Sub-Seaﬂoor Characterization and
Stability of Submarine Slope Sediments using
Dynamic and Static Piezocone Penetrometers

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Abstract

The exploitation of natural resources (e.g. oil, gas and gas hydrate), development of cable and pipeline routes, land reclamation activities in coastal and nearshore environments, and protection of coastal communities require a sound knowledge of the geotechnical properties and characteristics of sub-seafloor soils, especially when dealing with the stability of submarine slopes. In situ dynamic and static piezocone penetration tests (CPTU) are powerful cost- and time-efficient techniques for measuring geotechnical and stratigraphical properties. Such in situ tests directly determine the physical and geotechnical properties from measured CPTU parameters (cone penetration resistance, sleeve friction and pore pressure) without the use of time- and cost-intensive laboratory experiments, such as standard vane shear (v-s), fall cone penetration (fc) and direct simple shear (DSS) experiments.

The penetration process of a dynamic-CPTU device results in a non-linear decreasing penetration rate from an initial penetration rate of up to 10 m/s. In contrast, static-CPTU cones penetrate the soil with a constant penetration rate of usually 2 cm/s. This difference, known as strain-rate effect, causes elevated dynamic-CPTU parameters for fine-grained soils. The parameter values are up to 1.5 times higher for the corrected cone penetration resistance ($q_t$), less than 3 times higher sleeve friction ($f_s$) and 1.5 to 3 times higher for excess pore pressure ($\Delta u$) depending on the location of the pressure ports ($\Delta u_1$, $\Delta u_2$ and $\Delta u_3$). In order to investigate the strain-rate effect, a comprehensive MATLAB routine was developed including: (1) raw data processing, (2) dynamic data analyses, (3) in situ strain-rate correction, and (4) geological and geotechnical analyses. The MATLAB routine enables us to compare: (i) dynamic-CPTU profiles with static profiles, as well as v-s, fc and DSS datasets collected at different sites worldwide (e.g. France, Japan and Norway), and (ii) different strain-rate correction solutions (e.g. logarithmic, inverse sinh-hyperbolic, power-law, velocity ratio) in order to find the optimal solution and improve it by considering a modified non-dimensional velocity ratio. Visual observations and a simple statistical methods (e.g. two-sample Kolmogorov-Smirnov test), resulting in the coefficients of determination up to 0.82, demonstrate that the modified inverse
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The sin-hyperbolic equation is best suited to correct for the strain-rate effect in dynamic-CPTU tests. The soil-specific rate coefficients, part of the strain-rate correction solution, are determined for fine-grained soils to be 0.04 to 0.065 for \( q_t \), 0.2 to 0.6 for \( f_s \) and up to 0.65 for \( \Delta u \). In addition, the logarithmic, power-law and velocity ratio solutions exhibit reasonable agreement for most of the data.

The MARUM shallow-water dynamic-CPTU instrument (SWFF-CPTU) was developed to identify and characterize surficial weak layers in a slide-prone area off the village of Finneid fjord (Sør fjorden, northern Norway). A 0.45 m thick weak layer was found and characterized as soft, sensitive clays with a thin seam of sandy silt to sand. Several of the dynamic-CPTU tests are used: (i) to map the weak layer in a 2D sub-seafloor model, and (ii) to discuss the role of fluid flow in slope stability. A similar exploration was performed at the northeastern slope of the Gela basin (southern Sicily, Italy), where mainly undisturbed soils near two distinct landslides (northern and southern Twin slides) were investigated using the MARUM deep-water dynamic-CPTU instrument (DWFF-CPTU) and laboratory experiments. These investigations show an undrained shear-strength ratio \( (s_u/\sigma'_V) \) that is up to 1.5 times higher for the southern Twin slide compared to the northern one, which is interpreted to be due to an older father slide that affected only the southern Twin slide. Several morphological steps and gullies were found, which probably indicate surficial slow deformation processes along two embedded stiff fine-grained or coarse-grained layers encountered in an older sub-bottom profiler transect and in several DWFF-CPTU profiles.

A multi-disciplinary approach described by the combination of bathymetrical, geophysical, sedimentological and geotechnical datasets are used to develop an area-wide sub-seafloor model near the Nice international airport (southeastern France). The sub-seafloor model consists of a geometrical/sedimentological model using bathymetrical map, several core profiles and geophysical chirp transects, and a geotechnical/strength model using results from \( v-s \) and \( f_c \) experiments, high number of dynamic- and several static-CPTU tests. Based on the sub-seafloor model, the role of free gas in the soil and different failure geometries are discussed and demonstrated using 2D numerical slope stability assessments.
Abstract

_DWFF-CPTU_ tests and laboratory experiments on cored specimens were performed to characterize the surficial soils across mud volcanoes (MVs) in southern Crete (Greece) and southwestern Japan. Active and inactive mud volcanoes were identified and characterized by $s_u/\sigma_{V0}'$ less than 0.5 for the active MVs and 0.7 to 1.5 for the inactive ones. Moreover, in the vicinity of the city of Patras (western Greece), a pockmark field was studied using _SWFF-CPTU_ and previously collected core datasets. The _in situ_ and laboratory results indicate significant differences between active and inactive pockmarks, such as 5 times lower $s_u$ and low negative $\Delta u$ for the active ones. Active pockmarks are also underconsolidated with $s_u/\sigma_{V0}'$ less than 0.2 compared to inactive ones exhibiting a normally to slightly overconsolidated state with $s_u/\sigma_{V0}' > 0.2$. 
Zusammenfassung


Zusammenfassung

für die Arkussinus Hyperbolicus Funktion mit Verbesserung durch das nicht-dimensionale Geschwindigkeitsverhältnis. Für diese verbesserte Funktion wurden auch die bodenspezifischen Geschwindigkeitskoeffizienten bestimmt. Für feinkörnige Böden liegen diese Werte bei 0.04 bis 0.065 für den Spitzenwiderstand, bei 0.2 bis 0.6 für die Mantelreibung und nicht über 0.65 für den Porenwasserdruck. Die anderen Korrekturfunktionen sind jedoch grundsätzlich auch für die Korrektur von dynamischen Versuchen geeignet.


Die Kombination von bathymetrischen, geophysikalischen, sedimentologischen und geotechnischen Daten ermöglichte die
Zusammenfassung


Südlich von Kreta (Griechenland) und südwestlich von Japan wurden DWFF-CPTU-Messungen genutzt, um die oberflächliche Bodenabfolge von aktiven und nicht aktiven Schlammvulkanen zu untersuchen. Oberflächliche Böden von aktiven Schlammvulkanen haben einen normalen Konsolidierungsgrad mit einem undrainiertem Scherfestigkeitsverhältnis \( \frac{s_u}{\sigma'_{V0}} \) von weniger als 0.5. Die nicht aktiven zeigen sehr viel höhere \( s_u/\sigma'_{V0} \)-Werte in der Größenordnung von 0.7 bis 1.5. Zudem wurden SWFF-CPTU-Messungen und früher durchgeführte Versuche an Sedimentkernen verwendet, um ein Pockmark-Feld in der Nähe der Stadt Patras (westliches Griechenland) zu studieren. Ein signifikanter Unterschied in der Scherfestigkeit \( s_u \) von aktiven und nicht aktiven Pockmarks konnte festgestellt werden. Aktive Pockmarks weisen bis zu 5-mal geringere \( s_u \)-Werte und einen geringen negativen Porenwasserüberdruck auf. Zusätzlich sind die Bodenschichten bei aktiven Pockmarks unterkonsolidiert mit einem \( s_u/\sigma'_{V0} \)-Wert von weniger als 0.2. Im Vergleich dazu sind die nicht aktiven Pockmarks in der Regel leicht überkonsolidiert (z.B. \( s_u/\sigma'_{V0}>0.2 \)).
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Chapter 1

Introduction

1.1 Motivation

The growing settlement and development of subaqueous activities in coastal, nearshore and offshore regions requires a greater need for understanding the sedimentological and geotechnical properties of the submarine soils, which are ubiquitous in submarine engineering practice and landslide research. A submarine landslide occurs as a consequence of a wide variability of geological and geotechnical processes leading to

\[ \text{usually rapid down slope movement of a mass of rock, earth or artificial fill on a slope} \]

(international accepted definition of a landslide in Webster’s 3rd International Dictionary; Cruden 1991). These processes may include: (i) high sediment accumulation rates due to large amount of river discharge (e.g. Coleman et al. 1978), (ii) development of free gas as a result of decaying of organic matter or gas hydrates (e.g. Mienert et al. 1998, Sultan et al. 2010), (iii) pore pressure increase owing to high precipitation (e.g. Stegmann et al. 2011) and (iv) decrease of the soil’s mechanical strength related to the presence of weak layers (e.g. L’Heureux et al. 2007, 2010). An adequate investigation of these processes requires the use of geophysical, coring/sampling and in situ techniques, where each of these has their own advantages and limitations. The geophysical techniques allow developing 2D and/or 3D geometrical/geological models in surficial to deep sub-seafloor conditions using high-resolution reflection, seismic refraction and electrical resistivity systems. However, they provide more qualitative than quantitative results about the soil properties, and need to be ground truthed using in situ and coring/sampling measurements (e.g. Technical Committee 1, ISSMGE, Danson 2005). The coring/sampling techniques recover soil samples or core
segments, used for soil classification purposes and laboratory experiments to measure physical and geotechnical properties. The laboratory experiments are cost- and time-consuming tests and the quality of the derived properties is subject to disturbance of the soil specimens during the sampling procedure (e.g. Skinner and McCAve 2003). These limitations are eliminated for the in situ technique usually known as cone penetration testing with pore pressure measurement (CPTU). CPTU measures the soil properties using a cone mounted on metal rods and equipped with force and pressure transducers. The cone penetrate the soil with either: (i) a constant rate, commonly 2 cm/s, described as conventional, static-CPTU (de Ruiter and Fox 1975, Lunne et al. 1997), or (ii) a non-linearly decreasing penetration rate of up to 10 m/s, denoted as dynamic-CPTU (Stegmann et al. 2006, Stoll et al. 2007, Young et al. 2011). Both systems measure the cone penetration resistance \( q_c \), sleeve friction \( f_s \), pore pressure at the tip \( u_1 \) or behind the tip \( u_2 \) and behind the sleeve \( u_3 \). The combination of these parameters provides vital information about the lithology (soil classification), physical and geotechnical properties (e.g. in situ bulk density, permeability, pore pressure evolution, in situ undrained shear-strength, consolidation state). However, the handling and operational mode differs for both instruments. The static-CPTU instrument is parked on the seabed, and the cone is pushed into the soil until to the target depth by applying hydraulic cylinders or wheel drive. The dynamic-CPTU device is lowered in the water column with winch mode or in free-fall until the tip of the cone hits the seabed with an initial penetration rate \( v_0 \), and penetrates the soil by its own momentum until it reaches the terminal depth (Dayal et al. 1975, Stegmann et al. 2006). In clayey soils, the dynamic-CPTU device has an advantage in terms of measurement speed and does not require large, sophisticated vessels or platforms. However, the penetration depth is limited to less than 15 m (Dayal et al. 1975) in contrast to the static-CPTU instrument, where 100 m penetration depth is attainable (Randolph 2004).

The penetration rate profiles and magnitudes for dynamic-CPTU parameters are observed to be higher compared to the static-CPTU ones, known as the so-called strain-rate effect (Eyring 1936, Casagrande and Wilson 1951, Suklje 1957, Silva et al. 2006). Three different strain-rate correction methods are commonly used to correct laboratory experiments, such as vane shear, centrifuge and soil target tests (e.g. Dayal and Allen 1975, Biscontin and Pestana 2001, Randolph and Hope 2004) as well as numerical simulations (e.g. Silva et al. 2006, Nazem et al. 2012). These methods are logarithmic, inverse sin-hyperbolic and power-law equations that depend on the dynamic \( v_{dyn} \) and static penetration rate \( v_{ref} \) and soil-specific rate coefficients (SSCs)
(Dayal and Allen 1975, Mitchell and Soga 2005, Biscontin and Pestana 2001, Lehane et al. 2009). However, only the logarithmic equation is verified and validated for dynamic-CPTU tests (Dayal et al. 1975, Aubeny and Shi 2006, Young et al. 2011) and the associated SSCs are confirmed for: (i) low penetration depths of less than 2 m (e.g. Stoll et al. 2007, Stark et al. 2009) (ii) measurements without considering pore pressure effects (e.g. Dayal et al. 1975, Aubeny and Shi 2006) and (iii) laboratory tests on specific soil mixtures or grain-size classes using small tanks and containers (e.g. Dayal and Allen 1975).

The objectives of this doctoral thesis are: (i) to improve the data processing procedure for the analysis of dynamic-CPTU tests by developing a comprehensive and coherent analysis routine (MATLAB), (ii) to compare dynamic-CPTU tests with static-CPTU measurements, v-s, fc and DSS experiments. Novel and state-of-the-art strain-rate correction solutions are evaluated and the best suited one is suggested, including associated SSCs for various soil conditions, and (iii) analyze the dynamic and static-CPTU measurements in combination with intact undrained shear-strength (s_u) records derived from v-s, fc and DSS experiments on cored specimens to explore deep-water mud volcanoes in Greece and Japan, a coastal pockmark field in Greece, and offshore and coastal submarine landslides in France, Italy and Norway.

1.2 Study areas and objectives

1.2.1 Sørffjorden, the 1996 Finneidfjord landslide

The inner part of Sørffjorden basin is located near the village of Finneidfjord, northern Norway. This fjord basin is 1.5 km wide and up to 5 km long with water depth (WD) less than 60 m (Fig. 1.1). Several terrestrial landslides have been triggered in historical and prehistorical times (Fig. 1.1, Olsen et al. 2006) and on June 20th, 1996, a combined submarine/subaerial, retrogressive flow/quick clay landslide mobilized approximately 1*10^6 m^3 of soil (Longva et al. 2003). Consequently, this area is well suited to be a natural laboratory for testing high-resolution geophysical and geotechnical equipments (Vanneste et al. 2013), and this doctoral thesis addresses the following objectives:

- comparison of dynamic-CPTU records with static-CPTU measurements, vane shear (v-s), fall cone penetration (fc) and direct simple shear
1.2.2 **Ligurian margin, the 1979 Nice airport landslide**

The French part of the *Ligurian* margin contains a narrow continental shelf ~3 km width and is located in the northwestern *Mediterranean* Sea (Fig. 1.2). Approximately 250 landslide scars were detected at the shelf break and on
1.2. STUDY AREAS AND OBJECTIVES

Figure 1.2: Bathymetrical image illustrates the upper slope and shelf morphology of the Ligurian margin. The 1979 Nice landslide area, airport and airport/harbor extension within the study area are also shown. The across-track resolution of the bathymetry image is less than 25 m.

the upper continental slope between 20 and 1000 m WD using high-resolution hull-mounted and AUV bathymetrical records (Klaucke and Cochronat 1999, Migeon et al. 2012). Moreover, on 16th of October 1979, a tsunamigenic, submarine landslide mobilized approximately $8.7 \times 10^6$ m$^3$ of sediments (Assier-Rzadkiewicz et al. 2000). On the way to the Ligurian basin, the collapsed slide mass transformed into a debris flow, followed by a turbidity current (Dan et al. 2007). Consequently, a better understanding of the initiation and development of such landslides is necessary and will be targeted by the following investigations in this doctoral thesis:

- confirmation of free gas growth in the sediments using comparison of in situ CPTU records with v-s and fc experiments on cored samples taken from gc and cpc,

- development of an area-wide geotechnical/strength model taking into
consideration 37 dynamic-CPTU tests, eight static-CPTU measurements and intact undrained shear-strength datasets from gc and cpc,

- use of a multidisciplinary approach combining bathymetrical, geophysical and geotechnical measurements in order to develop 2D sub-seafloor models, and

- execution of 2D numerical slope stability assessments evaluating the potential risks for future landslides in this region.

1.2.3 Gela basin, the southern and northern Twin slides

Gela basin is a Neogene-Quaternary foredeep basin of the Maghrebian fold-and-thrust belt (southern Sicily, Italy). This basin is 20 km wide and up to 50 km long with WD between 600 and 700 m (e.g. Trincardi and Argnani 1990). The northeastern foreland of Gela basin is subject to several landslides that occurred in the Quaternary period (Fig. 1.3). Morphological investigations reveal evidence of a giant, old father slide and two recently exposed landslides, known as the northern and southern Twin slides (e.g. Minisini et al. 2007, 2009, Kuhlmann et al. 2014). The objectives for this study area are as follows:

- validation of the hypothesis that the excess pore pressure ($\Delta u$) records are directly correlated to the in situ intact undrained shear-strength ($s_u$) in fine-grained soils using dynamic-CPTU and laboratory datasets,

- evaluation of state-of-the-art logarithmic and inverse sin-hyperbolic strain-rate correction solutions, as well as a newly developed correction equation by comparing of in situ and laboratory intact undrained shear-strength profiles,

- sub-seafloor characterization of the surficial soils including the detection of stiff fine- and coarse-grained layers within the surrounding, homogenous soils, and

- development of a 2D sub-seafloor model taking into account in situ intact undrained shear-strength profiles derived from $\Delta u$ ($s_{u,\Delta u}$), a 2D sub-bottom profiler transect (Minisini et al. 2007), $v$-$s$ and $f_c$ strength profiles ($s_{u,v-s}$ and $s_{u,f_c}$) derived from cores.
1.2. STUDY AREAS AND OBJECTIVES

1.2.4 Mud volcanoes on the Mediterranean Ridge accretionary complex

The Mediterranean Ridge accretionary complex (MedRidge) is part of the Hellenic subduction zone, and it is more than 300 km wide and ~2000 km long (southern Crete, Greece). In the Olimpi field, located in the center part of the MedRidge, seven MVs were mapped in an area of ~100 km$^2$ and at up to 1950 m WD. The MV diameters are between 1.5 and 4.0 km and
Figure 1.4: Bathymetrical map of the mud volcanoes in the vicinity of the Olimpi field (southern Crete, Greece). The nomenclature of the different mud volcanoes and the inner deformation front (see Robertson and Kopf 1998 for details) are also described. The across-track resolution of the map is better than 50 m.

The heights are less than 200 m (Fig. 1.4 see also Camerlenghi et al. 1992, Kopf et al. 2002, 2003 for more details). In this study area, the following objectives are addressed:

- operational evaluation of a novel setup for the MARUM deep-water dynamic-CPTU equipment (DWFF-CPTU) for WD >1500 m,
- comparison of $s_u, \Delta u$, $s_{u,v}$ and $s_{u,fc}$ profiles to examine the empirical $\Delta u/s_u$ correlation and evaluate the strain-rate correction solutions and associated SSCs,
- geotechnical characterization including $\Delta u$ and $s_u, \Delta u$ profiles of the ejected clay-rich soils and the background soils close-located to the MVs in order to evaluate the fluid/gas conditions and consolidation state, and to determine if the MVs are currently active.
1.2. STUDY AREAS AND OBJECTIVES

1.2.5 Mud volcanoes in the northern Kumano basin

The Kumano basin is located in the eastern Nankai accretionary complex off the southwestern region of Japan. It is the largest forearc basin of the Nankai subduction zone extending over 7000 km² with a nearly flat seafloor and WD around 2000 m (Fig. 1.5, Morita et al. 2004). Several MVs and mud diapirs were identified with a maximum diameter of 2 km and up to 160 m height (e.g. Kuramoto et al. 2001, Sawada et al. 2002). The objectives are similar to those presented for the MedRidge (see section 1.2.4).

1.2.6 Pockmark field in the vicinity of the new harbor of Patras

In the Gulf of Patras, situated in the western part of Greece, an active pockmark field was identified near the new city harbor of Patras. This field
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Figure 1.6: Surface morphology of the pockmark field near the new city harbor of Patras (western Greece) including the engineering structures of the harbor. The contourlines with a spacing of 5m describe the water depth (modified after Christodoulou et al. (2003), Marinaro et al. (2006).

consists of 72 pockmarks and covers an area of 1.7km² within a WD less than 40 m (Fig. 1.6 Hasiotis et al. 1996, Christodoulou et al. 2003).

In 1993 and again 2008, two major earthquakes with magnitude 5.4 and 6.4 on the Richter scale occurred in the vicinity of the pockmark field (Hasiotis et al. 1996, Christodoulou et al. 2009). These earthquakes have probably activated several pockmarks leading to a temperature increase of 6°C at 10 m
above the seabed prior the event and gas bubble venting several days after
the earthquake (Christodoulou et al. 2003, Christodoulou et al. 2009). Con-
sequently, a geotechnical study is necessary in order to shed light on the
connection between seismicity and pockmark activation including:

- comparison of recently measured $s_{uqt}$ profiles with $s_{u,v,s}$ data taken
  from older cores in order to check the correlation between $in situ$ and
  laboratory strength measurements for sub-seafloor characterization,
- analysis of the pore pressure signal focusing on pressure anomalies prob-
  ably related to gas bubble emission, and
- evaluation whether the investigated pockmarks are currently active.

1.3 Structure of the doctoral thesis

This doctoral thesis was part of the MARUM - Center for Marine Environ-
mental Sciences and Faculty of Geosciences, University of Bremen research
project SD 3 "Rapid sediment mobilization" in the research area Sediment
Dynamics (SD). The incorporated datasets were mostly acquired within the
framework of cooperation between MARUM and several other research in-
stitutions, such as NGU (Geological Survey of Norway), NGI (Norwegian
Geotechnical Institute) and Ifremer (French Research Institute for Exploita-
tion of the Sea). The data processing and analyses were carried out within
the last three and a half years, and almost of the datasets are presented
in four manuscripts, where each includes an introduction, methods, results,
discussion and conclusion chapters, internal or external reports from other
research institutions and parts of scientific cruise reports (see also Appendix).

