin (Cremice)

(d) marine tephra deposits from the Cretan margin (Creteph)

(e) marine deposited volcanic silt-sized glass shards (GS) sampled in ∼83 mbsf from the Lesser Antilles at site U1400 [Expedition 340 Scientists, 2012].

(f) quartz dune sand (DS)

Grain-size distributions for each sample material besides the PS II were determined via laser diffraction analysis with a Coulter LS-13320 measuring grain size contents in 117 classes ranging from 0.04 to 2000 μm as a volume percent (vol%) (see [Syvitski et al., 1991] and [Loizeau et al., 1994] for details). The PS and ChiT samples were sieved to DS-equivalent grain-size distributions, to ensure that shear strength differences between these materials are investigated independent of grading. Sieving of the natural marine deposits Cremice and Creteph to DS equivalent grain size was not possible due to minor material abundance. Tests on natural marine deposited glass shard (GS) and the second pumice sand (PS II) enlarge the data set to volcanic material of smaller and larger grain size, respectively. The GS material forms an important part of this study due to the fact, that i) silts can be even more susceptible to liquefaction than sands [Singh, 1996] and ii) glass shards are common within the subduction fore arc [Fisher and Schmincke, 1984]. The results of standard classification tests on the sands are given in Table 3.2 and cumulative grain-size distribution curves are seen in Figure 6.1. Each material has been imaged with regard to angularity and roughness using scanning electron microscopy (SEM). SEM images of PS, ChiT, DS and GS (Figs. 6.1a-d, respectively) clearly show the differences in roughness and angularity of volcanic ash particles in contrast to the DS, independent of the grain size. The PS, ChiT and GS are characterized by high angularity and roughness and were visually interpreted as natural end-member in the estimation chart for roundness and sphericity of [Krumbein and Sloss, 1963]. Sharp angles and rough surfaces are visible down to the 10 μm scale. The glass shards highly resemble broken pieces of pumice sand and show typical convex, concave, and platy shaped particles. The ChiT sand shows vesicular structures that are partly filled with finer particles. The DS particles are well-rounded and spherical in comparison to the volcanic material, and their surface is smooth (Fig.6.1c), and serve as the opposite natural end-member.
6.1. STRENGTH OF VOLCANIC MATERIAL

Figure 6.1: Cumulative grain-size distribution curves of pumice sand (PS) pre- and post-shearing, glass shards (GS), Chilean tephra (ChiT), dune sand (DS), Cretan tephra (Creteph) and Cretan pumice (Creteice). SEM images of a) PS, b) ChiT c) GS and d) DS illustrating the difference in angularity and roughness of volcanic material vs a rounded and smooth quartz sand.
### Table 6.1: Mineralogical components of PS, PS II, ChiT and DS have been determined via semi-quantitative XRD analysis using a Philips X0Pert Pro multi-purpose diffractometer and the software packages Philips X0Pert HighScore and QUAX [Vogt et al., 2002].

<table>
<thead>
<tr>
<th>Material</th>
<th>Mineralogy†</th>
<th>Particle shape ‡</th>
<th>Surface ‡</th>
<th>Void ratio *</th>
<th>Gs **</th>
</tr>
</thead>
<tbody>
<tr>
<td>DS</td>
<td>quartz</td>
<td>rounded</td>
<td>smooth</td>
<td>0.47</td>
<td>0.77</td>
</tr>
<tr>
<td>PS</td>
<td>amorphous silica</td>
<td>angular</td>
<td>rough</td>
<td>1.62</td>
<td>2.09</td>
</tr>
<tr>
<td>PS II</td>
<td>amorphous silica</td>
<td>angular</td>
<td>rough</td>
<td>1.78</td>
<td>2.20</td>
</tr>
<tr>
<td>ChiT</td>
<td>65% plagioclase, 11% phillosilicates, ~15% amorphous parts, 9% other</td>
<td>angular</td>
<td>rough</td>
<td>1.01</td>
<td>1.49</td>
</tr>
<tr>
<td>GS</td>
<td>44% plagioclase, 30% amorphous silica, 16% expandable clays, 4% K-feldspar, 6% other</td>
<td>angular</td>
<td>smooth</td>
<td>n.d.</td>
<td>n.d.</td>
</tr>
<tr>
<td>Ceteph</td>
<td>phillosilicates, 3% quartz 3% amorphous silica</td>
<td>angular</td>
<td>rough</td>
<td>n.d.</td>
<td>n.d.</td>
</tr>
</tbody>
</table>

† Mineralogical components of PS, PS II, ChiT and DS have been determined via semi-quantitative XRD analysis using a Philips X0Pert Pro multi-purpose diffractometer and the software packages Philips X0Pert HighScore and QUAX [Vogt et al., 2002].

‡ Visual interpretation from SEM imaging.

* Minimum and maximum densities of the materials PS, PS II, ChiT and DS have been measured according to the standard DIN 1826 [Deutsches Institut für Normung, 1996].

** Determined with a helium pycnometer
6.1. STRENGTH OF VOLCANIC MATERIAL

to the tephra particles. Since the shear behavior of quartz sand is well documented in the literature [Bolton, 1986; Sadrekarimi and Olson, 2011], DS serves as reference material throughout.

6.1.3 Testing Procedure

6.1.3.1 Direct shear experiments

Cylindrical samples (25 cm$^2$, 2.5 cm height) were placed in a direct shear box where a vertical ram exerts the desired normal stress ($\sigma'_n$) upon the sample [Ikari and Kopf, 2011]. The PS, PS II, ChiT and DS were assessed at initial relative densities ($I_{d,0}$) of $\sim 50\%$ applying the 'air pluviation method'. Relative density is defined as:

$$I_d = \frac{e_{\text{max}} - e}{e_{\text{max}} - e_{\text{min}}}$$

(6.1)

Where $e_{\text{max}}$ and $e_{\text{min}}$ are the maximum and minimum void ratios and $e$ is the void ratio at the state of interest. In addition to the initial relative densities the density index at the start of shearing ($I_{d,s}$) and at 10 mm displacement ($I_{d,c}$) were determined. $I_{d,c}$ did not always represent the critical state due to continuous particle crushing as will be shown in the results. The difference between the relative density after consolidation ($I_{d,s}$) and the initial relative density ($I_{d,0} \sim 50\%$) will be denoted as $\Delta I_{d,1}$. The difference between the relative density at critical state ($I_{d,c}$) (or 10 mm displacement) and the relative density after consolidation will be denoted as $\Delta I_{d,2}$.

The silt-size glass shards (GS) were remolded prior to testing. The Cremice and Creteph material were poured into the direct shear box without determining the $I_{d,0}$ because of small sample volume available. Sample material was sheared under saturated and drained conditions at effective normal stresses ($\sigma'_n$) ranging from 100 to 950 kPa. The Creteph and Cremice were sheared exclusively at 200 kPa and 400 kPa $\sigma'_n$. The shear path for each experiment was at least 11 mm at a shear rate of 0.6 $\mu$m/s. The coefficient of peak and critical state friction, $\mu_p$ and $\mu_c$, were determined after Handin [1969]:

$$\mu_p = \frac{\tau'_p}{\sigma'_n}$$

(6.2)

$$\mu_c = \frac{\tau'_c}{\sigma'_n}$$

(6.3)
$\mu_c$ has been defined as average friction value from 10-11 mm shear path. Furthermore, DS was tested at its densest state ($I_{d,0}$ DS = 92%) and ChiTe sand and PS at their loosest possible state ($I_{d,0}$ ChiTe =19%), each at $\sigma'_{n}=100$ kPa in order to explore the frictional behavior of the end-members at the opposite extremes of pacing density. ChiTe sand was additionally tested at $I_{d,0}=75$ % for the entire range of $\sigma'_{n}$ (100 kPa - 950 kPa) in order to investigate the influence of pacing density on this volcanic, angular sand.

6.1.3.2 Triaxial Shear Experiments

Undrained static and cyclic shear experiments on PS, ChiTe, GS and DS were conducted using the MARUM Dynamic Triaxial Testing Device (DTTD) [Kreiter et al., 2010]. Cylindrical specimens (10 cm$^2$ surface, 7.2 cm height) were prepared at initial relative densities of $\sim$ 50 % using the air pluviation method. Volcanic glass shards (GS) were remolded and pre-consolidated to 100 kPa before being triaxially tested. Prior to each experiment, the sample was vacuum saturated to Skempton’s B-values $\geq$ 0.95 [Skempton, 1954] with de-ionized, de-aired water. Isotropic static and cyclic tests were set to an initial effective mean normal stress ($p'_0$) of 100 kPa and 300 kPa backpressure. The static shear rate was set to 0.1 mm/min. Isotropic cyclic stress tests were conducted under harmonic compression-extension mode ($r_c \sim 0.5$; [Galandarzadeh and Ahmadi, 2012]) with a frequency of 1 Hz. Anisotropic tests were conducted under an initial effective vertical and horizontal stress of 100 kPa and 50 kPa ($p'_0 = 75$ kPa), respectively, and without stress reversal ($r_c > 1$). The cyclic shear stress ratio (CSR) was calculated by normalizing the cyclic shear stress ($q_{cyc}/2$) to $p'_0$:

$$CSR = \frac{q_{cyc}}{2 \times p'_0} \quad (6.4)$$

$$q_{cyc} = \sigma_{1, \text{max}} - \sigma_{1,0} \quad (6.5)$$

$$p'_0 = \frac{\sigma'_{1,0} + \sigma'_{3,0}}{2} \quad (6.6)$$

Where, $\sigma_{1, \text{max}}$ is the maximum vertical stress in direction of the first principal axis $\sigma_1$. $\sigma'_{1,0}$ and $\sigma'_{3,0}$ are the initial effective vertical and horizontal stress at the end of consolidation.

Isotropic cyclic experiments were conducted on each material at CSRs ranging from 0.275 to 0.175. Anisotropic tests were solely conducted at a CSR
6.1. STRENGTH OF VOLCANIC MATERIAL

of 0.195. The failure criterion was a pore pressure ratio of 0.8 and/or 5% maximum strain, regardless of the initial test conditions.

6.1.4 Results

6.1.4.1 Direct Shear Experiments and Relative densities

Shear stress and vertical displacement of each material tested at $\sigma'_n = 100$ kPa and $\sigma'_n = 950$ kPa are plotted against the shear path in Figures 6.2a and 6.2b, respectively. At 100 kPa effective normal stress volcanic material shows significantly higher peak resistance than the DS. Peaks are reached at 2 - 4.7 mm shear path in volcanic material, whereas the DS reaches the peak after $\sim$1.5 mm shear. All samples dilate after a short period of compaction, however, GS remains in an overall compacted state (Fig. 6.2a). PS II juts out with extremely high peak shear strength and dilatation. At high effective normal stress ($\sigma'_n = 950$ kPa) (Fig. 6.2b) none of the tested materials except from DS show dilation. Peak and critical shear resistance of any volcanic material exceeds that of the quartz DS. PS II shows the highest resistance and prominent compaction/crushing during shear. The peak shear stress at 950 kPa is reached after $\sim$1.5 mm for DS, whereas all volcanic materials require larger shear paths than at $\sigma'_n = 100$ kPa indicating increasingly ductile behavior with increasing normal stress. Figure 6.2c shows the test on the DS at $I_{d,0}$ DS = 92%, the ChiT sand at $I_{d,0}$ DS = 19% and the PS at $I_{d,0}$ DS = 13% each loaded at $\sigma'_n = 100$ kPa. The density index after consolidation reached 95%, 28%, and 26%, respectively. Despite the difference in relative density (67%) between DS and ChiT, peak shear resistance differs insignificantly (2%). The loose volcanic sand (ChiT) still shows dilatational behavior. Critical shear resistance is higher in the ChiT sand than in the rounded, smooth quartz DS. PS is initially 69% less dense than the DS, and reaches a peak friction of 0.84 and a critical state friction of 0.76. PS contracts on the initial 3 mm of shear and then slightly dilates at larger strains, however, a net compaction is observed. $\mu_p$, DS is 8% higher than $\mu_p$, PS.

Results from direct shear tests (Fig. 6.3a, 6.3b) provide general observations on peak and critical friction: i) volcanic ashes show significantly higher values than DS, especially at $\sigma'_n < 400$ kPa; ii) friction coefficients of all materials decrease with increasing $\sigma'_n$, eminently in volcanic material; and iii) friction increases with increasing grain size. DS shows typical $\mu_p$ values ($\sim$0.8-0.7) of quartz sand in a medium dense state. Despite their difference in grain size and assessment method, PS and GS show similar peak friction ratios along the range of normal stresses tested, but significantly higher peak
Figure 6.2: a) Drained shear stress and vertical displacement of each material at 100 kPa effective normal stress b) Drained shear stress and vertical displacement of each material at 950 kPa effective normal stress c) Drained shear stress and vertical displacement of DS at high relative density (95%) and ChiT sand and PS at low relative density (28% and 26%, respectively) at 100 kPa effective normal stress

friction values than DS at low normal stress ($\mu_p \sim 1$ at $\sigma'_n = 100$ kPa). ChiT sand shows decreasing $\mu_p$ from $\sigma'_n = 100$ kPa to 200 kPa, an increase from 200 kPa to 600 kPa to $\sim 0.85$, and a decrease to $\sim 0.75$ at $\sigma'_n = 950$ kPa. PS II juts out with extremely high values of peak friction at low normal stress. It decreases from $\sim 1.5$ at 100 kPa normal stress to $\sim 0.77$ at 950 kPa normal stress. Marine tephra deposits from the Cretan margin (Creteph and Cremice) show peak friction values between PS and PS II at 200 kPa and 600 kPa normal stresses, which is mirrored by the grain size distribution (Fig.6.1). Critical friction values of volcanic material are in the range of 0.75-0.89 at low normal stress (100 kPa) and from 0.74-0.72 at high normal stresses (950 kPa) (Fig. 6.3b). Pumice sands with larger grain sizes (Cremice
6.1. STRENGTH OF VOLCANIC MATERIAL

and PS II, however, maintain high critical friction values even after 10-11 mm of shear. Figure 6.3c shows the peak and critical state friction of the ChiT sand at I_{d,0} = 75\% and 50 \% as a function of \( \sigma_n' \). The increase in initial relative density by 25\% results in an increase in peak friction ratio by \( \sim 0.2 \) at \( \sigma_n' = 100 \) kPa. With increasing normal stress this difference decreases to about 0.1 at \( \sigma_n' = 600 \) kPa. The critical state friction ratio does not show significant differences from 100-600 kPa normal stress. However, at \( \sigma_n' = 950 \) kPa difference in peak and critical state friction ratio of the sample at I_{d,0} = 50\% and 75\% can be seen. Note that the experiment with I_{d,0} = 50\% at \( \sigma_n' = 950 \) kPa was the only test showing compaction during shear (see also Fig. 6.4). The peak friction of silts and sands is known to be highly dependent on the initial state of relative density.

**Figure 6.3:** a) Coefficient of peak friction as a function of effective normal stress for each material b) Coefficient of critical state friction (or, if not reached, at 10-11 mm shear path) c) Coefficient of peak and critical state friction of ChiT sand at 100 kPa and 950 kPa effective normal stress

In Figure 6.4a \( \Delta I_{d,1} \) is plotted versus effective normal stress. It can be seen that DS increases its relative density by 14\% to 22\% when loaded to 100 kPa and 950 kPa normal stress, respectively. ChiT shows a \( \Delta I_{d,1} \) evolution similar to DS, but is slightly less compressible at low stresses (\( \Delta I_{d,1} \sim 12 \% \) at \( \sigma_n' \leq 200 \) kPa) and slightly more compressible at high stresses (\( \Delta I_{d,1} \sim 24 \% \) at \( \sigma_n' = 950 \) kPa). There is a drastic near-linear increase of \( \Delta I_{d,1} \) and \( \sigma_n' \).
for the pumice sands. $\Delta I_{d,1}^1$ of PS II increases from 20% at $\sigma'_n = 100$ kPa to 82% at $\sigma'_n = 950$ kPa, indicating prominent crushing during consolidation. Figure 6.4b shows $\Delta I_{d,2}$ as a function of $\sigma'_n$ indicating whether the material behaves in a dilatational ($\Delta I_{d,2} < 0$) or contractional manner ($\Delta I_{d,2} > 0$) during shear. $\Delta I_{d,2} = 0$ represents the relative density at critical state (if $\Delta I_{d,2} = 0$ then $I_d = I_{d,c}$). DS shows dilation in all tests. ChiT sand shows more significant dilation than the DS at 100 kPa, and compacts during shear only at a normal stress of 950 kPa. The pumice sands jut out with extreme compaction, i.e. grain crushing after 10 -11 mm shear. $\Delta I_{d,2}$ of PS II reaches 42% at 950 kPa normal stress, $\Delta I_{d,2}$ of PS reaches 30%. Particle crushing is further illustrated by the post-shearing grain-size distribution curve of PS (Fig.6.1).

**Figure 6.4:** a) Difference between relative density after consolidation ($I_{d,s}$) and initial relative density ($I_{d,0} \sim 50\%$); b) Difference between relative density at critical state (or 10 mm shear path) ($I_{d,c}$) and relative density after consolidation ($I_{d,s}$)

### 6.1.4.2 Triaxial Shear Experiments

*Monotonic undrained loading*

ChiT sand provides the highest resistance to monotonic undrained loading, PS the lowest (Fig. 6.5a). All volcanic materials tested show strain hardening behavior whereas DS softens discretely at $\sim 0.5\%$ strain (see inset of Fig. 6.5a) and thus, shows limited flow. The shear paths were too short to reach ultimate steady state conditions [Castro, 1975; Castro and Poulos, 1977; Mohamad and Dobry, 1986; Poulos et al., 1985; Yoshimine et al., 1999]. At higher strain ($> 10\%$) the volcanic materials, imminently GS, show strain softening (Fig. 6.5a), which is thought to be the initiation of decreasing deviator stress rates towards ultimate steady state conditions. The fact that this point is reached at 10-12 % strain solely in volcanic material might be
6.1. STRENGTH OF VOLCANIC MATERIAL

Figure 6.5: a) Undrained shear strength \( (q/2) \) of PS, DS ChiT sand and GS silt at 100 kPa confining stress plotted against axial strain \( (\epsilon_v) \) b) pore pressure \( (\Delta u) \) plotted against axial strain \( (\epsilon_v) \) c) effective stress paths in the \( p'-q \) space explained by particle crushing, internal particle pore water release and inherent pore pressure increase. The pore pressure evolution during shear is plotted in Figure 6.5b, where values reach a peak at \( \sim 1\% \) axial strain in less crushable material (DS and ChiT), \( \sim 2\% \) in GS, and \( \sim 4\% \) in PS. However, subsequent to the development of pore pressure peaks, phase transformation [Ishihara et al., 1975] occurs accompanied by a dilatational behavior and \( \Delta u \) starts decreasing with further strain accumulation in each material. PS builds up the highest pore pressure, which after the peak decreases less rapidly than in the other materials. This result could be related to excess pore pressure build-up due to decent grain crushing of soft PS particles. DS reaches negative pore pressures more rapidly than the volcanic materials, indicating facilitated dilation due to particle smoothness and roundness. In Figure 6.5c the effective stress paths are seen in the \( p'-q \) space. Here, ChiT sand is characterized by typical dilatational behavior [Vaid and Sivathayalan, 2000], which is in agreement with low pore pressure build up and strain hardening. In contrast, DS shows partial compaction [Mohamad and Dobry, 1986] and limited liquefaction [Castro, 1969] starting at \( p' \sim 56 \) kPa. This means that limited liquefaction behavior precedes the phase transformation point (PTP) [Ishihara et al., 1975], where dilatational behavior reduces the pore pressure and creates an increase in \( p' \) and \( q \). More fragile samples (PS
and GS) show a slightly different behavior, with \( p' \) clearly decreasing during shear as a result of partial compaction. However, no peak in shear stress and inherent limited liquefaction develops during partial compaction. The PTP corresponds to the peak shear stress before phase transformation. PS reaches the lowest \( p' \) at the PTP of all materials. Furthermore, volcanic materials have a steeper failure and phase transformation line than DS. The angles of peak internal friction (\( \mu_p \)) are \( \sim 45^\circ \) in volcanic material and \( \sim 39^\circ \) in DS. These angles correspond to \( \mu \) of \( \sim 1 \) and 0.81, respectively, which agrees with the peak friction during drained direct shear at 100 kPa \( \sigma_n' \) (Fig. 6.3b).

**Figure 6.6:** a) Axial strain (\( \epsilon_v \)) and pore pressure ratio (PPR) evolution during undrained isotropic cyclic loading at a CSR of 0.225; b) first loading cycles on PS, DS, GS and ChiT sand plotted in the stress-strain space c) first loading cycles on PS, DS, GS and ChiT sand plotted in the pore pressure ratio (PPR) - deviator stress (\( q \)) space; d) and e) are equivalent to b) and c), respectively at the 8th loading cycle.
6.1. STRENGTH OF VOLCANIC MATERIAL

Isotropic undrained cyclic loading

Figure 6.6a illustrates the axial strain ($\varepsilon_v$) and pore pressure ratio (PPR) evolution against the number of loading cycles of cyclic isotropic undrained triaxial shear experiments at a CSR of 0.225. The 1st and 8th cycle are shown in detail in the $\varepsilon_v$-q and PPR-q space (Figs. 6.2b - 6.6e). The 1st cycle was chosen in order to show the initial response to isotropic cyclic loading. The 8th cycle was chosen because DS and GS show first signs of failure. It can be seen that the pore pressure and axial strain increase is lowest in DS and highest in the GS during the 1st cycle (Figs.6.2b and 6.6c). DS reaches $\sim 0.05$ double amplitude axial strain (DAS); GS reaches $0.23\%$ DAS. The maximum PPR is $\sim 0.09$ in the DS and $\sim 0.27$ in the GS. PS and ChiT sand reveal intermediate axial strain and PPR loops to DS and GS with ChiT sand being closer to the DS and PS being closer to GS. During the 8th cycle the DAS in the DS and GS are $\sim 0.75\%$ and $2.4\%$, respectively. The major axial strain accumulation occurs during unloading. The loading phase of the 8th cycle of the test on GS is characterized by a PPR increases from $\sim 0.3$ to $> 0.8$. DS shows flow deformation at a PPR of $\sim 0.35$ and phase transformation at the end of the 8th cycle. PS and ChiT show significantly lower axial strain and pore pressure accumulation than DS and GS from the 1st to the 8th cycle. When plotting CSR against the number of cycles to failure of all isotropic cyclic tests (Fig. 6.7), it can be seen that any volcanic material bears a higher number of loading cycles than DS before failure - independent of the applied CSR. Analogously to the results of direct shear experiments (Fig.6.2 and 6.3), ChiT sand and PS are the most resistant materials, followed by GS. DS, however, has the least resistance against cyclic failure.

Figure 6.7: CSR against the number of cycles at failure for all isotropic undrained cyclic loading experiments.
Anisotropic undrained cyclic loading

Figure 6.8a shows the $\epsilon_v$ and PPR evolution against the number of loading cycles of anisotropic undrained triaxial shear experiments conducted under $K_0 = 0.5$, $p'_0 = 75$ kPa and a CSR of 0.195. Equivalently to the isotropic tests two cycles (5th and 305th cycle), one at the beginning of loading and one after a substantial number of loading cycles, were singled out (Fig. 6.8). It can be seen that DS accumulates significantly more axial strain and pore pressure than the volcanic material during the first cycles (Fig. 6.8b). After 300 more cycles of anisotropic loading the axial strain of DS is $\sim$19% whereas the volcanic material shows $\epsilon_v < 2.5\%$. The PPR increases from $\sim$0.3 after 5 loading cycles to $\sim$0.5 after 300 cycles in dune sand. Volcanic material shows a significantly lower increase independent of grain size. At the beginning of loading PS shows the highest PPR. After 300 more loading cycles, GS shows the highest PPR. Furthermore, DS behaves dilatationally during anisotropic loading. All volcanic material shows contractive behavior and PPR increase during the 5th loading cycle. After 300 more loading cycles GS and PS remain in a contractive regime, whereas the less crushable samples (DS and ChiT sand) dilate during the loading phase (Fig. 6.8e).

6.1.5 Discussion

In this study the mechanical strength of volcanic ash material was investigated in drained direct shear testing and undrained monotonic and cyclic triaxial experiments and compared to quartz sand. Samples were assessed at an initial medium density state ($I_{d,0} = 50\%$), which is assumed representative for sands in the marine realm based on the assumption that several processes enhance compaction at an early stage after deposition (e.g. sands at active margins may experience seismic strengthening; [Tokimatsu and Seed, 1987]). The volcanic materials and quartz sand tested were regarded as natural end-members of roughness and angularity (Fig.6.1). Furthermore the direct shear tests reveal a lower grain strength of volcanic material compared to DS, eminently in pumice sands (Fig. 6.4) [Orense et al., 2013]. Hence, the state parameter ($\psi$) introduced by Been and Jefferies [1985], which is the 'state of the art' parameter for the strength comparison of sands, turned out to be inadequate for volcanic sands as these tend to crush and unpredictably reduce their void ratio.