Note that the this doctoral thesis is not presented chronologically, be-
cause a later study is fundamental rather than specific in nature and thus
is presented at the beginning of this thesis. Furthermore, the geological and
geotechnical studies and ongoing projects may be more easily understood
with the fundamental study and an additional state-of-the-art section as
background material.

The state-of-the-art section is divided into three parts as follows: (i) de-
scription of CPTU-related issues, (ii) assessment of soil instabilities and land-
slides and (iii) introduction of submarine mud volcanoes and pockmarks. It
starts with a summary of commonly used static- and dynamic-CPTU equip-
ments focusing on penetration mechanisms, development date and company
CHAPTER 1. INTRODUCTION

Institution, name, advantage and limitation of the equipments (section 2.1). This summary is complemented by addressing the following two questions:

1. What are the differences between the two CPTU systems?

2. What is required in order to apply those systems for geological and geotechnical purposes?

In particular, special focus is given to the evaluation of intact undrained shear-strength, which is a crucial parameter for soil instability and landslides (section 2.1.4). The assessment of soil instability leading to mass movements is a complex matter requiring several data sets, such as potential types of mass movements, types of geomaterials, stages of mass movements and their development. The synthesis of these datasets enables a comprehensive geotechnical characterization and slope stability assessment, not only required for submarine landslides (section 2.2 and 2.3), but also in the broader sense relevant for the investigation of submarine mud volcanoes and pockmarks (section 2.4). Mud volcanoes and pockmarks are a sedimentological and hydrological window to deeper regions and may be used to obtain a better understanding of geomechanical and geochemical processes related to subduction zones occurring prior to or following earthquakes (section 2.4).

The exploration of geological and geotechnical processes related to submarine landslides, mud volcanoes and pockmarks require knowledge of soil properties, which can be collected using, for example, the MARUM SWFF- and DWFF-CPTU instruments. Since 2006, both instruments have been modified and adapted in order to be appropriate for the different sites where these novel devices have been deployed (section 3.1). Moreover, data processing is more demanding compared to static-CPTU instruments, thus requires a comprehensible and consistent procedure for the data analysis (section 3.2). This procedure contains four global tasks, which are described by: (i) raw data processing, (ii) dynamic data analyses, (iii) in situ strain-rate correction and (iv) geological and geotechnical analyses. The first two tasks depend on the deployed instrument and specifications, such as length and weight of the instrument, type of cone and other details. The third task depends on different CPTU parameters and soil conditions. Theoretical approaches are difficult to develop and the application to natural environments is associated with several limitations related to the heterogeneity of the sub-seafloor soils. However, in situ comparison between strain-rate corrected CPTU tests and records taken from static ones as well as laboratory experiments are the key to successful interpretation of data (section 3.2 and 3.3). The last task
is applied to the objectives for the different exploration sites presented in section 1.2 using the following datasets from:

- two scientific cruises in northern Norway with the R/V Seisma (collaboration between MARUM, NGU and NGI; section 4.1) and southern Sicily, Italy with the R/V Maria S. Merian (MSM 15/3; section 5.1),
- four scientific cruises in southeastern France with R/Vs Meteor (M 73), Poseidon (P 386, P 429) and L’Europe (collaboration between MARUM and Ifremer) (section 4.2),
- two scientific cruises in southern Crete, Greece with the R/V Poseidon (P 410 and P 429) and one cruise in southern Japan with the R/V Sonne (SO 222) (section 5.2), and
- one scientific cruise in western Greece (collaboration between MARUM and University of Patras) (section 5.3).

To summarize, this doctoral thesis presents: (i) a coherent and comprehensible approach for the data processing and strain-rate correction of \textit{in situ} dynamic-CPTU records, (ii) three geological and geotechnical applications related to the stability of submarine slope sediments including the identification and characterization of surficial weak layers as well as sub-seafloor modeling and numerical slope stability assessments, and (iii) soil characterization in the vicinity of deep-water mud volcanoes and coastal pockmarks.
Chapter 2

State-of-the-art

2.1 Cone penetration testing

Dynamic and static cone penetration testing (CPT) are powerful time and cost efficient instruments used to measure \textit{in situ} soil properties in onshore and offshore environments \cite[e.g.][]{Dayal1975, Lunne1997, Lunne2010}. Three different strategies are commonly used to measure these soil properties \cite[e.g.][]{Zuidberg1986}. The first and most frequently used strategy is to place a CPTU rig on the surface or seabed and push the cone until either the required terminal depth or refusal are reached, known as the so-called \textit{seabed mode} \cite[e.g.][]{Zuidberg1972, Eide1974}. Using the second strategy, the CPTU instrument is lowered into the water column at winch-controlled speed or in free-fall, and penetrates the soil with its own momentum, known as the so-called \textit{winch or free-fall mode} \cite[e.g.][]{Dayal1975, Stegmann2006}. The third strategy requires a borehole, within which the cone is pushed into the soil at the bottom of this hole, known as the \textit{down-hole or drilling mode} \cite[e.g.][]{Zuidberg1972, Berg1984}. This doctoral thesis focuses on new methods using dynamic- and static-CPTU tests employing the \textit{winch, free-fall or seabed modes} performed in coastal, nearshore and offshore environments. Consequently, \textit{in situ} tests carried out in onshore environments and/or using the \textit{down-hole mode} are outside of the scope of this thesis.

2.1.1 Static cone penetration equipments

From 1966 to 1972, the Dutch company \textit{Fugro} developed the first seabed CPT rigs, called \textit{Seabull} and \textit{Seacalf}, for the geotechnical profiling of soils in estuaries, shallow-water environments and North Sea \cite{Zuidberg1972, Zuidberg1974, Zuidberg1984}. 

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the hydraulic ram pushing system, called Stingray (McClelland 1975). In the mid 1970’s, a seabed platform for CPTs and soil drilling named Diving Bell was designed by Delft Soil Mechanics Laboratories (now Deltar es). CPT tests in Holland and Canada using this system reached more than 60 m penetration depth in 60 m water depth (WD) (Vermeiden 1977). Due to the low performance of CPT rigs on sands, in particular in the North Sea, two new heavy seabed rigs were developed for offshore site investigations of wind energy farms, platforms and subsea structures. In 1991, the GEO (Danish Geotechnical Institute) designed the GEOScope, which reached 45 m penetration depth at sites in the North Sea but is limited to 270 m WD (Den ver and Riis 1992). In 2010, a similar seabed rig was developed by MARUM called GOST (Geotechnical Offshore Seabed Tool; Fig. 2.1). It is capable to an operational WD of 4000 m and a penetration depth of up to 38 m in clay or sand (http://www.marum.de/GOST_- _System_ _Geotechnical_Offshore_Seabad_Tool.html). Both systems push a 5 cm$^2$ or 10 cm$^2$ cone into the soil in semi-continuous intervals using dual push/pull hydraulic cylinders powered by an autonomous hydraulic unit on the seabed tool.

In the 1980’s, a new generation of seabed rigs was introduced using roller wheels to drive the cone into the soil at a constant velocity. The APvandenBerg manufacturer in collaboration with D’Appolonia conducted the first successful tests at the Oseberg field in the North Sea by applying the so-called ROSON rig (Berg 1984). Following this strategy, McClelland, Fu-gro, Columbia University and Gregg Drilling & Testing designed their own seabed rigs using four synergistic hydraulic cylinders, with roller wheels or chain drive engines to accomplish continuous pushing (Amundsen et al. 1985, Zuidberg et al. 1986, Stoll 2005, Boggess and Robertson 2010). Penetration depths of 43 m or more were achieved in WDs between 350 and 3000 m at different sites in the North Sea. In deep-water environments, accurate measurements of the $q_c$ is difficult due to high water pressure acting on the tip compared to the low strength of the soil. APvandenBerg, Gregg Drilling & Testing, Ifremer and MARUM addressed this problem by designing oil balanced or hydrostatic pressure compensated shear load cones, which isolate the $q_c$ of the soil (Fig. 2.2). Meunier et al. 2004, Boggess and Robertson 2010, Lunne 2010).

Very long, straight rod setups require a tower or constant-tension winch
.onboard the vessel in order to compensate for vessel movement during operation (e.g. Lunne 2010). In 2011, TDI-Brooks developed a light-weight tower assembly, called Seabed CPT Stinger, which consists of a tubular weighted base and a tower up to 12 m long with a barrel containing the CPTU cone/rod (http://www.tdi-bi.com/field_services/geotech/CPT-main.htm, Jeanjean et al. 2012). The weighted base must be embedded into the surficial soil before the pushing process starts, hence use of this system is limited by weather conditions. This limitation of the operability, in particular for a rod lengths higher 30 m, lead to the design of seabed rigs with a full length coiled rods. Ifremer designed and constructed the Penfeld rig with an associated pressure compensated cone capable of operating in WD of 6000 m and a penetration depth up to 30 m (Fig. 2.3, Meunier 2000, Meunier et al. 2004). Moreover, combined drilling, coring and CPTU testing rigs were also developed to collect both core samples (which may be stored in a carousel), and CPTU records from one deployment. Three different systems are common, named PROD, Sarobin and GEOceptor, which can reach between 2 and 100 m penetration depth for the CPTU testing and coring in the range of 1 to 125 m (Hawkins and Marcus 1998, Randolph 2004).
Owing to the cost and limited availability of large, sophisticated vessels required for the handling of standard seabed rigs, miniature CPTU rigs were designed to be adequate for surficial or shallow sub-seafloor geological investigations, such as those used for communication and pipeline tracks. Miniature rigs with a weight less than 1.5 tons, called Seascout or Neptun 3000, can be operated from small vessels or platforms and are usually equipped with a coiled rod, 1 to 5 cm² cones and reach up to 12 m penetration depth (e.g. Power and Geise 1994). In addition, Fugro equipped a conventional ROV (remotely operated vehicle) with a miniature CPTU rig. This system operated down to 3000 m WD and its driving force was assisted by a suction anchor system located underneath the ROV (Geise and Kolk 1983).

In the last four decades, intensive developments were conducted to improve the performance of seabed rigs and mostly designed to improve the accuracy of the CPTU cone, maximum penetration depth and operational WD.
Figure 2.3: Ifremer’s Penfeld penetrometer (copyright Ifremer; Meunier 2000, Meunier et al. 2004)

Table 2.1 presents these developments including references, notes, name of equipment and companies/institutions, design dates, and penetration mechanisms as well as main advance developments.
Table 2.1: Summary of the main developments for seabed rigs (modified after Lunne 2010).

<table>
<thead>
<tr>
<th>penetration mechanism/main advance development</th>
<th>date</th>
<th>equipment</th>
<th>company and institution</th>
<th>note</th>
<th>reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>discontinuous push hydraulic cylinder</td>
<td>March 1972</td>
<td>dead weight operated from platform</td>
<td>NGI/McClelland</td>
<td>4 m penetration depth reached in dense sand</td>
<td>Eide (1974)</td>
</tr>
<tr>
<td>march 1972</td>
<td>Seacalf</td>
<td>Fugro</td>
<td></td>
<td>25 m penetration depth reached in 130 m WD</td>
<td>Zuidberg (1972)</td>
</tr>
<tr>
<td>1974</td>
<td>Stingray</td>
<td>McClelland</td>
<td></td>
<td>push on drill pipe, not on cone rod</td>
<td>McClelland (1975)</td>
</tr>
<tr>
<td>1976</td>
<td>Diving bell</td>
<td>Delft Soil Mechanics Laboratory (Deltas)</td>
<td>GEOscope</td>
<td>GEO, Denmark self leveling, 45 m penetration depth reached in the North Sea</td>
<td>Vermeiden (1977)</td>
</tr>
<tr>
<td>1991</td>
<td>GEOscope</td>
<td>GEO, Denmark</td>
<td></td>
<td>self leveling, 45 m penetration depth reached in the North Sea</td>
<td>Denver and Riis (1992)</td>
</tr>
<tr>
<td>2010</td>
<td>GOST</td>
<td>MARUM</td>
<td></td>
<td>1.5 m long straight rod, 2 cm$^2$ cone, 38 m penetration depth achieved in clay to sandy silt</td>
<td><a href="http://www.marum.de">www.marum.de</a></td>
</tr>
<tr>
<td>1984</td>
<td>modified BOR-ROS</td>
<td>McClelland</td>
<td></td>
<td>synopticated hydraulic cylinders</td>
<td>Amundsen et al. (1985)</td>
</tr>
<tr>
<td>1984</td>
<td>wheel drive</td>
<td>Seacalf</td>
<td>Fugro</td>
<td>roller wheels</td>
<td>Zuidberg et al. (1986)</td>
</tr>
</tbody>
</table>

Continued on next page...
<table>
<thead>
<tr>
<th>Date</th>
<th>Equipment</th>
<th>Company and Institution</th>
<th>Note</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>2004</td>
<td>STATPEN</td>
<td>Columbia University</td>
<td>straight rod, 10 cm$^2$ cone, 2 m penetration depth achieved in sand</td>
<td>Stoll (2005)</td>
</tr>
<tr>
<td>2010</td>
<td>DeepCPT</td>
<td>Gregg Drilling &amp; Testing</td>
<td>suction anchor; 200 kN trust capacity, 10 and 15 cm$^2$ cones</td>
<td>Boggess and Robertson (2010)</td>
</tr>
<tr>
<td>2011</td>
<td>Seabed CPT Stinger</td>
<td>TDI-Brooks</td>
<td>tower assembly, 15 cm$^2$ piezo-cone, max. 3000 m WD, 10-12 m penetration depth achieved</td>
<td><a href="http://www.tdi-bi.com/.../CPT-main.htm">www.tdi-bi.com/.../CPT-main.htm</a></td>
</tr>
<tr>
<td>Coiled rod (on full size rods)</td>
<td>2000</td>
<td>Penfeld</td>
<td>Ifremer</td>
<td>selfpowered by lead batteries, 30 m penetration depth reached in clay to silt</td>
</tr>
<tr>
<td>Seabed founded drilling, testing and sampling</td>
<td>2001</td>
<td>PROD</td>
<td>Benthic</td>
<td>rods stored in carousel on sea bottom</td>
</tr>
<tr>
<td>Combined rig</td>
<td>1997</td>
<td>Searobin</td>
<td>Fugro</td>
<td>take sample to 1 m and do 10 cm$^2$ CPT to 2 m in one deployment</td>
</tr>
<tr>
<td></td>
<td>2001</td>
<td>GEOceptor</td>
<td>GEO, Denmark</td>
<td>take sample to 6 m and do 10 cm$^2$ CPT to 10 m in one deployment</td>
</tr>
</tbody>
</table>

Continued on next page...
<table>
<thead>
<tr>
<th>Date</th>
<th>Equipment</th>
<th>Company and Institution</th>
<th>Note</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1992</td>
<td>Seascout</td>
<td>Fugro</td>
<td>coiled rod, weight &lt;1 tons, 1 and 2 cm² cones, 6 m penetration depth achieved</td>
<td>Power and Geise (1994)</td>
</tr>
<tr>
<td>2000</td>
<td>Neptun 3000</td>
<td>DATEM</td>
<td>coiled rod, weight &lt;1.5 tons, 2 and 5 cm² cones, 10 m penetration depth reached</td>
<td>Steggar (2009)</td>
</tr>
<tr>
<td>1999</td>
<td>MiniCPT</td>
<td>Gregg Drilling &amp; Testing</td>
<td>coiled rod, 2 cm² cone, 12 m penetration depth achieved</td>
<td></td>
</tr>
<tr>
<td>1983</td>
<td>ROV CPT</td>
<td>Fugro</td>
<td>5 cm² cone, 3 m penetration depth achieved</td>
<td>Geise and Kolk (1983)</td>
</tr>
</tbody>
</table>
2.1. CONE PENETRATION TESTING

2.1.2 Dynamic cone penetration equipments

Since the early 1970s lance-like devices, referred to as dynamic-CPT/CPTU hereafter, and projectile-like dynamic penetrometers have been deployed in several naval applications (e.g. mine burial), submarine geosciences (e.g. sediment remobilization, landslide and slope stability research) and engineering practice (e.g. exploration of cable and pipeline tracks, offshore foundation design). The first dynamic-CPT device, named impact penetrometer, was developed for laboratory use in order to supplement theoretical work in order to understand impact penetration phenomenon in dynamic penetration such as the so-called strain-rate effect (Dayal et al. 1973). This penetrometer was 0.6 m long including a conventional 10 cm$^2$ CPT cone, and was deployed in different soil targets such as clay and sand (Dayal and Allen 1975). Findings and experiments gained from these laboratory experiments were utilized to design a Marine Impact Penetrometer capable of reaching between 4.5 and 15 m penetration depth in clayey soils. It weighs 500 kg and can be operated in up to 180 m WD (Dayal et al. 1975). Two to three decades later, more lightweight systems were developed in order to classify and geotechnically characterize surficial soils within WD between 200 and 660 m. The Department of National Defense (Canada) in collaboration with the A.G.O. Environmental Electronics Ltd. designed and constructed the STING Mk II device (Poeckert et al. 1996). It records acceleration up to 10 g, but the internal data storage capacity was limited to maximum of 4 minutes; thus the datasets need to be downloaded after each deployment. This limitation and the lack of force measurements on the cone led to the development of the PROBOS device by the Columbia University (Stoll 2005). PROBOS monitors the acceleration and cone forces simultaneously and the setup geometry is similar to the STING device, including interchangeable cones with diameters of 5 to 40 cm$^2$ and an extendable shaft up to 3 m in length. A more robust system is the EFCPT instrument developed by Rolls-Royce Canada Limited, which was mainly designed to measure penetration acceleration and pore pressure (Olser et al. 2006). In 2010, University of Bremen and Fielax mbH developed a heavy (up to 2 tons) and robust system, called LIRmeter, mainly applicable for reconnaissance surveys of cable and pipeline routes in the North Sea (Stephan et al. 2012). The penetration depth is limited to 4 m and only acceleration and water pressure are recorded simultaneously.

Since 2005, intensive development and engineering were carried out resulting in a new generation of dynamic-CPT instruments. These instruments consist of a conventional, industrial CPTU cone that records $q_c$, $f_s$ and $u$, expendable metal rods and a pressure-tight housing containing the
electronics and power supply (Fig. 2.4, Stegmann et al. 2006). MARUM developed two different systems with a similar setup, but applicable for different WD. The shallow-water dynamic-CPTU device can be operated in up to 200 m WD (SWFF-CPTU; Fig. 2.4, Stegmann et al. 2006) and the deep-water instrument up to 4000 m WD (DWFF-CPTU; Fig. 2.5, Stegmann and Kopf 2007). Both instruments deploy a 15 cm$^2$ CPTU cone to a penetration depth of up to 6.5 m, and all data sets were recorded with a up to 1kHz data logger. In 2012, a similar tool, called Gravity CPT Stinger, was designed and constructed by TDI-Brooks using a 200Hz data logger and without a real-time communication system (http://www.tdi-bi.com/field_services/geotech/cpt-main.htm).

In addition to the dynamic-CPT(U) devices, projectile-like dynamic penetrometers, firstly introduced by Nikakhtar et al. (1982) and Ingram (1982), were deployed in naval applications (e.g. mine burial), engineering practice (e.g. seabed classification), paleolimnological and submarine research (e.g. support of acoustic record analysis, sediment remobilization processes). The

Figure 2.4: MARUM’s shallow-water dynamic piezocone penetrometer (copyright MARUM; Stegmann et al. 2006)
2.1. CONE PENETRATION TESTING

![Figure 2.5: MARUM's deep-water dynamic piezocone penetrometer (copyright MARUM; Stegmann and Kopf 2007)](image)

eXpendable Bottom Penetrometer (XBP) with a diameter of 40 cm² records only acceleration and is appropriate for a WD less than 200 m (Stoll and Akal 1999). It is deployed in free-fall mode and penetrates the soil with its own momentum. The Acadia University (Wolfville, NS) developed a similar free-fall penetrometer with a diameter of 20 cm², which is constructed for seabed characterization and classification of lakes and estuaries in WD of 15 m or more (Spooner et al. 2004). A more advanced version is the Nimrod penetrometer designed and constructed by MARUM in 2008. This tool measures not only acceleration, but also temperature and pore pressure at the tip (Stark et al. 2009). The operational WD is 200 m and the diameter is 95 cm², which provides better resolution when characterizing very soft soils or fluid mud.

The main limitation of dynamic instruments is the achievable penetration depth, which is usually lower compared to that of seabed CPTU rigs. Consequently, TDI-Brooks developed a novel instrument using the advantages of both dynamic- and static-CPTU devices (Young et al. 2011). This
instrument relies upon a conventional, industrial 15 cm^2 CPTU cone, which is appropriate for WD up to 3000 m. The operation starts with a dynamic sequence similar to a giant piston core deployment and is followed by a static pushing sequence with the inboard metal rods and CPTU cone. Following this procedure, a total penetration depth of 35 m was reached in clays (i.e. 20 m with the dynamic mode and 15 m with the static mode).

All these commonly used and novel instruments are summarized in Table 2.2 including references, notes, name of equipments and companies / institutions, design dates, penetration mechanisms as well as main advance developments.
### Table 2.2: Summary of the main developments for dynamic instruments.

<table>
<thead>
<tr>
<th>Penetration mechanism/main advance in development</th>
<th>Date</th>
<th>Equipment</th>
<th>Company and institution</th>
<th>Note</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lance-like dynamic penetrometer</td>
<td>1972</td>
<td>Laboratory Impact Penetrometer</td>
<td>University of Newfoundland</td>
<td>conventional 10 cm² CPT cone, 0.6 m penetration depth possible, 4.5 to 6 m/s penetration rate</td>
<td>Dayal et al. (1973)</td>
</tr>
<tr>
<td></td>
<td>1973</td>
<td>Marine Impact Penetrometer</td>
<td>University of Newfoundland</td>
<td>conventional 10 cm² CPT cone, max. 180 m WD, 4.5 to 15 m penetration depth reached in clays, up to 6 m/sec penetration rate</td>
<td>Dayal et al. (1973, 1975)</td>
</tr>
<tr>
<td></td>
<td>1995</td>
<td>STING Mk II</td>
<td>DND &amp; AGO</td>
<td>max. 200 to 300 m WD, 5 to 40 cm² cone, only acceleration measurement, 1 to 3 m penetration depth reached, 2 kHz sampling rate</td>
<td>Poeckert et al. (1996)</td>
</tr>
<tr>
<td></td>
<td>2000</td>
<td>PROBOS</td>
<td>Columbia University</td>
<td>modified STING Mk II, 5 to 40 cm² cone, only acceleration and cone penetration resistance measurement, 1 to 3 m penetration depth reached</td>
<td>Stoll et al. (2005)</td>
</tr>
</tbody>
</table>

Continued on next page...
<table>
<thead>
<tr>
<th>Date</th>
<th>Equipment</th>
<th>Company and Institution</th>
<th>Note</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>2004</td>
<td>FFCPT</td>
<td>Rolls-Royce</td>
<td>max. 660 m WD, 60 cm$^2$ cone, only acceleration and pore pressure measurement, 0.25 to 3 m penetration depth achieved in clay to sand soils</td>
<td>Olser et al. (2006)</td>
</tr>
<tr>
<td>2005</td>
<td>SWFF-CPTU</td>
<td>MARUM</td>
<td>max. 200 m WD, 15 cm$^2$ piezocone, up to 6.5 m penetration depth reached in clays, up to 1 kHz sampling rate</td>
<td>Stegmann et al. (2006)</td>
</tr>
<tr>
<td>2006</td>
<td>DWFF-CPTU</td>
<td>MARUM</td>
<td>max. 4000 m WD, 15 cm$^2$ piezocone, up to 4 m penetration depth achieved in clays, up to 1 kHz sampling rate</td>
<td>Stegmann and Kopf (2007)</td>
</tr>
<tr>
<td>2010</td>
<td>LIRmeter</td>
<td>Bremen University and Fiels mbH</td>
<td>max. 4500 m WD, 45 or 60 cm$^2$ cone, only acceleration and water pressure measurement, 4 m penetration depth achieved in clays, 500 Hz sampling rate</td>
<td>Stephan et al. 2012</td>
</tr>
<tr>
<td>2012</td>
<td>Gravity CPT Stinger</td>
<td>TDI-Brooks</td>
<td>max. 3000 m WD, 15 cm$^2$ piezocone, 3 to 6 m penetration depth achieved in clays, 200 Hz sampling rate</td>
<td><a href="http://www.tdi-bi.com/.../CPT-main.htm">www.tdi-bi.com/.../CPT-main.htm</a></td>
</tr>
</tbody>
</table>

Continued on next page...
### Table 2.2 - Continued

<table>
<thead>
<tr>
<th>Date</th>
<th>Equipment</th>
<th>Company and Institution</th>
<th>Note</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1998</td>
<td>XBP</td>
<td>Columbia University</td>
<td>Max. 200 m WD, 40 cm² cone, only acceleration measurement, up to 7 m/s penetration rate</td>
<td>Stoll and Akal (1999)</td>
</tr>
<tr>
<td>2003</td>
<td>Free-fall penetrometer</td>
<td>Acadia University</td>
<td>Max. 15 m WD, 20 cm² cone, only acceleration measurement</td>
<td>Spooner et al. (2004)</td>
</tr>
<tr>
<td>2008</td>
<td>Nimrod</td>
<td>MARUM</td>
<td>Max. 200 m WD, 95 cm² cone, acceleration and pore pressure measurement, 1 kHz sampling rate</td>
<td>Stark et al. (2009)</td>
</tr>
<tr>
<td>2010</td>
<td>CPT Stinger</td>
<td>TDI-Brooks</td>
<td>Max. 3000 m WD, 15 cm² piezocone, up to 20 m dynamic penetration and additional 16 m static penetration, dynamic deployment similar to Jumbo Piston Corer</td>
<td>Young et al. (2011)</td>
</tr>
</tbody>
</table>
2.1.3 Comparison of static and dynamic cone penetration tests

When comparing static- and dynamic-CPT(U)s, the different equipment and deployment modes result in differences in the recorded data between devices which have to be considered in the analyses of in situ tests (e.g. Stoll 2005). Parking of a heavy seabed rig on the surface causes an increase in the overburden stress and consolidation state, leading to overestimation of the strength properties within the first few meters. Such a phenomenon does not occur when using dynamic devices deployed by a winch or free-fall drop (e.g. Stegmann et al. 2006). Moreover, the seabed, winch and free-fall modes differ significantly in the penetration rates. In the seabed mode, the CPTU cone is pushed with a constant velocity of usually 2 cm/s into the soil. However, in the winch and free-fall modes, the instruments penetrate the soil at a non-linearly decreasing penetration rate in the order of less than 2 m/s in winch mode and up to 10 m/s in free-fall mode. Figure 2.6 illustrates typical examples of penetration rate profiles using the three deployment modes. The difference in achievable penetration depth between the winch and free-fall modes can also be seen.

Prior theoretical and experimental studies demonstrate that an increase in deformation rate can be directly correlated with an increase in strength properties (Eyring 1936, Casagrande and Wilson 1951, Sukje 1957). This so-called strain-rate effect was first addressed for dynamic penetrometers by Dayal and Allen (1975) using laboratory comparisons of remolded clays and sands at constant penetration rates up to 0.81 m/s. In addition, a few field experiments were conducted on land by dropping the Marine Impact Penetrometer (see also Table 2.2) into a target tank, and several trial tests were carried out in shallow water environments off St John’s, Newfoundland in order to confirm the laboratory findings (Dayal et al. 1975). From these laboratory and in situ tests, an empirical strain-rate correction method was derived, mathematically expressed as a logarithmic equation incorporating the SSC, \( v_{\text{dyn}} \) and \( v_{\text{ref}} \). Equations 2.1 and 2.2 describe the correction methods for \( q_c \) and \( f_s \):

\[
\frac{(q_{c,\text{dyn}} - q_{c,\text{ref}})}{q_{c,\text{ref}}} = \mu_{\text{CPTU},qc} \log \left( \frac{v_{\text{dyn}}}{v_{\text{ref}}} \right),
\]

\[
\frac{(f_{s,\text{dyn}} - f_{s,\text{ref}})}{f_{s,\text{ref}}} = \mu_{\text{CPTU},fs} \log \left( \frac{v_{\text{dyn}}}{v_{\text{ref}}} \right),
\]

where \( q_{c,\text{dyn}} \) and \( q_{c,\text{ref}} \) are the dynamic and static cone penetration resistances, \( f_{s,\text{dyn}} \) and \( f_{s,\text{ref}} \) are the dynamic and static sleeve frictions and \( \mu_{\text{CPTU},qc} \)
and $\mu_{CPTU,fs}$ are the SSCs for $q_c$ and $f_s$, which depend on undrained shear-strength (see Table 1 of Dayal and Allen 1975, Dayal et al. 1975). Several workers used these equations to correct *in situ* dynamic CPTs in clayey and sandy soils taking into account different SSCs (Aubeny and Shi 2006, Stoll et al. 2007, Stark et al. 2009, Young et al. 2011).

Two additional correction methods are commonly used in order to consider the strain-rate effect for laboratory experiments, in particular, for vane shear, consolidation and centrifuge tests (e.g. Biscontín and Pestana 2001, Mitchell and Soga 2005, Randolph 2004, Lehane et al. 2009). These methods are mathematically described by the inverse sinh-hyperbolic and power-law equations subject to the ratio of a particular and a reference strain-rate, and soil-specific coefficients $\mu'$ and $\beta$, expressed as follows:

$$\frac{q}{q_{ref}} = 1 + \mu'\text{arcsinh}\left(\frac{\epsilon}{\epsilon_{ref}}\right), \quad (2.3)$$
\[
\frac{q}{q_{\text{ref}}} = \left( \frac{\epsilon}{\epsilon_{\text{ref}}} \right)^\beta,
\]

where \( q \) is the strength value at failure at a particular strain-rate \( (\epsilon) \), and \( q_{\text{ref}} \) is the strength value at a reference strain-rate \( (\epsilon_{\text{ref}}) \).

### 2.1.4 In situ intact undrained shear-strength: field measurements vs. laboratory experiments

The \textit{in situ} intact undrained shear-strength \( (s_u) \) is determined in the laboratory using fc and v-s tests as well as anisotropically-consolidated undrained extension triaxial (CAUE), DSS and anisotropically-consolidated undrained compression triaxial (CAUC) experiments, either on core samples or \textit{in situ} using CPT/CPTUs or other field tests (e.g. \textit{in situ} vane shear measurements). However, the measured \( s_u \) can vary due to its dependence on the mode of shear failure, anisotropy of the soil, strain rate, strain history, as well as disturbance which can affect the sample quality (Lunne et al. 1997, Low et al. 2010). Consequently, when determining the \( s_u \) from CPT/CPTUs, it is crucial to mention which records were used as reference. For example, a \( s_u \) profile derived from \( q_t \) would be correlated with records from DSS experiments.

Since beginning of the 1940s, a large number of workers have dealt with the determination and analysis of \( s_u \) for fine-grained soils using CPT and CPTU records (e.g. Terzaghi 1943, Vesic 1972, Kjekstad et al. 1978, Senneset et al. 1982). Two approaches were deemed suitable; the first approach depends on theoretical calculations and the second considers empirical correlations.

**Theoretical solutions**

Four theoretical methods can be utilized to determine \( s_u \) from \( q_c \) (Konrad and Law 1987, Lunne et al. 1997, Yu and Mitchell 1998). Owing the complexity of the penetration problem, the solutions rely on several simplifications regarding soil behavior, failure geometry in the plastic zone and boundary conditions. Some examples include assumptions of incompressibility and full saturation.
2.1. CONE PENETRATION TESTING

The first method, namely soil bearing capacity theory, was derived from the original work of Prandtl (1921) and firstly applied to several soil mechanics problems by Terzaghi (1943). For CPT tests, it was assumed that $q_c$ is proportional to the failure load of a deep circular foundation and analytically described using limit equilibrium and slip-line methods. The limit equilibrium method first assesses the failure geometry and determines the associated failure load considering global equilibrium (Fig. 2.7a). The slip-line method combines a yield criterion (e.g. Mohr-Coulomb, Misses or Tresca law) with the limit equilibrium equation to derive several differential relationships describing the plastic equilibrium of the soil mass (Yu and Mitchell 1998). A differential relationship network is used to determine the global failure load (Fig. 2.7b). However, soil bearing capacity theory neglects the effect of stress-strain behavior of soil, thus the influence of $q_c$ on the soil stiffness is highly dependent on the failure geometry in the plastic zone (Konrad and Law 1987, Lunne et al. 1997), and compressibility cannot be ascertained (Yu and Mitchell 1998). This theory is mathematically described by:

$$s_u = \frac{q_c - \sigma_{V0}}{N_c},$$  \hspace{1cm} (2.5)

where $N_c$ is the theoretical dimensionless bearing capacity factor and $\sigma_{V0}$ is the in situ total overburden stress. $N_c$ varies between 7 and 10 (Terzaghi 1943, Meyerhof 1951, Caquot and Kerisel 1956).

The second method, cavity expansions theory, was first proposed by Bishop et al. (1945) for an elastic perfectly plastic material. Using a conical punch or cone, the pressures required to generate a deep hole is equal to the internal pressure in a spherical cavity of the same volume under similar conditions. Several workers have applied this theory to the geotechnical problem of depth-dependent soil bearing capacity and further developed it by incorporating the soil stiffness and semiapex angle ($\delta$) (Gibson 1950, Meyerhof 1951, Skempton 1951, Vesic 1972). The basic form is expressed as:

$$s_u = \frac{q_c - \sigma_{V0}}{\frac{4}{3} \left(1 + \ln \frac{E}{3s_u}\right)},$$  \hspace{1cm} (2.6)

where $E$ is the Young’s modulus. Vesic (1975) mentioned that for determining $q_c$, $\sigma_{V0}$ has to be replaced by the in situ total octahedral stress ($\sigma_{OCT}$), defined as:

$$\sigma_{OCT} = \frac{1}{3}(\sigma_{V0} + 2\sigma_{H0}),$$  \hspace{1cm} (2.7)

where $\sigma_{H0}$ is the in situ total horizontal stress.
CHAPTER 2. STATE-OF-THE-ART

Figure 2.7: (a) Failure geometry and associated failure load ($q_c$) considering global equilibrium and (b) slip-line network in order to describe the plastic equilibrium (modified after Yu and Mitchell 1998). Note: penetration depth ($D_f$), diameter (B), friction angle ($\varphi$), total overburden stress condition ($\sigma_{vo}$) and parameter of the differential relationships ($\omega_1$, $\omega_2$, $\delta_1$ and $\delta_2$).

The cone penetration process has also been approached as a steady-state problem (third method). Baligh (1975) presented an energy balance, in which the work of $q_c$ per unit distance is equal to the sum of the work required to push the cone at a constant penetration rate over a unit length and the work required to expand a cylindrical cavity behind the cone. Almost all steady-state equations were developed for cohesive soils using an elastoplastic law for the surrounding material, a perfectly plastic yield criterion (e.g. Tresca) and $\sigma_{H0}$ at the associated depth (Konrad and Law 1987, Teh and Houlsby 1991, Wittle 1992). The equation is defined as:

$$s_u = \frac{q_c - \sigma_{H0}}{1.2(5.71 + 3.33\delta + \text{cotan}[\delta]) + \left(1 + \ln \left[\frac{\omega}{\omega_u}\right]\right)}$$  \hspace{1cm} (2.8)

where $G$ is the undrained shear modulus.

The fourth method combines stress-strain curves obtained from laboratory triaxial tests and numerical analyses (Ladanyi 1963, Levadoux and Baligh 1980). Steady-state flow of a nonviscous medium combined with finite element calculations were used to derive the strain path and to calculate the stress path around the cone (strain path theory). Good agreements were achieved for Boston blue clays, however, this solution is highly influenced by
the boundary conditions of the model, as well as the number and type of elements (e.g. de Borst and Vermeer 1982).

Owing to the complexity of the penetration process and simplifications made in the theoretical regarding soil behavior, stress/strain field, boundary conditions, failure mode and geometry, the theoretical solutions need to be compared with results obtained in the laboratory in order to verify the assumptions and limitations used in the theoretical solutions. Consequently, empirical correlations are generally preferred in engineering practice and scientific research, however, the theoretical solutions can be viewed as a guideline (Konrad and Law 1987, Lunne et al. 1997).

Empirical correlations

The evaluation of $s_u$ using empirical correlations was categorized into three different groups defined by the total $q_t$, effective $q_t$ and $\Delta u$ (Lunne et al. 1997). Moreover, the empirical correlation factors ($N$-factors) are highly dependent on the results of the laboratory experiments used as a reference.

The first category utilizes the so-called total cone penetration resistance equation using $q_c$, $\sigma V_0$ and the empirical cone penetration resistance factor ($N_k$) for the calculation of $s_u$, resulting in $N_k$ values between 11 and 19 (e.g. Lunne and Kleven 1981). A modification and improvement were made by using the $q_t$ instead of the $q_c$ thus, the equation is defined as follows:

$$s_u = \frac{q_t - \sigma V_0}{N_{kt}}, \quad (2.9)$$

where $N_{kt}$ is the empirical cone penetration resistance factor related to $q_t$. Aas et al. (1986) pointed out that the $N_{kt}$ is directly related to the plasticity index ($I_p$) and suggested that $N_{kt}$ increases with rising $I_p$. Figure 2.8 shows these findings for on- and offshore Norwegian clays using CAUE, DSS and CAUC experiments as reference laboratory tests. Similar observations were made by Powell and Quartermann (1988), who found that $N_{kt}$ varied between 8 and 20 taking into account CAUC tests and $I_p$ less than 50%. In addition, high-quality block samples of Norwegian sediments were analyzed using CAUC experiments to evaluate the correlation between $N_{kt}$ and pore pressure ratio ($B_q$) (Karlsrud et al. 1996). An increase in $N_{kt}$ with decreasing $B_q$ has been observed and the $N_{kt}$ values are between 6 and 15, which are somewhat lower than those mentioned before and by Lunne et al. (1985). Complementing prior studies, a comprehensive study was presented by Low
et al. (2010) focusing on statistical evaluation of a worldwide database of cone, T-bar and Ball penetration tests as well as CAUE, DSS, CAUC and in situ v-s experiments on soil specimens, taken from three onshore and 11 offshore sites. Table 2.3 presents a sample of the results focusing on $N_{kt}$ and $N_{\Delta u}$ values; the latter are discussed in a later section of this chapter. Here, the first subscript characters indicate the measured parameter (e.g. "kt" for corrected cone penetration resistance and "$\Delta u$" for excess pore pressure) and the second subscript characters indicate the reference laboratory measurement ("$s_{u,c}$" for CAUC tests, "$s_{u,ave}$" for the average value of CAUE, DSS and CAUC experiments and "$s_{u,v-s}$" for in situ v-s measurements). In summary, the $N_{kt}$ values show a narrow range of 12 to 14 with a standard deviation less than ±2 (Table 2.3, Low et al. 2010).

The second category addresses the so-called effective cone penetration resistance ($q_e$) equation, which was firstly introduced by Senneset et al. (1982) and describes the difference of $q_c$ and $u_2$. In the same year, Campanella et al. (1982) suggested using $q_t$ instead of $q_c$ which results in the following $s_u$ expression:

$$s_u = \frac{q_t - u_2}{N_{ke}},$$  \hspace{1cm} (2.10)

where $N_{ke}$ is the effective empirical cone penetration resistance factor. Senneset et al. (1982) and Lunne et al. (1985) postulated that $N_{ke}$ values vary from 1 to 13 for normally to slightly over-consolidated clays. Moreover, a
2.1. CONE PENETRATION TESTING

Table 2.3: Statistics for N-factors (modified after Low et al. 2010)

<table>
<thead>
<tr>
<th>N-factor &amp; formula</th>
<th>no. of site</th>
<th>no. of data</th>
<th>range*</th>
<th>mean</th>
<th>SD</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>$N_{kt,s_u,c} = q_{net}/s_{u,c}$</td>
<td>19</td>
<td>71</td>
<td>8.61-15.31</td>
<td>11.90</td>
<td>1.63</td>
<td>0.14</td>
</tr>
<tr>
<td></td>
<td>(14)</td>
<td>(45)</td>
<td>(9.21-13.87)</td>
<td>(11.78)</td>
<td>(1.36)</td>
<td>(0.12)</td>
</tr>
<tr>
<td>$N_{kt,s_{u,ave}} = q_{net}/s_{u,ave}$</td>
<td>22</td>
<td>92</td>
<td>10.56-17.39</td>
<td>13.56</td>
<td>1.95</td>
<td>0.14</td>
</tr>
<tr>
<td></td>
<td>(14)</td>
<td>(56)</td>
<td>(11.08-16.35)</td>
<td>(13.43)</td>
<td>(1.68)</td>
<td>(0.13)</td>
</tr>
<tr>
<td>$N_{kt,s_u,v-s} = q_{net}/s_{u,v-s}$</td>
<td>27</td>
<td>225</td>
<td>10.87-19.89</td>
<td>13.30</td>
<td>2.23</td>
<td>0.17</td>
</tr>
<tr>
<td></td>
<td>(11)</td>
<td>(87)</td>
<td>(10.91-17.53)</td>
<td>(13.45)</td>
<td>(2.07)</td>
<td>(0.15)</td>
</tr>
<tr>
<td>$N_{\Delta u,s_u,c} = \Delta u/s_{u,c}$</td>
<td>19</td>
<td>71</td>
<td>3.29-8.76</td>
<td>5.88</td>
<td>1.23</td>
<td>0.21</td>
</tr>
<tr>
<td></td>
<td>(14)</td>
<td>(45)</td>
<td>(3.29-8.76)</td>
<td>(5.96)</td>
<td>(1.44)</td>
<td>(0.24)</td>
</tr>
<tr>
<td>$N_{\Delta u,s_{u,ave}} = \Delta u/s_{u,ave}$</td>
<td>22</td>
<td>92</td>
<td>3.29-12.02</td>
<td>6.86</td>
<td>2.20</td>
<td>0.32</td>
</tr>
<tr>
<td></td>
<td>(14)</td>
<td>(56)</td>
<td>(3.29-12.02)</td>
<td>(6.87)</td>
<td>(2.34)</td>
<td>(0.34)</td>
</tr>
<tr>
<td>$N_{\Delta u,s_u,v-s} = \Delta u/s_{u,v-s}$</td>
<td>27</td>
<td>226</td>
<td>4.79-11.90</td>
<td>7.12</td>
<td>1.91</td>
<td>0.27</td>
</tr>
<tr>
<td></td>
<td>(11)</td>
<td>(88)</td>
<td>(4.92-10.88)</td>
<td>(6.93)</td>
<td>(1.70)</td>
<td>(0.25)</td>
</tr>
</tbody>
</table>

Note: * Range of average N-factor for each site.