The influence of angularity, roughness, grain strength and grain size on mechanical strength of volcanic material compared to quartz dune sand is discussed in the following:
6.1.5.1 Drained direct shear experiments and relative density

The interpretation of consolidation and vertical displacement data of direct shear experiments towards particle crushability is summarized in Table 3.3. At low normal stress (< 400 kPa), volcanic materials have substantially higher peak friction coefficients than equivalent quartz sand (Figs. 6.1 and 6.2). High values of $\mu_p$ (~1) are interpreted to result from strong particle intercalation (i.e., pushing, rolling) and inherent sample dilation (Fig. 6.2), which is bound to high mobilizing friction [Sadrekarimi and Olson, 2011]. Dilation depends on initial density, normal stress [Been and Jefferies, 1985], particle surface roughness [Dietz and Lings, 2006] grain size and crushability...
Table 6.2: Crushability and grain size of volcanic material compared to DS.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Crushability</th>
<th>Grain-size</th>
</tr>
</thead>
<tbody>
<tr>
<td>PS</td>
<td>intermediate</td>
<td>= DS</td>
</tr>
<tr>
<td>ChiT</td>
<td>low</td>
<td>= DS</td>
</tr>
<tr>
<td>PS II</td>
<td>intermediate</td>
<td>&gt; DS</td>
</tr>
<tr>
<td>GS*</td>
<td>intermediate</td>
<td>&lt; DS</td>
</tr>
</tbody>
</table>

* no relative density data base

(Fig. 6.2a, Table 3.3). Focusing on the sands with equivalent grain sizes (PS, ChiT, DS) reveals that the least crushable volcanic sand (ChiT) shows the strongest dilation at 100 kPa normal stress and $I_{d,0} = 50\%$. PS shows intermediate dilation, which is interpreted as the net result of dilation due to particle surface roughness and crushing. When comparing DS and ChiT sand at low $\sigma'_{n}$, particle surface roughness is singled out as parameter influencing dilation as all other parameters, including crushability, are the same. Dilation may be expressed in energy or shear stress needed to overcome the mobilizing friction. The extreme extent of how angularity and roughness rules over relative density in the shear behavior at low normal stress is shown in Figure 6.2c. DS is nearly 70\% denser than the ChiT sand, however, the peak friction of DS is only 2\% higher. This experiment indicates that highly angular sands with low crushability at low $\sigma'_{n}$, dilate even in a loose density state. The loose PS shows partial dilation only after 4mm shear path when sheared in a loose state (Fig. 6.2c), indicating enhanced crushing and simplified abrasion of asperities resulting in net compaction. The critical state is preceded by dilatational behavior, and shear strength at low normal stress increases significantly with grain size, as shown by tests on PS II, Cremice and Creteph (Figs. 6.1 and 6.2. However, the silt-size glass shards (GS) show only slightly lower coefficients of peak friction than the PS and ChiT sand although the grain size is significantly lower, which indicates a non-linear
6.1. STRENGTH OF VOLCANIC MATERIAL

relationship between shear strength and grain size of angular particles. This interplay of angularity, roughness, crushability, and grain size at low normal stress needs to be kept in mind for interpretation of monotonic and cyclic undrained shear data. With increasing effective normal stress particle crushing increases during consolidation and shearing (Fig. 6.4), and peak friction decreases (Fig. 6.3) in volcanic material. Enhanced particle abrasion and crushing at high normal stresses is thought to reduce the mobilizing friction leading to an overall reduction of $\mu_p$. Thus, direct shear experiments reveal a decreasing influence of angularity and roughness with increasing normal stress in fragile, crushable volcanic sands. Similar observations have been made for pumice sand [Orense et al., 2012; Pender et al., 2006] and volcanic material involved in a landslide at Stromboli, Italy [Rotonda et al., 2010].

The fact that the plagioclase-dominated volcanic sand (ChiT) is less crushable than the amorphous silica samples is not surprising, as the crystalline structure of the particles reduces the crushability. Sharp edges of the particles may break off, but unlike pumice sand or glass shards the whole particle will not collapse. The critical state friction coefficient of DS ($\mu_c = 0.6$) is slightly lower than standard critical friction attributed to quartz sands ($\mu_r = 0.65$) from the sphericity and smoothness of the particles [Bolton, 1986]. Critical friction coefficients of ash is consistently but not abnormally high: $\mu_c$ of 0.7 - 0.8 have been observed in many, especially in feldspar-rich sands [Bolton, 1986; Sadrekarimi and Olson, 2011]. However, particle intercalation and continuous crushing may influence $\mu$ even at high strains and normal stress. Pender et al. [2006] found that the stress-strain-strength behavior of pumice sand is strongly dominated by particle crushing, which also explains why the critical state was not always reached in our experiments.

6.1.5.2 Undrained monotonic shear experiments

Monotonic undrained triaxial tests at low confining pressure reveal crushability as dominating factor over angularity and roughness. The least crushable volcanic sand (ChiT) shows strength differences equivalent to the non-crushing quartz sand in drained direct shear tests. Both fragile glass shards and highly porous and fragile PS, however, show significant weakening when compared to ChiT sand in drained shearing. This difference is explained by excess pore pressure build-up due to asperity abrasion and/or particle crushing and inherent expulsion of particle internal water of soft particles. GS shows an intermediate behavior to ChiT sand and PS with less pore pressure build-up than PS due to smaller internal particle voids. However, more experiments at higher effective confining pressure and changing relative densities are necessary to better understand the trade-off between strengthening
due to angularity and weakening due to crushing and inherent pore pressure build up. Hyodd et al. [1998] for instance show that dense crushable soils behave similar to loose, less crushable soils at high confining stress.

6.1.5.3 Undrained cyclic shear experiments

Cyclic loading experiments were carried out consistently with all parameters that may prominently influence liquefaction susceptibility kept constant. The differences in cyclic shear behavior (Figs. 6.6 and 6.8) thus attest that liquefaction susceptibility at low confining stresses is highly dependent on particle angularity [Y Vaid et al., 1985] and roughness. Particle crushability was shown to have no significant influence on the cyclic strength under either isotropic or anisotropic conditions. In fact particle intercalation seems to strengthen volcanic material compared to the DS under low effective normal stresses. Applied cyclic shear stresses may have been too low to create significant abrasion or crushing with inherent pore pressure build-up. Silty glass shards show higher liquefaction susceptibility than volcanic sands under isotropic loading conditions, which is coherent with the findings by [Singh, 1996] and related to smaller grain size. However, the smooth DS is still more susceptible to liquefy under the conditions tested than GS (Fig. 6.8). Beyond that, we show that angularity and roughness have an enhanced strengthening effect under anisotropic loading conditions in cyclic shear experiments without stress reversal. Cyclic shakedown was observed in volcanic materials, meaning that the materials reach an equilibrium state during cyclic loading where no axial strain and pore pressure is accumulated. Only DS accumulates strain although a constant pore pressure ratio is reached (Fig. 6.7). The particle shape and surface structure of dune sand is responsible for low friction at particle contacts, which allows the particles to move more easily relative to each other. This results in both a higher pore pressure ratio (Fig. 6.7) and the possibility for strain accumulation, the latter of which is enhanced by pore pressure. Volcanic ashes seem to hamper this process by particle intercalation and inherent stiffness at relatively low cyclic stress.

6.1.6 Conclusion

The abovementioned observations allow us to draw general conclusions concerning the monotonic and cyclic shear strength of highly angular volcanic material: Under monotonic drained conditions angularity and roughness lead to an increase in internal peak friction angles [Mair and Marone, 1999;
6.1. STRENGTH OF VOLCANIC MATERIAL

Figure 6.9: a) Axial strain ($\epsilon_v$) and pore pressure ratio (PPR) evolution during undrained anisotropic cyclic loading at a CSR of 0.195; b) 5th loading cycles on PS, DS, GS and ChiT sand plotted in the stress-strain space; c) 5th loading cycles on PS, DS, GS and ChiT sand plotted in the pore pressure ratio (PPR) - deviator stress ($q$) space d) and e) are equivalent to b) and c), respectively at the 305th loading cycle.
Sadrekarimi and Olson, 2011] of volcanic material. The influence of angularity and roughness is reduced with increasing normal stress and inherent particle crushing (Fig. 6.9a), while the monotonic undrained strength analogy to the static case at low confining pressure is conceptually illustrated in Figure 6.9b. The monotonic undrained failure behavior of rounded sands vs. volcanic material in $p'$-$q$ space essentially demonstrates that angularity and roughness lead to (i) a steeper failure line, (ii) a steeper phase transformation line (PTL) or quasi-steady state strength in case of low crushability, (iii) a steeper flow liquefaction surface (FLS), and iv) a higher quasi-steady state or steady state strength in case of low crushability. Thus, the cyclic mobility zone (CMZ) is larger for angular sands compared to well-rounded sands, making flow liquefaction less likely to occur. Also, more loading cycles are needed to hit the failure line because the distance to the failure line is larger at any given point in the $p'$-$q$ space. These factors equally affect the cyclic shear strength. If regarded in the geological context, an earthquake of a given magnitude could lead to flow liquefaction in a well-rounded sand whereas a volcanic sand of equivalent grain-size might experience cyclic mobility or even cyclic shake down.

In conclusion, this study attests an important strengthening effect due to angularity and roughness in volcanic material at low confining pressures. We show that the strengthening effect in fresh, unaltered volcanic ashes takes place under static as well as cyclic loading conditions, however volcanic ash sands and silts are susceptible to liquefy, and may act as glide planes in earthquake triggered submarine slides. To decide whether or not volcanic ashes preferentially act as gliding planes requires a statistical approach and is beyond the scope of this manuscript. However, we provide evidence for doubt that volcanic ashes constitute an end-member of silt and sand deposits along active margins that have low shear resistance and favor failure.

6.1.7 Acknowledgements

We thank Professor Tobias Mörz and Dr. Stefan Kreiter for providing customized triaxial cells funded by IWES Fraunhofer as well as valuable discussion. Matthias Lange is thanked for technical assistance with the static and dynamic shear apparatuses at MARUM Marine Geotechnics laboratory. We are grateful to Deutsche Forschungsgemeinschaft (Bonn, Germany) for funding MARUM - Center for Marine Environmental Sciences. Vulkatec GmbH is gratefully thanked for providing various pumice sands.
6.1. STRENGTH OF VOLCANIC MATERIAL

6.1.8 References


 Expedition 340 Scientists (2012), Lesser Antilles volcanism and landslides: implications for hazard assessment and long-term magmatic evolution of the arc.


Orense, R. P., M. J. Pender, and A. S. O'Sullivan (2012), Liquefaction char-
6.1. STRENGTH OF VOLCANIC MATERIAL

acteristics of pumice sands, Earthquake Commission, Wellington, N.Z.

Orense, R. P., M. J. Pender, M. Hyodo, and Y. Nakata (2013), Micro-

Pender, M. J., L. D. Wesley, T. J. Larkin, and S. Pranjoto (2006), Geotechni-

Poulos, S. J., G. Castro, and J. W. France (1985), Liquefaction evaluation pro-

Riley, C. M., W. I. Rose, and G. J. S. Bluth (2003), Quantitative shape mea-

Rotonda, T., P. Tommassi, and D. Boldini (2010), Geomechanical Char-


Singh, S. (1996), Liquefaction characteristics of silts, Geotechnical and Geo-
logical Engineering, 14, 19.


Sultan, N., P. Cochonat, M. Canals, A. Cattaneo, B. Dennielou, H. Hafidason, J. S. Laberg, D. Long, J. Mienert, and J. Trincardi (2004), Trig-
ering mechanisms of slope instability processes and sediment failures on continental margins: a geotechnical approach, Mar Geol, 213(1-4), 291-321.

Syvitski, J. P. M., K. W. Asprey, and D. A. Clattenburg (1991), Princi-
ples, design and calibration of settling tubes, Cambridge University Press, New York.


6.2 Altered volcanic ash deposits as potential slope failure planes?

G. Wiemer$^1$ and A. Kopf$^1$

$^1$MARUM - Center for Marine Environmental Sciences, Bremen, Germany.
gwiemer@marum.de - akopf@marum.de

Submitted to Geophysical Research Letter (GRL)

6.2.1 Abstract

Particle angularity and surface roughness of fresh, granular volcanic fall-out ash leads to high static frictional resistance compared to common granular materials such as sands at low stress ($<0.5$ MPa), and may only be overcome by liquefaction. Authigenic clay mineral formation in volcanic fall-out ash layers causes a transition from induced to inherent weakness of the material due to an increase in the weak phase smectite. This manuscript presents drained direct shear experiments ($\sigma'_n < 0.5$ MPa) on altered volcanic material and quartz sand-clay mixtures for reference in order to investigate whether altered volcanic ash may act as slope failure planes due to successive weakening. Frictional resistance of volcanic fall-out ash layers decreases from extremely high values ($\mu_{c,a}$) at 0% clay mineral to the lowest values encountered in classical soil mechanics ($\mu_{c,a} \sim 0.17$ in pure smectite). However, results attest the key role of particle shape and angularity in sediment stability at clay mineral contents < 40%. Decreased failure susceptibility of altered volcanic material compared to quartz sand-clay mixtures at equivalent clay mineral content is attributed to the inefficiency of clay minerals located in cavities of rough and angular volcanic particles. Major weakening due to the dominance of a clayey matrix occurs at clay mineral contents beyond $\sim 40\%$, which corresponds to an advanced stage of alteration at anticipated depths below that commonly observed for slope failure initiation. We hence suspect volcanic ashes to be susceptible to failure and inherent submarine slope failure initiation only under cyclic shaking conditions and inherent liquefaction even at advanced alteration and inherent smectite abundance.

Keywords: Slope failure, volcanic ash alteration, shear strength, roughness, angularity
6.2.2 Introduction

Submarine landslides along active margins present frequent dynamic sedimentary processes that may directly damage seabed infrastructure and generate destructive tsunamis [Hampton et al., 1996]. Over the past decades, earthquake shaking along active margins was singled out as one of the most effective short-term triggering mechanisms for slope failure initiation [Biscontin et al., 2004; Sultan et al., 2004]. Earthquake shaking may induce liquefaction failure in loosely deposited granular sediments such as sands and silts [Ishihara et al., 1975] or weaken clayey soils due to excess pore pressure build up. Volcanic arcs provide a source for granular, potentially liquefiable ash deposits (i.e. sand and silt) along active margins [Fisher and Schmincke, 1984], which in turn are frequently subjected to seismic shaking. Fresh volcanic fall-out ashes have been identified as sliding planes in several earthquake related slope [Expedition 340 Scientists, 2012; Harders et al., 2010; Sassa et al., 2012; Wiemer et al., (submitted)] failures, where liquefaction has been shown to be the most likely sediment failure mechanism. It is shown in Wiemer and Kopf [(submitted)] that fresh volcanic fall-out ashes are susceptible to liquefaction despite the fact that they are relatively strong and resistive to cyclic shaking. Mechanical strength of volcanic material is inherent [Brown et al., 2003], but additionally caused by high particle angularity, roughness, and inherent intercalation of the particles. Hence, failure of fresh volcanic ashes under static, gravitational conditions is considered as unlikely due to extremely high overall coefficients of friction. Nevertheless, it is generally accepted that the breakdown of volcanic glass leads to the formation of authigenic clay minerals (predominantly smectite), in particular in a marine environment [Hein and Scholl, 1978; Hodder et al., 1993; Matthews, 1962; Ross and Hendricks, 1945]. The process of volcanic glass break-down is known to be one of the most rapid weathering processes on earth. Volcanic ashes that contain a prominent amount of amorphous silica as stiff particle component may undergo a significant change in geotechnical properties as the amount of stiff granular particles (glass) is weathered to cohesive smectite. Fresh volcanic fall-out ashes may undergo a successive transformation from a purely granular, liquefiable deposit to a cohesive, clay-dominated deposit, which is usually characterized by low liquefaction susceptibility, low frictional resistance and intrinsic weakness. Lupini et al.[1981] show that the residual frictional resistance of sand-clay mixtures is more or less steady in the range of 0-15% clay fraction. Sand-clay mixtures bearing ~15% to ~50% clay fraction experience a drastic decrease in frictional resistance with increasing clay content. An additional increase in clay content to ~100% leads to a slight supplementary reduction in frictional resistance.
This manuscript investigates to what extent authigenic clay mineral formation impacts the frictional resistance of volcanic fall-out ashes in comparison to common sand/silt-clay mixtures. The working hypothesis of our study is that altered volcanic ash sands and silts deposited in marine settings are more resistant to shear stresses than common sand-clay or silt-clay mixtures at low smectite content. Roughness and angularity lead to high frictional resistance in fresh volcanic ashes at low stress, which is possibly maintained even at an advanced stage of alteration and inherently elevated clay content given the habitus of the glass particles. However, the frictional resistance behavior of volcanic fall-out ash is thought to successively change from sand-like to clay-like behavior associated with a reduction in frictional resistance over time.

6.2.3 Methods

Drained direct shear experiments were conducted using i) a series of generic volcanic ash-clay mixtures and quartz sand-clay mixtures of different angularity and roughness, and ii) a set of naturally altered volcanic fall-out ash deposits.

6.2.3.1 Sample material and intrinsic characterization

The generic quartz sand-clay mixtures and volcanic ash-clay mixture were prepared by mixing with a natural clay end-member in different ratios. The sample materials comprise:

(a) A well rounded quartz sand (DS)

(b) A pumice sand (PS)

(c) A volcanic tephra (ChiT)

(d) An angular, industrially crushed quartz sand (M4)

(e) A natural clay containing $\sim 60\%$ clay minerals (smectite and illite)

A detailed geotechnical characterization of the end-members DS, PS and ChiT sand is given in [Wiemer and Kopf, submitted]. DS, PS and ChiT sand were sieved to equivalent grain size distributions in order to make sure that frictional differences are independent of grading. Furthermore, some of the data on the M4-clay mixtures are published in [Wiemer et al., 2012] and is pointed out as such in the corresponding Figures. Raw natural sample materials include several marine deposited and partially altered volcanic fall-out ashes as well as a terrestrially altered volcanic soil:
(a) Marine deposited pumice sand from the northern Cretan margin sampled during RV Poseidon Expedition P336 [Kopf et al., 2006] (Cremice) (further geotechnical description in [Wiemer and Kopf, submitted])

(b) Marine tephra deposits from the Cretan margin sampled during RV Poseidon Expedition P336 [Kopf et al., 2006] (Creteph) (further geotechnical description in [Wiemer and Kopf, submitted])

(c) Marine deposited volcanic silt-sized glass shards (GS) sampled in ∼83 mbsf from the Lesser Antilles at site U1400 [Expediton 340 Scientists, 2012] (further geotechnical description in [Wiemer and Kopf, submitted]).

(d) Altered terrestrial pumice sand (APS) sampled on the flank of Volcano Villarrica, (South-Central Chile). This material reached the stage of palagonitization [Stroncik and Schmincke, 2002]. It is of yellowish-brown color and grain crushing leads to the formation of a gel-like texture.

(e) Altered rhyolitic silt (Harhy), sampled at an outcrop in the Ilfeld Basin, Harz, Germany

(f) Five partially altered fall-out marine tephra deposits from the Lau Basin (ODP expedition Leg 135) [Parson et al., 1992], deposited in 14.5-141 mbsf.

Grain-size distributions of each bulk sample material, but the M4-clay mixture series, were determined via laser diffraction analysis with a Coulter LS-13320 measuring grain size contents in 117 classes ranging from 0.04 to 2000 μm as a volume percent (vol%) (see [Syvitski et al., 1991] and [Loizeau et al., 1994] for details). Grain size distributions of M4-clay mixtures were analyzed by aerometer principle after Bouyoucos-Casagrande, following DIN 12790 [Deutsches Institut für Normung, 1979; Wiemer et al., 2012]. Each natural sample material and the generic end-member sample materials were imaged with regard to their alteration stage and/or angularity and roughness using scanning electron microscopy (SEM). The quartz sand end-members (DS and M4) and the volcanic sand end-members (PS and ChiT sand) were mixed in different ratios with the natural clay end-member. Thereby we were vigilant about mixing the materials in identical volume ratios. Pumice sand (PS) is known to have a very low bulk density compared to quartz sand due to particle internal voids [Pender et al., 2006]. However, as PS and Chit sand were sieved to DS-equivalent grain size distributions and weights at maximum and minimum void ratio were determined [Wiemer and Kopf, (submitted)], it was possible to prepare PS-clay, ChiT sand-clay and DS-clay
mixtures with equivalent sand to clay-matrix ratios. The results of standard classification tests on the pure sands (DS, PS and ChiT) are given in [Wiemer and Kopf, (submitted)]. Results of standard classification tests subjected to M4-clay mixtures containing 20-60% clay are shown in [Wiemer et al., 2012]. Furthermore, the natural clay end-member was purified in clay fraction by separating the clay fraction from the silt and sand portion in a centrifuge. Using this method, we obtained an illite and smectite bearing clay with a clay-size fraction of 95% (Fig. 6.10). Additionally we purified smectite-clay in the same manner and obtained smectite material with 100% < 2 μm particles. The mineralogical components of the natural ash samples (GS, APS, Harhy and all five samples of ODP Leg 135) were determined via semi-quantitative XRD analysis using a Philips X0Pert Pro multi-purpose diffractometer and the software packages Philips X0Pert HighScore and QUAX [Vogt et al., 2002].

6.2.3.2 Drained direct shear experiments

Cylindrical samples (25 cm², 2.5 cm height) were placed in a direct shear box where a vertical ram exerts the desired effective normal stress ($\sigma'_{n}$) upon the sample. The testing procedure conducted on purely granular material (materials ii - viii) is described in [Wiemer and Kopf, submitted]. Naturally altered ash samples and generic mixtures of volcanic or quartz sand with clay were remolded and saturated with de-aired, de-mineralized water prior to the assessment. Each sample material, besides the PS-clay, DS-clay and ChiT sand-clay (40% and 55% clay), was sheared at three different effective normal stresses ($\sigma'_{n}$) ranging from 100 to 450 kPa in order to determine the Mohr-Coulomb failure line according to DIN 18137-3 [Deutsches Institut für Normung, 2002]:

$$\tau'_{c} = c_{c} + \sigma'_{n} \times \tan(\phi_{c})$$

(6.7)

Where $\tau'_{c}$ is the shear stress, $c_{c}$ is the cohesion in cohesive samples or apparent cohesion in purely granular, non-cohesive sample materials. $\phi_{c}$ is the angle of internal friction, each at critical state. The PS-clay, DS-clay and ChiT sand-clay were sheared at $\sigma'_{n} = 300$ kPa exclusively. In that case, cohesion is regarded as part of the absolute shear strength and tests are described via the critical apparent friction, which is given by:

$$\tau'_{c} = \sigma'_{n} \times \mu_{c,a}$$

(6.8)

Where, $\mu_{c,a}$ is the critical coefficient of apparent friction. The shear path for each experiment was at least 11 mm. Critical friction values were determined as average friction from 10-11 mm shear path, regardless of whether
or not the critical state [Wood, 1990] was actually reached. This convention
needed to be applied because especially pumice bearing samples are charac-
terized by prominent crushing and do not necessarily reach the critical state
within the range of shear paths possible in direct shear testing [Wiemer and
Kopf, (submitted)]. Shear rates were set to i) 0.6 \( \mu \text{m/s} \) for purely granular
material ii) 0.13 \( \mu \text{m/s} \) for material with up to 30% clay and iii) 0.03 \( \mu \text{m/s} \)
for material with more than 30% clay.