The values in the bracket are the statistics for sites with parallel piezcone and T-bar penetration tests.

$\text{mean}$ is the average value.

$SD$ is the standard deviation.

$COV$ is the covariance.

correlation with $B_q$ was observed and confirmed by Karlsrud et al. (1996) using CAUC tests on Norwegian clays. The comparison with CAUC tests exhibits a large scatter between 2 and 9. However, in the $B_q$ diagram, a narrow range of $B_q$ values between 30 and 95% was observed (Karlsrud et al. 1996). The main disadvantage of this approach is that the difference between $q_t$ and $u_2$ is small, in particular for soft, fine-grained soils where it could be less than 10%. Consequently, this approach is more sensitive to small errors in $q_t$ or $u_2$ and using this approach to determine $s_u$ in soft, fine-grained soils is not recommended (Lunne et al. 1997).

According to the cavity expansion theory and semi-theoretical approaches, $\Delta u$ was also considered in estimations of $s_u$ and represents the third category (e.g. Vesic 1972, Campanella et al. 1985). This correlation is described by the following equation:

$$s_u = \frac{u - u_0}{N_{\Delta u}},$$

where $u$ is the pore pressure, $u_0$ is the hydrostatic water pressure and $N_{\Delta u}$ is the empirical excess pore pressure factor. Lunne et al. (1985) and Karlsrud et al. (1996) found that the $N_{\Delta u}$ for normally to slightly over-consolidated clay-rich deposits using CAUC strength as reference strength, is much smaller than 10 and decreases with decreasing $B_q$ to a value of 4 for a $B_q$ of 30%. A
large number of statistical tests confirmed these findings and indicate average $N_{\Delta u}$ values in the range of 6-7 using a worldwide database and reference strengths taken from CAUE, DSS, CAUC and \textit{in situ} v-s experiments (Table 2.3, Low et al. 2010).

2.2 Landslides and soil instabilities

Landslides are complex geological phenomena describing the downslope movements of natural or artificial materials which occur on land and in coastal, nearshore and offshore environments (e.g. Hampton et al. 1996, Locat and Lee 2000). Landslides occur due to the instability of soil masses and quantified by the factor of safety (FoS), defined by the sum of the resisting forces divided by the sum of the driving forces. For an unstable slope, FoS is equal or less than one. Over the last two decades, a diversity of styles, processes and causes of submarine landslides have been discovered (e.g. Hampton et al. 1996, Lee et al. 2009), and specific causes were identified as: (i) reduction of the soil strength (e.g. growth of gas) and (ii) increase of the \textit{in situ} stress conditions (e.g. mud emission by diapirs and mud volcanoes) (Fig. 2.9, Locat and Lee 2000, 2005).

A quarter century ago, intensive efforts were made to develop a coherent and comprehensive geotechnical system for assessing geohazard and risk assessments related to mass movements (Faure et al. 1988). This system needed to consider: (i) all types of mass movements which may occur under diverse climatic and hydraulic conditions, (ii) the variety of the geomaterials and (iii) the constitutive laws, physical and mechanical properties controlling movement. All this information was combined in a geotechnical characterization of mass movements first introduced for onland landslides by Vaunat et al. (1994) and Leroueil et al. (1996), and slightly adapted for submarine landslides by Leroueil et al. (2003). A schematic representation of the geotechnical characterization of mass movements is illustrated in the form of a 3D magic cube, where the different axes define the types of mass movements, types of geomaterials and stages of movements, described in detail hereafter (Fig. 2.10). Moreover, for each element a characterization sheet is prepared including: (i) controlling laws and parameters, (ii) predisposition factors, (iii) triggering or aggravating factors, (iv) revealing factors and (v) consequences (see Leroueil et al. 2001, 2003 for details).
2.2. LANDSLIDES AND SOIL INSTABILITIES

\[ \text{FoS} = \frac{\sum \text{resisting forces}}{\sum \text{driving forces}} \]

<table>
<thead>
<tr>
<th>reduction of the soil strength</th>
<th>increase of the in situ stress condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>natural trigger</td>
<td></td>
</tr>
<tr>
<td>earthquake</td>
<td>earthquake</td>
</tr>
<tr>
<td>wave load</td>
<td>wave load</td>
</tr>
<tr>
<td>tide</td>
<td>tide</td>
</tr>
<tr>
<td>sedimentation</td>
<td>sedimentation</td>
</tr>
<tr>
<td>gas and gas hydrate</td>
<td>glaciation</td>
</tr>
<tr>
<td>groundwater seepage</td>
<td>erosion</td>
</tr>
<tr>
<td>glaciation</td>
<td>diapir</td>
</tr>
<tr>
<td>man-made trigger</td>
<td></td>
</tr>
<tr>
<td>gas hydrate and reservoir depletion</td>
<td>excavation</td>
</tr>
<tr>
<td></td>
<td>loading</td>
</tr>
<tr>
<td></td>
<td>subsidence</td>
</tr>
</tbody>
</table>

Figure 2.9: Possible causes and triggers for submarine landslides (after Locat and Lee 2000, 2005).

### 2.2.1 Types of mass movements

Mass movements, for example landslides, occur in various forms and are strongly influenced by geological, climatic and morphological processes, thus developing a uniform and rigorous classification is difficult. Hutchinson (1968) introduced the first classification system using the material involved and morphological signature, in particular, to distinguish the mode of movement. Since the 1970s, several classification systems have been postulated for terrestrial landslides, including, for example, those of Varnes (1978), Brunsden (1979) and Hutchinson (1988). In addition, a Multilingual Landslide Glossary was published by the International Geotechnical Societies’ UNESCO Working Party for World Landslide Inventory (WP/WLI 1993, English version by D.M. Cruden) and recommended by the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) Technical Committee on Landslides (TC-11) (Cruden and Varnes 1996). This glossary presents five basic types with several subgroups applicable to submarine landslides, including turbidity currents (Fig. 2.11 Locat and Cruden 1997). It is noteworthy that all types are independent and mutually exclude one another, therefore a mass movement classified as "flow" cannot also be one of the other types.
Figure 2.10: Geotechnical characterization of mass movements in the form of a 3D magic cube (after Vaunat et al. 1994, Leroueil et al. 1996).

Figure 2.11: Types of submarine mass movements according to the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) Technical Committee on Landslides (TC-11) (after Locat and Cruden 1997).
2.2. LANDSLIDES AND SOIL INSTABILITIES

Slides

A slide is the downslope and outward movement of a mass of soil, rock or artificial material. This usually occurs along a rupture surface or a thin distinct zone of weakness, which separates the slide mass from the underlying undisturbed material (e.g. soft, sensitive clay, quick clay). Two subgroups, called translational and rotational slides, were introduced to describe the shape of the rupture or distinct zone. The translational slide mass fails along a flat surface (Fig. 2.12a, USGS 2004), while the mass of a rotational slide moves along a concave surface (Fig. 2.12b, USGS 2004).

Topples

A topple is a forward rotation, out of the slope, of a unit or units of soil, rock or artificial material about a pivotal point or axis located below the center of gravity. This type is controlled by gravity and excess pore pressures acting in adjacent cracks or fissures (Fig. 2.12c, USGS 2004).

Spreads

A spread is a lateral extension of a unit or units of soil, rock or artificial material in conjunction with a subsidence of the involved material into weaker underlying units. This type occurs normally on very even slopes. Spreads are caused by extrusion of the weaker components and liquefaction, which defined as the transformation from a solid (e.g. loose sand or silt) into a fluid, but they are not associated with localized shear displacement in a distinct zone (Fig. 2.12d, USGS 2004).

Falls

A fall is the sudden movement of soil, rock or artificial material from a steep slope or cliff. The movements happen by bouncing, falling, rolling and saltation of a mass with little or no shear displacement along a surface. The partition is induced by gravitational processes and takes place along fissures, cracks and bedding planes (Fig. 2.12e, USGS 2004).
Figure 2.12: Schematic illustrations of the different types of submarine mass movements (Locat and Cruden 1997). (a) translational slide, (b) rotational slide, (c) topple, (d) lateral spread, (e) fall and (f) flow (after USGS 2004).
2.2. LANDSLIDES AND SOIL INSTABILITIES

Table 2.4: List of flow types and their characteristics according to Varnes (1978), Cruden and Varnes (1996).

<table>
<thead>
<tr>
<th>flow type</th>
<th>movement mechanism</th>
<th>material type</th>
<th>velocity of movement</th>
</tr>
</thead>
<tbody>
<tr>
<td>debris flow</td>
<td>flow in steep canyon and gully systems</td>
<td>fully saturated non-plastic debris</td>
<td>usually &gt;1 m/s</td>
</tr>
<tr>
<td>debris avalanche</td>
<td>flow on a steep slope without confinement in canyon and gully systems</td>
<td>fully saturated non-plastic debris</td>
<td>up to 5 m/s</td>
</tr>
<tr>
<td>earthflow</td>
<td>intermittent flow-like movement developing elongate or lobate forms</td>
<td>plastic, clayey material or earth</td>
<td>less than 1.8 m/h</td>
</tr>
<tr>
<td>mudflow</td>
<td>flow in a canyon or gully with higher amount of water compared to solid material</td>
<td>at least 50% of sand-, silt- and clay-sized material</td>
<td>up to 5 m/s</td>
</tr>
<tr>
<td>creep</td>
<td>imperceptibly slow, steady, downslope movement</td>
<td>plastic, clayey material</td>
<td>centimeter or tens of centimeter per year</td>
</tr>
</tbody>
</table>

Flows

A flow is a very slow or fast continuous movement of a composite of soil, rock, water, organic and artificial material mobilized as slurry. The slurry unit consists of a large number of shear surfaces, which are short-lived, closely spaced and frequently preserved, and its behavior is similar to a viscous fluid (Fig. 2.12f, USGS 2004). Five different categories were observed depending on their movement mechanisms, material properties and velocities of the movements (Table 2.4, Varnes 1978, Cruden and Varnes 1996, Hungr et al. 2001).

2.2.2 Types of geomaterials

The physical and mechanical behavior of the slope mass depends on the both the type of material and in situ conditions (e.g. Leroueil et al. 2001). Vaunat et al. (1994) postulated ten different types of onland mass movements
Figure 2.13: Decision tree illustrates different types of geomaterials according to the geotechnical characterization (after Leroueil et al. 2003).

including several rock, soil, intermediate and residual classes. Leroueil et al. (2003) modified this classification system for submarine conditions replacing unsaturated soil classes by gassy soils (Fig. 2.13).

2.2.3 Stages of mass movements and their development

For onland landslides, Vaunat et al. (1994) and Leroueil et al. (1996) first proposed four different stages of mass movements designated as pre-failure, failure, post-failure and reactivation. Leroueil et al. 2003 suggested similar taxonomy for submarine landslides, which was defined by the chronology of the displacement rate of mass movements (Fig. 2.14).

Beginning with the pre-failure stage, all physical and mechanical processes result in a decrease of the FoS, i.e. the driving forces approach the resisting forces (e.g. D’Elia et al. 1998). The displacement rate is relatively low at the beginning of a mass movement, and it is controlled by: (i) soil or rock mechanical properties and behavior, (ii) accumulated strains and strength degradation associated with creep failure, and (iii) variations in stress conditions (e.g. Leroueil et al. 2001, Silva and Booth 1984). However, when approaching failure, a slight increase in displacement rate is observed as a consequence of the generation of microcracks and isolated shear-zones. The continuous enlargement of the shear-zones though time leads to a fully
developed shear surface over the entire slope mass. This shear surface combined with a FoS equal to unity describes the onset of mass movement at the failure stage, i.e. a slope failure is on the verge of occurring.

The failure mechanism is characterized by the formation of individual shear surfaces, called Riedel shears and thrust shears, as well as displacement discontinuities (Riedel 1929, Skempton and Petley 1967). From a mechanical point of view, this mechanism depends on the effective stress-shear strength relation, or failure envelope (e.g. Mohr-Coulomb; Leroueil 2001).

The failure stage is followed by the post-failure stage which is signaled by movement of the slope mass. The displacement rate starts to significantly increase and is followed by a large amount of the mass mobility. Petley et al. (2005) developed a novel conceptual approach describing all different stages of a progressive mass movement in cohesive materials. Figure 2.15 illustrates this model in terms of the evolution of the FoS, simple description of the shear-surface and interaction between the reciprocal of the landslide displacement rate ($\Lambda$) and time to describe quantitatively the initial failure of a shear-surface mass movement. The behavior of the moving mass depends on the physical and mechanical properties of the soil and the size of the potential energy, which leads to reworking and disturbance processes if high enough. Consequently, the moving mass may become a flow and behave as a viscous fluid, requiring the introduction of a rheological model for the analysis (e.g. Locat 1997). The post-failure stage is also influenced by the
Figure 2.15: Novel conceptual approach for the development of progressive mass movement failure in cohesive soils. Plots dealing with the factor of safety (SF) against time are illustrated for each step. In addition, illustrations shows the state of development of the shear-surface (upper) and reciprocal of the landslide displacement rate (Λ) - time (t) space (lower). (a) The development of microcracks is initiated, however, not shear-surface are generated, (b) the formation of the shear surface starts according to the decrease of the SF, (c) stress concentration and acceleration of mass movement due to growth of the shear surface and a clear linearly decreasing trend can be seen in the Λ-t space and (d) shear surface is fully developed and the SF is equal unity. (after Petley et al. 2005).

distribution of activity, for example, the extension of an initial mass movement (e.g. retrogressive landslide; Bjerrum 1955, Quinn et al. 2007).

As a complement to the three stages introduced above, the fourth stage describes the reactivation of mass movements on pre-existing shear surfaces. The material properties and behavior are dominated by the residual sliding
for soils that have undergone large displacements. Consequently, the dis-
placements are rather low compared to the post-failure stage (see Fig. 2.14, D’Elia et al. 1998).

2.3 Slope stability assessment

Before introducing different engineering methods appropriate for assessing the stability of natural or engineered slopes, the individual terms in the expression slope stability need to be described according to the definitions of Kliche (1999):

The term slope may be defined as any inclined surface cut in natural material or as the degree of inclination with respect to horizontal. ... The term stability may be defined as the resistance of a structure, slope, or embankment to failure by sliding or collapsing under normal conditions for which it was designed; e.g. bank stability and slope stability. Hence, slope stability may be defined as the resistance of any inclined surface, as the wall of an open cut or cut, to failure by sliding or collapsing.

Once the geometrical and stratigraphic conditions and appropriate soil properties have been determined and evaluated, slope stability assessments are currently carried out using almost exclusively limit equilibrium (e.g. single-free-body and slices procedures) and numerical techniques (e.g. continuum, discontinuum and coupled modeling) in contrast to previously used graphical approaches such as charts and tables (e.g. Kliche 1999, Griffiths and Lane 1999). Limit equilibrium and numerical techniques usually require complex computer programs which are applied to: (i) detect safe areas, (ii) explore possible failure modes and mechanisms, (iii) fulfill requirements for an engineering structure regarding reliability and economics, and (iv) design appropriate retaining measures for deep excavations and embankments (e.g. Kliche 1999).

2.3.1 Limit equilibrium technique

The limit equilibrium technique is based on the fulfillment of static equilibrium conditions, i.e. (1) equilibrium of the sum of all vertical forces, (2) equilibrium of the sum of all horizontal forces and (3) equilibrium of the sum
of all moments (e.g. Duncan and Wright 2005). For each equilibrium condition, associated equations were formulated in order to establish the FoS for a single-free-body or the full slope using several slices (e.g. Taylor 1948, Bishop 1955). Consequently, two different limit equilibrium groups are common in research and engineering practice, known as the single-free-body procedure (e.g. infinite slope, logarithmic spiral, Swedish slip circle) and the procedure of slices (e.g. ordinary method of slices, simplified Bishop, force equilibrium) (see Duncan and Wright 2005 for details). Here, the infinite slope and simplified Bishop procedures are presented as examples of the general formulation of each group.

The infinite slope procedure considers a single-free-body at equilibrium shown in Figure 2.16 and assumes that: (i) the plane of the slope is infinitely long, and (ii) mass movement occurs along a slip plane downslope (e.g. Taylor 1948). From the equilibrium equations considering the Mohr-Coulomb constitutive law and assuming full saturation of the soil mass, the FoS is:

\[
\text{FoS} = \frac{c' + (\gamma'z\cos^2[\alpha] - \Delta u)\tan[\varphi']}{\gamma'z\cos[\alpha]\sin[\alpha]},
\]

where \(c'\) is the effective cohesion, \(\gamma'\) is the \textit{in situ} submerged unit weight of the soil, \(z\) is depth below seabed, \(\alpha\) is the slope angle, and \(\varphi'\) is the effective friction angle (e.g. Dugan and Flemings 2002). Moreover, the soil strength is influenced by the state of consolidation (i.e. drained and undrained conditions) during the shear process. Equation 2.12 satisfies drained conditions, i.e. the soil mass had sufficient time for drainage of the pore-water. It can also be adapted for undrained conditions, i.e. for a certain amount of excess pore pressure, using \(\varphi'\) equal to zero and replacing \(c'\) by \(s_u\). For further consideration of the variations in pore pressure, seepage conditions and earthquake loading in the infinite slope procedure, the reader is referred to Bishop and Morgenstern 1960, Hampton et al. 1996, Duncan and Wright 2005, Ten Brink et al. 2009.

Using the simplified Bishop procedure, the slope mass above the shear surface is divided into several vertical segments assuming a circular shear surface, in contrast to an arbitrary (noncircular) shear-surface implemented in other procedures (Fig. 2.17 Bishop 1955). For all individual segments, the FoS is calculated considering the Mohr-Coulomb constitutive law, full saturation of the soil segment, and equilibrium of moments and effective
2.3. SLOPE STABILITY ASSESSMENT

![Diagram of a slope unit with forces and stresses](image)

Figure 2.16: Schematically illustration of an infinite slope unit and associated forces, for example weight \( W \), normal and resisting force (e.g. Taylor 1948). The dashed arrows cancel each other. **Note:** slope angle \( \alpha \) and depth below the seafloor \( z \).

The stresses:

\[
FoS = \frac{\sum \left( c' \Delta l \cos[\alpha] + (W_i' - \Delta u \Delta l \cos[\alpha]) \tan[\varphi'] \right)}{\cos[\alpha] + \frac{\sin[\alpha] \tan[\varphi']}{Fos}}, \quad (2.13)
\]

where \( \Delta l \) is the length of the segment base, and \( W_i' \) is the submerged weight of each individual segment. Moreover, the procedure of slices allows the implementation of additional driving and resisting forces. The additional driving forces may be loading from traffic or deposited/stockpiled soil and pseudostatic loads from seismicity (e.g. Seed 1979, Duncan and Wright 2005) while further resisting forces may be reinforcement and anchoring measures (e.g. Wright and Duncan 1991).

In summary, limit equilibrium techniques are common in engineering practice and research. However, several assumptions need to be made, such as the shear surface geometry, constitutive laws and basic equilibrium equations used to calculate the FoS. Moreover, complications can arise due to the model geometry and stratigraphy, neglecting stress conditions and deformability, and implementation of external loads (e.g. Duncan 1996).
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Figure 2.17: Schematically illustration of an individual segment of the simplified Bishop procedure and associated forces, for example weight, normal and resisting force (e.g. Bishop 1955). The dashed arrows cancel each other. **Note:** orientation of the slide circle \( O \), radius of the slide circle \( r \), submerged weight of each individual segment \( W'_i \), slope angle \( \alpha \) and the length of the segment base \( \Delta l \).

2.3.2 **Numerical technique**

More complex and advanced slope stability assessments can be performed using continuum, discontinuum and hybrid/coupled numerical techniques (e.g. Griffiths and Lane 1999, Eberhardt 2003). These techniques allow the inclusion of: (i) complex geometrical conditions, (ii) anisotropy of the material, (iii) non-linear material behavior, (iv) *in situ* stress variability, and (v) coupling processes, such as pore pressure distribution and seismic loading (e.g. Eberhardt 2003).

Continuum numerical modeling is best suited for slope stability problems in rock environments with a very high number of discontinuities, intact rock, weak rock and mainly soil conditions. Clough and Woodward (1967) firstly presented finite element models to evaluate the stability and performance of slopes and embankments using nonlinear stress-strain relationships. Finite element and finite difference approaches discretize the entire slope mass (Fig. 2.18) and these elements obey continuum mechanics principles. In continuum mechanics, an element is quantified by the internal stresses \( \sigma \), external loads \( p \), internal strains \( \varepsilon \) and external displacements \( u_{\text{disp}} \), which are formulated by the following three differential equations:

\[ L^T \sigma + p = 0, \]  

(2.14)
2.3. SLOPE STABILITY ASSESSMENT

Figure 2.18: Examples of different types of finite elements common in numerical modeling. (a) 2D triangle elements with 3 and 6 nodes, (b) 2D rectangle elements with 4 and 8 nodes and (c) 3D box elements with 8 and 20 nodes.

\[ \varepsilon = Lu_{disp}, \]  
(2.15)

\[ \sigma = D\varepsilon, \]  
(2.16)

where \( L \) is the differential operator, \( L^T \) is the transpose of a differential operator, and \( D \) is the constitutive law matrix (e.g. elasto-plasticity, strain-softening and hardening, elasto-viscoplasticity) (e.g. Duncan 1996). The differential equations are solved for each integration point (see points in Fig. 2.18) using Gauss or fast Lagrange integration methods (e.g. Malvern 1969, Brinkgreve 2002).