6.2.4 Results
6.2.4.1 Grain-size distribution, XRD and SEM imaging

Grain-size distribution curves of sand and clay end-member materials are
presented in Figure 6.10a. It can be seen that the sample materials cover
the grain-size range from pure sand to pure clay. The quartz sand (DS), the
pumice sand (PS) and the Chilean tephra (ChiT) show very similar grain-
size distributions, which was intended by sieving. DS, PS and ChiT sand are
uniformly graded with \( d_{50} \sim 330 \mu \text{m} \). The light lines without symbols located
within the grey corridor adjacent to the lines of DS, PS and ChiT sand show
the curves of each of these sands mixed with 5% (vol.) of the clay end-
member. The second quartz sand end-member (M4) is a silty-sand with \( d_{50} \sim 45 \mu \text{m} \).
M4 is an industrially crushed quartz sand that contains \( \sim 5\% \) clay fraction. It is deduced from SEM imaging and from product information
that these 5\% consist of quartz, i.e. M4 contains no clay minerals. The
natural clay end-member has a clay fraction of \( \sim 60\% \) and \( \sim 40\% \) silt-sized
particles. The large grey corridor in the centre of the grain-size distribution
plot (Fig. 6.10a and 6.10b) frames the end-members of the M4-clay mixture
series. Light lines show the distribution of different mixtures of M4 and the
natural clay (see [Wiemer et al., 2012] for detail). \( d_{50} \) values of the M4-clay
mixtures range from \( \sim 1-45 \mu \text{m} \). Furthermore, Figure 6.10a presents the
grain-size curves of the purified natural clay and the purified smectite with
clay fractions of \( \sim 95\% \) and 100\% respectively. The \( d_{50} \) values are \( \sim 0.4 \) and
\( \sim 0.3 \mu \text{m} \), respectively. Cumulative grain size distribution curves of natural
samples are presented in Figure 6.10b. It is shown that the marine deposited
Cretan pumice sand and tephra are close to the grey corridor of the DS, PS
and ChiT sand mixed with 5\% (vol.) clay. All other natural samples fall into
the grey corridor covered by the M4-clay mixture series. \( d_{50} \) values of the
natural samples range from \( \sim 6-20 \mu \text{m} \). These samples present clay fractions
ranging from 0-27\%. For samples with available XRD data the percentage of
clay fraction was compared to the percentage of clay minerals that resulted
from semi-quantitative XRD analysis. In case XRD analysis revealed less
6.2. STRENGTH OF ALTERED VOLCANIC MATERIAL

clay content then could be anticipated from clay-size fraction, friction values were plotted against minimum clay content from XRD analysis. Our results, which are presented as a function of clay mineral content in the following, can hence be regarded as conservative.

Figure 6.10: Cumulative grain-size distribution curves of a) generic sample material and b) natural ash sample material

SEM images of the generic sample materials are shown in Figure 6.11. The DS vs. ChiT sand and PS (Fig. 6.11a, 6.11b, and 6.11c, respectively) were visually interpreted as end-members in angularity and roughness in the estimation chart for roundness and sphericity of [Krumbein and Sloss, 1963]. The effect of roughness and angularity on the drained and undrained monotonic and cyclic shear strength of these sands is presented in [Wiemer and Kopf, submitted]. Furthermore, the M4 quartz sand particle morphology (Fig. 6.11d) is of high angularity. The surface texture of the broken pieces is smooth. The difference in grain-size between DS and M4 (Fig. 6.10) is also visualized in the SEM images (Fig. 6.11a, 6.11d). Figure 6.11f presents the particle matrix assemblage of the natural clay end-member. Typical aggregates of platy clay particles can be seen. However, in the centre of Figure 6.11f a rounded, silt-size particle is completely surrounding by clay, which confirms the existence of silt-size fraction from grain-size analysis (Fig. 6.10). The SEM image of a mixture of the M4 and natural clay end-member
at ∼ 30% (vol) clay (Fig. 6.11e) attests that the silty-sand M4 is completely coated by clay. Interparticle contacts of the M4 material are partially present. Angularity of M4 particles can be seen clearly despite the clay matrix.

Figure 6.11: SEM images of generic sample material: a) DS, b) ChiT sand, c) PS, d) M4, e) M4+30% clay mineral, f) natural clay end-member.

Figure 6.12 shows SEM images of six examples of tested altered volcanic ashes, where each material was photographed using two different magnifications: one demonstrating particle morphology and the other showing surface texture and stage of alteration. Figure 6.12a, shows glass shards from the IODP expedition 340 [Expedition 340 Scientists, 2012] with vesicular, convex and concave shaped particles. The larger particles (silt-size) resemble pumice particles. XRD analysis revealed 16% expandable clay minerals (mainly smectite). Figure 6.12a’ shows an aggregate of smectite clay minerals within fragments of glass shards. Figure 6.12b shows a palagonized pumice sand particle. Glass particles are typically highly angular with a convex and concave surface morphology and abundant veins. Upon crushing these particles gain a gel-like texture typical for the early stage of glass alteration and palagonite [Stroncik and Schmincke, 2002]. On the surface of the hemispherical particle portion there are aggregates of supposedly palagonite (Fig. 6.12b’). The altered rhyolitic silt sample is presented in Figure 6.12c and 6.12c’. Particle morphology is angular, but other than in the pumice/tephra particles there are no deep carving hemispherical gaps. Furthermore the particles are loosely arranged but completely coated in clay of illite and smectite composition. Similar to the natural clay end-member (Fig.
6.2. STRENGTH OF ALTERED VOLCANIC MATERIAL

6.11f), single platy clay particles can be seen surrounding granular particles. Figure 6.12d, 6.12e and 6.12d show three examples of the marine deposited and altered fall-out ash deposits cored during ODP Expedition Leg 135. All materials presented consist of majorly glass/pumice particles. Smectite has coated parts of glass particle walls, while internal particles voids are partially filled with smectite aggregates (see also [Christidis, 2001; Ghiara and Petti, 1995]). The alteration stage obviously increases from shallow/young to deeper/older parts in the sediment sequence (Fig. 6.12d vs. 6.12e and 6.12f).

**Figure 6.12:** a) IODP Exp. 340 GS from \(\sim 82\) mbsf, b) APS, c) Harhy, d, e and f) ODP Leg 135 site 837-A, 834-A and 834-A sampled at \(\sim 14.5, \sim 78\) and \(\sim 62\) mbsf, respectively.
6.2.4.2 Direct Shear Experiments

Coefficients of peak and critical apparent friction at 300 kPa effective normal stress are presented in Figure 6.13a and 6.13b, respectively. The grey corridor in Figure 6.13a (and 6.13c) shows the lower and upper limit of residual friction coefficients published in [Lupini et al., 1981] (and references therein). The M4-clay mixture series shows friction values that lie within the range of expected values. Only at ~10% clay mineral content the M4-clay mixture presents an unusually high friction value which is still lower than in altered volcanic material. $\mu_{c,a}$ of generic sand-clay mixtures increases with angularity and roughness. DS shows friction values of 0.58, M4 of 0.63 at 0 % clay mineral content which is within the usual range for quartz [Bolton, 1986]. Pure volcanic sands (0% clay) have higher coefficients of friction than quartz ($> 0.7 < 0.85$) (see also [Wiemer and Kopf, submitted], with PS and ChiT sand sporting the highest frictional resistance. With increasing clay content from 0-10% the coefficients of friction increase in each generic series, subsequently $\mu_{c,a}$ decreases with increasing clay mineral content. From 0-20% clay each volcanic material lies beyond the range of usual friction values, i.e. outside the grey-shaded zone. At about 20% clay content $\mu_{c,a}$ of angular volcanic samples (Harz-rhyolite, ChiT sand-clay series) with less cavities than PS, approach the M4-clay mixture series and are located within the grey shaded area. The altered rhyolite which is similar to the Chit sand in terms of grain-shape, is characterized by $\mu_{c,a} = 0.55$ and lies close to the ChiT sand clay mixture series. Nevertheless, frictional hierarchy in dependence of particle angularity and roughness is maintained from 0-50% clay content with PS at the high end, and DS at the low end of frictional resistance. PS is ~25 % stronger than DS from 0-40% clay mineral content. PS and the natural ash samples which contain predominantly glass shards or pumice, lie beyond the usual range of frictional values until reaching ~50% clay content. At ~55% clay content the influence of particle shape, roughness and grain-size seems to be negligible as each generic sample material shows similar friction values ($\mu_{c,a} \sim 0.3$). The purified natural clay and smectite present $\mu_{c,a}$ values of 0.29 and 0.17, respectively and are equivalent to residual friction values of illite and smectite determined at high strain with a ring shear device [Brown et al., 2003]. Figure 6.13b and 6.13c show the Mohr-Coulomb friction parameters tan ($\phi_c$) and $c'$ (cohesion), respectively as a function of clay content. This data results from three tests at effective normal stresses ranging from 100-450kPa on each sample material. It can be seen that friction values are generally slightly lower compared to the apparent friction at $\sigma_n' = 300$ kPa (Fig. 6.13a) which is related to the fact that (apparent) cohesion is not integrated in the angle of internal friction ($\phi_c$) as it is in the normal stress
6.2. STRENGTH OF ALTERED VOLCANIC MATERIAL

dependant coefficient of apparent friction ($\mu_{c,a}$). Angles of internal friction ($\phi_c$) follow a similar trend as described for the coefficient of critical apparent friction (Fig. 6.13a). However, four of the natural sample materials plot within the grey shaded area. Note, that the data presented in [Lupini et al., 1981] which determines the grey area presents apparent friction data including (apparent) cohesion. It is shown in Figure 6.13c that the apparent cohesion of the purely granular material is not zero even in the well-rounded quartz sand (DS). Cohesion of DS is thought to be related to the influence of cell friction which can be significant at low effective stress [Deutsches Institut für Normung, 2002]. Nevertheless, the apparent cohesion values shown in Figure 6.13d present a significant increases with angularity and roughness of the particles. Again, PS and ash samples bearing pumice or glass shard present the highest values of apparent cohesion (30-55 kPa) even at elevated clay content ($\sim 25\%$). The M4-clay mixture series shows a near linear increase in cohesion with increasing clay content that is low ($\sim 2$ kPa) at 5% clay content and high ($\sim 17$ kPa) in its purified state ($\sim 95\%$ clay). The significantly altered Harz-rhyolite sample (Harhy) also has angular particle geometries and cohesion values slightly higher than the M4-clay mixture with equivalent clay content.

6.2.5 Discussion and conclusion

The tested materials comprise (volcanic) sands, silts and clays in terms of their grain-size distribution (Fig. 6.10). In essence Figure 6.10 can be viewed as illustration of the grain-size evolution of a volcanic fall-out ash deposit undergoing successive glass break-down. Large particles of volcanic glass (sand) experience a reduction in size while the fine fraction (silt and clay) successively increase. The friction data presented here show that fresh and slightly altered volcanic material may be regarded as end-members with high frictional resistance compared to common sands or sand-clay mixtures. The general strengthening effect related to roughness and angularity of (volcanic) particles [Sadrekarimi and Olson, 2011; Wiemer and Kopf, (submitted)] seems to be annulled at a clay mineral content of $\sim 55\%$ (vol.) (Fig. 6.13a) regardless of the degree of angularity and roughness of the stiff particle end-member (Figure 6.11 and 3). Materials containing mainly pumice sand or glass shards show the highest frictional resistance even at elevated clay mineral content (20%). Pumice-sand particles and glass-shards mostly show hemispherical cavities at the surface and internal voids that may be connected to the outer surface [Heiken and Wohletz, 1985]. It is thought that clay minerals located in internal voids or within hemispherical cavities (Fig. 6.12) may be detected in grain size and/or XRD analysis, but do not
influence the frictional resistance significantly. Those clay minerals are inefficient because they are located at positions, which do not allow to lubricate the interparticle contacts. Only when the clay matrix forms a prominent coat around the entire particle, intercalation is prevented and the effect of angularity and roughness is annihilated. The amount of efficient clay minerals to frictional resistance reduction is higher in a well-rounded quartz sand than it is in a highly angular and porous pumice sand (Fig. 6.14) at equivalent absolute clay mineral content which may be detected in XRD and/or grain-size analyses. Moreover, it is shown in Figure 6.13b and 6.13c that the strength related to particle intercalation is partially expressed in apparent cohesion (Fig. 6.13c). However, even if apparent cohesion is subtracted, the angles of internal friction of volcanic sands are still higher than the apparent friction coefficients presented in [Lupini et al., 1981] (and references therein). Our data further signify that by mineral breakdown, volcanic ash that initially contains mainly amorphous silica (i.e. glass) may show a dramatic decrease in static frictional resistance from $\mu_{c,a} \sim 0.85$ (PS) to $\mu_{c,a} \sim 0.17$ (pure smectite) (Fig. 6.13a). This process, however, seems insignificant for
slope failure initiation since relatively high temperature at depth (\( \sim 25^\circ C \) [Hill et al., 1993]) or interstitial water with slightly lower salinity than in the marine realm are required for the formation of significant amounts of smectite in shallow marine sediments (50% clay in 4.5 mbsf [Hodder et al., 1993]). For instance, volcanic fall-out ash samples from ODP Leg 135 and IODP Expedition 340 are sampled in a depth range (7.5-141 mbsf), which is representative for the depth at which submarine slides along active margins may be initiated [McAdoo et al., 2000]. However, the alteration stage reached at that depth (see also [Hein and Scholl, 1978; Hill et al., 1993]) is not sufficiently advanced (max 23% clay) to impact the resistance to static frictional forces.

In conclusion, we propose to enlarge the range of apparent friction values presented in [Lupini et al., 1981] to higher values at clay mineral contents < 40% (Fig. 6.13a) in order to include fresh and slightly altered marine volcanic sediments. Furthermore, we find volcanic ash alteration to be insignificant for submarine static slope failure initiation unless the setting is characterized by a high geothermal gradient and/or brackish water.

![Figure 6.14: Schematic illustration of the effect of angularity, roughness and the presence of cavities on the frictional resistance at different stages of identical clay mineral content.](image)

### 6.2.6 Acknowledgments

Matthias Lange and Christian Zöllner are thanked for technical assistance with the direct shear apparatuses at MARUM Marine Geotechnics laboratory. We are grateful to Deutsche Forschungsgemeinschaft (Bonn, Germany) for funding MARUM - Center for Marine Environmental Sciences. Vulkatec GmbH is acknowledged for providing various pumice sands.
6.2.7 References

Biscontin, G., J. M. Pestana, and F. Nadim (2004), Seismic triggering of submarine slides in soft cohesive soil deposits, Mar Geol, 203(3-4), 341-354.


Deutsches Institut für Normung (1979), Laborgeräte aus Glas; Aräometer, Grundlagen für Bau und Justierung, in DIN-Norm 12790, edited, Berlin, Beuth Verlag.


Expedition 340 Scientists (2012), Lesser Antilles volcanism and landslides: implications for hazard assessment and long-term magmatic evolution of the arc.


6.2. **STRENGTH OF ALTERED VOLCANIC MATERIAL**


6.2. **STRENGTH OF ALTERED VOLCANIC MATERIAL**


7.1 Subaqueous landslide in earthquake-prone South-Central Chile: The role of sediment composition and its behaviour under dynamic loading conditions on slope failure initiation

1 Gauvain Wiemer, 2,3 Jasper Moernaut, 4 Nina Stark, 3 Philipp Kempf, 3 Marc De Batist, 5 Mario Pino, 6 Roberto Urrutia, 2 Michael Strasser and 1 Achim Kopf

1 MARUM, Centre for Marine Environmental Sciences, Bremen, Germany
2 Geological Institute, ETH Zürich, Switzerland
3 Renard Centre of Marine Geology, Ghent University, Belgium
4 Department of Civil and Environmental Engineering, Virginia Tech, Blacksburg, USA
5 Instituto de Geociencias, Universidad Austral de Chile, Chile
6 Centro EULA, Universidad de Concepción, Chile

Submitted to International Journal of Earth Sciences (IJES)

7.1.1 Abstract

Details of subaqueous slope failure mechanisms are still poorly understood. Compared to marine settings landslides in lakes are prime targets given their
smaller scale and versatile access. Lake Villarrica (South-Central Chile) experienced prominent slope failures and served here as study area for subaqueous landslide initiation mechanisms in geodynamically active settings. The MARUM free-fall CPTU has been deployed adjacent to coring sites. Cores and CPTU testing pierced through all lithological units involved in the slope failure. Three types of soils (diatomaceous ooze, volcanic ash, quick clay) were analyzed for their tributary to slope failure, and earthquake shaking was identified as trigger mechanism using geotechnical methods, such as pseudo-static factor of safety analysis and cyclic triaxial testing. The investigated landslide was two-phased: initial failure occurred above a tephra layer that did not liquefy, but helped to reduce the shear strength at the boundary to the overlying sediment. The slide scarp extended further shoreward by retrogression along a second failure plane located in a stratigraphically deeper, extremely sensitive lithology (quick clay). Results attested that liquefaction of buried volcanic fall out ash layers was unlikely, but such layers might have still contributed to shear strength reduction at the contact surface to neighboring sediment. Furthermore, pore pressure build-up in diatomaceous ooze during cyclic loading may be similar to the behavior in granular soils. We generally infer that observed mechanisms regarding dynamic behavior of diatomaceous ooze and intercalated volcanic ash, as abundantly present along active continental margins, are equally important for landslide initiation in submarine settings.

**Key words:** landslide, earthquake shaking, tephra, quick clay, diatomaceous ooze, CPT, cyclic loading

### 7.1.2 Introduction

Submerged slope sediments located in seismically active settings are known to produce failures which may entail devastating and dramatic aftermaths for onshore and offshore infrastructure as well as coastal communities [Hampton et al., 1996]. For the purpose of slope stability risk assessment and landslide-caused damage mitigation it is of primary importance to understand the initiation mechanisms of diverse past subaqueous slope failure events in all their detail and complexity [Leroueil et al., 1996]. A detailed analysis of the failure plane and investigation of the failure mechanism often turns out to be an expensive and challenging enterprise in the marine realm. Drilling is usually the only possibility to reach crucial lithologies at depth. Drilling platforms are not available in many projects. Hence, data from surface lithologies are extrapolated based on assumptions and approximations, which may lead
to uncertainties and deviations in the assessment of the 'in situ' conditions and failure plane. Particularly, active margins comprise sediment types that are poorly understood from a geotechnical point of view, namely diatomaceous ooze and volcanic ash. Volcanic ashes for instance have been proposed as preferential failure planes or weak layers in earthquake prone submarine setting due to their high liquefaction potentials [Harders et al., 2010; Sassa et al., 2012], however their mechanical strength remains rudimentarily investigated.

Figure 7.1: Morphological setting of Lake Villarrica modified after Moernaut et al. (2009). Lake surrounding topography derived from SRTM data. Slope shader illumination from the North. Moraine locations (“Llanquihue” moraine belt) based on [Laugenie, 1982]. Lahar pathways derived from Laugenie (1982) and satellite pictures. Lake Villarrica: Bathymetric contours every 10 m. Deepest part: 167 m below lake level (based on SHOA, 1987)

Unlike in the marine realm, access to failure planes in lakes is often easier and more cost effective [S. Stegmann et al., 2007; Strasser et al., 2007]. Lake Villarrica (Fig. 7.1) is a glaciogenic lake (ca. 20 x 10 km²) located at the foot of Villarrica Volcano in the Andes, in south?central Chile. High magnitude earthquakes occur on a regular base, due to the subduction of the Nazca plate under the South American continent. The sediment sequence comprises
diatomaceous oozes and volcanic ash layers, and several large sublacustrine slope failures were identified along bedding-parallel slip planes via a seismostratigraphic analysis [Moernaut et al., 2009]. Earthquake shaking, one of the most cited short term triggering mechanisms for landslide initiation [Sultan et al., 2004], has been recognized as most probable cause. This study focuses on the reappraisal of that hypothesis by investigating geotechnical properties of Lake Villarrica’s sedimentary succession, including its failure planes at highest spatial resolution. Centre point of the study is a confined area with a well pronounced landslide. We present a broad set of 'in situ' and laboratory geotechnical data, including earthquake shaking simulation via cyclic triaxial testing. The pseudo-static factor of safety is calculated on the basis of a continuous data set of 'in situ' undrained shear strength. Embedded within a spatial network of seismic reflection and side-scan sonar data, this data set allows us to carry out a high resolution analysis of a subaqueous landslide and explain why the slope failed where it failed.

7.1.3 'In situ' Data acquisition

In addition to the data acquired in 2001 and 2007 [Moernaut et al., 2009] additional side-scan sonar and single channel high-resolution (pinger, 3.5 kHz) seismic data were acquired in 2011 for a higher spatial resolution within the slide area. Navigation and positioning was done using stand-alone GPS (horizontal accuracy of ± 5m) on the Huala II, the small research vessel from the Universidad Austral de Chile. The KLEIN 3000 side-scan sonar simultaneously recorded in two frequency bands of 100 and 500 kHz using SonarPro (Klein) software. Towfish depth regulation was performed manually. The resulting mosaic was created in SonarWiz.Map (Chesapeake Technology) after manual water column removal and applying automatic gain control. The acoustic signal of the pinger penetrated up to 25 – 30 ms TWT (∼20 m) of the sedimentary fill with a vertical resolution of 15 – 20 cm and shot-point spacing of about 1 – 1.5 m. Data were recorded digitally on a TRITON-ELICS Delph-2 acquisition system. Seismic-stratigraphic interpretation was done using SMT’s Kingdom Suite after applying a frequency band-pass filter. Water and subsurface depths were calculated using a mean acoustic velocity of 1500 m/s.

7.1.3.1 Coring and Cone Penetration Testing

Three main coring and CPT sites where selected based on the seismic investigation. Multiple hammer-driven piston cores [Bertrand, 2005] (VillaR 1 to 5) and gravity cores (Villa 754-1 to 754-4) have been taken on a plateau of
unfailed sediment (site 1) and within the slide scarp (site 2 and 3) (see Fig. 7.2 for exact positioning). The MARUM Free-Fall Piezocone Penetrometer (FF-CPTU) [Sylvia Stegmann et al., 2006] has been deployed to obtain the 'in situ' geotechnical properties of the unfailed and disturbed stratigraphic units. A total of 28 measurements were conducted on the plateau and within the slide area (see Fig. 7.2 for exact positioning). Cone resistance, sleeve friction and pore pressure were measured down to a maximum depth of ∼ 8.1 m. The penetration rate effect [Dayal and Allen, 1975] on cone resistance and sleeve friction has been taken into account, making the data comparable to standard pushed CPTU data [Steiner et al., 2012]. The 'in situ' undrained shear strength \( s_u \) was derived using a standard cone factor \( (N_k) \) of 12 - 16 for normally and overconsolidated clays, respectively (for details see [Lunne et al., 1997]). \( s_u \) has subsequently been normalized with respect to the effective overburden stress \( \sigma'_{v,0} \) assuming zero excess pore pressure in order to analyze the state of consolidation of the sediment. The normalized 'in situ' undrained shear strength has been calculated as follows:

\[
\frac{s_u}{\sigma'_{v,0}} = \frac{s_u}{\gamma' \times z} \quad (7.1)
\]

where \( \gamma' \) is the submerged unit weight and \( z \) is the 'in situ' depth. \( \gamma' \) was set to 3.9 kN/m\(^3\) as it has been deduced from MSCL data (see section 7.1.3.1). The shear strength of normally consolidated organic and inorganic soils is considered to be in the range of 0.2 - 0.3 \times \sigma'_{v,0} [Karlsson and Viberg, 1967]. However, fossiliferous soils, may be characterized by a normalized undrained shear strength of up to ∼ 0.4 at normal consolidation [Tanaka et al., 2012]. Thus, any soil with \( s_u > 0.4 \times \sigma'_{v,0} \) is considered as overconsolidated and any soil rich in diatom frustules (> 70 %) with \( s_u < 0.3 \times \sigma'_{v,0} \) may be considered as underconsolidated.

### 7.1.3.2 Pseudo static factor of safety analysis

\( s_u \) has subsequently been integrated in a one-dimensional, undrained, infinite slope stability analysis [Morgenstern and Price, 1965] where earthquake induced shear stresses are considered as constant [Hampton et al., 1996]. The pseudo-static factor of safety (FS) is the ratio of resisting forces to driving forces acting on slope sediment during earthquake shaking. If FS > 1, slope sediment may be considered as stable whereas FS < 1 indicates failure under tested conditions [Sultan et al., 2004]. FS is defined as:

\[
FS = \frac{s_u}{\sigma'_{v,0}(\sin(\alpha)\cos(\alpha) + k(\frac{\sigma}{\gamma}))(\cos^2(\alpha))} \quad (7.2)
\]
Where $\gamma$ is the bulk unit weight, $k$ is the pseudo static coefficient and $\alpha$ the slope angle. Yet, the FS is critically dependent on the k-value [Kramer, 1996] that accounts for the additional shear stress induced by the propagation of seismic shear waves. In order to select an appropriate k-value, past earthquake intensities have been linked to k-values. In Villarrica, the Modified Mercalli Intensity (MM) of the earthquake with the largest ever recorded magnitude, the 1960 Great Chilean Earthquake, corresponded to VII 1/2 (Very Strong) [Lazo Hinrichs, 2008]. The 2010 Chile earthquake lead to an intensity of VI 3/4 (MM) [USGS, 2010]. A MM of VI to VIII can be induced by earthquakes with a peak ground acceleration (PGA) of 2.5 % g to 20 % g, respectively [Medvedev, 1977]. To link the PGA to k-values the following relation has been used [Hynes-Griffin and Franklin, 1984]:

$$k = 0.5 \frac{PGA}{g}$$

(7.3)

The FS has been calculated for k-values of 0.0125, 0.0375, 0.1 and $\alpha = 3^\circ, 5^\circ$ and $7^\circ$ in order to explore the influence of slope angle and earthquake magnitude variation. The most adequate slope angle however is $5^\circ$ as reconstructed from reflection seismic profiles. The bulk density was set to 1.4 g/cc according to MSCL data (see section 7.1.3.1). The k-values we chose for the FS analysis represent a range of scenarios (PGA of 2.5, 7.5 and 20% g) that may reoccur in the near future. However, no k-value considers earthquake induced excess pore pressure build up and inherent sediment shear strength reduction as it can be simulated in laboratory shear experiments (see section 7.1.3.2).

7.1.4 Laboratory Data acquisition

7.1.4.1 Index Properties

Magnetic susceptibility, Gamma Ray Attenuation bulk density and compressional wave velocity where measured on the closed cores (whole round -WH) using the GEOTEK Multi-Sensor Core Logger (MSCL), at the Limnology Laboratory, ETH Zürich. All cores were then transported to the geotechnical laboratories of MARUM (Centre for Marine Environmental Sciences, Bremen) where they were split, photographed, sedimentologically described and further analyzed: Right after splitting the sediment’s natural water content ($w_h$ %) was measured at $\sim$ 25 cm intervals. Samples were oven-dried for at least 24 hours at 60 °C and solid volume was determined using a helium pycnometer to derive grain density, bulk density and void ratio. Grain-size distributions were measured via laser diffraction analysis with a Coulter LS-13320
that quantifies grain size contents in 117 classes ranging from 0.04 to 2000 μm as a volume percent (vol%) (see [Syvitski et al., 1991] and [Loizeau et al., 1994] for details). Undrained shear strength ($\tau_u$) was measured on the split cores using i) a Wykeham-Farrance cone penetrometer WF 21600 according to British Standards Institutions BS-1377 [British-Standard-Institute, 1977; Hansbo, 1957] and ii) a motorized Wykeham-Farrance vane shear apparatus [Blum, 1997] with a spacing of ~5 cm and ~50 cm, respectively. Remolded shear strength at w$_n$% of selected samples was measured using a Haake RV-20 Vane Shear Rotovisco Apparatus in order to derive sensitivity, which is defined as the ratio of undrained shear strength ($\tau_u$) over remolded undrained shear strength ($\tau_{ru}$) [Chaney and Richardson, 1988]. Atterberg Limits were determined with a Casagrande apparatus according to [ASTM Standard D4318-05, 2000]. Scanning Electron Microscopy (SEM) was performed on samples representative for the different lithologic units of VillaR 1 (Fig. 7.3).

7.1.4.2 Advanced geotechnical laboratory shear experiments

Crucial lithologic units where tested for their drained static and undrained cyclic shear strength in order to characterize the sediment's behavior under static gravitational and earthquake loading conditions, respectively. On particular tephra layer has been reproduced by sieving tephra that was sampled on the Villarrica Volcano flank to equivalent grain-size distribution. By doing so we obtained sufficient material to determine the minimum and maximum density according to DIN-Norm 1826 [Deutsches Institut für Normung, 1996]. Shear tests could thus be conducted on lake-equivalent tephra material and at approximately equivalent density index. The original tephra will be called 'failure plane tephra' (FPT) (see section 7.1.4.2) and the reproduced material will be called FPT-equivalent.

Direct shear experiments

Direct shear tests on cylindrical samples of 56 mm in diameter and ~ 20 mm height were placed in a GIESA RS5 (see [Ikari and Kopf, 2011] for details) and conducted according to DIN-Norm 18137-3 [Deutsches Institut für Normung, 2002]. A vertical ram exerts the desired effective normal stress ($\sigma'_n$) upon the samples that were sheared under saturated and drained conditions with $\sigma'_n$ ranging from 50 to 300 kPa. The shear path for each experiment was at least 10 mm. Effective normal stress ($\sigma'_n$), effective shear stress ($\tau'$), and vertical and horizontal displacement were measured at a frequency of 0.1
Hz. These measurements follow the Mohr-Coulomb constitutive law:

$$\tau_p = c' + \sigma'_n \times \tan(\phi)$$  \hspace{1cm} (7.4)

Where $c'$ is the effective cohesion, $\phi$, the angle of internal friction, $\tau'_p$, the peak effective shear stress, and $\sigma'_n$, the effective normal stress, both at failure. In granular materials $c'$ is considered zero [Handin, 1969].

**Cyclic triaxial shear experiments**

Earthquake shaking was simulated via undrained cyclic shear strength experiments using the Dynamic Triaxial Testing Device [Kreiter et al., 2010]. Seed and Idriss [1971] developed a simplified procedure to convert any arbitrary earthquake signal into uniform loading cycles of equivalent cyclic shear stress amplitude. This method consists in taking 65% of the maximum earthquake induced shear stress ($\tau_{\text{max}}$) in the field as uniform cyclic shear stress amplitude in laboratory testing ($\tau_c$):

$$\tau_c = 0.65 \times \tau_{\text{max}}$$  \hspace{1cm} (7.5)

$$\tau_{\text{max}} = \frac{PGA}{g} \times \sigma'_{v,0d}$$  \hspace{1cm} (7.6)

where $r_d$ is a reduction factor that accounts for the variation of cyclic shear stresses with depth. Under laboratory conditions cyclic shear stress is induced by cyclic, vertical loading and unloading of a cylindrical sample at constant lateral stress. The cyclic shear stress is defined as:

$$\tau_c = \frac{q_c}{2}$$  \hspace{1cm} (7.7)

$$q_c = \sigma'_{1,\text{max}} - \sigma'_{1,0}$$  \hspace{1cm} (7.8)

where, $q_c$ is the cyclic deviator stress, $\sigma'_{1,0}$ is the initial effective vertical stress, $\sigma'_{1,\text{max}}$ is the maximum vertical stress during cyclic loading.

Cylindrical specimens with an area of 10 cm$^2$ were stitched out of WH-cores; tephra samples were air-pluviated and tapped to desired densities and assembled in a triaxial cell. Initial effective vertical ($\sigma_{1,0}$) and lateral ($\sigma_{3,0}$) stress were set to 80 kPa and 40 kPa, respectively. All samples were loaded under initial anisotropic stress conditions, in harmonic, purely compression mode (i.e. $0 < q_{\text{min}} < q_c$) with a frequency of 1.0 Hz. The vertical displacement, cyclic stress, lateral stress and excess pore pressure evolution ($\Delta u$) were recorded during cyclic loading. The pore pressure ratio (PPR) is defined as:

$$PPR = \frac{\Delta u}{p_0}$$  \hspace{1cm} (7.9)
Where, $p_0$ is the initial mean effective normal stress. The number of cycles at failure were determined with i) the onset of liquefaction, defined by a PPR of 0.9, or ii) the gain of 5% vertical axial strain ($\epsilon_a$). Exact testing conditions, including the reversal coefficient ($r_c$) [Galandarzadeh and Ahmadi, 2012], $q_0$, $q_c$, $K_0$, Skempton's B-Value [Skempton, 1954], density index ($I_D$), initial sample height ($H_0$) and number of cycles to failure are given in Table 3.4.

In our experiments we applied a cyclic stress ($\tau_c$) of 12.5 kPa and used $rd = 0.95$ (~20 m depth) [Seed and Idriss, 1971]. This corresponds to an earthquake with a local PGA of ~25 % g, i.e. a Modified Mercalli Intensity (MM) of VIII [Medvedev, 1977]. These undrained cyclic shear tests shed light on the dynamic response of the sediment to earthquake shaking, where the pore pressure evolution and inherent sediment weakening is primary output information.

### 7.1.5 Data and Results

#### 7.1.5.1 Seismic analysis and coring location

Reference CPT and coring site VillaR 1 (Fig. 7.2) is located at the centre of the unfailed plateau that is almost fully surrounded by a complex and irregular head scarp of the slide (Fig. 7.2). The core VillaR 3 (site 3) and the gravity core Villa 754 (site 2) were taken where slope failure occurred. CPT site 2 and 3 correspond with these coring sites (Fig. 7.2). The seismic profiles A (W-E transect) and B (N-S transect) (Fig. 7.2) show perpendicular cross sections from the unfailed plateau (site 1) to the North (site 2) and East (site 3), respectively, and cut through the head scarp of the slide. It can be seen on the N-S profile B that the reflections in the uppermost seismic unit are parallel and continuous with low-to-high reflection amplitudes building a uniform drape. Profile A shows continuous, sub-parallel stratification of plateau sediment in W-E direction and reveals spatial differences in unit thickness. As we have good control for stratigraphic correlation the different thicknesses of U2 can be related to different sedimentation rates. Combining all seismic lines revealed the stratigraphic horizon of the failure plane at site 2 is much deeper in the seismic stratigraphy than at site 3, indicating that the landslide event was initiated at two distinct stratigraphic levels. In the following these two failure planes will be termed 'lower failure plane' (site 2) and 'upper failure plane' (site 3). From seismic data alone it cannot con-
CLUSIVELY BE INTERPRETED, ALONG WHICH STRATIGRAPHIC LEVEL FAILURE WAS INITIATED FIRST. COMPETING INTERPRETATIONS ARE I) RETROGRESSIVE FAILURE OR II) THAT FAILURE INITIATED UP-SLOPE AT A DEEPER STRATIGRAPHIC LEVEL AND STRATIGRAPHICALLY STEPPED-UP TO A SHALLOWER LEVEL WHEN PROGRESSING DOWN SLOPE (STRATIGRAPHIC 'STEP-UP' SEE [MOERNAUT AND DE BATIST, 2011]) THE FAILURE ALONG THE UPPER SLOPE IS DIVIDED INTO A NORTHERN AND SOUTHERN PART, BOTH SEPARATED BY THE UNFAILED PLATEAU (FIG. 7.2). BOTH PARTS OF THE UPPER SLIDE SCAR ARE CHARACTERIZED BY AN IRREGULAR HEAD SCARP AND A 'BOTTLE-NECK-SHAPED' OUTLET. THE LOWER HEAD SCARP AREA, WHERE THE UPPER STRATIGRAPHIC LEVEL ACTED AS FAILURE PLANE, IS MORE REGULAR AND CONFINED. CORE VILLAR 1, TAKEN ON THE UNDISTURBED PLATEAU PROVIDES INTACT SAMPLES FROM THE UPPER AND LOWER STRATIGRAPHIC LEVEL, THAT ACTED AS FAILURE PLANE ON THE SLOPE.

7.1.5.2 SEDIMENT CHARACTERIZATION

REFERENCE CORING SITE 1: UNDISTURBED PLATEAU

The sediment core VillaR1 is ~8 m long. Three distinct units were identified via abrupt changes in physical and lithological properties measured in cores, which correlated to the distinct depositional geometry and acoustic facies observed in reflection seismic data (Fig. 7.2). Figure 6.12 shows the composite plot of all data obtained at the reference site VillaR 1. The core comprises the units 1 to 3 (U1-U3) from top to bottom. Distinct tephra layers suggest that it covers the past ~12 kyr. These tephra deposits include the ~3.8 kyr Pucón event [Heirman, 2011; Silva Parejas et al., 2010] and the ~11 kyr Neltume Pumice [Echegaray et al., 1994]. Furthermore, AMS radiocarbon dating was performed at the Ion beam Physics Lab of ETH Zürich (Switzerland) on a leaf found in VILLAR1 at a sediment depth of 7.29 m (ETH-52078), very close to the U2-U3 boundary. It provided a 14C age of 10841±42 yrs BP which is calibrated (IntCal09) to 12600-12770 cal yrs BP (1σ).

Unit 1 (U1) stretches from 0 - 2.85 mblb (meters below lake bottom) and mainly consists of greenish, brownish, finely laminated, diatomaceous ooz. Thin (0.1-1.5 cm) tephra layers occur regularly and can be considered as part of the background sediment. U1 is a sandy-silt with ~75% silt-size, diatom frustules. The amount of sand size detrital particles decreases gradually from ~19% to ~8% towards the bottom of U1. Dry natural water content (w_n %) ranges from 119% to 166% with a bulk density of ~1.4 - 1.5 g/cc. Peaks in bulk density are consistent with peaks in magnetic susceptibility and can be correlated to tephra layers. The Liquid Limit (LL) of U1 (~90 %) was tested at ~1.25 mblb. It is lower than the w_n %. The plasticity index (PI) is
7.1. SUBAQUEOUS LANDSLIDE AND VOLCANIC ASH

~ 50%. Sensitivity values are > 16 and thus, U1 may technically be termed medium quick clay [Rosenqvist, 1953]. The influence of diatom frustules on the Atterberg Limits and sensitivity will be discussed in section 7.1.5. The 'in situ' undrained shear strength ($s_u$) increases nearly linearly from 0 - 15 kPa from 0 to 2.85 mblb. $s_u$ is higher than 40% of the effective overburden stress ($\sigma'_v$). The laboratory-based undrained shear strength ($\tau_u$) shows an equivalent quasi-linear increase in fall cone and vane shear data, but shows peak values exceeding 30 kPa. Hence, U1 is overconsolidated.

Figure 7.2: Bathymetric map of the SW part of Lake Villarrica including location of seismic profile A and B and CPT positions (modified after Moernaut et al. 2009). Isobaths every 5 m. Red area: extent of the upper failure plane that developed within silty diatomaceous ooze (Unit 2) immediately overlying a tephra (FPT). Dusky pink: extent of the quick clay (U3) sliding surface. Seismic profile A: W-E orientated, CPT- sketch illustrates CPT position and penetration depth. light blue zone: post failure Unit 1. dark blue: pre failure U1. green zone: Unit 2, dusky pink zone: Unite 3. red dotted line: extent of the FPT. Seismic profile B: see seismic profile A for color code.
Unit 2 (U2) (2.85 - 7.25 m blb) has a similar lithological composition as U1, but different physical properties. The sand fraction of U2 is reduced by 5 % whereas the silt fraction (mainly coarse silt, i.e. diatom frustules) is increased by 5 % in comparison to U1. Peaks in the sand fraction within the grain-size data are related to tephra layers. The main sediment constituent is equally a finely laminated diatomaceous ooze, with fewer detrital sand particles than in U1 (see SEM images in Fig. 7.3). The clay content and bulk density show a slightly increasing trend with depth, and reach values of \( \sim 20 \% \) and 1.5 g/cc, respectively. The \( w_n\% \) within the upper meter of U2 is \( \sim 275 \% \). Values within the lower portion range from \( \sim 90\% \) to \( \sim 160\% \). LL at 3.95 and 6.7 mblb is \( \sim 75\% \) and 110\%, with LL < \( w_n\% \). PI being 45 % and 22 %, respectively. Sensitivity shows an increasing trend with depth (137 - 266). U2 may thus technically be termed extra quick clay [Rosenqvist, 1953]. The 'in situ' undrained shear strength shows a slightly decreasing trend from \( \sim 15 \pm 5 \) kPa to 10 kPa with depth. Peaks that reach 40 kPa or more, can be related to the presence of tephra layers. Three prominent tephra layers have been identified at \( \sim 3.1\) m (\( s_u = 45 \) kPa; Pucon event), at 6.05 m (\( s_u = 40 \) kPa) and at \( \sim 7\) m (\( s_u = 92 \) kPa; Neltume Pumice). The tephra layer at 6.05 m depth has been called 'failure plane tephra' (FPT) as it is located immediately below a failure plane (see section 7.1.4.1) and Fig. 7.2, Fig. 7.3). In contrast to the CPT data, the vane shear strength of U2 shows a slightly increasing trend from \( \sim 7.5 \) kPa at the top to \( \sim 10 \) kPa at the bottom. The state of U2 overconsolidation decreases with depth and reaches \( \sim 0.3 - 0.4 \) * \( \sigma'_{o,0} \) at the bottom. Unit 3 (U3) comprises the lowermost portion of Core VillaR1. It is characterized by an increase in clay size fraction by \( \sim 20\% \) and represents proglacial extra-quick clay. Sensitivity reaches its maximum with a value of 370. \( w_n\% \) is < 100 % and decreases with depth. LL reaches \( \sim 80 \% \) (< \( w_n\% \)). Bulk density jumps to values > 1.5 g/cc. SEM images reveal an open and flocculated, detritic particle arrangement and a significant decrease in the amount of diatom frustules. \( s_u \) drops to values < 10 kPa whereas vane shear and fall cone data show higher values (\( \tau_u = 10 - 20 \) kPa). The fact that \( \tau_u \rightarrow s_u \) may be related to the penetration process of the CPT. Extremely low cone resistance values are measured in quick clay [Lundström et al., 2009], despite the fact that the peak shear strength of quick clay is not particularly low [Rosenqvist, 1966]. Hence, the \( s_u \) would be underestimated and explain the discrepancy between laboratory and 'in situ' shear strength. The state of consolidation would equally be underestimated.
Figure 7.3: Index properties of the sediment core VillaR 1 (site 1). For visualization issues the shear strength data is clipped at 40 kPa.
Coring site 2 - Lower Failure Plane

The sediment core Villa 754 is a short gravity core of 0.9 m length. It was recovered within the 'deep slide scar' and contains the lower failure plane (Fig. 7.2). Villa 754 covers partly the post failure sediment drape (U1) and about 0.3 m of unfailed U3. The excavated slope sequence, comprises parts of U1, all of U2 and parts of U3 and is marked as 'slide mass A' in Figure 7.3. Figure 7.4a shows the composite data plot of the core Villa 754. Here U1 and U3 are separated by a slump deposit of ~38 cm thickness and are further distinguished by equivalent differences in grain size, water content and bulk density as described above on the undisturbed plateau. Bulk density is > 1.5 g/cc in U3 and w_n% is < 100%. The clay content increases by ~20 % at the transition from the slump to U3 (Fig. 7.4a). U3 is characterized by the highest amount of clay size fraction, bulk density and sensitivity and lowest w_n%. The lower failure plane, on which the slide mass A had slid, is located within the quick clay of U3. Below the lower failure plane brittle 'in situ' deformation was observed. From core-to-seismic correlation we deduced that the lower failure horizon was not recovered at the reference coring site 1.2.

Figure 7.4: Index properties of the sediment core VillaR 754 (site 2) and VillaR 3 (site 3). For visualization issues the shear strength data is clipped at 40 kPa.
Coring site 3 - Upper Failure plane

Core VillaR 3 was taken within the stratigraphically upper slide scar (Fig. 7.2). It is \( \sim 4 \) m long and covers the post failure sediment drape (U1), the upper failure plane, \( \sim 1.25 \) m of unfailed U2 and \( \sim 1 \) m of U3 sediment. The upper failure plane is very distinct and located just above the sandy silt tephra layer, already named FPT, that is located at \( \sim 6.05 \) mblb at site 1.2 (Fig. 7.5). As due to the impedance difference, indicated by a peak in bulk density, this layer correlates to a strong reflection in the seismic profiles (Fig. 7.2). The slid sediment sequence is marked as 'slide mass B' in Figure 7.3. Figure 7.4 b shows the composite data plot of CPT and coring site 3. The lower section of the core (U3) has been kept as WH for further geotechnical experiments. U1 and U2 are separated by a slump deposit (see also Fig. 7.5) of \( \sim 25 \) cm thickness and are further distinguished by differences in grain size similar to those described for core VillaR 1. Bulk density ranges from 1.4 - 1.5 g/cc and \( w_n \% \) is \( > 100 \% \). The upper failure plane does not show major differences to U2 sediment. The grain size, natural water content, bulk density and shear strength are very similar to the over and underlying U2 sediment. The only observed anomaly is a peak in sensitivity within U2. Indeed, the upper failure plane is located immediately above a tephra layer (Fig. 7.5). However, there are several other tephra layers within U2 with properties comparable to the FPT (Fig. 7.5) (macroscopic observation). The location of the upper failure plane can neither be explained solely by its physical properties observed in core VillaR 1 nor can the proximity of the FPT be the sole reason for the development of the upper failure plane.

![Figure 7.5](image)

**Figure 7.5:** a) pictures showing the FPT and overlying sediment from core VillaR 1 (undisturbed plateau site 1.2) b) pictures showing the FPT underliying the distinct upper failure plane and the slump deposit from core VillaR 3 (Coring site 3, within the slide scarp).
132  CHAPTER 7. CASE STUDY

7.1.5.3 CPT-plateau-transect and pseudo-static Factor of Safety analysis

‘In situ’ CPTU deployments were carried out to shed light on the spatial variation in shear strength on the undisturbed plateau and its significance for failure localization. Figure 7.6a shows the ‘in situ’ undrained shear strength ($s_u$) with depth along a W-E transect across the unfailed plateau (Fig. 7.2). Due to local sedimentation rate variations (see Fig. 7.2 profile A) the CPT reached different stratigraphic levels, although the absolute penetration depth does not vary significantly at the three sites. U3 was only reached at reference coring site 1.2. At site 1.1 and 1.3 CPT penetration stops within U2, right in the Neltume pumice layer (located at 7 m blb at site 1.2) and $\sim$20cm above the FPT (Fig. 7.3). At all three sites the shear strength increases from 0 kPa to $\sim$15 kPa towards the bottom of U1. U2 shows a decreasing trend with depth at all sites, the gradient of which becomes steeper towards sites located closer to the head scarp. Close to the head scarp the shear strength above the FPT ($\sim$ 5kPa) is about two times lower than at the reference site 1.2 ($\sim$11.5 kPa). Values as low as 5 kPa are reached only within U3 at the reference site, but these values might not represent the correct peak ‘in situ’ undrained shear strength as mentioned before in section 7.1.4.2 Furthermore, the shear strength decrease within U2 trends toward values that would be expected in normally consolidated sediment (as indicated by the gray shaded normal consolidation lines in Figure 7.6a and 6b). Figure 7.6b shows the shear strength normalized with effective overburden stress assuming zero excess pore pressure. Dashed vertical lines indicate the range of normal consolidation from 0.2 - 0.4 * $\sigma'_{v,0}$. U1 and the top of U2 are overconsolidated (OC). This OC decreases gradually with depth. At site 1.1 and 1.2, U2 sediment shows values of $s_u/\sigma'_{v,0} > 0.3$ apart from the sediment segment right below the FPT at site 1.1. At site 1.3, U2 sediment shows values of $s_u/\sigma'_{v,0} < 0.3$ below $\sim$3.9 mblb. 20 cm above the FPT the normalized shear strength decreases to values $< 0.2$, indicating underconsolidation of the sediment. As the shear strength generally decreases with depth at all sites, we can expect even higher underconsolidation deeper in the strata at site 1.3.

The factor of safety was calculated for different k-values (i.e. earthquake intensities) and slope angles at site 1.1 to 1.3 (Fig. 7.6c and 6d) using equation 2: Regardless of the considered scenario of k-value and/or slope angle, the FS generally decreases gradually with depth at all sites, which is coherent with the decrease in OC. Peaks in the FS are related to peaks in cone resistance, corresponding to higher $s_u$ values of tephra layers. At a constant slope angle of 5° and varying k-values (Fig. 7.6c) it can be seen that the influence of the k-value variation (width of grey shaded area) decreases with depth. Figure
7.1. SUBAQUEOUS LANDSLIDE AND VOLCANIC ASH

Figure 7.6: a) 'in situ' undrained shear strength across the unfailed plateau transect. Grey shaded area indicates the undrained shear strength of normally consolidated sediments ($0.2-0.3 \times \sigma'_{v,0}$). Red lines: moving average over 151 data points. Grey dotted line (CPT 1.2): moving average over 7 values of laboratory based undrained shear strength (fall cone). For visualization purposes data are clipped at 20 kPa. b) Normalized 'in situ' undrained shear strength $\leq 2$. Grey shaded area: $s_u/\sigma'_{v,0} = 0.2 - 0.3$. c) Red vertical lines in figure 6c and 6d indicate the limit of stability (FS $>1$) and instability (FS $<1$).
CHAPTER 7. CASE STUDY

7.6d shows equivalent results for varying slope angles at constant k-value, however, the comparison of Figure 7.6c and 6d shows that the influence on the FS of the k-value is significantly more important than the influence of the slope angle. Within U2 the FS decreases both with depth and with increasing sedimentation rate analogue to the undrained shear strength. The analysis shows that an earthquake of MM ~ VIII (k=0.1) would lead to FS < 1 at all sites at slightly different levels, however, site 1.3 stands out with low FS values regardless of the MM. Considering a MM of ~ VII, i.e. an event comparable to the 2010 Chile earthquake, FS <1 would only be reached at site 1.3 ~ 20 cm above the FPT. An earthquake of MM ~ VI (k =0.0125) leads to FS values > 1 at all sites, however at site 1.3 ~ 20 cm above the FPT, FS = 1.2, i.e. close to failure. The spatial variance along the plateau transect shows that i) the FS is lower close to the head scarp and ii) close to the head scarp the FS is lowest slightly above the FPT. However, as $s_u$ and FS-values surrounding the upper failure plane at site 1.1 and 1.2 are even lower than $s_u$ of the upper failure plane, another mechanism must have taken place in order to reduce the shear strength and create failure at the exact horizon above the FPT, i.e. the upper failure plane. This observation leads us to the hypothesis that the tephra layer right below the upper failure plane might have facilitated slope instability. Furthermore, given the uncertainty about the $s_u$ in U3, advanced geotechnical experiments helped to reveal whether the quick clay or the U2 sediment is more susceptible to failure under static and cyclic loading than the tephra. The approach will further discriminate how the sediments involved in mass wasting respond to dynamic forcing, an aspect which has been ignored in the FS analysis.