Discontinuum and hydrid/coupled numerical models were developed to simulate slope movements and failure caused by discontinuity properties and behavior, flow processes in channel structures and gullies, and high groundwater pressure on soil and weak rock slopes (e.g. Eberhardt et al. 2002, Eberhardt 2003).

In summary, the advantages of numerical techniques compared to limit equilibrium techniques are:

- shear surface shape and location are automatically generated depending on the stress-strength relationship,

- the slope mass is not subdivided in several slices, thus no solution related to the forces acting at the shear-surface is required,
• deformation, consolidation, swelling of the slope mass and potential construction stages are considered in a realistic manner, and

• progressive failure and overall stability are realistically simulated (i.e. location of the initial development of the shear-surface).

Finally, it is important to note that numerical techniques are a time- and effort-consuming process that requires detailed knowledge of the technique (e.g. mesh and boundary effects, used constitutive law), and the soil properties and behavior from advanced laboratory tests (e.g. one-dimensional compression (CT), DSS and CAUC experiments) in order to avoid misinterpretation of the results (e.g. Duncan 1996, Griffiths and Lane 1999).

2.4 Mud volcanoes and pockmarks

2.4.1 Mud volcanoes

Mud volcanoes, diatremes and mud diapirs are well-known geological phenomena occurring in on- and offshore environments and have been explored in many places around the world (e.g. Higgins and Saunders 1974, Robertson 1996, Kopf 1999). Kopf (2002) published a detailed summary of the "significance of mud volcanism" including the distribution of MVs on earth and nature of these geological phenomena, for example geometrical conditions related to the mechanisms of eruption, models of mud diapirs, mud flux and fluid discharge, source of gaseous, aqueous and solid phases, and quantitative aspects. Moreover, Kopf (2002) also presented a glossary that includes all relevant terminology with respect to the MVs, diatremes and mud diapirs (Fig. 2.19 Kopf 2002), which are used in this doctoral thesis.

Mud volcanoes: Surface expression of mud that originated from depth. Depending on the geometry of the conduit and the physical properties of the extrusive, the feature may be a dome (cone; see Figure 3a of Kopf 2002) or a pie with low topographic relief.

Diatreme: Type of mud extrusive feature that evolved from a violent eruption of overpressured mud, cross-cutting the overlying strata like a dyke.

Mud diapir: Intrusive body of shale or clay that does not reach the surface.
2.4. MUD VOLCANOES AND POCKMARKS

Figure 2.19: Schematic illustration of a mud diapir, mud volcano extrusions and diatremes. Possible fluid sources are also mentioned (numbers 1 to 8) (after Kopf 2002).

Offshore MVs usually occur in compressional tectonic settings (i.e. convergent margins), which provide a "tectonic window" into the deeper parts of the earth's crust (e.g. subduction zones) and transport gaseous, fluid and solid composites from several kilometers depth to the surface of the earth. Conventional drilling techniques are usually expensive, time-consuming and difficult to implement in such deep-water environments (e.g. Robertson 1996). Consequently, the installation of long-term observatories, monitoring fluid discharge and temperature, sampling and in situ testing of undisturbed fluids, gases and clay-rich soils are well-suited for improving global mass flux estimates and understanding the link between seismicity and mud volcano emission (e.g. Kopf 1999, 2002, Kopf et al. 2012a, 2012b).
2.4.2 Pockmarks

In the mid-1960s, pockmarks were firstly observed at the Emerald and Roseway basin (Nova Scotia, Canada) and described as V-shaped indentations located along the seabed using sub-bottom profiler data (e.g. King 1967). King and MacLean (1970) performed detailed investigations in this area (Emerald Basin at 43°45’N and 62°45’W [Nova Scotia, Canada]) using side-scan sonar records and stated that:

The term **pockmark** is applied herein to concave, crater-like depressions that occur in profusion on mud bottoms across the Scotian Shelf.

Pockmarks can occur both individually or in groups (Fig. 2.20) and are linked with deep-seated stratigraphical units and characteristically host fluid flow and/or gas venting processes (e.g. Hovland et al. 2002). Several studies show the occurrence of pockmarks upslope in undisturbed soil successions and/or adjacent to landslide-prone regions (e.g. Gardner et al. 1999, Baraza et al. 1999). Consequently, the pressure distributions measured with short-term and long-term monitoring systems have the potential to act as stability indicators for submarine slopes (e.g. Hovland et al. 1998, 2002). Moreover, several observations, mainly visual in nature, suggest that pockmarks are activated prior to and during earthquakes, characterized by an increase in the rates of fluid flow and/or gas venting (e.g. Dando et al. 1995, Hasiotis et al. 1996). Consequently, an observation of a pockmark field in an earthquake-prone area may be a key to better prediction of earthquake occurrence (e.g. Hovland et al. 2002).

To summarize, sampling and **in situ** investigations at regular intervals or using long-term instrumentation are an important way to obtain a better understanding of the triggers and causes of submarine landslides, and the connection between seismicity and the activity of such geological phenomena.
Figure 2.20: Schematic picture of possible types of pockmarks (after Hovland et al. 2002).
Chapter 3

Dynamic piezocone penetration testing

This doctoral thesis deals mainly with three different MARUM dynamic-CPTU instruments (one deep-water and two shallow-water systems) firstly developed in 2005 and 2006 (Table 2.2), and introduced by Stegmann et al. (2006), Stegmann and Kopf (2007). Since this time, several technical modifications have been carried out and additional tools (e.g. fast speed winch) have been developed which modify the configuration of the instruments, described hereafter in order to present the most current status.

3.1 MARUM dynamic-CPTU instruments

3.1.1 Shallow-water instrument

Since 2011, MARUM has operated two lightweight dynamic-CPTU instruments for use in shallow water environments (SWFF-CPTU) up to a WD of 500 m. They consist of an industrial 15 cm$^2$ piezocone (dynamic-CPTU cone) and an aluminum pressure-tight housing containing a microprocessor (Avissaro AG), logging unit, including a standard secure digital memory card (SD), power supply (battery package), tiltmeter, accelerometer and data communication unit using a RS 485 interface (Fig. 3.1b). The pressure-tight housing is designed to withstand a confining pressure of up to 6 MPa (approx. 600 m WD). Four different dynamic-CPTU subtraction cones, constructed by Geomil equipment B.V., can be deployed housing strain gauges and pressure sensors with capacities of 25 to 100 MPa for $q_c$, 0.25 to 1 MPa for $f_s$ and 1 to 5 MPa for $u$. In addition, pore pressure can be measured at the tip ($u_1$)
and behind the tip \((u_2)\) using exchangeable 60° conical tips and filter rings made of sintered metal (Fig. 3.1b). The accuracy of the dynamic-CPTU parameters is better than \(\pm 10\) kPa based on the technical specification of the piezocone (accuracy class 1; ISO 22476-1). An Analog Device - dual-axis inclinometer is installed which can record data within \(\pm 45^\circ\) relative to vertical during the penetration process. Five different accelerometers from Analog Device - iMEMS with different ranges, for instance \(\pm 1.7\) g, \(\pm 18\) g, \(\pm 35\) g, \(\pm 50\) g and \(\pm 120\) g, monitor the deceleration behavior of the dynamic-CPTU instrument during the deployment (Stegmann et al. 2006). The frequency of data acquisition is variable and can be adjusted depending on the operation purpose of the dynamic-CPTU instruments. For sub-seafloor profiling, a logging rate of 1 kHz is commonly used and long-term pore pressure dissipation tests (e.g. over-night dissipation tests) are recorded at a frequency of 10 Hz. The maximum error in penetration depth is estimated to be 2% based on the accuracy of the accelerometers. Consequently, the vertical resolution of CPTU profiles is better than 1 cm for penetration rates of less than 10 m/s using a 1 kHz logging rate. The recorded binary data are stored on a SD memory card with a capacity of 8 GB, and downloaded and processed after the deployment on a personal computer (PC) with MATLAB routines (Stegmann et al. 2006). The data analysis and post-processing are described in detail in section 3.2. The three non-volatile battery packs and the capacity of the SD memory card provide can operate for up to 24 hours.

The length of the dynamic-CPTU instruments can be adjusted from 0.5 m to a maximum length of 8.5 m depending on what type of soil is anticipated. Metal rods with 1 m length containing data/power umbilical cables are utilized for the extension. Hence, the weight of the dynamic-CPTU instruments range from approx. 40 kg to 140 kg. In order to achieve deeper targets, modular 15 kg weight pieces can be fixed to the top of the pressure-tight housing, increasing the maximum weight to 200 kg for the entire device (Fig. 3.2). The dynamic-CPTU instrument is deployed as an individual measurement or pogo-style, and remains in the sub-seafloor for approx. 15 minutes or longer to obtain dissipation records (Stegmann et al. 2006). Moreover, for the shallow-water instruments, a data communication system in conjunction with a high-speed, velocity-controlled hydraulic winch was developed in order to facilitate real-time data transfer between the dynamic-CPTU instrument resting in the surficial soil, and the deck-interface unit on the vessel or platform. Consequently, the remaining dissipation time can be adjusted depending on the pore pressure signal, which reaches a steady state condition when fully dissipation is achieved. A schematic illustration of this communication/telemetry system is presented in Figure 3.3b, and a comparable sys-
3.1. MARUM DYNAMIC-CPTU INSTRUMENTS

Figure 3.1: (a) Aluminum pressure-tight housing and main electronic of the MARUM shallow-water dynamic-CPTU instrument (SWFF-CPTU) are presented (see also Stegmann et al. 2006, Steiner et al. 2012 for details). (b) The SWFF-CPTU cone with two pore pressure configurations ($u_1$ - port at the tip and $u_2$ - port behind the tip) are also illustrated. The measured cone penetration resistance ($q_c$), measured sleeve friction ($f_s$) and measured pore pressure ($u_1$ and $u_2$) are dynamic parameters according to the strain-rate effect (see section 3.2) (copyright MARUM).

Since beginning of 2012, a high-speed, velocity-controlled hydraulic winch with an associated diesel power unit, designed and constructed by NCS Hydraulics, has been used to deploy both shallow-water instruments (Fig. 3.4). The lightweight design of the winch and power unit (700 kg for the winch and 900 kg for the power unit) allows transport via small tracks and operation using medium-size vessels (e.g. ≥25 m length). The winch is capable of supporting a force of 40 kN using a 10 mm steel wire rope with an inner conductor, for use in a maximum WD of 200 m. The inner conductor is used for the real-time data transfer between the dynamic-CPTU instrument and deck-interface unit using RS 485 communication protocol. However, the main advantage of this winch, compared to standard winches which are fixed on different research vessels, is the maximum achievable penetration rate.
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Figure 3.2: The SWFF-CPTU instrument is shown including the main components and SWFF-CPTU piezocone.

...of 6 m/s in contrast to conventional 1.5 m/s. Consequently, fully velocity-controlled deployments can be executed allowing a wider range of surficial exploration targets of interest.

3.1.2 Deep-water instrument

The dynamic-CPTU instrument for deep-water (up to 4000 m WD) marine use (DWFF-CPTU) consists of an either industrial or self-designed 15 cm$^2$ piezocone and a stainless pressure-tight housing containing a micro-processor unit (Avisar AG), logging unit with SD memory card, battery package, tiltmeter, accelerometer, absolute and differential pore pressure sensors, and a power and data interface module (PDIM) (Fig. 3.5a). The pressure-tight housing is constructed to withstand 45 MPa confining pressure (~4500 m WD). Three different types of dynamic-CPTU cones are in use, which contain two self-designed dynamic-CPTU cones; the first one records only $u$, the second measures $q_c$ and $u$, and a third industrial dynamic-CPTU cone, developed by Geomil equipment B.V., measures $q_c$, $f_s$ and $u$ (Fig. 3.5b). However, the industrial piezocone is limited to an operational WD of 1000 m and the self-designed piezocones can achieve 4000 m. All DWFF-CPTU cones are equipped with two pore pressure ports, which can be designated $u_1$ or $u_2$ by utilizing exchangeable 60° conical tips and $u_3$ (i.e. port behind the sleeve), which are connected to differential pore pressure transducers (VALI-
Figure 3.3: Schematic illustration of the communication/telemetry systems used for the MARUM deep-water (a) and shallow-water instruments (b). The deep-water telemetry system consists of a deck unit (Seabird Electronics - SBE36 CTD) and power and data interface module (PDIM) while the shallow-water communication system contains a newly developed telemetry unit. The shallow-water instrument can be operated with an associated high-speed, velocity-controlled hydraulic winch.

DYNE P55D) via stainless steel tubes having an inner diameter of 2 mm. The strain gauges measuring the CPTU parameters are capable of recording stresses and pressures between 25 and 45 MPa for $q_c$ and 0.25 MPa for $f_s$. The accuracy of the parameters range within ±10 kPa according to the technical specification of the piezocone (accuracy class 1; ISO 22476-1). The reference pore pressure port at the pressure-tight housing is equipped with a maximum 40 MPa (400 bar) water pressure sensor (Wika ECO-1). Differential pore pressure changes can be monitored over a range of 85 kPa (12.5 PSI) to 140 kPa (20 PSI) with a resolution between 8 and 15 Pa (Fig. 3.6). The pressure transducers are protected with valves in case high excess pore pressures are encountered (e.g. owing to blocked hydraulic tubes). They are further used to bleed the tubing if gas becomes trapped inside, for example during the initial phase of deployment when the instrument is lowered through the water column (Stegmann and Kopf 2007). The configuration of the tiltmeter
and accelerometer, as well as the data post-processing and analysis, are the same as that of the shallow-water dynamic-CPTU instruments and therefore have a similar maximum error in penetration depth and vertical resolution (see sections 3.1.1 and 3.2 for details). The dynamic-CPTU instrument is used in an autonomous mode, at which all sensor and transducer records are saved on a SD-memory card with a very high sampling rate of 1kHz and are downloaded via W-LAN to a PC after the operation. The two available non-volatile battery packs provide performance lifetimes of about eight to 12 hours, respectively. A self-developed deck interface box is used to download the recorded data and to charge the battery packs. In addition, a data transmission telemetry system (Seabird Electronics) is used to monitor all sensor and transducer parameters on board the research vessel in real time (for a lower frequency of 1Hz). The telemetry system consists of a deck unit (SBE36 CTD) and a PDIM and is schematically illustrated in Figure 3.3a. It provides real-time data acquisition and control of the instrument (e.g. operation of the valves) via PC using self-developed LabVIEW control software.

The length of the deep-water dynamic-CPTU instrument varies from 4 m to a maximum length of 7 m depending on the soil type (Fig. 3.7). Extension is accomplished by adding 1.5 m long metal rods, internal data/power umbilical cables and steel tubing. The weight of the instrument ranges from
3.1. MARUM DYNAMIC-CPTU INSTRUMENTS

Figure 3.5: (a) Stainless pressure-tight housing, main electronic, power and data interface module (PDIM) of the MARUM deep-water dynamic-CPTU instrument (DWFF-CPTU) are presented (see also Stegmann and Kopf 2007). (b) The industrial and self-designed DWFF-CPTU piezo cones with three pore pressure ports ($u_1$ - port at the tip, $u_2$ - port behind the tip and $u_3$ - port behind the sleeve friction) are also illustrated. The measured cone penetration resistance ($q_c$), measured sleeve friction ($f_s$) and measured pore pressure ($u_1$, $u_2$ and $u_3$) are dynamic parameters according to the strain-rate effect (see section 3.2) (copyright MARUM).

Figure 3.6: The differential pore pressure transducers and associated valves (VALIDYNE), battery package and absolute water pressure sensor (WIKA) are shown.
~500 kg to 550 kg, and the deployment modes are the same as that of the shallow-water devices. However, the deep-water instrument can only be deployed at a winch speed of 2 m/s for safety reasons.

### 3.2 Post processing of dynamic-CPTU records

The post-processing and analysis of dynamic-CPTU measurements are more demanding due to the instrument’s autonomous nature (i.e. dynamic-CPTU parameters that depend on time are continuously recorded during multiple deployments), difficulty in choosing the appropriate starting and ending points of an individual deployment, and the need for strain-rate corrections (i.e. divergence between static-CPTU and dynamic-CPTU measurements). Consequently, a coherent and comprehensible post-processing and analysis procedure is required in order to obtain adequate results useful for geological and geotechnical purposes (e.g. landslide research, offshore engineering design). Such a procedure requires four main tasks: raw data processing, dynamic data analysis, \textit{in situ} strain-rate correction and geotechnical analysis, in order for the dynamic-CPTU raw data to be relevant to fundamental geological processes and geotechnical procedures; for instance, sub-seafloor
3.2. POST PROCESSING OF DYNAMIC-CPTU RECORDS

modeling and slope stability risk assessment.

3.2.1 Raw data processing

The MARUM instruments store digital electrical dynamic-CPTU raw data, such as the electrical records of the forces, pressures, decelerations and tilts, on SD memory cards as single, consecutive log-files (i.e. binary data) representing a 15-minute-long dataset in order to facilitate effective and safe data handling (Fig. 3.8). The log files are converted to ascii-files (i.e. numerical data) using an internally developed LabVIEW routine (Stegmann et al. 2006). Standard and advanced spreadsheet or programming routines (e.g. Excel, MATLAB) are best suited for the further data processing, which is schematically shown in Figure 3.8.

Several ascii files, for example, individual sub-seafloor profiles with dissipation measurements, are loaded into such a routine and then an offset correction is applied for all recorded, electrical values. The offset correction removes error in the initial state of the dynamic-CPTU parameters, e.g. such that the strain gauges and pressure transducers measure zero at the sealevel, and the instrument is vertically oriented for the deceleration and inclination. This zeroing procedure is followed by a calibration of the recorded electrical values taking into account the sensor and transducer specification and calibration equations (i.e. the digital electrical dynamic-CPTU raw data [e.g. millivolt] is converted into forces [kN], pressures [kPa], decelerations [g] and inclinations [°], known as mechanical dynamic-CPTU raw data). The me-

![Figure 3.8: Schematic illustration of the process from the digital, electrical dynamic-CPTU raw data to the mechanical dynamic-CPTU raw data, named as raw data processing.](image-url)
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Mechanical dynamic-CPTU raw data are the fundamental basis for the dynamic data analysis of an individual dynamic-CPTU deployment, discussed hereafter.

3.2.2 Dynamic data analyses

The main goal of dynamic data analysis is to obtain accurate geological and geotechnical sub-seafloor profiles using the MARUM instruments (Fig. 3.9). The mechanical dynamic-CPTU raw data depend on measurement time, however, the time period of a single deployment has to be established (i.e. choice of the penetration start and stop time). The penetration start time is defined as the moment at which the instrument hits the sub-seafloor; the penetration stop time is defined as the moment at which terminal depth is achieved. The correct choice is crucial for the dynamic data analysis, consequently, a direct correlation between $q_c$, $u$ and acceleration records are required to analyze transient changes in the parameter gradients in order to pick the correct penetration start and stop times. The deceleration records within this time period allow the researcher or engineer to calculate the penetration rate and depth by 1st and 2nd integrations over time (Fig. 3.10, Stegmann et al. 2006). In order to verify the initial penetration rate and terminal depth, a visual direct comparison with depth derived from a photo documentation during the cruise, combined with records of the winch velocity when using a winch mode deployment, is recommended. In case significant differences between calculated and documented values are detected due to incorrect choice of the penetration start and stop time, accelerometer defect, or wrong calibration and offset correction, a detailed analysis and debugging are required. Such checks have to be carried out as long as the differences are significant, however, if no mistake is found these data are to be taken with caution or should be excluded (see Fig. 3.9). After this iterative process, combining the depth profile with the force and pressure records results in the primary dynamic-CPTU profiles (Fig. 3.10). As an additional consideration, the different sensors and transducers of the piezocone are mobilized at different locations (i.e. during a deployment the $q_c$ will firstly be activated, followed by $u$ and $f_s$). This geometrical difference has to be corrected by including an offset shift on the order of $\leq 4$ cm for $u$ (related to the pressure port location) and $\sim 10$ cm for $f_s$, but need to be selected for each deployment according to the start time of mobilization (i.e. point at that the sensor or transducer will be mobilized by the soil).

The secondary dynamic-CPTU profiles or parameters are derived from
3.2. POST PROCESSING OF DYNAMIC-CPTU RECORDS

Figure 3.9: Schematic representation of the procedure to obtain from the mechanical dynamic-CPTU raw data the secondary dynamic-CPTU profiles, named as dynamic data analysis.