7.1.5.4 Advanced geotechnical shear experiments

Drained direct shear tests on all units did not reveal significant differences (Fig. 7.7). The internal friction angle of U1, U2 and U3 sediment range from ~ 33.5° - 37°. There are no major differences in the drained shear strength of the lower and upper failure plane sediment. For comparative reasons we also tested the FPT-equivalent tephra for its drained shear strength, which emerged to be very high with ~ 45°. Figure 7.8 shows the cyclic triaxial data simulating an earthquake of modified Mercalli intensity VIII with a PGA of 0.25 g for all three units and the FPT equivalent material. Pore pressure ratio (PPR) and axial strain ($\epsilon_a$) evolution are presented. Exact test conditions are specified in Table 7.2. During cyclic loading, PPR increases rapidly in U1 and U2 samples and in the FPT equivalent. Within the first 20 loading cycles PPR reach 40-50 % of $p_0$. After 50-70 cycles the PPRs reach their maximum of ~65 -75 %. However, these sample didn’t reach the liquefaction
criteria (PPR = 90 %), which is related to the anisotropic testing conditions. In contrast to the behavior of U1, U2 and the FPT equivalent material, the pore pressure build up in U3 quick clay is very slow, reaching $\sim 20 \%$ of $p_0$ after 100 loading cycles. The $\epsilon_a = 5 \%$ failure criterion is first reached by the U2 sample, followed by U1 and U3, and finally the FPT equivalent. Although pore pressure evolution within the U1, U2 and tephra samples is similar, the axial strain evolution differs significantly. After an initial settling phase $\epsilon_a$ of the tephra samples increase approximately linearly, and reaches $5 \% \epsilon_a$ after more than 100 cycles. U1 and U2 sediment build up strain and pore pressure rapidly within the first 0 - 20 cycles, and subsequently reduce the PPR and $\epsilon_a$ increase per cycle. U3 sediment increases its $\epsilon_a$ gradient with increasing number of cycles. Progressive failure and inherent remolding of the sediment seem to favor $\epsilon_a$ accumulation so that $20 \% \epsilon_a$ is first reached by U3 quick clay and then by U2 diatomaceous ooze, which is coherent with the observations made on sensitivity.
CHAPTER 7. CASE STUDY

7.1.6 Discussion

In this study a subaqueous landslide has been investigated using advanced geotechnical methods with the objective of a detailed understanding of the failure initiation and propagation mechanisms. The focus of this study was to identify the failure planes and work out the reason for failure at these exact horizons within the sediment sequence. The combination of seismostratigraphic analysis, 'in situ' cone penetration testing and laboratory geotechnical measurements of sediment cores, allowed a comprehensive discussion of the complex slope failure initiation process. In the following sections we discuss the key aspects of presented data toward interpretation of the mechanical dynamic stability of the main lithologies and their interconnections in landslide initiation, propagation and retrogression along different stratigraphic levels.

Figure 7.8: Anisotropic cyclic triaxial tests. See table 7.1 for details
7.1. SUBAQUEOUS LANDSLIDE AND VOLCANIC ASH

### Table 7.1:

<table>
<thead>
<tr>
<th>Sample</th>
<th>$r_s$</th>
<th>$q_0$</th>
<th>$q_t$</th>
<th>$K_s$</th>
<th>B-value</th>
<th>ID</th>
<th>$H_0$ (mm)</th>
<th>N°</th>
</tr>
</thead>
<tbody>
<tr>
<td>U1</td>
<td>1.133</td>
<td>40.268</td>
<td>25</td>
<td>0.500</td>
<td>0.967</td>
<td>n.d</td>
<td>74.643</td>
<td>20</td>
</tr>
<tr>
<td>U2</td>
<td>1.193</td>
<td>40.816</td>
<td>25</td>
<td>0.496</td>
<td>0.974</td>
<td>n.d</td>
<td>76.269</td>
<td>10</td>
</tr>
<tr>
<td>U3</td>
<td>0.853</td>
<td>20.752</td>
<td>25</td>
<td>0.675</td>
<td>0.950</td>
<td>n.d</td>
<td>73.971</td>
<td>25</td>
</tr>
<tr>
<td>FPT-equivalent</td>
<td>1.163</td>
<td>40.048</td>
<td>25</td>
<td>0.502</td>
<td>0.975</td>
<td>0.721</td>
<td>72.057</td>
<td>252</td>
</tr>
<tr>
<td>FPT-equivalent</td>
<td>1.173</td>
<td>40.426</td>
<td>25</td>
<td>0.498</td>
<td>0.983</td>
<td>0.551</td>
<td>72.287</td>
<td>136</td>
</tr>
</tbody>
</table>

**7.1.6.1 Failure Planes and Sediment Physical Properties**

**Sensitivity**

Geotechnical laboratory tests and sediment characterization revealed that the only parameter in which the upper and lower failure planes were clearly silhouetted against the rest of the sediment sequence was the sensitivity (i.e. the ratio of peak to remolded shear strength). Extremely sensitive clay (quick clay) is defined as sediment that has a higher $w_n$% than its LL [Mitchell and Soga, 2005]. The term 'clay' here refers to grain size and not to mineralogy. Clay size particles with low activity (quartz, feldspar, amphibole, hydrous mica and chlorite) may arrange in a high void ratio, flocculated structure that resists consolidation and thereby remains at near-constant water content during burial. This structure is typical for post-glacial, fine-grained sediments [Torrance, 1983]. U3 sediment is a paradigm of quick clay (sensitivity > 300, Fig. 7.3).

U2 sediment has a $w_n$% > LL (Fig. 7.3), and thus may technically be termed 'quick clay', but as it contains ~45-50% coarse silt and sand (Fig. 7.3), the flocculation of particles is very unlikely. U2 has special physical properties due to the high amount of diatoms frustules (SEM pictures Fig. 7.3). Diatoms are hollow silicious microfossil shells bearing internal water, which explains high natural water contents and low bulk densities [Day, 1995]. During remolding and inherent partial crushing of the diatoms, the shell-internal water becomes interstitial water and leads to liquefaction of the material. The LL is reached once most of the 'excess' shell-internal water is expelled. Hence, LL < $w_n$% makes the sediment highly sensitive, and similar to classical quick clay once remolded.

High sensitivity as single indicator for sudden failure susceptibility is rather dubious, because it is a ratio that is mainly determined by extremely low
remolded shear strength. In order to produce failure and subsequently remold the sediment, peak shear strength must be transcended in the first place. Sensitivity can be a leading parameter in a progressive failure process under quasi-static conditions [Urciuoli et al., 2007], but this eventuality has been rejected as potential failure mechanism due to i) high internal friction angles ($\phi > 30^\circ$) (Fig. 7.7) in all slope charging sediments that make static failure on a 5° slope impossible, ii) high ‘in situ’ undrained shear strength ($s_u$) values (Fig. 7.6a), and iii) $FS \sim 1$ only if strong earthquake shaking is presumed. 'In situ' undrained and cyclic shear strength are the leading parameter in slope stability analysis and especially those factors need to be taken into account.

In situ undrained shear strength and state of consolidation

The 'in situ' undrained shear strength has been determined along a W-E transect on the undisturbed plateau (Fig. 7.2 and Fig. 7.6). This allows the identification of changes in shear strength in a 2D space. Considering soils with $s_u/\sigma'_{v,0} \sim 0.4$ as normally consolidated, the U1 and U2 (top) sediment is 'apparently overconsolidated'. Apparent Overconsolidation (AOC) typically occurs within the uppermost sediment cover of 0 - 5 m [Sultan et al., 2000]. The AOC is thought to be related to weak interparticle bonds, bioturbation, strengthening by currents and wave action or seismic shaking. In Villarrica, currents and wave action are caused by a strong wind called Puelche that hits the lake regularly in E-W direction [Meruane Naranjo, 2005]. A common decrease in the AOC with depth [Busch and Keller, 1981; Lee and Baraza, 1999; Suess et al., 1990] can be interpreted as the reason for decreasing $s_u/\sigma'_{v,0}$ with depth within U1 and the top of U2. Deduced from Figure 7.6, potential shear failure is restricted to the zone below the AOC. With continues sedimentation, the AOC is moving up-section, and a layer or stratigraphic contact with the potential to become a failure plane must first be buried to a certain depth before it may be triggered as critical failure layer for landslide initiation.

However, the negative trend in $s_u$ with depth (Fig. 7.6a) at all three sites can only be related to overpressure. There is evidence for remaining overpressure, particularly close to the head scarp, at site 1.3 where the sediment is underconsolidated ($s_u/\sigma'_{v,0} < 0.2$ and the FS is $< 1$ given an earthquake of MM $\sim$ VII). It seems likely that the overpressure is locally constrained and may be provided by focused fluid escape from lower stratigraphic levels. Sites 1.2 and 1.3 are only $\sim$50 m apart, but the shear strength and normalized shear strength profiles differs significantly. Such strong spatial variation, as observed on the E-W transect, would not be expected in case of earthquake
shaking induced excess pore pressure. Furthermore, sedimentation rate differences from site 1.2 to 1.3 are not considered relevant for overpressure and inherent shear strength differences because neither extreme sedimentation rates nor high sediment clay-contents that would reduce the permeability have been encountered. Therefore we hypothesize a fluid source at depth to create overpressure in shallow depth [Dugan and Germaine, 2008].

In any case, low $s_u$ values shortly above the FPT at site 1.3 may directly be related to excess pore pressure. Yet, this factor does not fully explain the exact location of the upper failure plane because even lower $s_u$ values can be expected deeper in the stratigraphy. This can be deduced from the continuously decreasing $s_u$ below the FPT at site 1.1 and 1.2. Hence, the excess pore pressure build up within the FPT during earthquake shaking (Fig. 7.8) must have reduced the resisting forces at the boundary to the U2 sediment in such extend that it becomes the weakest spot in the sequence.

The role of ash

Tephra layers are presumed to have high liquefaction susceptibility and may represent preferred failure planes of submarine slides in active geological settings [Harders et al., 2010; Sassa et al., 2012]. However, not every tephra layers is a failure plane and not every failure plane is close to a tephra layer in such settings. Our data from Lake Villarica clearly indicates that neither the tephra layer at 3.1 mblb, nor the pumice layer at 7 mblb ever acted as a failure plane in our study area. Since the pumice layer at 7 m is older than the FPT, it experienced significantly more high magnitude earthquakes than the FPT, and must have had the opportunity to liquefy many more times, but it did not. The discrepancy can be explained with the anisotropic, undrained, cyclic triaxial data, that show three main results: i) U2 sediment builds up significant axial strain and pore pressure ii) the failure plane FPT-equivalent only slowly accumulated axial strain, but pore pressure increases very fast in comparison to U2 and U3 sediment and iii) the U3 quick clay neither quickly accumulates axial strain nor does the pore pressure rise significantly during initial cyclic loading (Fig. 7.8). This data gives evidence for the fact that flow liquefaction did not occur within the FPT. Instead, localization of slope failure initiation immediately above the FPT (Fig. 7.5) must be caused by the coupling of the pore pressure build up and high hydraulic conductivity of the FPT induced by grain size, with the weakness of U2 sediment just above it. Consequently the FPT is only responsible for the ultimate strength reduction of the U2 sediment at the interface of U2 and the FPT. I.e. excess pore pressure within the FPT is directly transmitted to the interface and reduces the resisting forces, but flow liquefaction of the ash was not the reason
for failure. Once the peak shear strength was transcended, high sensitivity of the failure horizon lead to a clear slope parallel shear plane and translational slide (Fig. 7.9).

**The role of quick clay**

Quick clay slides are known to be predominantly retrogressive in nature. In Lake Villarrica, there is multiple evidence that the lower failure plane (U3 quick clay) was activated after the failure of the upper failure plane (U2 horizon) had occurred: i) From a morphological point of view the 'bottle-neck-shaped' outlet (Fig. 7.2) points toward retrogression [Ter-Stepanian, 2000]. ii) Neither in static nor in cyclic loading experiments (Fig. 7.7 and Fig. 7.8) the quick clay juts out with low shear strength values. Progressive failure in cyclic loading experiments (Fig. 7.8) forebodes retrogression and even liquefaction, which again is needed to produce a 'bottle-neck-shape' outlet.

Retrogression as the dominant mechanism to explain the observed geometries with two failure planes and distinct scarp is explained with the following scenario (Fig. 7.9): After triggering of the upper failure plane and inherent mass movement, the backstop for the quick clay unit (U3) was locally removed and local strain could accumulate (Fig. 7.8). With progressive strain accumulation (brittle failure) and shear strength reduction of U3 sediments, progressive slope failure was reached. Down slope transport led to a total remolding of the entire sediment sequence, which ended in mass wasting along the 'bottle-neck-shaped' channel.

### 7.1.7 Conclusion

This manuscript testifies unambiguously that some of the complexity associated with natural hazards can only be discovered and subsequently understood with a coupled, 'in situ' and laboratory geotechnical approach. Our results show that static gravitational failure of the investigated slope under drained conditions would be unfounded because high angles of internal friction (Fig. 7.7) make failure on a 5° slope impossible. Instead, an additional external trigger is required, which we have approximated in this study as an earthquake of Modified Mercalli Intensity (MM) of ~ VII or higher, which lead to sediment failure within U2 diatomaceous ooze just above a tephra layer. Pore pressure increase and inherent reduction of effective stresses at the contact surface of the tephra layer and the weak, underconsolidated diatomaceous background sediment triggered a first failure phase. Subsequently quick clay failure occurred in retrogression as a result of the first failure phase that
Figure 7.9: Simplified model for landslide process (left). Illustration of undrained shear strength variation due to remaining excess pore pressure along the W-E transect and influence of earthquake induced pore pressure (right). a) shear strength decrease with local sedimentation rate and inherent excess pore pressure increase. b) earthquake shaking leads to pore pressure increase in the tephra layer (FPT) and ultimately reduces the shear strength at the interface of U2 and the FPT. c) retrogressive and progressive failure in quick clay unit due to backstop removal.
facilitated ramp-out after backstop removal (Fig. 7.9). The hypothesis that the subaqueous slides in Lake Villarrica were earthquake triggered was based on interpretation of seismic reflection data [Moernaut et al., 2009] and is now supported by geotechnical data.

Our study of landslide-prone Lake Villarrica allows drawing a number of additional conclusions with global significance for larger submarine slope failures from the geotechnical experiments:

- Full liquefaction in deeply buried tephra is unlikely due to anisotropic stress conditions and cyclic strengthening effects in which repeated shaking from non-failure earthquakes progressively dewater and thus strengthen the sediment, although more data are needed for confirmation.

- Diatomaceous earth has fundamentally different physical properties compared to organic or inorganic soils, and might not completely be characterized by conventional geotechnical methods. Further examination is mandatory for landslide initiation studies as diatomaceous sediment occur frequently in combination with volcanic ash on active margins [Hamme et al., 2010]. Furthermore it has a similar potential to build up pore pressure as silts or sands. However, we agree with Locat and Tanaka [2001] to introduce a new class of soil for fossiliferous soils according to their abnormal behavior.

7.1.8 Acknowledgement

Matthias Lange is thanked for technical assistance with the static and dynamic shear apparatus at MARUM Marine Geotechnics laboratory. We are grateful to Deutsche Forschungsgemeinschaft (Bonn, Germany) for funding MARUM - Center for Marine Environmental Sciences. We thank Alejandro Peña, Robert Brümmer and Koen De Rycker for their logistic and technical support of the geophysical surveys. This work was financially supported by the Research Foundation Flanders (FWO-Vlaanderen) and the Swiss National Science Foundation (grant 133481).
7.1.9 References


Bertrand, S. (2005), Sédimentation lacustre postérieure au dernier maximum glaciaire dans les lacs Icalma et Puyehue (Chili méridional): réconstitution de la variabilité climatique et des événements sismo-tectoniques, Université de Liège, Liège.


CHAPTER 7. CASE STUDY


Karlsson, R., and L. Viberg (1967), Ratio of c/p in Relation to Liquide Limit and Plasticity Index, with Special reference to Swedish Clays Rep., 43-
7.1. *SUBAQUEOUS LANDSLIDE AND VOLCANIC ASH*

47 pp, Oslo.


Moernaut, J., M. De Batist, K. Heirman, M. Van Daele, M. Pino, R. Brümmer, and R. Urrutia (2009), Fluidization of buried mass-wasting deposits in lake sediments and its relevance for paleoseismology: Results from a reflection seismic study of lakes Villarrica and Calafquén (South-Central Chile), Sediment Geol, 213, 121-135.


Rosenqvist, I. T. (1966), Norwegian research into the properties of quick clay—a review, Engineering Geology, 1(6), 445-450.


7.1. SUBAQUEOUS LANDSLIDE AND VOLCANIC ASH


Torrance, J. K. (1983), Towards a general model of quick clay development, Sedimentology, 30(4), 547-555.


7.2 The influence of excess pore pressure, fluid flow and depositional patterns on subaquatic slope stability: a detailed case study of Lake Villarrica (South-Central Chile)

1,5 Jasper Moernaut, 2 Gauvain Wiemer, 1 Anna Reusch, 4 Nina Stark, 3 Marc De Batist, 5 Mario Pino, 6 Roberto Urrutia, 7 Bruno Ladrón de Guevara, 1 Achim Kopf and 2 Michael Strasser

1 Geological Institute, ETH Zürich, Switzerland
2 MARUM, Centre for Marine Environmental Sciences, Bremen, Germany
3 Renard Centre of Marine Geology, Ghent University, Belgium
4 Department of Civil and Environmental Engineering, Virginia Tech, Blacksburg, USA
5 Instituto de Geociencias, Universidad Austral de Chile, Chile
6 Centro EULA, Universidad de Concepción, Chile
7 BENTOS, Santiago, Chile

In preparation for the International Journal of Earth Sciences (IJES)

7.2.1 Abstract

A dense network of seismic-reflection, multibeam bathymetry, sedimentological and geotechnical data provides a detailed characterization of a subaquatic slope in Lake Villarrica (Chile). This allowed investigating the interplay of excess pore pressure, focused fluid escape and subaquatic slope failures in unprecedented detail. In-situ undrained shear strength and formation pore fluid pressures were documented by free-fall piezocone penetrometer (CPTU) deployments and subsequent pore pressure dissipation tests. We show four independent lines of evidence for high overpressure ratios within the sedimentary slopes: i) pockmarks and other fluid escape features on seismic profiles and bathymetric maps, ii) in-situ pore pressure data via CPTU dissipation tests, iii) downward decrease in undrained shear strength within uniform lithologies, and iv) hydro-fractured glacio-lacustrine sediments. These data are embedded within the depositional history of this glacigenic lake derived from seismic-stratigraphic analysis. We infer that excess pore pressure originates or is transferred within rapidly-deposited glacier-proximal units
capped by relatively impermeable glacio-lacustrine clays and fine silts. Our study confirms the major role of fluid pressure for preconditioning subaqueous slopes to failure, especially at formerly-glaciated marine and lacustrine areas.

7.2.2 Introduction

One of the key challenges in submarine landslide research is to understand and quantify the role of different preconditioning factors that affect subaqueous slope stability. Pore water overpressure is regarded as one of the most important preconditioning factors as it decreases the effective stresses and thus the resisting forces of a sedimentary sequence against downslope driving forces. It has often been invoked to explain the initiation and regressive behavior of extensive submarine landslides on relatively low (<3°) slope gradients (e.g. Field et al., 1982; Bryn et al., 2005), the process of which has been experimentally justified (Vendeville and Gaullier, 2003; Lacoste et al., 2012). Additionally, many studies suggest an ultimate short-term triggering mechanism, such as earthquake shaking, for slope failure to actually take place (e.g. Canals et al., 2004; ten Brink et al., 2009).

It was thought for a long time that overpressure in fine-grained material could only exist at significant subsurface depths (>1.5 km; Mello et al., 1994). During the last decade, the application of in-situ piezometers and pore pressure penetrometers (Flemings et al., 2008; Lafuerza et al., 2009; Stegmann et al., 2011) and laboratory consolidation experiments (Dugan and Germaine, 2008) provide more and more evidence for shallower overpressure within the uppermost hundreds of meters. In young sedimentary basins, the main process for shallow overpressure generation is considered to be disequilibrium compaction due to rapid sedimentation of compressible low-permeability material, which effectively prevents full drainage of excess pore fluids. Consequently, the pore fluid supports some of the overburden weight and pore pressure rises above hydrostatic values (Dugan and Sheahan, 2012 and references therein). Differences in burial rate can result in overpressure gradients, causing lateral flow of pore fluids through high-permeability layers from areas with a thick overburden to those with thinner overburden (Dugan and Flemings, 2000). Hence, shallow overpressures can prevail in areas of low sedimentation rate as a result from fluid transfer from deeper areas via more permeable layers and vertical conduits.

When focused fluid flow reaches the seafloor, it may create characteristic morphological features such as pockmarks (craters) and mud volcanoes. These features are often found in association with landslides and are used as indicators of overpressures down in the sedimentary sequence (Hovland et al., 2002). A genetic link between focused fluid escape and landslide initiation...
in different regions was suggested by: i) the presence of pockmark fields and mud volcanoes in and near the failed slope (Piper et al., 1999; McAdoo et al., 2000; Naudts et al., 2006), ii) headwall scarps that spatially link with presumed pockmarks (Lastras et al., 2004) or iii) reflection-seismic anomalies such as enhanced reflections and chimneys/pipes of acoustic blanking, respectively indicative of accumulation and vertical transfer of fluids (Bünz et al., 2005; Gee et al., 2007; Sultan et al., 2010; Berndt et al., 2012; Riboulot et al., 2013). If fluid flow is a persistent or recurrent phenomenon, it may allow for repeated failure at a single location, as is hypothesized for example at the “Ana Slide complex” in the Eivissa Channel, western Mediterranean Sea (Berndt et al., 2012). Here, free gas is supposed to be a major factor preconditioning the slope to fail, but sediment cores or penetrometers did not evidence gas-charged sediments in the present-day sequence (Laueferza et al., 2012). This clearly illustrates the challenges related to reconstructing past slope stability as the failed slope (i.e. the object of study) and its former pore pressure state are no longer present, and only the adjacent unfailed portions of the slope are left to be explored. In this light, the best understanding of slope failure processes can be gained by investigating relatively young (Holocene) and small-scale landslide events.

Figure 7.10: Modified from Moernaut et al., submitted. A) Setting of our study in South-Central Chile with indication of the main rupture areas (co-seismic slip > 5m) of the 2010 (Mw 8.8) and 1960 (Mw 9.5) megathrust earthquakes. B) Lake Villarrica and its drainage basin with indication of active volcanoes (triangles).
Lake basins offer this opportunity as sub-recent landslides of different sizes are common components of their infill (Moernaut and De Batist, 2011). Their small dimensions and shallow water depths allow for very-high-resolution datasets and the critical layers that acted as sliding surfaces during slope failure can be accessed with relatively simple and cost-effective methods (e.g. Stegmann et al., 2007; Strasser et al., 2011). Nevertheless, lacustrine slope stability studies are very scarce and detailed characterization of in-situ stress conditions is lacking.

In the present study, we characterize in unprecedented detail the morphology, stratigraphy, in-situ shear strength and ambient pore pressure state of a remnant sublacustrine slope and below a failed slope section. We document the spatial relationship between shallow overpressures, focused fluid escape and slope failure, and embed this within the depositional architecture of the glacigenic lake basin.

7.2.3 Setting

Lake Villarrica (39°15' S; 72°02' W; 214 m a.s.l.) is a large (21 km x 9 km) glacigenic lake with a maximum depth of 167 m (Fig. 7.10, 7.11). It is located at the western piedmont of the volcanically-active Chilean Andes in a region that has experienced strong shaking during several subduction megathrust earthquakes; the most well-known of which happened in 1960 (Mw 9.5) along the Valdivia Segment and in 2010 (Mw 8.8) along the Maule Segment. The lake basin originated from glacial valley overdeepening and the formation of successive frontal moraine belts during the Late Quaternary glaciations (Laugenie, 1982). Regional deglaciation is believed to have started around 17.5–17.1 ka BP in concert with several discrete warming pulses (McCulloch et al., 2000), with exception of Lake Puyehue where ice-free conditions were already present at 18 ka BP or even earlier (Heirman et al., 2011). Nowadays, the lake basin consists of a deep central basin and a shallower area in its southwestern part which is made up of several rock basement peaks, moraine ridges and sedimentary sub-basins (Fig. 7.11, 7.12; Moernaut et al., 2009). The main river in- and outflow are located at its eastern and western extremities, respectively. Lahars originated on the flanks of Villarrica Volcano frequently entered the central basin of the lake (Van Daele et al., in press). Since the last deglaciation, a gradual incision of the outflowing river within the frontal moraine ridges created a generally decreasing trend in lake level (Laugenie, 1982). In historical times, this open-lake systems has not exhibited large (>5 m) lake-level fluctuations due to its relatively large drainage basin and the overall high rainfall regime in the region (2000 mm/yr).
In the southwestern area, the irregular basement morphology is directly overlain by a complex rapidly-deposited sedimentary infill (up to 120 m thick) deposited during early deglaciation. This thick infill is covered by a more continuous and thinner drape (10-20 m) of hemi-pelagic sediments, deposited in an open, post-glacial lake environment during the Holocene (Moernaut et al., 2009; Heirman, 2011). The Holocene background sediments can be classified as diatomaceous ooze, with a small fraction of terrestrial organic matter, dispersed volcanic ash and terrigenous clays/silts. These sequences are frequently interrupted by thin tephra-fall layers and lahar deposits (Moernaut et al., 2009; Van Daele et al., in press). Seismic-stratigraphic analysis showed that several voluminous landslides (1-6 \times 10^6 m^3) took place in the sedimentary slopes of the southwestern extremity of the lake and were deposited in a stacked manner in the sub-basins (Moernaut et al., 2009).

Our present study target is an unfailed sedimentary “platform” which is almost completely surrounded by failed slope sections (Fig. 7.12). Here, geotechnical and sedimentological studies were conducted in order to characterize the strength of the sedimentary sequences involved in past slope failure when subjected to static and cyclic loading (Wiemer et al., submitted). This study documented three main depositional units that represent the post-glacial sediment drape (see also Fig. 7.13, 7.14) (from lake floor down): U1: sandy-silt diatomaceous ooze, U2: silt diatomaceous ooze and U3: detrital “quick” clay. This study suggests that the investigated landslide failed initially at the contact of relatively weak Unit 2 sediments overlying a prominent
sandy-silty tephra layer. The failure further developed retrogressively within the stratigraphically-deeper quick clays of Unit 3. The unfailed platform was also characterized by free-fall piezocone penetrometer tests (CPTU) to calculate the in-situ shear strength of the sediment sequence. This data showed a striking downward decrease in shear strength, which we will further investigate by spatially extending the dataset and including in-situ pore pressure data. Moreover, we embed this geotechnical data within the detailed seismic stratigraphy of this area and newly-acquired high-resolution bathymetry data.

7.2.4 Methods

7.2.4.1 Bathymetry and sub-surface imaging

High-resolution bathymetry data was acquired in 2013 using an R2Sonic 2024 multibeam system (200-400 kHz) deployed on R/V Bentos Surveyor of the survey company BENTOS (Santiago, Chile). The position of the boat was determined by real time kinematic (RTK) corrected GPS with horizontal accuracy of a decimeter (OmniSTAR G2). Multibeam swath angle and frequency was modified on-line depending on water depth and data quality. Data processing (corrections for sound velocity, ship motion and orientation, sensor offsets, spike removal, etc.) and visualization was carried out using CARIS HIPS and SIPS and standard GIS software. All presented bathymetric maps have a horizontal grid size of 3 m. For illustrative cleanness, color scale and hill-shade effects were varied for each Figure. Side-scan sonar imagery was acquired in 2011 for a better resolution of the slope failure features using a KLEIN 3000 sonar (100 and 500 kHz) and SonarPro software.

Dense networks of reflection seismic data (SI-Fig. 7.10) were acquired in 2001, 2007 and 2011 using R/V Huala Il of the Universidad Austral de Chile. Navigation and positioning was done using stand-alone GPS and the data was acquired on a TRITON-ELICS Delph-2 system. In order to image the entire infill of the lake we used a CENTIPEDE multi-electrode “sparker” (400-1500 Hz) as seismic source and a single-channel high-resolution streamer as receiver. For a high-resolution characterization of the subsurface, we used a “pinger” source/receiver (sub-bottom profiler; 3.5 kHz), which was mounted on a Cataract system and towed behind or at the side of the vessel. Seismic-stratigraphic interpretation was done using IHS Kingdom Suite after applying a frequency band-pass filter (more details in Moernaut et al., 2009). Water and subsurface depths for the uppermost seismic units were calculated using an acoustic velocity of 1500 m/s. As this is an underestimation for the lower seismic units typically encountered in glacigenic lakes (Finckh et al., 1984),
we report the thicknesses for the general seismic stratigraphy in milliseconds Two-Way Travel Time (TWT).

### 7.2.4.2 Sediment cores

Several hammer-driven piston cores were taken from an anchored platform on the unfailed and failed slope (Fig. 7.14). The composite cores consist of successive 3-m sections. The closed cores were scanned in a GEOTEK multi-sensor core logger at ETH Zürich for obtaining the density, magnetic susceptibility and P-wave velocity of the sediments. Detailed sedimentological (grain-size, macroscopic description, water content, scanning electron microscopy), and geotechnical parameters (undrained shear strength, Atterberg Limits, sensitivity) of these sediments were obtained at the Centre for Marine Environmental Sciences (MARUM), Bremen, and are presented in Wiemer et al. (submitted).

### 7.2.4.3 In-situ geotechnical measurements

The MARUM free-fall Piezocone Penetrometer (CPTU; details in Stegmann et al., 2006) was deployed from an anchored platform on the coring sites and on E-W and N-S transects in the vicinity. In this way, we obtained in-situ geotechnical parameters of the Late Glacial to Holocene sedimentary sequences in different environments, such as at an unfailed slope, under a failed slope and areas of fluid escape down to more than 8 m below lake bottom. The CPTU measured cone resistance, sleeve friction, deceleration and absolute pore pressure up to a sub-surface depth of about 8 m. Penetration depth of the CPTU is derived from double integration of the deceleration signal over time. Different stratigraphic levels were studied by varying the free-fall penetration velocities. Pogo-style measurements were made on-the-go in a short lance mode (3 m long) from the vessel Huala II. At each site, at least three drops were made in order to select the most representative dataset. Undrained shear strength was derived from pore-pressure and strain-rate corrected cone resistance using the following geotechnical solution:

\[
s_{u, \text{quasi-stat}} = \frac{q_{t, \text{quasi-stat}} - \sigma'_{v,0}}{N_{kt}}
\]  

(7.11)

Where \(s_{u}\) is the undrained shear strength, \(q_{t, \text{quasi-stat}}\) is the pore pressure and strain-rate corrected cone resistance which is comparable to standard pushed CPT cone resistance, \(\sigma'_{v,0}\) is the hydrostatic vertical effective stress and \(N_{kt}\) is an empirically determined cone factor (Lunne et al., 1997), here set to 14. For details on MARUM CPTU data processing and post-processing, including strain-rate correction see Steiner (2013).
The evolution of the pore pressure signal induced by probe penetration and inherent displacement of sediment and fluids, towards ambient formation pressures designates a dissipation test (details in Stegmann et al., 2011). Thirteen pore pressure dissipation tests were carried out of which dissipation duration ranged from 27 to 49 minutes (Table 7.2). The shape of dissipation curves is generally dependent on the hydraulic conductivity and the behavior of the sediments and pore fluids during insertion. Over-consolidated fine-grained soils—as in our study area (Wiemer et al., submitted)—typically show dilatant shearing behavior and give rise to sub-hydrostatic pore water pressure recorded at the u2 position during penetration. Hence we are confronted to ‘non standard’ dissipation curves in which the initial excess pore pressure values at the beginning of dissipation are negative. These values can either rise to a peak and subsequently slowly decrease toward ambient conditions like a standard curve, or slowly rise to a “plateau” without recording any peak in pore pressure (Sully et al., 1999; Stegmann et al., 2006; Chai et al., 2012). Partial clogging of the pore pressure filter with fine sediments can oc-

**Figure 7.12:** Multibeam bathymetric map for the SW part of Lake Villarrica (location: see Fig. 7.11) with indication of the geotechnical study area (CPT sites). The map exhibits outcropping rock basement highs, a moraine ridge and headwall scarps. Detailed maps of the headwall scarps are provided in Fig. 7.17. MTD: Mass-transport deposit.
7.2. PRECONDITIONS OF SUBAQUATIC SLOPE FAILURE

cur during successive drops, resulting in a delayed response in measured pore pressure during free fall and penetration. Therefore, we consider the highest recorded pore pressure at the moment of lake bottom contact for all drops at the same site as the most realistic hydrostatic lake bottom pressure. As lake level—and thus hydrostatic pressure at the bottom—is variable, we made this correction independently for each survey day. The obtained hydrostatic values are comparable to those based on the site’s water depth derived from the high-resolution multibeam data. The hydrostatic pressure at maximum CPT penetration depth was calculated by adding the hydrostatic pressure at the lake bottom to the hydrostatic pressure induced by the water column along the penetration depth (assuming water-saturated sediments). Due to the relatively long dissipation time, we are confident that the measured pore pressure values at the end of dissipation testing are only influenced by the insertion pressure and the ambient pore pressure and not by other possible short-lived deployment artifacts. However, due to the characteristics of the dynamic penetration process (Stegmann et al., 2006), it is rather unreliable to use the style of the dissipation curve or the half-time value (t50) between peak insertion pressure and ambient pressure as indicators of in-situ horizontal permeability. Therefore, we omitted this approach from our study.

In case pore pressure values at the end of dissipation did not reach steady state, we extrapolated the gradient of the ultimate part of the curve using the “inverse time” approach, which was successfully tested and applied in different marine settings (Flemings et al., 2008; Sultan et al., 2009). The extrapolation techniques 1/t and 1/√t were empirically compared (see SI-Fig. 7.11). Based on two tests at a single site showing contrasting dissipation profiles, we found that the “inverse time” approach is only applicable in case of a decreasing pore pressure at the end of the test. This comparison also revealed that the 1/t method systematically overestimates the ambient pore pressure value for our sediments (cfr. Long et al., 2007), whereas the 1/√t method systematically underestimates it. We subtract the hydrostatic pressure (u_h) of the “ambient” pore pressure to calculate the overpressure (u*) at specific stratigraphic intervals. The overpressure ratio (λ*) (see Flemings et al., 2008) represents the amount of excess pore pressure relative to the hydrostatic vertical effective stress (σ'_v,0), which is calculated from the density data obtained on the sediment cores. For sites without core data, we used the average density values for the different units (U1-U3) and applied these to a reconstructed lithological profile derived from the seismic profiles and the su profiles (Table 7.2). The “ambient” pore pressure values and overpressure ratios (λ*) are reported (Table 7.2) as either: i) one value, in case of a complete dissipation; ii) a minimum value, i.e. the last data point of a rising dissipation curve; or iii) a range, in case of a decreasing dissipation
curve. Minimum and maximum possible pore pressure of the range is based on extrapolation via the $1/\sqrt{t}$ and $1/t$ techniques, respectively.

Table 7.2: CPTU pore pressure dissipation data at the different sites used in this article (see section 3.3 for details) with $u_h = \text{hydrostatic pressure}$; $\sigma_{v,0}' = \text{hydrostatic vertical effective stress}$; $u^* = \text{overpressure}$; $\lambda^* = \text{overpressure ratio}$. The $u_h$ is calculated for fresh water for the depth reached by the CPTU probe. $\sigma_{v,0}'$ is derived from the $\gamma$-density data. The used density value for CPT2 (mainly U3) and CPT5 (mix of U1-U2-U3) is 1.6 g/cm$^3$ and 1.5 g/cm$^3$, respectively. This is based on seismic-stratigraphic determination of the depositional unit and its typical $\gamma$-density value obtained on the coring sites. Due to lake level fluctuations between different measurement days, $u_h$ at the lake bottom at a single site can range within 10 kPa.

7.2.5 Results

7.2.5.1 Sedimentary infill of Lake Villarrica

Sparker seismic profiles illustrate that the southwestern part of Lake Villarrica is characterized by a complex sedimentary infill (Fig. 7.13). We divide the subsurface into four principal seismic units deposited on top of the rock basement. Detailed identification of this basement is generally hampered by limited acoustic energy reflected at these sub-surface depths. The lower seismic unit SU-A is made up of parallel sub-horizontal to slightly undulating reflections which exhibit vertical wipe-outs and large lateral amplitude changes. It is the thickest unit (up to 140 ms) and has an infilling depositional geometry.
Figure 7.13: Sparker seismic profile (west-east) illustrating the complete succession of characteristic seismic units overlying the rock basement (see section 7.2.4.1). Recurrent landsliding is evidenced by stacked mass-transport deposits (MTDs) within unit SU-D and an evacuated (failed) slope. Pockmarks were identified in the westernmost part of the profile. Identification of the rock basement reflector is hampered by “multiple” reflections and loss of seismic wave energy with depth.
Its upper boundary is an irregular surface, locally cutting into the unit’s reflections and is interpreted as an erosional unconformity. In the southwestern extremity of the lake, this unconformity and the acoustic basement is overlain by a massive unit SU-B which is characterized by a chaotic acoustic facies and an undulating eastwards-dipping upper boundary composed of several peaks and troughs. SU-B has its maximum thickness in its western half (up to 90 ms). Both units SU-A and SU-B are overlain by SU-C which has a similar acoustic facies as SU-A and partly levels out the pre-existing morphological variability. SU-C is divided in two sub-units: i) SU-C1 which is only present in the basin and its reflectors present large lateral amplitude changes, and ii) SU-C2 which drapes the entire area and has more uniform reflection amplitudes. SU-C2 is covered concordantly by the continuous, parallel and high-amplitude reflections of SU-D. Its reflectors are locally intercalated by lenses of chaotic-to-transparent facies interpreted as landslide deposits (Moernaut et al., 2009). In the southwest part of the lake, Units SU-C2 and SU-D were evacuated at many places during such slope failure processes. In the basinal area where these units are intact, both SU-C and SU-D are around 25-30 ms thick. We interpret this succession of seismic units by comparing them with seismic-stratigraphic studies of several glacigenic lakes worldwide (Van Rensbergen et al., 1998; Eyles et al., 2000; Heirman et al., 2011; Pinson et al., 2012; Ndiaye et al., in press). The location, morphology and seismic facies of SU-B resembles in all aspects the lowermost unit described in these studies and is generally interpreted as a complex and heterogeneous assemblage of glacier-edge deposits and landforms, such as subaquatic outwash fans, frontal dump moraines, glacier push moraines, etc. In South-Central Chile, these glacigenic assemblages were rapidly deposited during the last glacial maximum and during the onset of last deglaciation, and form a wide variety of sedimentary facies ranging from deformed glacio-lacustrine clays to gravels and boulders (Denton et al., 1999). In South-Central Chilean lakes, this seismic unit can be relatively massive (Heirman et al., 2011) and its westernmost location just offshore the frontal moraine belts (Fig. 7.11) confirms its intimate association with glacier-proximal processes during the initial phase of deglaciation. Units SU-C and SU-D can be interpreted as the typical succession of depositional units related to a retreating glacier. Unit SU-C represents glacio-lacustrine sediments deposited in a proglacial setting. When the glacier was still in close contact to the studied basin, coarse outwash sediments were deposited in the basin (SU-C1). The glacier front progressively retreated out of the lake basin, into the Andes. Meltwater-laden rivers provided large volumes of fine-grained glacially-eroded sediments (SU-C2) to the proglacial lake (“glacial flour”). When temperatures culminated at the initiation of the Holocene, the catchment became essentially ice-free
and hemipelagic lacustrine sedimentation started draping the lake bottom and formed SU-D. This seismic-stratigraphic interpretation of the upper two units is confirmed by the sediment core VILLAR1 (Fig. 7.14B) taken on an unfailed portion of the slope. This core contains 7.3 m of diatomaceous ooze (U1 and U2 of Wiemer et al., submitted; SU-D) on top of detritic fine silt and clay (U3; SU-C2). Many sub-vertical fractures with small offsets are present in the rhythmically-layered sediments of U3. This strongly fractured pattern was also encountered in other cores in the lake and at the border of a comparable lake in the region (Fig. 7.15). Therefore, we suggest that it may be a general characteristic of this glacio-lacustrine depositional unit. One AMS radiocarbon date (Fig. 7.14B) and some well-studied tephra layers in this core (Wiemer et al., submitted) place the transition of SU-C2 to SU-D at around 12.7 ka BP and infer sedimentation rates between 0.4 and 0.8 mm/yr for the hemipelagic Holocene sediments at this specific location. Deglaciation of the Chilean Lake District is believed to have started around 17.5-17.1 ka BP at most places (McCullogh et al., 2000). Applying this date to the boundary of SU-B and SU-C2 at the coring site, we obtain a sedimentation rate of about 3 mm/yr for unit SU-C2 which is significantly higher than for SU-D. The main difference between the sedimentary infill of Lake Villarrica and the glacigenic lakes of abovementioned studies is the presence of a thick unit SU-A below the glacial unit SU-B. Given its similar seismic facies as SU-C, it can be regarded as a glacio-lacustrine unit deposited in a proglacial environment during former interstadials within the last glacial period. The local high-amplitude reflections may represent coarser deposits resulting from sporadic glacier meltwater outbursts or short-lived episodes with a closer glacier terminus. During the Last Glacial Maximum, this unit was overridden by an advancing glacier, scouring into the unconsolidated SU-A sediments and dumping a glacier-proximal unit (SU-B) on top of it. Apparently, the advancing glacier did not fully erode the pre-existing lake sediments, a phenomenon which has not been documented often, except maybe in the Chilean Lake Puyehue (see Heirman et al., 2011).

7.2.5.2 Focused fluid escape and headwall scarps

The newly-acquired multibeam bathymetry data (Fig. 7.12, 7.16, 7.17) constrains the location and morphology of the headwall scarps of the landslides presented in Moernaut et al. (2009). The side-scan sonar imagery complements it with a higher resolution and shows the exposed stratigraphic layering within the headwall scarps (Fig. 7.16E-F). We select two study areas (area 1 and 2; Fig. 7.12) where slope failures took place. The southern area 2
Figure 7.14: A) Pinger profile on the “unfailed platform” with the projected location of the main CPTU measurements and sediment core VILLAR1. Seismic unit SU-D corresponds to sedimentary units U1 and U2 of Wiemer et al. (submitted), whereas SU-C2 corresponds to U3. B) Sediment core VILLAR1: lithology and grain-size data (1 cm sample every 25 cm) allow differentiation of three lithological units. Peaks in $\gamma$-density (at 0.5 cm resolution) correlate with high-amplitude reflections and allow core-to-seismic correlation. Strong reflector R2 corresponds with tephra layer T2 (“FPT” in Wiemer et al., submitted). C-D) Pinger profile near CPT4 site and core VILLAR8. The upper part of U2 and lower part of U1 is evacuated (“unconformity”) by a sliding event. Note the very similar grain size characteristics between VILLAR8 and VILLAR1 for the different sedimentary units. Data type and resolution as in subfigure A-B. E) Location of the shown seismic profiles and core/CPT sites plotted on the multibeam bathymetry map.
7.2. PRECONDITIONS OF SUBAQUATIC SLOPE FAILURE

Figure 7.15: Brittle deformation (hydrofractures?) in sediment cores, representing the glacio-lacustrine rhythmites of U3. The right picture is taken at an outcrop offshore Lake Rupanco (40.788° S; 72.688° W) which has a similar setting as Lake Villarrica.

contains an unfailed sedimentary platform where we acquired most of our geotechnical data. The sliding plane for large parts of the slope failures is located on top of a distinct strong reflector (R2) which corresponds to a basin-wide fine-sandy tephra layer (T2) examined in Wiemer et al. (submitted) and called the “failure plane tephra” (FPT). Locally, acoustic wipe-out features and reflection discontinuities are observed in the seismic-stratigraphic sequences and interpreted as zones of focused fluid escape (cfr. Chapron et al., 2004; Moernaut et al., 2009). In more extreme cases, the stratigraphic layering is clearly disrupted by fluid escape processes (e.g. “breached horizon” on Fig. 7.16A). Headwall scarps developed either i) above such areas of focused fluid escape, or ii) above distinct slope breaks (downslope steepening) of the failure plane (Fig. 7.16B). Fluid escape features also underlie bathymetric depressions bordered by undisturbed sediment sequences (Fig. 7.16C). In area 1, these depressions have circular or irregular morphology, are 10-80 m wide and up to 1-4 m deep. Their bottom is overall horizontal but with many irregularities. Their association with fluid escape features in the subsurface allows us to interpret these depressions as composite pockmarks (cfr. Hovland et al., 2002). In area 2 it seems that pockmarks of similar dimensions link up with the headwall scarps of the main landslides. This association explains why the headwall scarps have a very irregular trace in map view and seem to locally cut back and forth in the slope sediments (Fig. 7.16D). Several distinct areas of fluid escape within the failed slope (slide scar) were identified on the seismic profiles and by using a slope gradient map of the bathymetric data (Fig. 7.17A-C). Here, fluid escape areas are characterized by discrete sub-horizontal areas with irregular morphology.
Figure 7.16: Characterization of headwall scarps. A) Pinger profile at the “unfailed platform” showing a headwall scarp which is associated to a zone of focused fluid escape. Fluid escape is identified by acoustic wipe-outs and the breaching of reflector R2. The post-landslide stratigraphy is also disturbed indicating that fluid escape (also) took place after the slope failure event. B) Pinger profile where the headwall scarp is located above a slope break in the R2 horizon. C) Pinger profile indicating focused fluid escape below pockmarks (see Fig. 7.17) and at the headwall scarp. D) Location and morphology of headwall scarps visible on the multibeam bathymetry map. Location of profiles A-C is indicated in blue. Areas 1 and 2 are presented in more detail on Fig. 7.17. E-F) Side-scan sonar images of the “fresh” headwall scarps in which the cut-off stratigraphy is exposed.
(green patches on Fig. 7.17C) within a generally eastward dipping slope. Sub-horizontal areas in the shallower unfailed areas correspond to unconformities on the seismic profiles and are probably created by wave/current erosion during storms and non-deposition. Remarkably, most of the fluid escape areas are located on top of morphological highs in SU-B (Fig. 7.17D), the massive unit deposited in close proximity of the glacier front. This association suggests that topographic focusing of fluids in SU-B took place and locally pierced through the overlying sediments. Therefore, SU-B is considered as the source layer or transport pathway of the expelled fluids at the lake floor.

7.2.5.3 In-situ undrained shear strength and focused fluid escape

The pogo-style CPTU transects on and around the unfailed platform reveal spatial differences in undrained shear strength ($s_u$) of the upper 1-2 m of sediments (Fig. 7.18). The post-failure drape on top of the sliding plane yields values of about 60-70% of those on the unfailed platform, although both penetrated the relatively recent, undisturbed U1 sediment drape. CPTU drops within the areas of fluid escape on the failed slope generate $s_u$ values in U1 as low as 10-35% of those of the unfailed platform. To estimate the consolidation state of the sediments, we compare our CPTU data with the general gradient of $s_u$ with overburden depth documented for normally-consolidated organic and inorganic soils ($s_u/\sigma'_{v,0} = 0.2-0.3$) (Karlsson and Viberg, 1967) and pure diatoms ($s_u/\sigma'_{v,0} = 0.4$) (Tanaka et al., 2012). We assume an intermediate gradient of $s_u/\sigma'_{v,0} = 0.3-0.4$ for normally-consolidated U1-U2 diatomaceous ooze which contains a small fraction of terrestrial organic matter, volcanic ash and terrigenous clays/silts.

In this view, it is inferred that the sediments on the unfailed platform at a water depth of about 19-25 m are “apparently overconsolidated” which may be related to currents and wave action during strong storms (Wiemer et al., submitted) or progressive “seismic strengthening” due to recurrent earthquake shaking and intermittent excess pore pressure dissipation (e.g. Locat and Lee, 2002). The sediments on the failed slope—which are located in larger water depths (30-40 m)—show less “apparent overconsolidation”, whereas sediments within the fluid escape areas seem to be normally consolidated (Fig. 7.18). Regardless of the exact cause for the apparent overconsolidation of shallow sediments, disturbance of the sediment structure (i.e. remolding) by focused fluid escape processes apparently seems to reset them into a normal consolidation state expressed in a drastic loss of strength by 40-80% compared to the adjacent areas 50-100 meters away on the failed slope (blue vs. green data on Fig. 7.18). Moreover, the consistent pattern in our measure-
CHAPTER 7. CASE STUDY

Figure 7.17: Pockmarks and other areas of focused fluid escape identified by multibeam bathymetry data and pinger seismic profiles (see legend). A) Isolated pockmarks upslope of a failed slope in study area 1. B) Irregular trace of the headwall scarp due to linkage with pockmarks. The mapped fluid escape features on the seismic profiles match with the fluid escape areas identified on figure C. C) Slope gradient map used for mapping all areas of fluid escape in study area 2 (dotted closed lines). These fluid escape areas are identified by their irregular, overall horizontal topography illustrated by green patches. Green areas in the shallower parts are associated to shallow water erosion and non-deposition (see Fig. 7.20C). D) Paleo-bathymetric map of the top of SU-B based on interpolation of seismic profiles (pinger, sparker). Many areas of fluid escape (red dotted lines) coincide spatially with peaks in SU-B. The “unfailed platform” is located where the upper boundary of SU-B is relatively flat.
ments implicates that the very time-efficient pogo-style CPTU measurements can be used as a tool for detecting areas of focused fluid escape, even when their bathymetric or subsurface expression is below the detection limit of the geophysical survey data.