The primary dynamic-CPTU profiles using state-of-the-art geotechnical solutions, for instance, the dynamic corrected cone penetration resistance \( \left( q_{c,dyn} \right) \), dynamic excess pore pressure \( \left( \Delta u_{dyn} \right) \) and dynamic in situ intact undrained shear-strength \( \left( s_{u,dyn} \right) \). Due to the "inner" geometry of the dynamic-CPTU cone, the pore pressure effect on dynamic cone penetration resistance \( \left( q_{t,dyn} \right) \), or the so-called unequal area effect, is considered in an expression for \( q_{t,dyn} \):

\[
q_{t,dyn} = q_{c,dyn} + u_{dyn}(1 - a),
\]

(3.1)

where \( a \) is the net area ratio, assumed here to be 0.6-0.7 (Campanella et al. 1983, Roberson 1990, Lunne et al. 1997). The \( \Delta u_{dyn} \) and \( s_{u,dyn} \) are based on the solutions presented in section 2.1.4. In conclusion, the use of secondary dynamic-CPTU profiles allows the researcher or engineer to perform preliminary geological and geotechnical analyses, such as the detection of specific targets (e.g. weak layer), and gives an overview of soil homogeneity and ordinary soil properties.
Figure 3.10: (a) Example of a deceleration record (decel), and associated penetration rate ($v_{dyn}$) and depth profiles obtained from the double integration of the deceleration over time, and (b) combination of the depth profile and the dynamic corrected cone penetration resistance ($q_{t,dyn}$) resulting in a sub-seafloor geotechnical profile.

### 3.2.3 In situ strain-rate correction

The secondary dynamic-CPTU profiles and parameters are not appropriate for detailed sedimentological and geotechnical analyses, and cannot be utilized for ground modeling or slope stability and risk assessment due to the strain-rate effect introduced in section 2.1.3. Consequently, a proper strain-rate correction solution has to be selected which includes SSCs appropriate for the corresponding CPTU parameters and the prevalent soil in the survey area (Fig. 3.11). Due to the high variability of soil conditions at the different sites on earth, at several locations, quasi-static CPTU profiles have to be directly compared with static-CPTU profiles collected with a conventional CPTU equipment and/or fc, v-s, DSS, CAUE or CAUC experiments performed in the laboratory on soil specimens. If the comparison between each
3.2. POST PROCESSING OF DYNAMIC-CPTU RECORDS

![Diagram](image)

**Figure 3.11:** Schematic illustration of the process to calculate the primary quasi-static CPTU profiles from the secondary dynamic-CPTU profiles, known as *in situ* strain-rate correction.

In *in situ* tests and laboratory experiments using samples obtained at the same location show equal results, the strain-rate correction and associated SSCs can be considered adequate, and the profiles obtained from dynamic-CPTU tests are defined as primary quasi-static CPTU profiles. In case of differences between the test results, the SSCs have to be modified to obtain the best possible fit between the datasets using statistical tests in order to verify and validate the new SSCs. This iterative procedure is necessary due to the complexity and heterogeneity of the soil conditions, meaning that a definition of a uniform strain-rate correction solution is likely insufficient.

In summer of 2010, I received an offer to participate on a research cruise with the R/V *Seisma* to the *Finneidfjord* landslide-prone area (northern Norway). This cruise was funded through the SEABED project (*Norwegian DeepWater Programme*) and the International Centre for Geohazards (*ICG*) in close cooperation between the parties involved (*NGU*, *NGI*, *UiB*, and *MARUM*). In total, 38 dynamic-CPTU tests at 21 sites were performed. Almost all dynamic-CPTU deployments, v-s and DSS experiments on cored specimens were taken to obtain a better understanding of the strain-rate effect observed in data obtained with *MARUM* instruments in clayey soils. The findings were presented in a research article in the *Canadian Geotechnical Journal*. 
**In situ** dynamic piezocone penetrometer tests in natural clayey soils - a reappraisal of strain-rate corrections

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In Review: *Canadian Geotechnical Journal*

**Abstract**

Cone penetration testing with pore pressure measurements (CPTU) is a cost and time efficient way of collecting *in situ* geotechnical parameters of near-surface marine soils for cable and pipeline tracks, offshore foundations, and geohazard identification. The measured dynamic-CPTU parameters (cone penetration resistance, sleeve friction, pore pressure) are higher than the measured static-CPTU parameters. This mismatch is caused by the different penetration rates used for dynamic- and static-CPTU tests (the dynamic tests have up to five hundred times higher penetration rates than the static tests; i.e. using a commonly 2 cm/s penetration rate). This study presents comprehensive *Calypso* piston-and gravity core as well as dynamic- and static-CPTU datasets acquired in the landslide-prone Sørfjorden area (Finneidfjord, northern Norway). The fjord-marine sediments at the study site are characterized as normally- to slightly over-consolidated clay-dominated soils with embedded layers of sandy silt to sand. The dynamic-CPTU results were corrected to match the nearby static-CPTU (i.e. distance less than 10 m) using strain-rate factors (SF) derived from three known correction methods. Based on a statistical test and visual comparison of dynamic- and static-CPTU profiles, the modified inverse sin-hyperbolic correction method is found to be best suited for the strain-rate correction of dynamic-CPTU tests, and results in SF less than 1.35 for the corrected cone penetration resistance and up to 2.4 for the sleeve friction. Our
data illustrate a positive correlation between penetration rate and penetration depth, which is governed by the consolidation state of the clays. The good agreement between SF-corrected dynamic-CPTU data from 34 deployments (acquired within less than 36 hrs. shiptime) with data obtained from static-CPTU and laboratory experiments on sediment cores further demonstrates that the MARUM dynamic-CPTU device is a powerful tool for characterizing the properties of surficial seafloor sediments in shallow-water environments.

Key words: dynamic-CPTU, static-CPTU, clayey soils, in situ comparison, Finneidfjord, weak layer, strain-rate effect, soil-specific rate coefficient.

Introduction

During the past four decades, dynamic- and static-CPTU instruments have been deployed in offshore environments (e.g. Dayal et al. 1975, de Ruiter and Fox 1975). The deployment of the static, seabed going CPTU rigs requires time, a large vessel with a heavy winch, a high A-frame, and good weather conditions. The penetration depth is up to 100 m in order to achieve shallow to deep sub-seafloor targets (e.g. Meunier et al. 2004, Randolph 2004). In contrast, dynamic-CPTU probes can be handled and operated faster from small vessels and platforms due to their light-weight configuration with a weight less than 200 kg, and are less prone to bad weather conditions (e.g. Stegmann et al. 2006). Such probes reach superficial targets in less than 15 m depth (e.g. Dayal et al. 1975), and allows the engineer to collect a large number of soil properties within a very short period. Such information is primordial for area-wide geotechnical sub-seafloor characterization, slope stability and/or risk assessments.

The dynamic-CPTU probe is lowered through the water column in winch or free fall mode, impacting the seafloor with an initial penetration rate \( v_0 \) controlled by the weight of the device, buoyancy, winch speed or drag of the steel wire (e.g. see details in chapter 2.1 of Stark 2010). Penetration into the soil is driven by the momentum of the probe at a non-linearly decreasing velocity, whereas the conventional, static-CPTU instrument penetrates the soil with constant penetration rate \( v_{ref}=2 \text{ cm/s} \). The dynamic- and static-CPTU instruments measure the cone penetration resistance \( q_c \), sleeve friction \( f_s \) and pore pressure behind the tip \( u_2 \). The combination of these parameters provides information for calculating soil mechanical parameters (e.g. intact undrained shear-strength, sensitivity, OCR [over-consolidation ratio]) and inferring lithology (soil classification) (i.e. see for overview Robertson and Campanella 1983, Lunne et al. 1997, Robertson 2009, Lunne 2010).
Previous theoretical studies demonstrate that soil properties depend on the rate at which the soils are deformed (strain-rate). This correlation was derived from the theory of absolute reaction rates, and the relationship between stress-strain behavior and strain-rate (Eyring 1936, Glasstone et al. 1941, Suklje 1957, Ladd et al. 1977). Consequently, geotechnical measurements performed at different penetration rates must be corrected due to this strain-rate effect. Similar findings were observed in ring shear tests, centrifuge experiments and soil target measurements by Casagrande and Shannon (1949), Casagrande and Wilson (1951), Lefebvre and LeBoeuf (1987), Sheahan et al. (1996), Randolph and Hope (2004), Lehane et al. (2009). Other studies have investigated this effect using finite element simulations (e.g. Silva et al. 2006, Nazem et al. 2012), while some workers compared dynamic-CPTU records with static-CPTU measurements, vane shear (v-s) and fall cone penetration (fc) experiments measured in the laboratory (e.g. Young et al. 2011, Steiner et al. 2012). However, some of those studies are only applicable when exploring the surficial soil properties in natural geological settings according to the following reasons: (i) small penetration rate or penetration depth (e.g. Stoll et al. 2007), (ii) lack of pore pressure consideration (Dayal and Allen 1975, Osler et al. 2006), and (iii) restriction to certain grain size classes, specifically homogeneous soft, and clay-rich sediments in Christian et al. (1993), Aubeny and Shi (2006), Lehane et al. (2009).

To date, only the logarithmic strain-rate correction method has been rigorously tested for dynamic penetrometers (e.g. Dayal et al. 1975, Aubeny and Shi 2006, Stoll et al. 2007). Consequently, comparative work including other methods is necessary for translating dynamic data to quasi-static values common in soil engineering (e.g. Stark et al. 2009). A second method based on an inverse sinhyperbolic equation (Mitchell and Soga 2005, Randolph and Hope 2004, Chung et al. 2006) and a third based on a power-law equation (Biscontin and Pestana 2001, Lehane et al. 2009) are examined. All corrections are predicated on the ratio of the dynamic penetration rate \(v_{\text{dyn}}\) to the \(v_{\text{ref}}\). In addition, these corrections can be adjusted for soil-specific variability in the soil properties by employing soil-specific rate coefficients (SSCs) (Dayal et al. 1975, Stoll et al. 2007, Mahajan and Budhu 2008) or penetration rate multipliers (Randolph and Hope 2004, Aubeny and Shi 2006).

This study assesses the three state-of-the-art strain-rate corrections by comparing datasets from a high-quality dynamic-CPTU, static-CPTU, fc and direct simple shear (DSS) experiments on cored samples collected in normally- to overconsolidated silty clay off the landslide-prone region around the village of Finneidfjord, northern Norway. The strain-rate corrections, previously published and calculated SSCs are used to determine strain-rate corrected, quasi-static CPTU parameters from dynamic ones. The quasi-static CPTU parameters are directly compared with the other datasets taken nearby to: (i) assess the performance of
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the strain-rate correction methods, and (ii) determine the best suitable strain-rate correction for dynamic-CPTU tests with very high $v_0$ of up to 7.8 m/s by using a statistical test. Moreover, a direct correlation between dynamic-CPTU tests with various $v_dyn$ and their penetration depths sheds light on the influence of the clay’s consolidation state on the total penetration depth of dynamic-CPTU tests. The $fc$, DSS, dynamic- and static-CPTU records are directly used to: (i) determine the intact undrained shear-strength ($s_u$) of the soil, and (ii) detect a thin, weak and landslide-prone layer.

Regional setting

The dynamic-CPTU tests were carried out in the inner part of Sørjorden located off the village of Finneidfjord, northern Norway. At this location, the fjord basin is 1.5 km wide and up to 5 km long with water depth (WD) less than 60 m (Fig. 3.12). The whole area has been subject to intensive glacio-isostatic rebound since the late Precambrian period (~9000 years BP), and the marine limit is located 120-128 meters above modern sea level (Olsen et al. 2006). The sedimentary sequence is composed of an up to 200 m thick succession of Holocene marine clays and silts overlain by fluvioglacial and/or littoral soils (Olsen et al. 2001, Olsen et al. 2006). Following their exhumation in the Holocene, the marine clay deposits were leached by fresh groundwater and this resulted in the formation of very sensitive clays. Several terrestrial landslides have been triggered in these marine sensitive clays in historical and prehistorical times (e.g. Fig. 3.12, Olsen et al. 2006, L’Heureux et al. 2012). Some of the largest events resulted in deposition of distinct event beds in the fjord (L’Heureux et al. 2012, Vardy et al. 2012).

Review of strain-rate corrections and derivation of quasi-static CPTU parameters

State-of-the-art strain-rate corrections Three different strain-rate corrections were suggested, all described by the normalized strength factor like $s_u/s_{u,ref}$, $q/q_{ref}$ or $SF$ (e.g. Dayal and Allen 1975, Mitchell and Soga 2005, Biscontin and Pestana 2001, Lehane et al. 2009). However, only one of these correction methods is commonly used for dynamic penetrometers deployed in natural homogeneous and heterogeneous soils (Eq. 3.2, Dayal et al. 1975, Aubeny and Shi 2006, Stoll et al. 2007, Steiner et al. 2012). This correction is based on soil strength properties under dynamic loads taking into account the soil bearing capacity theory (Terzaghi 1943). Casagrande and Shannon (1949) and Casagrande and Wilson (1951) demonstrate a significant increase in strength under dynamic loading. This increase is mathematically described by a logarithmic equation comprising the $v_{dyn}$
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Figure 3.12: Surface morphology of the region near the village of Finneidfjord, Sørfjorden, northern Norway, based on high-resolution swath bathymetry with a 1 m bin size, including an ordinary assignment of previous landslide event locations. Symbols indicate the location of the Calypso piston and gravity cores, static- and dynamic-CPTU tests. The contourlines with a spacing of 5 m describe the water depth. Dynamic-CPTU tests measured in over-consolidated clay (OC) or with winch mode (MW) are marked (see also Fig. 3.23 for details).

versus the $v_{ref}$ and soil-specific rate coefficient ($\mu_{CPTU}$), defined as:

$$SF_{log} = 1 + \mu_{CPTU} \frac{v_{dyn}}{v_{ref}},$$  \hspace{1cm} (3.2)

where $SF_{log}$ is the strain-rate factor. The $\mu_{CPTU}$ is used as a proxy for the shear viscosity of the soils during failure (Dayal and Allen 1975, Dayal et al. 1975, Steiner et al. 2012) and/or as a soil dependent material constant (Perlow and Richards 1977, Biscontin and Pestana 2001, Stoll et al. 2007). Other workers describe this parameter as a non-dimensional rate factor related to the $v_{ref}$ based on in situ penetrometer tests, and centrifuge and vane shear experiments (Randolph and Hope 2004, Aubeny and Shi 2006). A summary of different methods for evaluating $\mu_{CPTU}$ based on soil type, type of test, and soil mechanical properties like $s_u$, $q_c$, $q_t$, $f_s$ is given in Table 3.1.

Divergent behavior of the strain-rate factor has been observed when penetration rate ratios ($v_{dyn}/v_{ref}$) are much lower than 1 due to the mathematical form of
3.2. POST PROCESSING OF DYNAMIC-CPTU RECORDS

Table 3.1: List of existing studies of the soil-specific rate coefficient ($\mu_{CPTU}$) for the logarithmic strain-rate correction. (Note: cone penetration resistance ($q_c$), sleeve friction ($f_s$), net cone penetration resistance ($q_{c,net}$), soil bearing capacity ($q_u$), corrected cone penetration resistance ($q_t$) and intact undrained shear-strength ($s_u$))

<table>
<thead>
<tr>
<th>soil type</th>
<th>test type</th>
<th>output parameter</th>
<th>$\mu_{CPTU}$</th>
<th>reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>remolded clay (Pottery clay)</td>
<td>CPT</td>
<td>$q_c$</td>
<td>1.50-0.03 (for $s_u=3-80\text{kPa}$)</td>
<td>Dayal and Allen (1975), Dayal et al. (1975)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$f_s$</td>
<td>0.93-0.38 (for $s_u=3-80\text{kPa}$)</td>
<td></td>
</tr>
<tr>
<td>Kaolin clay</td>
<td>CPT</td>
<td>$q_{c,net}$</td>
<td>0.10-0.15</td>
<td>Randolph and Hope (2004)</td>
</tr>
<tr>
<td>water-sturated sand</td>
<td>CPT</td>
<td>$q_c$</td>
<td>0.7-1.5</td>
<td>Stoll et al. (2007)</td>
</tr>
<tr>
<td>quartz and carbonate sand</td>
<td>Nimrod</td>
<td>$q_u$</td>
<td>1.0-1.5</td>
<td>Stark et al. (2009)</td>
</tr>
<tr>
<td>clay to silty clay</td>
<td>CPTU</td>
<td>$q_t$</td>
<td>0.13</td>
<td>Steiner et al. (2012)</td>
</tr>
<tr>
<td>clay silt</td>
<td>CPTU</td>
<td>$f_s$</td>
<td>0.45</td>
<td></td>
</tr>
<tr>
<td></td>
<td>vane shear</td>
<td>$q_t$</td>
<td>0.11</td>
<td>Young et al. (2011)</td>
</tr>
<tr>
<td>Bentonite-Kaolinite clay</td>
<td>vane shear</td>
<td>$s_u$</td>
<td>0.21-0.36</td>
<td>Perlow and Richards (1977)</td>
</tr>
<tr>
<td>clay</td>
<td>XBP</td>
<td>$s_u$</td>
<td>0.15</td>
<td>Biscontin and Pestana (2001)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Aubeny and Shi (2006)</td>
</tr>
</tbody>
</table>

the logarithmic equation (Randolph 2004, Mitchell and Soga 2005) and potentially transition from completely undrained to partially drained conditions resulting in an increase of the strength properties (e.g. DeJong et al. 2011). Bemben and Myers (1974) and Roy et al. (1982) observe also a non-linear increase of the strength properties of up to $\sim 20\%$, most prominently at low rates towards the terminal depth of the deployment, using static-CPTU tests with penetration rates $v_{ref}=0.25$ to 4 cm/s. Steiner et al. (2012) and Stoll et al. (2007) confirm these observations for the last portion (i.e. less than 5cm) of the dynamic-CPTU profiles affecting $\sim 1\%$ of the entire data points.

A second strain-rate correction is based on studies dealing with rate processes
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using the theory of absolute reaction rates (Glasstone et al. 1941, Kuhn 1987). This theory is premised on statistical mechanics, and sheds light on the functional forms of factors affecting the soil behavior and nature of soil strength (Eyring 1936, Glasstone et al. 1941). In particular, the shear rate-dependent increase of viscosity is expressed as:

\[ SF_{\text{arcsinh}} = 1 + \mu'_{\text{CPTU}} \text{arcsinh} \frac{v_{\text{dyn}}}{v_{\text{ref}}}, \]  

(3.3)

see Eyring 1934 or Mitchell and Soga (2005) for details. This strain-rate correction is described by the inverse sinh-hyperbolic equation defined by the penetration rate ratio and the soil-specific rate coefficient \( \mu'_{\text{CPTU}} \) expressed as:

\[ \mu'_{\text{CPTU}} = \frac{\mu_{\text{CPTU}}}{\ln(10)}. \]  

(3.4)

If \( v_{\text{dyn}}/v_{\text{ref}} \) greater than 1, Equations 3.2 and 3.3 are approximately equivalent (e.g. Randolph 2004, Chung et al. 2006) and if \( v_{\text{dyn}}/v_{\text{ref}} \) less than 1, Equation 3.2 show a significant decrease in \( SF \), which is in Equation 3.3 a \( SF \) near unity. Several workers use Equations 3.3 and 3.4 for the correction of the strain-rate in laboratory experiments, in particular, to correct centrifuge measurements and vane shear data (e.g. Randolph and Hope 2004, Chung et al. 2006, Abelev and Valen 2009).

The third strain-rate correction assessed here is based on the assumption that the behavior of soils in one-dimensional compression is affected by strain-rate, bulk viscosity, compressibility, and drainage effects (Ladd et al. 1977, Graham et al. 1983, Leroueil et al. 1985, Leroueil and Hight 2003). The rheological model described as the so-called stress-strain-strain rate relation was initially proposed by Suklje (1957). The first application focused on the determination of the rate-dependent preconsolidation pressure (e.g. Mesri and Godlewski 1977, Soga and Mitchell 1996, Leroueil and Marques 1996). Further studies show that this strain-rate correction is valid for centrifuge experiments and vane shear tests (e.g. Perlow and Richards 1977, Biscontin and Pestana 2001, Rattley et al. 2008, Lehane et al. 2009). It has a power-law form encompassing the ratio of \( v_{\text{dyn}} \) and \( v_{\text{ref}} \) and a rate exponent \( (\beta) \):

\[ SF_{\text{power}} = \left( \frac{v_{\text{dyn}}}{v_{\text{ref}}} \right)^\beta, \]  

(3.5)

where \( \beta \) accounts for lithological variations of the sediments soils (e.g. Mitchell and Soga 2005). Table 3.2 presents an overview of previous studies addressing this strain-rate correction, including soil type, laboratory test, output parameters, and \( \beta \)-factor.