The deeper reaching CPTU tests at three positions on the unfailed platform show a striking downward decrease in $s_u$ for U2 (Fig. 7.20E) (Wiemer et al., submitted). This is in contrast to the general increase of $s_u$ with depth in shallow marine sediments and cannot be explained by mineralogical changes as the background sediments in U2 have very uniform sedimentological properties along core (Fig. 7.14). Here we add a CPTU test (CPT4 on Fig. 7.20D-E; Table 7.2) obtained in the vicinity of the main headwall scarp and where U1 and the upper part of U2 were evacuated during the slope failure events (“sliding surface U2a” on Fig. 7.18). Since then, about 1.80 m of U1 accumulated on top of the exposed sliding surface. The main part of U2 is still in place and exhibits the lowest $s_u$ values (down to 4 kPa) for this unit of all study sites (Fig. 7.20E). It is also the only site where U2 seems to be significantly underconsolidated. Comparison of cores VILLAR1 and VILLAR8 (Fig. 7.14) suggest that inter-site variability in the U2 sedimentology is minimal and that other factors need to be in play to explain the significant $s_u$ variability in U2 over distances of less than a hundred meter.

### 7.2.5.4 Focused fluid flow and Pore pressure based on dissipation tests

The 13 dissipation tests (Table 7.2; Fig. 7.19) are grouped within the different types of pore pressure response documented by Sully et al. (1999) for overconsolidated fine-grained soils. Type I represents the “standard” dissipation profile with an immediate peak due to insertion of the probe and a monotonic decay of pore pressure with time (Fig. 7.19D). This style was only found in the deep basin at the reference site CPT6, which is outside our main study area (Fig. 7.12). The “unloading” type II was not encountered in our measurements as this style is characteristic for pore pressure filters located on the cone surface (u1 position). Type III (3 tests) and type IV (3 tests) correspond to increasing pore pressures shortly after stopping penetration and subsequent decay like a “standard” curve. The pore pressures immediately at the end of penetration are above (type III) or below the hydrostatic value (type IV). Type V response (6 tests) is characterized by initial negative excess pore pressures which monotonically rise to a “plateau” without recording any distinct peak in pore pressure.

Only two dissipation tests reached a stable value at the end of dissipation. These were taken within sandy-gravelly sediments (e.g. pumice layer; Table
Figure 7.18: A) Map of area2 presenting the different stratigraphic levels used as sliding surface (update of Wiemer et al., submitted), the location of fluid escape areas. The sites were pogo-style CPTU measurements were taken (squares) are color coded as in Figure B. Arrows indicate the directions of mass-transport. B) Undrained shear strength ($s_u$) of the uppermost 2 m of sediments in and around the “unfailed platform”. For illustrative clarity, a running average (19 points) is presented. Measurements in the “unfailed platform” (red) have higher $s_u$ values than those in the post-failure sediments (green). Fluid escape areas (blue) have a significantly lower $s_u$ which plots close to the “normal consolidation line” for diatomaceous sediments (see section 7.2.4.3).

7.2; Fig. 7.14B) which likely have a larger hydraulic conductivity than the silty background sediments of U2, or the fine silty-clayey U3. Determination of the ambient pore pressure of the other tests was made via extrapolation (Fig. 7.19B-C; Table 7.2) which shows that most investigated stratigraphic levels at different sites are characterized by formation overpressures ($\lambda^* > 0$). In some rare cases, the range of extrapolated overpressure values slightly exceeds the hydrostatic vertical effective stress ($\sigma_{v,o}^*$). This can be considered unrealistic and is likely due to inaccuracies related to the used extrapolation techniques. On the main study site CPT1.2, $\lambda^*$ values vary for different levels. The measurement at 7 m subsurface ($\lambda^*: 0.42$) is by far the most reliable as it is obtained from a complete dissipation (type V). The 5.5 m level shows the highest $\lambda^*$ value of 0.78 or more.

For investigating the spatial distribution of excess pore pressure, we compare the $\lambda^*$ values obtained in the lower part of the stratigraphy (lower U2 and within U3) between different CPT sites (Fig. 7.20). Only sites CPT1.1
and 1.2 on the unfailed platform show $\lambda^*$ values below 0.5. An intermediate value of 0.53-0.8 is obtained at CPT 1.3, located on the unfailed platform close to the headwall scarp. Sediments underlying the failed slope sequence (CPT2, 3, 4) and a fluid escape area (CPT5) are characterized by the highest $\lambda^*$ values between 0.7 and 1. The most reliable measurement ($\lambda^*$: 0.88) was from a completed type V profile from CPT site 2, probably located within a tephra layer (i.e. strong continuous reflector) in U3 and underlying the failure scar (Fig. 7.20B, D).

The presence of excess pore pressure decreases the effective stress of the sediment, and consequently leads to reduced undrained shear strength (Dugan and Sheahan, 2012 and references therein). We compared in-situ su values and $\lambda^*$ between sites where we have a good stratigraphic control, i.e. where the seismic-stratigraphic horizons can be determined and where cores are available (Fig. 7.20E). The west-east transect at the unfailed platform clearly shows that the highest $\lambda^*$ (at CPT 1.3 and CPT 4) relate to the lowest values in su in the lower U2 and within U3. At CPT4, the high $\lambda^*$ explain the underconsolidated nature of the lower U2 sediments. At sites where overpressure ratios are not so high (<0.5; CPT1.1 and CPT1.2), the down-sequence negative trend in shear strength is much less pronounced and most of U2 is overconsolidated.

7.2.6 Discussion

7.2.6.1 Earthquake shaking, overpressure and hydrofractures

Strong earthquake shaking can induce significant overpressures in shallow sediments which can take days to years to dissipate (Biscontin et al., 2004). Earthquake-induced overpressures have been documented in the submarine slope sediments near the Sunda Trench (offshore Sumatra) about seven months after the $M_w$ 9.1 Sumatra-Andaman in 2004 (Sultan et al., 2009). Our survey on Lake Villarrica was conducted only 2 years (22 months) after the 27 February 2010 Maule earthquake ($M_w$: 8.8) in Central Chile, the seismic intensity of which was nearly VII at Lake Villarrica and several liquefaction-related phenomena (sand boils, lateral spreading, etc.) were observed at the lake shore (Moernaut et al., submitted). The question arises whether the overpressures encountered in our in-situ measurements are remnants of possible excess pore pressures generated during the 2010 Maule earthquake. Therefore, Wiemer et al. (submitted) used dynamic triaxial tests to decipher the response of the different lithologies in the Villarrica slope sequences when subjected to cyclic loading under an overall compressive regime. In this way, strain and pore pressure build-up during earthquake shaking was simulated.
for near in-situ stress conditions. These tests showed that the diatomaceous ooze layers (U1, U2) and tephra layers at our study sites generate a significant amount of overpressure when subjected to seismic shaking corresponding to a Modified Mercalli Intensity of around VIII. Pore pressure buildup in the quick clays of U3 was much slower. Consequently, it seems unlikely that earthquake shaking on its own induced the large overpressure ratios ($\lambda^*$: 0.7-1.0) encountered in the upper part of U3. Also, it would be hard to explain the differences in $\lambda^*$ in only short distances (100-200 m).

Figure 7.19: Examples of CPTU dissipation tests to estimate the ambient pore pressure state. A, D, E, F) Four of the five types of Sully et al. (1999) were observed in our dataset (see section 7.2.4.4). B-C) Extrapolation via the "inverse-time" technique. We used the $1/t$ and $1/\sqrt{t}$ methods to estimate an upper and lower bound for the ambient pore pressure state (see section 7.2.3.3 and SI-Fig. 7.11). $u_h =$ hydrostatic pressure; $\sigma'_{v,0} =$ vertical total stress. For illustrative purposes, $\sigma'_{v,0}$ is used instead of $\sigma'_{v,0}$. 
7.2.6.2 Origin of overpressure and focused fluid escape

The fractured U3 sediments in the cores, outcrops and the acoustic wipe-outs on the seismic profiles somewhat resemble those found in the fine-grained glacio-lacustrine sediments of Lake Superior (Wattrus et al., 2003), which were interpreted as an immature polygonal fault system. In both lakes, fracturing occurred within the youngest rapidly-deposited glacio-lacustrine unit when the glacier already retreated far back into the catchment, and hence, is not linked to stresses and meltwater pumping associated to advancing glaciers overriding the sedimentary sequences, as is often hypothesized for other areas (e.g. Philipp et al., 2007). Our data does not allow revealing the exact geometry and genesis of the U3 fractures but the documented elevated pore pressures let us speculate that hydraulic fracturing is the most likely candidate to create these. Hydraulic fracturing can take place when the pore pressure in the sediments exceeds the least principal stress (i.e. horizontal) and the tensile strength of the sediments. Effective stress drops significantly and mode I fractures can form under tensile stresses applied on relatively consolidated lake sediments that are stiff enough to undergo brittle failure. This hypothesis fits with the complex network of steeply-oriented fractures observed in the cores and in the outcrop (Fig. 7.15). Possibly, the ultimate pore pressure increase needed for hydrofracturing was induced by earthquake shaking, as was hypothesized for explaining slope failures in the Alpine Lake Lucerne (Stegmann et al., 2007). However, normal fault displacement with vertical throws up to several centimeters to a decimeter suggest that processes other than hydraulic fracturing were also involved in the deformation process, such as e.g. the downslope gravitational stress acting on the 1-5° dipping slopes. In any case, once the fractures are formed, vertical permeability is locally enhanced and excess fluids preferentially reuse this pathway. In this way, fluid venting at the lake floor reoccurs at the same localities, partially inhibiting sedimentation, and creating long-lived pockmark features. Sediments within such fluid escape areas may be less consolidated than the surrounding sediments (Fig. 7.18)

Unloading: Many marine studies hypothesized that pockmark fields within and near a landslide scar are created shortly after slope failure took place (Reiche et al., 2011; Boe et al., 2012). Ebulition of dissolved gases in the pore spaces occurred due to the sudden drop in overburden pressure when unloading the sediments. These gas bubbles can vent to the sea bottom and create pockmarks in an abrupt way. Steepened pore-pressure gradients are created adjacent to scarps due to the evacuation of slope sediments and can induce fluid flow towards these scarps (Orange et al., 1997). However, in
Lake Villarrica, the occurrence of isolated pockmarks (Fig. 7.17A) relatively far away from the modestly-sized landslide scars implies that fluid escape is not triggered by slope failure.

**Sedimentation rate**  : Disequilibrium compaction due to rapid sedimentation (>1mm/yr) of low permeability ($\kappa < 10^{-16}$ m$^2$) material is generally considered as one of the main cause for shallow overpressure (e.g. Ursa Basin: Dugan and Germaine, 2008). At our study site in Lake Villarrica, sedimentation rates in U1-U2 are about 0.4-0.8 mm/yr which is relatively low. Moreover, permeability may be too high in these coarse silty sediments which would prevent the development of overpressures in U1-U2 by sedimentary processes alone. This inference is reinforced by the absence of overpressures at the reference site CPT6 (Fig. 7.12) where sedimentation rates in U1 were about double as high as on the unfailed platform (Heirman, 2011).

**Free gas**  : Elevated pore pressures and non-standard dissipation profiles have often been attributed to the presence of free gas in offshore sediments (Hirst and Richards, 1977; Stegmann et al., 2006). In Lake Villarrica, no indicators of free gas such as acoustic turbidity/blanking or enhanced reflections were found on the pinger and sparker profiles at our main study sites. No evidence of cracks and holes by degassing of the sediment cores were observed. However, the organic matter quantities (6-10 %) for U1 measured in a different area of the lake (Heirman, 2011) are high enough to induce methanogenesis. In glacialicgenic sediments such as in U3 (e.g. Lake Puyehue: Bertrand et al., 2009), lack of organic matter makes this impossible. In combination with the seismic-reflection data, we can conclude that free gas is not present in our studied units and thus cannot explain the documented elevated pore pressures.

In the westernmost, shallower area of the lake, acoustic blanking was detected just below the lake bottom in some pockmarks (SI-Fig. 7.12). We think that shallow biogenic gas may be locally present in these depressions by trapping of coastal organic material due to locally reduced bottom currents in an overall dynamic littoral system (cfr. Bussmann et al., 2011).

**Fluid transfer from below**  : The spatial association of focused fluid escape features with the topographic highs in the underlying SU-B (Fig. 7.16, 8) let us infer that excess pore pressures in U3 mainly originate via fluid transfer from below. In this respect the fine-grained glacio-lacustrine U3
7.2. PRECONDITIONS OF SUBAQUATIC SLOPE FAILURE

Figure 7.20: A-C) Pinger profiles with stratigraphic location of dissipation test. Numbers represent the overpressure ratio ($\lambda^*$). Red: $\lambda^*$ values above 0.5. Bold and underlined: most reliable values based on complete dissipation during testing. D) Location of shown profiles and $\lambda^*$ value for the lower U2 or U3 documented at different sites. E) West-east transect of CPTU sites where both su and $\lambda^*$ could be determined. The down-core negative trend in su for U2 can be explained by the documented $\lambda^*$ values. The higher the $\lambda^*$ value, the lower the corresponding su (see CPT4). The dotted line represents the range in which sediments are considered to be normally consolidated.
sediments (Fig. 7.16, 7.20) can be regarded as a permeability barrier for vertical fluid flow. Indeed, this unit contains an elevated clay content of about 35% (Fig. 7.14) which should result in relatively low permeability. The permeability just above the boundary of U3 and SU-B may have been further reduced in the initial phase of U3 deposition due to downward fluid movement from U3 into the coarse and permeable sediments of SU-B, resulting in enhanced consolidation of the lower U3. Such localized permeability reduction may have enhanced the capping efficiency of U3 (process described in Henriet et al., 1989).

SU-C2 (U3) is thinner on top of the topographic highs and above the steepest slopes in SU-B (Fig. 7.16, 7.20). This results in a reduced vertical overburden stress and a steeper pore pressure gradient which allows fluid focusing towards the peaks of SU-B. Additionally, horizontal compressive stresses are lowest above these peaks due to the radial pattern of the downslope component of gravitational stress. Consequently, the combination of enhanced overpressures due to flow focusing and enhanced tensile stress above the SU-B peaks make these the preferential location for hydrofracturing to take place. These hydrofractures further increased the local vertical permeability and allowed fluid escape at the lake floor.

Pockmarks and other fluid escape structures in Lake Villarrica were only found where SU-B is present in the subsurface (Fig. 7.13, 7.17). This unit consists of a heterogeneous assemblage of rapidly-deposited coarse sediments deposited at the glacier edge. At continental margins, it is shown that glacigenic sediments (till and glacigenic debris flows) are typically characterized by a low permeability and compressibility compared to hemipelagic sediments, and act as relatively impermeable fluid flow barriers. This can create moderate overpressures in shallow sediments and high overpressure ratios (Llopart et al., 2014). However, in ice-contact and ice-proximal sedimentary units, meltwater pumping can actually impede the consolidation process (Boulton and Dobbie, 1993) and can create hydrofracture networks filled with coarse sediments (Phillips et al., 2007) which remain more permeable fluid flow pathways during post-glacial lake conditions. The absence of direct sampling of SU-B makes it impossible to assess its permeability and how this is distributed spatially. Nevertheless, our combined datasets let us assume that permeability within this heterogeneous unit should be somewhat higher than in the overlying SU-C2.

Temperate warm-based glaciers—such as in Northern Patagonia during the Last Glacial (Denton et al., 1999)—erode their beds more effectively and are able to rapidly dump a vast amount of coarse material at their snouts. Consequently, their associated glacial landforms can be very extensive. In Lake Villarrica, glacial loading and rapid dumping of the glacigenic SU-B on
7.2. PRECONDITIONS OF SUBAQUATIC SLOPE FAILURE

top of the thick glacio-lacustrine SU-A may have interrupted normal sediment consolidation, created overpressures in SU-A and/or SU-B in the SW part of the lake (Fig. 7.21). Pore pressure dissipation was partly impeded by the low-permeability SU-C2, and resulted in overpressured fluids, flow focusing and pockmark development.

7.2.6.3 Influence of overpressure and focused fluid flow on slope stability

For spatially uniform sediment sequences in lacustrine and marine settings, headwall scarps mostly develop above downslope-steepening slope breaks in the stratigraphic level acting as sliding surface (e.g. Strozyk et al., 2010; Moernaut and De Batist, 2011). This is simply due to a lower downslope gravitational stress upslope of the slope break, resulting in more stable sediment sequences. Headwall scarps associated to slope breaks were also found in Lake Villarrica (Fig. 7.16B) on those places where there are no fluid escape features in the underlying sequences. In map view these scarps are relatively linear and smooth, as they are controlled by the geometry of the stratigraphic horizons (Fig. 7.16D, 7.17A). Irregular segments of the headwall scarp correlate with areas of focused fluid escape (Fig. 7.17B-C) and the underlying sediments show characteristic wipe-outs and breached horizons on the seismic profiles (Fig. 7.16A, 7.16C, 7.17C). We cannot determine if pockmarks were already present before slope failure took place or, alternatively, if the headwall scarp did cut back into areas weakened by focused fluid escape (cfr. Lastras et al., 2004). In any case, our data let us suggest that focused fluid flow driven by overpressure gradients influences the location of slope failure, the morphology of the headwall scarp, and consequently the total volume that is included in the slope failures.

Effective stress and hence undrained shear strength (su) decreases when overpressure ratio ($\lambda^*$) increases. This relationship is clearly evidenced in the W-E transect at the unfailed platform (Fig. 7.20). Wiemer et al. (submitted) quantified the pseudo-static factor of safety (FoS) against slope failure for sites CPT1.1, 1.2 and 1.3 when subjected to seismic intensities of about VI-VIII, and concluded that the FoS in U2 was about two times higher in CPT1.2 than in CPT1.3. As su is one of the key parameters governing slope stability, we suggest that the lower $\lambda^*$ values (< 0.5) in CPT1.1 and CPT1.2 values prevented this area from failure during the landslide events which evacuated the areas north, east and south of the platform. CPT1.3, located closely to the headwall scarp must have been close to failure during that paleo-earthquake. These geotechnical considerations agree perfectly with the absence of fluid escape indicators in the subsurface of the “unfailed platform”
(except close to CPT1.3, Fig. 7.20A) whereas these are abundant in the surrounding regions (Fig. 7.16A, 7.17B-C).

At CPT site 1.2, high overpressure ratios ($\lambda^*$: 0.78+) were only found about 0.5 m above the failure plane tephra (FPT = T2) within U2, whereas the other stratigraphic levels at this site have considerable lower $\lambda^*$ values (Fig. 7.20A). Remarkably, the boundary of the FPT and the overlying U2 sediments formed the main sliding plane for slope failures in this area. Wiemer et al. (submitted) hypothesized that this sliding plane developed due to earthquake-induced pore pressure increase in the FPT and -due to its high permeability- a rapid transfer of pore pressure to the overlying U2 sediments. This sudden pore pressure rise may have significantly weakened the U2 sediments, allowing large-scale landsliding. Accordingly, the elevated in-situ pore pressure values measured about 0.5m above the FPT-U2 contact may be a long-lived relict of this process.

7.2.6.4 General implications

Fluid escape features within glacio-lacustrine sediments located above glacigenic “ice-contact” sediments were also documented in Lake Puyehue, Chile (Heirman et al., 2011), Flathead Lake, Montana (Hofmann et al., 2006) and Mazinaw Lake, Ontario (Eyles et al., 2003). Different speculations were put forward ranging from glacier-induced overcompaction to delayed melting of trapped ice (kettle holes). Similar features can also be seen on published seismic profiles from other glacigenic lakes (e.g. Figure 5 in Pinson et al., 2012), but they are never studied in full detail. Therefore, the exact formation mechanism is not known for this commonly-encountered phenomenon.

Lake Villarrica's depositional sequence comprising fine-grained glacio-lacustrine deposits overlying coarser and more heterogeneous glacier-proximal deposits is analogue to many glacigenic basins worldwide (see 4.1).

We suggest that the postulated mechanism for creation of excess pore pressure may be a general process at formerly glaciated margins and basins, and may have played a key role in the creation of subaquatic landslides in fjords (e.g. Hjelstuen et al., 2009) and at passive margins, e.g. the Storegga Slide offshore Norway. In the latter, the alternation of rapidly-deposited glacial sediments (tills, debris flows) with fine-grained hemipelagic marine deposits resulted in high excess pore pressure ratios (Kvalstad et al., 2005) which significantly contributed to the initiation and development of this giant landslide on a gentle slope.
7.2. CONCLUSIONS

We characterize overpressure, fluid escape features and slope (failure) morphology in a glacigenic lake in Chile where large subaquatic landslides took place. High-resolution seismic profiles and multibeam bathymetry show that headwall scarps and areas of focused fluid escape (e.g. pockmarks) locally link up. Focused fluid escape locally weakened sedimentary slope sections, which contributed to the development of large-scale landsliding and irregular headscarp morphology. Free-fall penetrometer measurements were used to calculate the in-situ undrained shear strength and the pore water pressure of the sediments. Extrapolation techniques were tested and used to estimate the ambient overpressure ratio and let us conclude that most sites were characterized by high overpressures ($\lambda^* > 0.5$). $\lambda^*$ values of 0.3-0.4 are present in an unfailed platform where no fluid escape features were found. We discovered a striking correlation of highest $\lambda^*$ values with lowest undrained shear strength and underconsolidated sediments. Overpressures are interpreted to originate from coarse glacier-proximal depositional units underlying fine-grained glacio-lacustrine sediments which may act like a permeability barrier. This inference is supported by the spatial correlation of fluid escape areas with topographic highs in the glacier-proximal units, and the careful evaluation of other possible mechanisms for overpressure such as earthquake shaking, free gas, sediment unloading and rapid sedimentation. Earthquake shaking may have created an ultimate pore pressure pulse needed for hydrofracturing the sediments and for landsliding to take place above a sandy tephra layer as a predefined sliding surface.

This detailed case study is one of the few studies on an offshore sediment sequence where both shear strength and pore pressure were measured in-situ in an accurate and independent manner, and where the theoretical postulation of the influence of excess pore pressure on sediment shear strength is verified by empirical in-situ data. Moreover, it reveals a significant variability in overpressure ratio and slope stability over very short distances, and highlights the necessity for complementary geophysical and in-situ geotechnical datasets of very high spatial resolution to successfully understand subaquatic slope failure processes.
Figure 7.21: Conceptual model for the deposition of the different seismic units since the start of deglaciation and the creation and focusing of excess fluids and landslide development. The different depositional stages are presented in section 7.2.4.1 and the overpressure and fluid flow is discussed in section 7.2.5.2. Stages A-D took place from 17.5 to 12.7 ka BP and sedimentary processes are tightly related to the presence of a glacier in the basin (A-C) or in the catchment (D). Stages E represents the hemipelagic open lacustrine sedimentation during the Holocene, and stage F the slope failure event discussed in this article.
7.2. PRECONDITIONS OF SUBAQUATIC SLOPE FAILURE

7.2.8 Acknowledgements

We thank the Chilean survey company BENTOS for the cost-effective logistical and technical support of the multibeam survey on Lake Villarrica. We further acknowledge the support of Francisco Martin, capitán Juan, Koen De Rycker, Alejandro Peña, Manuel Novoa, Philipp Kempf, François Charlet and Robert Brümmer during the geophysical and geotechnical field campaigns. IHS Kingdom and OmniSTAR are acknowledged for their educational grant programs providing seismic interpretation software and high-accuracy satellite positioning data, respectively. This research is funded by the Swiss National Science Foundation (grant 133481), a research grant of the Research Foundation Flanders (FWO-Vlaanderen), and the MARUM - Center for Marine Environmental Sciences via the Deutsche Forschungsgemeinschaft (Bonn, Germany).

7.2.9 References


CHAPTER 7. CASE STUDY


7.2. PRECONDITIONS OF SUBAQUATIC SLOPE FAILURE

RG3001.