Drained, partially drained and undrained penetration  Results from dynamic- and static-CPTU tests are both affected by the penetration rate, drainage
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Table 3.2: List of existing studies of the soil-specific rate coefficient ($\beta$) for the power-law strain-rate correction. (Note: preconsolidation pressure ($\sigma'_p$) and intact undrained shear-strength ($s_u$))

<table>
<thead>
<tr>
<th>soil type</th>
<th>test type</th>
<th>output parameter</th>
<th>$\beta$</th>
<th>reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>sensitive clay, organic silt, peat</td>
<td>Oedometer</td>
<td>$\sigma'_p$</td>
<td>0.025-0.10</td>
<td>Mesri and Godlewski (1977)</td>
</tr>
<tr>
<td>soft clay</td>
<td>Oedometer</td>
<td>$\sigma'_p$</td>
<td>0.02-0.07</td>
<td>Mesri and Castro (1987)</td>
</tr>
<tr>
<td>inorganic clay</td>
<td>Oedometer</td>
<td>$\sigma'_p$</td>
<td>0.029-0.059</td>
<td>Leroueil and Marques (1996)</td>
</tr>
<tr>
<td>silt</td>
<td>vane shear</td>
<td>$s_u$</td>
<td>0.08-0.13</td>
<td>Perlow and Richards (1977)</td>
</tr>
<tr>
<td>Pierre shale</td>
<td>vane shear</td>
<td>$s_u$</td>
<td>0.05</td>
<td>Sharifoumasab and Ullrich (1985)</td>
</tr>
<tr>
<td>Bentonite-Kaolinite clay</td>
<td>vane shear</td>
<td>$s_u$</td>
<td>0.055</td>
<td>Biscontin and Pestana (2001)</td>
</tr>
<tr>
<td>Kaolin clay</td>
<td>centrifuge</td>
<td>$s_u$</td>
<td>0.05</td>
<td>Rattley et al. (2008)</td>
</tr>
</tbody>
</table>

Condition and evolution of pore pressure during penetration. The soil response varies between fully undrained and fully drained as the penetration rate decrease. For a drained penetration, no pore pressure occurs due to high permeability of the tested soil. Conversely, the lack of drainage and shear induced pore pressure for undrained penetration cause weakening of the tested soil. Therefore, the measured in situ parameters show higher values for fully drained and lower values for fully undrained conditions, resulting in an under- or overestimation of the potential risks and geohazards for marine environmental (e.g. House et al. 2001). Several workers favor the use of the "non-dimensional velocity" for assessing the degree of consolidation during penetration (Randolph and Hope 2004, Randolph 2004, Chung et al. 2006, Kim et al. 2008, Lehane et al. 2009, Robertson 2010). The non-dimensional dynamic velocity ($V_{dyn}$) and non-dimensional static velocity ($V_{ref}$) are defined as:

$$V_{dyn} = \frac{v_{dyn}d_{dyn}}{c_v},$$

$$V_{ref} = \frac{v_{ref}d_{ref}}{c_v},$$

where $v_{dyn}$ and $v_{ref}$ are the dynamic and static penetration rates, $d_{dyn}$ and $d_{ref}$ are the diameters of the dynamic- and static-CPTU probes, and $c_v$ is the coefficient of consolidation. The CPTU probe diameters vary between 2.5 and 4.4 cm.
Non-dimensional velocity limits for undrained and drained penetrations were obtained from centrifuge, in situ and calibration chamber tests on different soils. Table 3.3 presents a summary of studies investigating the non-dimensional velocity and the boundaries between fully undrained and partially drained and from partially drained to fully drained penetration as a function of testing method and lithology. For the purpose of this study, a value of \( V_{\text{dyn}} \) or \( V_{\text{ref}} \) ≥ 30 represents fully undrained (e.g. Finnie and Randolph 1994, Randolph 2004), and a value of \( V_{\text{dyn}} \) or \( V_{\text{ref}} \) ≤ 0.3 represents fully drained conditions (DeJong et al. 2013). Both \( V \) values define the most likely limits taken from Table 3.3.

Here, the formulation of the non-dimensional velocity is used to accommodate the different diameters of the CPTU cones used in this study. The ratio of \( v_{\text{dyn}} \) and \( v_{\text{ref}} \) (Eqs. 3.2, 3.3, 3.5) in the strain-rate correction methods is replaced using the following expression:

\[
\frac{V_{\text{dyn}}}{V_{\text{ref}}} = \frac{v_{\text{dyn}} d_{\text{dyn}}}{v_{\text{ref}} d_{\text{ref}}},
\]

(3.8)

Table 3.3: Overview of transition limits from fully undrained to partially drained and partially drained to fully drained represented by the non-dimensional velocity (\( V \)). (Note: net cone penetration resistance (\( q_{\text{c,net}} \)), pore pressure measured behind the tip (\( u_2 \)), cone penetration resistance (\( q_c \)), sleeve friction (\( f_s \)), normalized cone penetration resistance (\( Q \)) and excess pore pressure measured behind the tip (\( \Delta u_2 \))

<table>
<thead>
<tr>
<th>soil type</th>
<th>test type</th>
<th>( V )</th>
<th>reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>calcareous sand, silt, clay</td>
<td>centrifuge</td>
<td>&gt;30 for ( q_{\text{c,net}} )</td>
<td>&lt;0.01 for ( q_{\text{c,net}} )</td>
</tr>
<tr>
<td>natural soil (clay)</td>
<td>in situ</td>
<td>&gt;100 for ( u_2 ), ( q_{\text{c,net}} )</td>
<td>~0.1 for ( u_2 )</td>
</tr>
<tr>
<td>Kaolin clay</td>
<td>centrifuge</td>
<td>&gt;30 for ( q_{\text{c,net}} )</td>
<td>-</td>
</tr>
<tr>
<td>clay and silty clay</td>
<td>in situ</td>
<td>&gt;100 for ( q_{\text{c,net}} )</td>
<td>-</td>
</tr>
<tr>
<td>natural soil (silty clay)</td>
<td>in situ</td>
<td>4-10 for ( q_c )</td>
<td>-</td>
</tr>
<tr>
<td>sand-clay mixture</td>
<td>chamber</td>
<td>~10 for ( q_c )</td>
<td>~0.05 for ( q_c )</td>
</tr>
<tr>
<td>sand-clay mixture</td>
<td>centrifuge</td>
<td>&gt;20 for ( q_c ) and ( u_2 )</td>
<td>&lt;0.01 for ( q_c ) and ( u_2 )</td>
</tr>
<tr>
<td>Kaolin clay</td>
<td>in situ</td>
<td>&gt;30 for ( Q ) and ( \Delta u_2 )</td>
<td>~0.3 for ( Q ) and ( \Delta u_2 )</td>
</tr>
</tbody>
</table>
In this "non-dimensional velocity ratio" ($V_{\text{dyn}}/V_{\text{ref}}$) the following assumptions are made: (i) $c_v$ is assumed by the highest value for the complete soil profile, and (ii) $c_v$ is less than $2 \times 10^{-5} \text{m}^2/\text{s}$, which results in $V$ values $>30$ for the static-and dynamic-CPTU tests. Consequently, if these assumptions are abided then the dynamic- and static-CPTU tests have been conducted under fully undrained conditions.

**Quasi-static CPTU parameter** This study quantifies the $SF$ for the $q_t$ and $f_s$ by incorporating strain-rate corrections (Eqs. 3.2, 3.3, 3.5) and the $V_{\text{dyn}}/V_{\text{ref}}$ (Eq. 3.8) of comprehensive dynamic-CPTU datasets collected in marine clays. The $SF_{\text{log}}, SF_{\text{arcsinh}}$ or $SF_{\text{power}}$ are determined for each recorded data point taking into account the associated $v_{\text{dyn}}$ and the constant $v_{\text{ref}}$ in order to calculate the individual $V_{\text{dyn}}/V_{\text{ref}}$. The different $SF$s are denoted by the shortcut of the strain-rate corrections, and are represented by the logarithmic (log), inverse sinh-hyperbolic (arcsinh) and power-law (power) equations. The quasi-static CPTU parameters ($q_{t,q-s}, f_{s,q-s}$) are obtained by dividing the measured dynamic properties ($q_{t,dyn}, f_{s,dyn}$) by the instantaneous $SF_{\text{log}}, SF_{\text{arcsinh}}, SF_{\text{power}}$ (Eqs. 3.9, 3.10):

$$q_{t,q-s} = \frac{q_{t,dyn}}{SF_{\text{log}}[\text{arcsinh}][\text{power}]},$$

$$f_{s,q-s} = \frac{f_{s,dyn}}{SF_{\text{log}}[\text{arcsinh}][\text{power}]}.$$  

The quasi-static CPTU parameters are connected to the associated penetration depth in order to obtain a continuous quasi-static CPTU profile comparable to a conventional static-CPTU record.

**Methodology**

**Static-CPTU devices** Two different static-CPTU systems were used in this study. Both systems pushing the piezocone with 2 cm/s penetration rate into the soil, but differ significantly in WD of operation and terminal sub-seafloor penetration depth.

The CPTU device GOST (Geotechnical Offshore Seabed Tool) is a novel instrument to characterize the sub-seafloor. The application of GOST is offshore site investigation with a maximum operational depth of 4000 m (Fig. 3.13). The system operates from the seabed by pushing 1.5 m long straight steel rods into the soil to a maximum depth of 40 m. The vertical resolution is 0.4 cm resulting from a sampling rate of 5 Hz. The maximum error is less than 2% of the actual penetration depth. A "digital subtraction piezocone" of 5 cm$^2$ area with $u_2$ pore pressure port was used on a rod with a 2 cm diameter. This piezocone has a maximum capacity of 70 MPa for $q_c$, 1 MPa for $f_s$ and 1 MPa for $u_2$. It will be pushed into the soil in
semi-continuously 25 cm intervals using two push-pull hydraulic cylinders powered by a hydraulic unit on the seabed tool. Each push phase is followed by a break in order to move the cylinder to the start position. This break is less than 15 sec long. A static clamp is holding the rod, preserving the stress at the cone/tip and avoiding unloading of the entire rod assembly. Consequently, the records are not affected negatively (i.e. no anomalies in the datasets are observed; T. Mörz, pers. comm., 2013). The CPTU parameter accuracy is limited by an error band of better than ±25 kPa (accuracy class 1; ISSMGE). It should be noted that the accuracy specification was measured using an internal calibration chamber and calibrated force transducers (e.g. HBM S9M).

Two static tests were performed in the study area in 1997 by the Norwegian road authorities using a conventional onshore CPTU system (Environmental Mechanics AB [ENVI]), which was operated from a barge. This instrument deploys a standard, industrial 10 cm$^2$ Memocone MKII piezocone with sensors monitoring $q_c$, $f_s$ and $u_2$. This piezocone logs the three CPTU parameters simultaneously with a sampling rate of 0.5 Hz. The maximum capacity is 50-100 MPa for $q_c$, 1 MPa for $f_s$ and 2 MPa for $u_2$. The CPTU parameter low range accuracy varies within an error band less than ±20 kPa, and is 0.1 to 0.5% for the full range (accuracy class 1; ISSMGE). The accuracy specifications are based on the Memocone MKII piezocone data sheet. The two conventional CPTU test achieved penetration depths of less than 25 m.
3.2. POST PROCESSING OF DYNAMIC-CPTU RECORDS

**Dynamic piezocone penetrometer** The *MARUM* shallow-water dynamic-CPTU (*SWFF-CPTU*), with a maximum depth range of \( \leq 500 \text{ m} \), has a \( 15 \text{ cm}^2 \) *Geomil* subtraction piezo cone recording \( q_c, f_s, u_2 \) and tilt/temperature. The piezocone is mounted on 1 m long metal rods built to a maximum length of 8.5 m and hosts a pressure-tight housing containing a microprocessor, standard SD-card, battery packages, tiltmeter, accelerometers and data interface module (Fig. 3.14, see Stegmann et al. 2006 for details). It has a maximum capacity of 25 MPa for \( q_c \), 0.25 MPa for \( f_s \) and 1 MPa for \( u_2 \).

The *Analog Devices* - dual-axis tilt sensor monitors the penetration angle within \( \pm 45^\circ \) relative to vertical. Five different accelerometers from *Analog Devices* (*iMEMS*) with different ranges (\( \pm 1.7 \text{ g}, \pm 18 \text{ g}, \pm 35 \text{ g}, \pm 70 \text{ g} \) and \( \pm 120 \text{ g} \)) provide information about the changes in impact and penetration velocity. These data allow the calculation of the penetration rate and penetration depth from the 1\(^{st}\) and 2\(^{nd}\) integration of the acceleration over time. The frequency of data acquisition was 1 kHz. The maximum error in penetration depth is estimated to be 2\% according to the accuracy of the accelerometers. The vertical resolution of CPTU profiles is \(<1 \text{ cm} \) at penetration rates less than \( 10 \text{ m/s} \). The accuracy of the dynamic-CPTU

![Figure 3.14: Deployment of the *MARUM* shallow-water dynamic-CPTU and sketch of the piezo cone including a simple schematic of the dynamic-CPTU parameters (copyright *MARUM*).](image-url)
parameters, recorded with force sensors and pressure transducers, ranges within an error band of better than $\pm 10\text{kPa}$ and was derived from the technical specification of the piezocone (accuracy class 1; ISO 22476-1). Moreover, an important issue in the data processing is to detect the correct point at that the probe hits the seafloor. The direct correlation between $q_c$, $u_2$ and acceleration records is used to analyze transient changes in the parameter gradients. The starting point/time of each test (seafloor impact) usually corresponds to a significant increase of the three parameters (i.e. $q_c$, $u_2$ and acceleration). This increase is observed at nearly the same time in all three records considering the time difference between seafloor contact of the tip and seafloor contact of the pore pressure port (i.e. equivalent to $\sim 4\text{cm}$).

The dynamic tests were carried out in two different deployment modes, either controlled by winch with a constant velocity up to $2\text{m/s}$, or nearly decoupled from the ship’s winch with up to $v_0=7.8\text{m/s}$. Penetration depends on the velocity before impact, the weight of the instrument and the resistance of the soils. This results in a variable penetration depth and a non-linearly decreasing penetration rate (e.g. Stegmann et al. 2006, Steiner et al. 2012).

**Sediment sampling and laboratory techniques** In 2009, a set of 12 gravity cores (up to 3m long) were retrieved from areas within the landslide deposit, areas of exposed slide planes, and from the undisturbed seafloor outside the area influence by the 1996 landslide event using a standard gravity corer from the R/V *Seisma*. In 2010, two longer cores (12 and 14m, respectively) were collected in areas of intact seafloor immediately adjacent to the landslide using a French *Calypso* piston corer (cpc) from the R/V *G.O. SARS* [http://wwwold.nioz.nl/nioz_nl/b1c6b6914e59dfb0be67e948b12858.php](http://wwwold.nioz.nl/nioz_nl/b1c6b6914e59dfb0be67e948b12858.php; Haffkla-son et al. 2010). The cpc system is considered one of the most useful and efficient marine coring systems, designed so that soil disturbance is minimal. The corer weighs 7-10 tons, and is equipped with a 40 to 60m iron lance with a 5 inches 1/2 diameter and an internal high-pressure PVC liner with a 10cm diameter, a mechanical trigger and a piston to ensure uniform extraction of the sediment within the lance during the final free fall, from about 1m above the seabed.

The laboratory program for the material retrieved from the gc included x-ray imaging, visual description and measurement of geotechnical index properties. The index properties include grain size ($gs$), magnetic susceptibility, water content, consistency limits and intact fall cone penetration strength ($s_{u,fc}$). Grain size analyses for material collected from the gc were performed using the *falling drop method* (Moum 1965).

For the cpc, the laboratory program was developed using whole-core conven-
3.2. POST PROCESSING OF DYNAMIC-CPTU RECORDS

Post processing of dynamic-CPTU records was performed following traditional X-ray images in combination with multi-sensor core logging results, the latter yielding measurements of gamma density, P-wave velocity and magnetic susceptibility in very-high vertical resolution of 0.5 cm. Grain size analyses on the cpc samples were performed using the falling drop method (Moum 1965). Geotechnical tests targeted the different lithofacies and particularly weak units of importance for slope stability. Measurements included density, water content, Atterberg limits, $s_{u,fc}$, intact direct simple shear-strength ($s_{u,DSS}$), and permeability tests on DSS samples. For more rigorous tests, the samples were reconsolidated to approximate in situ stress levels according to the sample depth and assuming hydrostatic conditions. All tests were performed according to international standards (e.g. British and European Standards, American Society for Testing and Materials) and a complete summary of the testing procedures and laboratory results is presented in Vanneste et al. (2013).

Results

Soil properties A summary of physical and geotechnical properties is given in Table 3.4 for the normally- to over-consolidated silty clays found in the study area. Grain size analyses were carried out on soil specimens from core 01_cpc within depth ranges of 2.9 to 3.5 m and 7.6 to 8.4 m. Figure 3.15 illustrates the cumulative distributions according to the European Standard (ISO 14688-1), indicating a narrow range of gs classes within the field of clay to fine silt. According to the plasticity chart, the soils are classified as slightly- to moderately plastic clay (BS 5930, 1999).

For core 01_cpc, the $s_{u,fc}$ is between 4 and 15 kPa in the upper 3 m and shows a broader range below 3 m penetration depth (Fig. 3.16). Representative undisturbed values for $s_{u,DSS}$ and the associated $s_{u,DSS}/\sigma'_{ac,DSS}$ range between 7 and 10 kPa with an assumed error band of ±0.5 kPa and 0.31-0.41 at the depth between 3.0 and 3.5 m, respectively. The experiments on core 04_gc follow this strength trend (see Table 3.4, L’Heureux et al 2012, Steiner et al. 2012). However, the $s_{u,fc}$ profile of the gc is up to 60% higher compared to the cpc profile, probably related to the spatial variability of both cores (i.e. core 04_gc is located near the shoreline, while the core 01_cpc is on the fjord bottom) (Fig. 3.16). The $c_{uc,DSS}$ is determined using the consolidation phase of the DSS, which results in values from $7.3*10^{-8}$ to $6.8*10^{-5}$ m²/s as a first-order estimate (see results in Vanneste et al. 2013).

The soil physical and geotechnical properties are used to constrain and validate the different $\mu_{CPTU}$, $\mu'_{CPTU}$ and $\beta$ shown in Tables 3.1, 3.2 and calculated using Eq. 3.4. The static-CPTU data and laboratory records serve as reference profiles that characterize the natural stratigraphy and geotechnical properties.
Table 3.4: Soil properties of the normally- to over-consolidated silty clay of core specimens taken from Calypso piston and gravity cores.

<table>
<thead>
<tr>
<th>soil properties</th>
<th>values</th>
</tr>
</thead>
<tbody>
<tr>
<td>total unit weight, $\gamma_{bulk}$ (kN/m$^3$)</td>
<td>15.7-19.1</td>
</tr>
<tr>
<td>intact direct simple shear-strength, $s_{u, DSS}$ (kPa)</td>
<td>6.7-9.5</td>
</tr>
<tr>
<td>intact fall cone penetration strength, $s_{u, fc}$ (kPa)</td>
<td>5.5-11.0 (gc$^3$)</td>
</tr>
<tr>
<td>undrained shear-strength ratio ($s_{u, DSS}/\sigma'_{ac, DSS}$)</td>
<td>4-15 (cpc$^4$)</td>
</tr>
<tr>
<td>in situ undrained shear-strength ratio ($s_{u, qt}/\sigma'_{V0}$)</td>
<td>0.2-0.5</td>
</tr>
<tr>
<td>moisture content, $\omega$ (%)</td>
<td>35-70</td>
</tr>
<tr>
<td>liquid limit, $\omega_L$ (%)</td>
<td>25-36</td>
</tr>
<tr>
<td>plastic limit, $\omega_P$ (%)</td>
<td>10-22</td>
</tr>
<tr>
<td>plasticity index, $I_P$ (%)</td>
<td>15</td>
</tr>
<tr>
<td>liquidity index, $I_L$ (%)</td>
<td>120-130</td>
</tr>
<tr>
<td>sand proportion &gt;0.063 mm (%)</td>
<td>2-8</td>
</tr>
<tr>
<td>silt proportion &gt;0.002 to 0.063 mm (%)</td>
<td>62-80</td>
</tr>
<tr>
<td>clay content &lt;0.002 mm (%)</td>
<td>18-30</td>
</tr>
<tr>
<td>coefficient of consolidation for fine-grained soils, $c_v, DSS$ (m$^2$/s)</td>
<td>$7.3<em>10^{-8}$-$1.1</em>10^{-6}$</td>
</tr>
<tr>
<td>coefficient of consolidation for coarse-grained soils, $c_v, DSS$ (m$^2$/s)</td>
<td>$6.0<em>10^{-6}$-$6.8</em>10^{-5}$</td>
</tr>
</tbody>
</table>

Notes: 1 DSS = Direct Simple Shear, static test (NGI),
2 fc = fall cone penetration experiment, post cruise,
3 gc = gravity core,
4 cpc = Calypso piston core.

Comparison of dynamic-CPTU with static-CPTU systems A total of 34 dynamic-CPTU tests were carried out in the study area in WD ranging between 12 and 46 m (see Fig. 3.12, Table 3.4 for details). The $v_0$ values vary between 0.9 and 7.8 m/s, and penetration depths are up to 5 meters below seafloor (mbsf). Test SW27_fin was collected at less than 10 m from the static-CPTU measurement (947_envi) at a WD of 14.5 m. Both datasets are compared in Figure 3.17. The dynamic test was conducted in free-fall mode with a $v_0$ of 5 m/s (Fig. 3.17a, Table 3.5). The dynamic penetration rate shows a non-linear decrease starting from $v_0$ and decelerates to zero with a terminal penetration depth ($z_{depth}$) of 2.75 m. The $u_2$ from the 947_envi shows low quality, but adequate reliability to use it for the pore pressure correction of the qc. Hence, only $q_{t, ref}$ and $f_{s, ref}$ profiles are compared. The $q_{t, ref}$ characterizes a homogeneous soil succession (Fig. 3.17b). The parameters vary between 150 and 300 kPa, including a significant increase in the first meter and a slightly increasing gradient after the first meter. The $f_{s, ref}$ profile is nearly constant in the range of 4 to 7 kPa (Fig. 3.17b). The difference between dynamic- and static parameters in the first 0.25 m might be related to
3.2. POST PROCESSING OF DYNAMIC-CPTU RECORDS

Figure 3.15: Cumulative grain-size distribution curves (ISO 14688-1) for the homogeneous soil succession of the studied area for specimens from Calypso piston core 01_cpc.

internal effects of the static-CPTU probe, but cannot be reconstructed from the records. The \( q_{t,dyn} \) profile illustrates also homogeneous soils with a slightly increasing gradient, except positive peaks up to 400 kPa at 0.3-0.7 m and 2.1-2.2 m depth. A significant peak of 16 kPa in the depth range 0.5-0.7 m and a decreasing behavior below 2.5 m depth can be seen in the \( f_{s,dyn} \) profile. In general, the \( q_{t,dyn} \) is up to 1.35 times higher than the \( q_{t,ref} \) (Fig. 3.17b), while \( f_{s,dyn} \) is 2.1 to 2.4 times higher than its static counterpart (Fig. 3.17c).