7.2. PRECONDITIONS OF SUBAQUATIC SLOPE FAILURE


Ndìaye, M., Clerc, N., Gorin, G., Girardclos, S., Fiore, J., in press. Lake Neuchâtel (Switzerland) seismic stratigraphic record points to the simultaneous Würmian deglaciation of the Rhône Glacier and Jura Ice Cap. Quaternary Science Reviews, in press.


SHOA (Servicio Hidrográfico y Oceanográfico de la Armada de Chile) (1987), Lago Villarrica, scale 1:40000, Santiago, Chile.


Chapter 8

Conclusion

This dissertation aimed at the i) geotechnical characterization and shear behavior definition of fresh volcanic material in comparison to common sands/silts, ii) effect of authigenic clay mineral formation on the shear behavior of volcanic material, and iii) in situ verification of results gained through laboratory experimentation within a case study on a subaqueous slide that involved volcanic material. The following conclusions could be drawn from laboratory experimentation: SEM imaging and consolidation prior to shearing revealed that volcanic fall-out ash particles differ in i) shape ii) surface texture iii) strength and iv) porosity from common sands. Ash particles can be i) angular, ii) rough iii) highly porous and iv) soft and crushable. Nevertheless, a differentiation needs to be made between volcanic particles that combine angularity-roughness-softness and particles that combine angularity-roughness-hardness. When it comes to shearing of ashes angularity and roughness have a strengthening effect while crushability and porosity may have a weakening effect. In a sense angularity and roughness are competing crushability and porosity. Depending on the drainage conditions and shear stress amplitudes one or the other pair of factors dominates the shear behavior and leads to either strengthening or weakening relative to common sands. In drained monotonic shearing at low effective normal stress (< 400 kPa) angularity and roughness lead to strong particle intercalation which entails:

- dilatational behavior even in a loose state
- high apparent critical friction coefficients
- high apparent cohesion
- high internal friction angles (Fig.8.1a)
The amplitude of each of these effects increases with grain size. An increase in normal stress is accompanied by enhanced and progressive crushing of soft particles which entails

- highly contractive behavior during compaction and shearing
- higher plasticity
- a decrease in apparent friction coefficients to values observed in plagioclase or feldspar dominated sands ($\mu \sim 0.7$)
- the necessity of large-strain experiments (ring shear) in order to reach critical state conditions that were not reached in direct shear testing on soft particles.

However, crushability plays a secondary role in drained shearing because particle internal pore water that may suddenly be released by crushing during drained shearing, does not lead to pore pressure build up and inherent effective stress reduction. Particle intercalation reigns over crushability at low effective stress and drained shearing conditions (Fig.8.1a). An increase in clay mineral content due to authigenic clay mineral formation becomes relevant to shear resistance at higher clay mineral contents in volcanic ash than in common sands. It is differentiated between an efficient and inefficient clay matrix for the reduction of frictional resistance. Clay minerals located in the internal pore space or deeply carved hemispherical cavities are inefficient for lubrication of inter-particle contacts of the stiff particle phase. Strong particle intercalation and its influence is maintained even at elevated clay mineral content and leads to high frictional resistance under static, drained conditions (Fig.8.1a). At $\sim 50\%$ clay mineral content the effect of particle intercalation is annulled because inter-particle contacts are reduced to a minimum.

In undrained monotonic shearing intercalation of particles still leads to an increase in undrained shear strength ($s_u$), however only if particles are hard-grained. Crushable particles with internal voids and high porosity (pumice sands and glass shards) release excess pore pressure by crushing, which reduces effective stresses and inherently $s_u$. Hence, highly angular particles that are easily crushable present lower undrained shear strength than common sands even at low effective mean stress (Fig.8.1b).

In undrained cyclic shearing the amplitude of effective mean stress and cyclic shear stress is relevant for an overall strengthening by angularity and
Figure 8.1: a) Schematic illustration of the coefficient of apparent friction as a function of effective normal stress ($\sigma'_n$) in common sand-clay mixtures and volcanic ash-clay mixtures. Schematic illustration of b) the undrained shear strength of volcanic ash ($s_u$) and c) the Cyclic Resistance Ratio (CRR), both as a function of particle shape complexity and crushability in reference to well rounded quartz sand.
roughness or weakening due to crushability like in undrained monotonic shearing. If effective mean stress is low (100 kPa) and cyclic shear stresses are of medium amplitudes (CSR = 0.25), crushing is not prominent enough to lead to a inherent excess pore pressure build-up. Angularity and roughness under these conditions have a strengthening effect similar to drained shearing. Nevertheless, volcanic sands are susceptible to liquefaction and may show liquefaction failure during earthquake shaking. Flow liquefaction requires higher static shear stresses in volcanic sands than in more common sands, which increases the probability of cyclic mobility in volcanic sands (Fig. 8.1c). Failure susceptibility has been shown to increase with decreasing grain size, i.e. a silt-sized volcanic ash is more susceptible to failure than a volcanic sand.

These laboratory results imply a lower failure probability in volcanic sands compared to more common sands. In the field cyclic mobility failure requires higher earthquake intensities and/or earthquake duration while flow liquefaction failure requires higher static shear stresses, i.e. steeper slope angles compared to common sands. Furthermore, authigenic clay mineral formation seems to be irrelevant for slope stability because the clay mineral content reached in depths relevant for submarine slope failure is insufficient to overcome strengthening by particle intercalation.

The overall findings from the generic work in this thesis were tested in a case study in Lake Villarrica, South-Central Chile. Results are in line with generic studies regarding shape, surface texture, crushability, efficient clay content, and drained monotonic and undrained cyclic strength of volcanic material. The presence of a volcanic fall-out ash layer is not per se sufficient to initiate liquefaction and sliding as a consequence of cyclic shaking. Preconditions and factors such as volcanic ash grain-size, earthquake intensity, fluid flow, excess pore pressure and effective overburden stress were shown to interplay and define the failure probability at depth. Even the largest ever instrumentally recorded earthquake, the Great Chilean Earthquake from 1960 (Mw 9.5, Intensity VII 1/2 in Villarrica) the 2010 Chile earthquake (Mw 8.8, Intensity VI 1/2 in Villarrica) could not fail the strong ashes. However, volcanic ash layers may building up excess pore pressure and inherently reduce effective stress conditions at the interface to overlying sediment, or even liquefy. Nevertheless, there is no unambiguous geotechnical evidence for volcanic ashes to act preferentially as glide planes in submarine landsliding along active margins.
Chapter 9

Ongoing study - towards a refined understanding of the shear behavior of diatomaceous oozes

It was beyond the scope of this dissertation on shear behavior of volcanic material to undertake a comprehensive geotechnical investigation of the shear behavior of diatomaceous oozes. Diatomaceous oozes often occur in equivalent geological setting as volcanic ashes because of the abundance of SiO$_2$ in solution in the water. Volcanic ashes may fuels the productivity of diatoms [Hamme et al., 2010; Verdugo, 2008], which may play a major role in submarine landslide initiation processes [Volpi et al., 2003]. Diatoms are hollow siliceous particles with particle internal pore water that may be set free upon breakage and lead to excess pore pressure. Diatomaceous oozes are known to be characterized by atypical geotechnical properties as for instance high water content, low bulk density, high frictional resistivity, high plasticity and high compressibility. It has been shown, that small amounts of diatoms (~10%) in a marine hemi-pelagic sediment can lead to significant deviations from geotechnical properties of common hemipelagic sediments [Day, 1995; Volpi et al., 2003]. The monotonic and cyclic shear behavior of pure diatomaceous oozes and mixtures of diatoms with hemi-pelagic sediments remains insufficiently investigated to estimate the role that these materials may play in submarine landslide initiation processes.

The focus of the case study (Chapter 7.1) was set on volcanic ashes; however the investigated sedimentary sequence is dominated by diatomaceous oozes which have been tested in drained direct shear and undrained, cyclic triaxial shear experiments. Furthermore, a pure diatomaceous ooze from the for arc region of the South-Sandwich Islands (Polarstern Expedition XXIX) was tested for monotonic undrained and drained shear strength and
Figure 9.1: blue frame: Unit 1 sediment, top and bottom; green frame: Unit 2 sediment, top and bottom; see Chapter 7.1
cyclic undrained shear strength at the Marine Geotechnology laboratory of MARUM within a Master thesis conducted by Ricarda Dziadek [Dziadek, 2014].

The combination of both unpublished data sets provides a base line for a detailed generic study on the shear behavior of diatomaceous oozes mixed with hemipelagic sediment in different ratios. Preliminary results indicate a similarity to soft-grained volcanic ashes in drained direct shear testing. Figure 9.1 presents the stress-strain and vertical displacement-strain data of the sediment samples tested in the course of the case study (Chapter 7.1) at 100, 200 and 300 kPa effective normal stress. From top to bottom of Figure 9.1 the ratio of diatoms to detrital particles increases (Fig. 7.3). It is shown that the tested diatomaceous oozes reach a plateau in shear strength after ~5 mm shear path which indicates high plasticity. These samples are insensitive, expressed by the fact that there is little or no difference between peak and ’critical’ shear resistance (or resistance at 10-11 mm shear path). The critical state is actually never reached in non of the tested samples. Each sample presents significant compaction during shearing which might indicates excessive pore pressure build-up and effective stress reduction under undrained conditions.

However, undrained isotropic cyclic loading experiments suggests a stiffness increase with decreasing detrital component. Figure 9.2 shows two isotropic, undrained, cyclic triaxial shear experiments on a sample from Unit 1 (Top) and Unit 2 (Bottom), each consolidated to $\sigma'_c = 100$ kPa and subjected to cyclic shear stresses of 25 kPa (CSR = 0.25). The 1st and 12th are shown in detail in the stress-strain and stress-pore pressure ratio space. It is shown that both samples fail in dilation. The unloading phase has more effect on pore pressure and strain accumulation than the loading phase. The 12th cycle in U1 is purely dilatational and reaches ~4% axial strain. U2 at that point shows still near symmetric strain oscillating from -0.25-0.75% axial strain. The PPR is ~0.22 at its maximum in U2. U1 reaches 0.475 at the maximum of the 12th cycle.

One possible explanation for these observations could be, that a detrital component leads to a less imbricated framework of the diatoms with higher inter-particle void ratio and an enhanced potential for pore pressure build-up and strain accumulation. Nevertheless, a more detailed analysis and more data are required before expressing final conclusions on the shear behavior of diatom-silt or diatom-clay mixtures.
Figure 9.2: Two cyclic, isotropic, undrained triaxial tests on unit 1 and unit 2 sediment from Lake Villarrica (South-Central, Chile) (see chapter 7.1) presented in the stress-strain and stress-ppr space. The 1\textsuperscript{st} and 12\textsuperscript{th} cycle are shown in detail.
Chapter 10

References

(cited outside the manuscripts)


Biscontin, G., J. M. Pestana, and F. Nadim (2004), Seismic triggering of submarine slides in soft cohesive soil deposits, Mar Geol, 203(3-4), 341-354.


Casagrande, A. (1936), Characteristics of cohesionless soils affecting the stability of slopes and earth fills, Journal of the Boston Society of Civil Engi-
neers, 257–276.


Saharan debris flow: an insight into the mechanics of long runout submarine debris flows, Sedimentology, 46(2), 317-335.


Kramer, S. L. (1996), Geotechnical earthquake engineering, Prentice Hall,


Leroueil, S., and D. Hight (2003), Behaviour and properties of natural soils and soft rocks, Characterisation and engineering properties of natural soils, 1, 29-254.


Masson, D. (1996), Catastrophic collapse of the volcanic island of Hierro
15 ka ago and the history of landslides in the Canary Islands, Geology, 24(3), 231-234.


Petley, D. N., T. Higuchi, D. J. Petley, M. H. Bulmer, and J. Carey (2005),
CHAPTER 10. REFERENCES

Development of progressive landslide failure in cohesive materials, Geology, 33(3), 201-204.


Wood, D. M. (1985), Some fall-cone tests, Geotechnique, 38, 64-68.


Chapter 11

Acknowledgements

Ladies and Gentleman,

foremost, I would like to express my sincere gratitude to Achim Kopf for excellent guidance, total freedom of action and outstanding support, even around the clock and globe if necessary, always coupled with exemplary fun, irony and undogmatic thinking.

Furthermore, I would like to thank Achim Kopf and Tobias Mörz for reviewing this thesis. Tobias Mörz is moreover gratefully thanked for supporting my work by providing laboratory equipment with trust and without hesitation.

Michael Strasser, is thanked for fantastic collaboration.

Engineers Mathias Lange, Christian Zöllner, Marc Huhndorf, Wolfgang Schunn and Koen De Rycker are gratefully thanked for their assistance, implementations of ideas, help and lessons all around engineering issues.

Daniel Otto is thanked for patient lab. assistance, introduction to the dynamic triaxial testing device and exchange of ideas for triaxial testing.

My sincere thanks is also owed to Alois Steiner for numerous CPT instructions.

Andre Hüpers and Matt Ikari, thank you very much for excellent scientific advice and not to forget, outstanding jokes.

Stefan Kreiter, a great portion of my geotechnical understanding was billed
by discussion with you. Thank you very much for sharing knowledge.

Jasper Moernaut, Sylvia Stegmann and Nina Stark, apart from all the support and advice, I’d like to sincerely thank you for the exchange of thoughts that go beyond science.

Timo Fleischmann and Mary Belke Brea are thanked for helping out whenever needed and for numerous grain-size analyses, respectively.

I’d also like to thank Petra Renken for helping with administrative issues.

It’s been a pleasure.
Chapter 12
Appendix

12.1 Supplementary data

The following chapter presents supplementary data to technical or scientific issues tackled in this thesis. This chapter is organized in sections related to former Chapters/sections. Supplementary data is not discussed or interpreted in detail as it is data that reinforces interpretations and conclusions already made in respective sections. Nevertheless, this data is shown because it is considered relevant to this dissertation although it may be partially irrelevant for publication purposes.

12.1.1 # to Chapter 4
12.1.1.1 Triaxial cell-internal vs. external force sensor

The force and strain controlled triaxial experiments conducted in the course of this thesis were controlled and/or monitored via a cell-external force transducer (Chapter 4). Only latest experiments could be conducted with a cell bearing an internal force sensor which however, only records data passively. The following subchapter shows to what extent the data collected by the internal force sensor differs from the data collected by the external force sensor and allows estimation on general data quality. Note that each test has been conducted in the identical triaxial cell using identical sealing oil and only slightly deviating sample heights which result in slightly different piston length outside the triaxial cell.

Monotonic loading: Figure 12.1 shows two effective stress paths of each material tested under monotonic undrained conditions, namely DS, PS, GS

207
and ChiT sand (see Chapter 6.1). One effective stress path of each material results from data collected by the internal force sensor; the other results from data collected by the external force sensor. It is shown that the interpretation of data collected by the external sensor always leads to an overestimation of the quasi-steady-state shear strength. The inclination of the failure line is not significantly affected. Only in the case of the DS, which is the test that shows the highest deviation between internal and external sensor, the failure line is $\sim 5$ kPa higher than the actual failure line. Furthermore, external force sensor data interpretation leads to an overestimation of the dilatational behavior. Maximum deviations in $q_{int}$ and $q_{ext}$ range from 10-25 kPa (5-12.5 kPa $q/2$). Deviations are thought to be related to frictional forces acting on the cell piston. The magnitude of frictional resistance may change depending on how well the triaxial cell is centered below the hydraulic piston. This data illustrates the importance of a cell internal force transducer for triaxial testing.

**Figure 12.1:** Effective stress paths of DS, PS, GS and Chit sand determined via internal and external force sensor.
12.1. SUPPLEMENTARY DATA

Cyclic loading: A cyclic triaxial test for internal vs. external force sensor comparison was conducted on a very dense sand under drained conditions in order to prevent early sediment failure. Figure 12.2a shows the deviatoric stress as a function of number of cycles resulting from the internal and external force sensor. The deviatoric stress was increased in steps of 5 kPa from 10 kPa to 60 kPa cyclic load (Fig. 12.2a) in order to cover the same stress range as applied in cyclic triaxial testing in the course of this thesis. For a

![Figure 12.2](image-url)
better illustration the loading and unloading peaks collected by the internal and external force sensor are presented in figure 12.2b. Figure 12.2c shows the difference of loading and unloading peaks between the external and the internal force sensor, respectively. Peaks in loading differ by $\sim 5$ kPa from 15-45 kPa cyclic load (external sensor). The difference in peak-unloading values increases with increasing applied cyclic load from $\sim 2.5$ kPa to $\sim 10$ kPa. At 45 kPa cyclic load the difference between internal and external force sensor unloading-peaks seems to be steady at $\sim 8$ -10 kPa. Peaks towards zero difference in internal and external deviator stress are related to a slight delay in force transmission. This data again shows how important it is to control and measure the major principle stress via an internal force sensor. However, this data also signifies, that a deviation of $q$ by $\pm 5$ kPa leads to a reduction of the CSR by 0.025 at $\sigma'_c = 100$ kPa, which is acceptable. The latest CSR data presented in Chapter 6.1 has been corrected for friction by reducing the CSR that was determined via the external force sensor by 0.025. Nevertheless, it remains unclear whether this is an optimal result due to a well centered triaxial cell below the hydraulic piston (see above), or if higher differences between internal and external sensor can be expected in case the cell is not well centered in the DTTD.

A modification of the DTTD basement with less degree of freedom to position the triaxial cell has already been proposed and will be undertaken as soon as possible. Furthermore this problem should be solved by conducting force controlled triaxial experiments via cell internal force sensors.

### 12.1.1.2 Direct shear force sensor calibration

The following plots (Fig. 12.3) show the calibration data the vertical and horizontal force placed in Giesa 1, Giesa 2 and Giesa 3. This data may be used as reference for further calibrations. Furthermore, it will be possible in the future to investigate whether the force transducers change their properties through time and gain in plastic strain deformation due to frequent usage close to limit capacities.
12.1. SUPPLEMENTARY DATA

Figure 12.3: Linear relations between voltage (mV) and Newton (kN) of all force transducers in the direct shear devices Giesa 1 (G1), Giesa 2 (G2) and Giesa 2 (G2)

12.1.2 # To section 6.1

Figure 12.4 presents grain size distribution curves of three pumice sands (PS I, PS II and PS III). These sands were tested for their drained monotonic frictional resistance (Fig. 12.4 b and 12.4 c) as described in Chapter 6.1. For reasons of clarity this data was not presented before. Nevertheless, the coefficient of friction data of the large-grained pumice sands fully supports the observations and conclusions made in Chapter 6.1.
Figure 12.4: a) Cumulative grainsize distribution curves of three pumice sands, b) peak apparent friction as a function of effective normal stress c) critical apparent friction as a function of effective normal stress

12.1.3 #To section 6.2

The coefficients of peak apparent friction at 300 kPa effective normal stress of fresh and altered volcanic material are plotted as a function of clay content in Figure 12.5, analogue to the coefficients of critical apparent friction presented in chapter 6.2. It can be seen that peak friction decreases with increasing clay mineral content in each tested material. Sample materials angular amorphous silica (pumice or glass shard) present the highest coefficients of peak friction throughout the range of clay mineral content. In contrast to critical apparent friction, peak friction seems to be grain size dependant. The M4-clay mixture is characterized by lower peak friction at elevated clay content (≤ 20%) than the quartz sand (DS) with larger grains and low angularity. Peak strength increases with grain size and angularity. At ∼ 40% clay mineral content the Chit sand and DS-clay mixtures present equivalent peak shear resistance that is significantly higher than that of the M4-clay mixture at 40% clay mineral content. Peak friction of the Chit sand, DS and PS (sands with equivalent grain size distribution) present equivalent peak shear...
12.1. SUPPLEMENTARY DATA

resistance at $\sim 55\%$ which is also the clay content at which these sands reach equivalent critical shear resistance (see chapter 6.2). However, this data combined with the data on critical apparent friction indicates increased sensitivity in samples with larger grains. Figure 12.6 shows shear stress and vertical displacement as a function of shear path of tested materials at 0, 60 and 100% clay content. Details on tests conducted at 0% clay content are given in chapter 6.1. However, with increasing clay content the sand-clay mixtures successively adapt and their shear behavior and contractive behavior match increasingly. It is shown that at 55-60% clay mineral content the sands DS, PS and Chit sand have almost identical stress-strain curves and vertical displacement curves which stands in contrast to the behavior at lower clay contents. Furthermore the PS, DS and Chit sand mixtures at $\sim 55\%$ clay content present significant strain softening which is thought to be related to successive arrangement of clay minerals parallel to the shear surface. At $\sim 100\%$ clay mineral content the smectite dominated clay is characterized by lower frictional resistance than the clay bearing illite and smectite. No peak is to be observed in the smectite dominated clay.

12.1.4 #To chapter 7.1

Figure 12.7a and 12.7b presents the isotropic undrained cyclic triaxial test data conducted on the different units and the failure plane tephra (FPT) of the sediment sequence cored in Lake Villarrica at coring and CPT site 1.2. Figure 12.7a shows the cyclic stress ratio (CSR), vertical axial strain ($\epsilon_v$) and the pore pressure ratio (PPR) as a function of loading cycles at a CSR of 0.25. It is shown that each sample representative for each unite fails in dilation. Only the FPT reaches failure due liquefaction after 6 cycles. Unit 1, Unit 2 and Unit 3 sediment build up significant excess pore pressure, but reach 5%
axial strain before reaching the liquefaction criteria (PPR = 0.9). Unit 1 fails after $\sim 16$ cycles, Unit 2 sediment after 48 cycles and unit 3 sediment fails after $\sim 190$ cycles. Hence, the resistivity increases with increasing depth of the units in the sediment sequence, whereby the failure plane tephra (FPT) presents the lowest resistivity to cyclic loading. This observation is confirmed for cyclic stress ratios ranging from $\sim 0.2-0.325$ (Fig. 12.7b).

The data presented in the manuscript of chapter 7.1 cover only the anisotropic triaxial test data conducted on identical units and the FPT. That data stand in total contrast to the isotropic cyclic triaxial data presented here. It must be noted, that isotropic triaxial testing does not take into account the lateral earth pressure coefficient ($K_0$) (see chapter 4) which is significant in sediment with high frictional resistance (see chapter 4). Moreover, initial isotropic cyclic loading conditions leads to enhanced pore pressure build up. Both
factors provide the reason for interpretation of anisotropic cyclic loading data only in chapter 7.1.

**Figure 12.7:** a) Cyclic stress ratio (CSR), vertical axial strain ($\epsilon_v$) and pore pressure ratio (PPR) as a function of loading cycles at a CSR = 0.25, b) CSR at failure vs. number of cycles at failure for all samples tested under isotropic undrained cyclic loading conditions of unit 1, 2, 3 and the failure plane tephra (FPT) in lake Villarrica

### 12.2 List of Cruises and Field Campaigns

This dissertation involved the following Cruises and field campaigns:
<table>
<thead>
<tr>
<th>Date</th>
<th>Location</th>
<th>Vessel/Platform</th>
<th>Expedition</th>
<th>Chief scientist</th>
<th>Methods</th>
</tr>
</thead>
<tbody>
<tr>
<td>March 2011</td>
<td>Mediterranean Ridge Accretionary Complex</td>
<td>R/V Poseidon</td>
<td>P 410</td>
<td>A. Kopf (MARUM)</td>
<td>CPT, vane shear, fall cone penetrometer</td>
</tr>
<tr>
<td>Dec. -Jan.11</td>
<td>Lake Villarrica (South-central Chile)</td>
<td>Platform from University of Ghent</td>
<td>-</td>
<td>J. Moernaut (ETH Zürich, Univ. Ghent)</td>
<td>CPT, Piston-coring</td>
</tr>
<tr>
<td>March 2012</td>
<td>Mediterranean Ridge Accretionary Complex, Nice Slope (France)</td>
<td>R/V Poseidon</td>
<td>P 429</td>
<td>A. Kopf (MARUM)</td>
<td>CPT, vane shear, fall cone penetrometer</td>
</tr>
<tr>
<td>May 2013</td>
<td>Lake Neuchatel (Switzerland)</td>
<td>Platform from ETH Zürich</td>
<td>-</td>
<td>M. Strasser (ETH Zürich)</td>
<td>CPT</td>
</tr>
</tbody>
</table>

**Table 12.1:** List of cruises and field campaigns
Chapter 13

Erklärung

Hiermit versichere ich, dass ich

1. die Arbeit ohne unerlaubte fremde Hilfe angefertigt habe.

2. keine anderen als die von mir angegebenen Quellen und Hilfsmittel verwendet habe.

3. die den benutzten Werken wörtlich oder inhaltlich entnommenen Stellen als solche kenntlich gemacht habe.

Bremen, den 25.02.2014

(Gauvain Wiemer)