Results from the SW27_fin are corrected for strain-rate effects using the three approaches presented in section [State-of-the-art strain-rate corrections] (Figs. 3.18, 3.19). For the logarithmic and power-law equations, previously reported SSCs (\( \mu_{CPTU} \) and \( \beta \)) for clay-rich soils were applied (Tables 3.1, 3.2). The quasi-static corrected cone penetration resistance \( (q_{t,g-s}) \) is calculated based on Equation 3.2 and \( \mu_{CPTU} \) values varying between 0.1 and 0.25 (Table 3.1). However, the latter \( \mu_{CPTU} \) value is derived from the soil viscosity correlation presented in Table 1 of Dayal et al. (1975) and assuming a somewhat higher value for \( s_u \) (9 kPa) than those found in the other studies (Table 3.1). Calculations of the \( \mu_{CPTU} \) (Eq. 3.4) results in values of 0.043 to 0.109. For clay, the power-law exponent \( \beta \) defines a broad range of parameters varying between 0.02 and 0.07 (Table 3.2). Similar corrections were performed for the sleeve friction data (Fig. 3.19). Here, the \( \mu_{CPTU} \) value is
set at 0.9 (see Table 1 of Dayal et al. 1975), while 0.45 was used in other studies (Table 3.1). Values of $\mu'_{CPTU}$ are set to 0.391 and 0.195. Given that the power-law exponent $\beta$ has not been established for clay, an iterative parameter variation was conducted to obtain an approximate match between $f_{s,q - s}$ and $f_{s,ref}$, resulting in $\beta=0.13$ (Fig. 3.19e). The $q_{t,q - s}$ profiles considering higher SSCs of $\mu_{CPTU}=0.25$ and $\mu'_{CPTU}=0.109$ show significantly lower values compared to those from static tests (Figs. 3.18b, 3.18b). For the $f_{s,q - s}$, a similar underestimation, when using the higher values of $\mu_{CPTU}$ and $\mu'_{CPTU}$, can also be seen (Figs. 3.19b, 3.19b). In order to validate the best correlation between dynamic- and static-CPTU tests, a two-sample Kolmogorov-Smirnov test is performed (Eadie et al. 1971). This test results in the coefficient of determination ($R^2$) using the relationship defined as follows:

$$R^2 = 1 - \max | F_1(x) - F_2(x) |,$$

(3.11)

where $F_1(x)$ and $F_2(x)$ are the cumulative distribution of the dynamic- and static-CPTU records. For all selected soil-specific rate coefficients such a statistic test were performed and the associated $R^2$ values are calculated and illustrated in the different figure panels (e.g. Figs. 3.18, 3.19). Moreover, if using the SSCs summarized in Table 3.6 all $SF$-corrected $q_t$ and $f_s$ profiles show appropriate to good agreement with the static datasets. Table 3.6 further illustrates the associated $R^2$ values showing that the inverse sin-hyperbolic correction results in a best match...
3.2. POST PROCESSING OF DYNAMIC-CPTU RECORDS

Figure 3.17: Strength profiles of the uncorrected dynamic-CPTU tests ($q_{t,dyn}$ and $f_{s,dyn}$; SW27_fin) and static-CPTU measurements ($q_{t,ref}$ and $f_{s,ref}$; 947_envi) obtained from the same location in a slightly eroded region closely located to the 1996 landslide (see Fig. 3.12 and Table 3.5 for location and details). Also shown is the non-linear penetration rate profile ($v_{dyn}$).

Table 3.5: Summary of dynamic-CPTU measurements, static-CPTU tests and Calypso piston / gravity cores (cpc/gc) used for the correction and empirical parameter analysis. (Note: water depth (WD) and initial penetration rate ($v_0$))

<table>
<thead>
<tr>
<th>$v_0$ [m/s]</th>
<th>dynamic-CPTU</th>
<th>static-CPTU</th>
<th>coring</th>
<th>WD [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.1</td>
<td>SW01_fin</td>
<td>01_gost</td>
<td>01_cpc &amp; 02_cpc</td>
<td>44.5</td>
</tr>
<tr>
<td>3.6</td>
<td>SW02_fin</td>
<td>-</td>
<td>-</td>
<td>45.5</td>
</tr>
<tr>
<td>4.3</td>
<td>SW14_fin</td>
<td>-</td>
<td>-</td>
<td>45.2</td>
</tr>
<tr>
<td>5.8</td>
<td>SW15_fin</td>
<td>-</td>
<td>-</td>
<td>44.9</td>
</tr>
<tr>
<td>2.9</td>
<td>SW03_fin</td>
<td>-</td>
<td>04_gc</td>
<td>10.2</td>
</tr>
<tr>
<td>3.2</td>
<td>SW04_fin</td>
<td>-</td>
<td>-</td>
<td>11.7</td>
</tr>
<tr>
<td>2.7</td>
<td>SW05_fin</td>
<td>-</td>
<td>03_gc</td>
<td>17.0</td>
</tr>
<tr>
<td>3.4</td>
<td>SW06_fin</td>
<td>-</td>
<td>-</td>
<td>17.4</td>
</tr>
<tr>
<td>6.2</td>
<td>SW26_fin</td>
<td>947_envi</td>
<td>(03_gc)</td>
<td>14.5</td>
</tr>
<tr>
<td>5.0</td>
<td>SW27_fin</td>
<td>-</td>
<td>-</td>
<td>14.9</td>
</tr>
<tr>
<td>4.7</td>
<td>SW28_fin</td>
<td>205_envi</td>
<td>-</td>
<td>20.5</td>
</tr>
<tr>
<td>4.7</td>
<td>SW29_fin</td>
<td>-</td>
<td>-</td>
<td>21.0</td>
</tr>
</tbody>
</table>
for the \( q_t \) data while the power-law correction leads to a best fit for \( f_s \). However, in all \( f_s,q-s \) profiles, a decrease in the final (i.e. lowest) 15cm was detected. Moreover, \( q_t,q-s \) and \( f_s,q-s \) profiles utilizing the logarithmic and power-law equations show unrealistic, positive spikes for low penetration rates if \( v_{\text{dyn}} \) is much lower than \( v_{\text{ref}} \), and is applicable for the last couple of data points (i.e. approximately 1% of the entire data points) before the CPTU instrument comes to a complete halt. In order to evaluate this fact, additional statistical tests were carried out for the \( q_t,q-s \) and \( f_s,q-s \) profiles considering the logarithmic and power-law corrections without the last couple of data points. Table 3.6 shows the associated \( R^2 \) values resulting in a minor improvement of less than 1%. In contrast, \( q_t,q-s \) and \( f_s,q-s \) profiles corrected with the inverse sin-hyperbolic equation do not exhibit such irregularities (Figs. 3.18, 3.19).

From a geotechnical point of view, the inverse sin-hyperbolic correction leads to a \( q_t,q-s \) profile similar to the \( q_t,\text{ref} \) profile, except for the sections 0-0.7m and 2.1-2.4m. In the slope apron soils (0-0.7m), two distinct coarse beds or overconsolidated silty clays with an embedded fine-grained layer with \( q_t,q-s \) up to 300kPa are observed. At 2.1-2.4m depth, an alternating sequence of coarse- and fine-grained beds with individual thicknesses between 5 and 10cm is detected and quantified by \( q_t,q-s \) up to 350kPa for what are likely sandy silts, and at least 210kPa for what are likely sensitive clayey soils. This sequence potentially describes a weak layer exhibiting a 30-40% strength reduction in the clay-rich beds compared to the surrounding soils (Fig. 3.18). A similar trend is observed in \( f_s \).

<table>
<thead>
<tr>
<th>dynamic-CPTU test output parameter</th>
<th>SSC</th>
<th>( R^2 )</th>
<th>( R^2 ) without last data points</th>
</tr>
</thead>
<tbody>
<tr>
<td>SW27_fin</td>
<td>( q_t )</td>
<td>( \mu_{\text{CPTU}}=0.10 )</td>
<td>0.67</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( \mu'_{\text{CPTU}}=0.043 )</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( \beta=0.05 )</td>
<td>0.70</td>
</tr>
<tr>
<td></td>
<td>( f_s )</td>
<td>( \mu_{\text{CPTU}}=0.45 )</td>
<td>0.45</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( \mu'_{\text{CPTU}}=0.195 )</td>
<td>0.40</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( \beta=0.13 )</td>
<td>0.49</td>
</tr>
<tr>
<td>SW14_fin</td>
<td>( q_t )</td>
<td>( \mu_{\text{CPTU}}=0.10 )</td>
<td>0.79</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( \mu'_{\text{CPTU}}=0.043 )</td>
<td>0.82</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( \beta=0.029 )</td>
<td>0.82</td>
</tr>
</tbody>
</table>
3.2. POST PROCESSING OF DYNAMIC-CPTU RECORDS

Figure 3.18: Comparison of quasi-static corrected cone penetration resistance ($q_{t,q-s}$; SW27_fin) with static-CPTU data ($q_{t,ref}$; 947_envi), see Fig. 3.12 and Table 3.5 for location and details. The strain-rate corrections are performed using the logarithmic, inverse sinh-hyperbolic and power-law equations as well as associated SSCs reported previously, see Eqs. 3.2 to 3.5 and Tables 3.1 to 3.2.
Figure 3.19: Comparison of quasi-static sleeve friction ($f_{s,q-s}$; SW27_fin) with static-CPTU data ($f_{s,ref}$; 947_envi), see Fig. 3.12 and Table 3.5 for location and details. The strain-rate corrections are conducted using logarithmic, inverse sinh-hyperbolic and power-law equations as well as associated SSCs derived from previous work and empirical best match procedures, see Eqs. 3.2 to 3.5 and Tables 3.1.
for the weak layer, with a minimum value of 4 kPa in the fine-grained beds and a prominent peak of 9 kPa in the coarse-grained bed (Fig. 3.19d).

The second tests were conducted in an undisturbed region representing normal consolidation. The SW14_fin record is compared to the 01_gost test collected with the GOST system, and core 01_cpc in a WD of 44.5 m (see Table 3.5, Fig. 3.12 for location and Vanneste et al. 2013). The free-fall mode was used for the dynamic deployment with a \( v_0 \) of 4.3 m/s and a \( z_{\text{depth}} \) of 4.35 m (Fig. 3.20a). The soils below the seafloor in Finneidfjord are generally homogenous in depth and characterized as silty clay/clayey silt. However, from 2.85-3.30 m a distinct event bed is found. This event bed is characterized from top to bottom by a 15 cm thick clay layer overlying 20 cm of sandy silt to sand and another 10 cm grey clay. This sequence, which was deposited following a turbidity current at the mouth of the river, correlates to the slide plane of the 1996 landslide. It has therefore been interpreted as a weak layer (see L’Heureux et al. 2012, Steiner et al. 2012 for details). In such a weak layer, pore pressure or artesian groundwater pressure can be build up resulting in a decrease of the effective stresses and reduce the slope stability. The clay-sized content varies between 18 and 25% for the grey clay with a decrease to less than 10% in the sandy silt to sand beds (see also Fig. 3.15, Table 3.4). The coarse fraction is generally \( \sim \)5% and shows a significant increase from greater than 20% (sandy silt) up to 60% in the sand bed (see grain size in Fig. 3.20). In general, the \( q_{t,\text{ref}} \) profile varies between 100 and 400 kPa and displays a linear dependence on increasing \( \sigma'_{\text{V}_0} \) (Fig. 3.20b).

The \( q_{t,\text{dyn}} \) profile is 1.25 to 1.30 times higher than \( q_{t,\text{ref}} \) profile and illustrates a slightly increasing gradient (Fig. 3.20b). The static excess pore pressure (\( \Delta u_{2,\text{ref}} \)) profile varies between -20 and 60 kPa and shows high variability. The prominent negative peaks are particularly difficult to explain but could be the result of dilatancy (Fig. 3.20b). In contrast, the dynamic excess pore pressure (\( \Delta u_{2,\text{dyn}} \)) profile is nearly constant over the full profile and distinct excursions across thin individual layers encountered in the \( \Delta u_{2,\text{ref}} \) profile were not detected, probably due to the high penetration rate of \( v_0=4.3 \) m/s or imperfect saturation of the pore pressure sensor. Consequently, an adequate comparison of \( \Delta u_{2,\text{dyn}} \) with \( \Delta u_{2,\text{ref}} \) was not possible.

The \( \mu_{\text{CPTU}} \), \( \mu'_{\text{CPTU}} \) and \( \beta \) values appropriate for calculating \( q_{t,s} \) were determined using logarithmic, inverse sin-hyperbolic and power-law equations. The visual correlations between SF-corrected and static-CPTU measurements are illustrated in Figure 3.21. Results compare well for all \( q_{t,s} \) profiles using SSCs presented in Table 3.6. The \( R^2 \) value of 0.82 describes a best fit for the inverse sin-hyperbolic and power-law equations (Table 3.6). Consequently, SSCs are similar to the ones presented for SW27_fin, except for \( \beta \), which is slightly lower, but oth-
Figure 3.20: Sedimentological description and grain-size distribution of the 01_cpc as well as corrected cone penetration resistance and excess pore-water profiles of the SF-uncorrected dynamic-CPTU tests ($q_{t,dyn}$; SW14_fin) and static-CPTU measurements ($q_{t,ref}$ and $\Delta u_{2,ref}$; 01_gost) collected from a landslide in the undisturbed area. The non-linear penetration rate profile ($v_{dyn}$) is also shown, see Fig. 3.12 and Table 3.5 for location and details.

otherwise in better with the $q_{t,ref}$ profile (Fig. 3.21c). The unrealistic positive peaks for low penetration rates when $v_{dyn}$ is much lower than $v_{ref}$ are only observed for the logarithmic and power-law equations (Figs. 3.21a, 3.21c). This effect influences only 1% of the measured data points (i.e. centimeters of $z_{depth}$) and does not appear in the correction using inverse sin-hyperbolic equation (Fig. 3.21b). The $R^2$ values for the logarithmic and power-law corrections are nearly unchanged when omitting the last 1% of data points (Table 3.6).

The $q_{t,q-s}$ profile correlates with $\sigma'_{V0}$ and results in values up to 310 kPa, except the sand bed of the weak layer (Fig. 3.21b). A prominent decrease of 40% is detected in the grey clay beds compared to the homogeneous silty clay. In these clay beds, the $q_{t,q-s}$ profile starts at 150 kPa and increases slightly towards the sandy silt bed. A significant increase in $q_{t,q-s}$ is observed in the sandy silt and sand beds, showing a maximum value of 410 kPa (Fig. 3.21b). Using $q_{t,q-s}$ for the clay beds, an average empirical cone penetration factor ($N_{kt}$) of 14 was calculated ($N_{kt}$=10-18; Lunne 2010). The $N_{kt}$ was confirmed by back-calculations using in situ and DSS datasets (see also Table 3.5). The in situ intact undrained shear-strength ($s_{u,qt}$) of the grey clays is around 7 kPa and the sand bed exhibits values up to 24 kPa. The
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Figure 3.21: Comparison of quasi-static corrected penetration resistance ($q_{t,q-s}$; SW14_fin) with static-CPTU data ($q_{t,ref}$; 01_gost) including the lithological description of the potential weak layer, see Fig. 3.12 and Table 3.5 for location and details. The strain-rate corrections are performed using logarithmic, inverse sinh-hyperbolic and power-law equations as well as previously published SSCs, see Eqs. 3.2 to 3.5 and Tables 3.1, 3.2.

$s_{u,qt}/\sigma'_V$ is 0.24 for the clays, which is consistent with soft, normally-consolidated soils. In addition, the weak layer is also identified in the $\Delta u_{2,ref}$ profile by a massive drop in the sandy silt to sand beds and two positive peaks up to 40 kPa in the grey clays (Fig. 3.20c).

In summary, the comparison of dynamic- and static-CPTU tests show that the strain-rate correction using the power-law and inverse sinh-hyperbolic equations results in good accordance with $R^2$ values of 0.75 to 0.82 for $q_t$. Reasonable consistency is found for $f_s$ with a $R^2$ value of 0.49 using the power-law equation (Table 3.6). Moreover, grey clay beds containing a thin seam of sandy silts to sands show a significant drop in strength and increase of the excess pore pressure. This sequence may be interpreted as weak layer (see core log in Fig. 3.20).

Comparison of SWFF-CPTU with laboratory data  The $s_{u,qt}$ derived from the SW14_fin record using an average $N_{kt}$ parameter of 14 ($N_{kt}=10-18$; Lunne 2010) is compared with the laboratory $f_c$ and undrained DSS experiments on core 01_cpc ($s_{u,f_c}$ and $s_{u,DSS}$). In order to geotechnically characterize the
Figure 3.22: Comparison of quasi-static *in situ* intact undrained shear-strength ($s_{u,qt}$; SW14_fin) with laboratory intact undrained shear-strength ($s_{u,lab}$) derived from fall cone penetration (fc) and direct simple shear (DSS) experiments on core specimens of the 01_cpc, see Table 3.4. The strain-rate corrections are performed using the inverse sin-hyperbolic equation as well as a SSC of 0.043, see also Eq. 3.3 and Fig. 3.21. An average $N_{kt}$ of 14 is used.

weak layer, five undrained DSS experiments were performed, which show almost a perfect agreement with the SF-corrected $s_{u,qt}$ and $s_{u,fc}$ (Fig. 3.22), except the 30% lower DSS parameter compared to the *in situ* data in the sand bed. A reasonable agreement can be seen comparing the $s_{u,fc}$ profile with the dynamic-CPTU records, except for the uppermost 2.5m of soils. The $s_{u,fc}$ of the uppermost core section is less than 70% lower than $s_{u,qt}$ and may be influenced by sample disturbance (i.e. core transport/handling, storage time) and material loss during core recovery.

In summary, the comparison between the dynamic-CPTU records and results from fc and undrained DSS experiments confirms that the inverse sin-hyperbolic correction with a $\mu'_{CPTU}$ of 0.043 results in a better match than the other two methods (see also section [Comparison of dynamic-CPTU with static-CPTU systems]).

**Initial penetration rate and maximum depth** The (initial) penetration rate is not only important for the strain-rate correction, but also for the assessment of the maximum penetration depth, which is crucial for the planning of expeditions.
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Table 3.7: Coefficients \((k, d)\) used in conjunction with Equation (3.12) in giving the maximum penetration depth \(z_{\text{depth}}\) dependent to the initial penetration rate and consolidation state.

<table>
<thead>
<tr>
<th>state of consolidation</th>
<th>basic parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>normally-consolidated</td>
<td>1.10 2.86</td>
</tr>
<tr>
<td>normally-to over-consolidated</td>
<td>1.21 1.40</td>
</tr>
<tr>
<td>over-consolidated</td>
<td>1.16 0.91</td>
</tr>
</tbody>
</table>

(i.e. achievable targets). Figure 3.23 illustrates an empirical relationship between \(v_0\) and the maximum penetration depth including the differences between \textit{winch mode} (denoted by WM in Figure 3.12) and \textit{free-fall mode}, and the consolidation state of the clayey soils (in our case given by \(s_{\text{u,qt}}/\sigma'_{V0}\)). Two dynamic-CPTU tests were usually deployed at nearly the same location (i.e. distance less than 5 m). The seafloor morphology was also used to confirm the consolidation state of the tested areas. \textit{In situ} tests in over-consolidated clays are found under erosion surfaces or within transported blocks and denoted by OC in Figure 3.12 (see also Longva et al. 2003, L’Heureux et al. 2012). The maximum \(z_{\text{depth}}\) is empirically described by:

\[
z_{\text{depth}} = kl\ln(v_0) + d, \quad (3.12)
\]

where \(k\) and \(d\) depend on the state of consolidation (Table 3.7). The \(k\) parameter varies from 1.1 to 1.21, while the \(d\) parameter ranges from 0.91 to 2.86 for normally- to over-consolidated soils, respectively. This relationship illustrates that \(z_{\text{depth}}\) of free-fall deployments is 1.6 to 2.25 times deeper than that of winch-guided dynamic deployments in normally- to over-consolidated soils. Overall, the \textit{MARUM} dynamic-CPTU probe with \textit{free-fall mode} is well suited to achieve \(z_{\text{depth}}\) up to 5 m in clay-rich soils.

Discussion

\textbf{Strain-rate correction} In total, 34 dynamic-CPTU tests were collected in the vicinity of the 1996 Finneidfjord landslide area. An \textit{in situ} comparison of eight dynamic tests with three contiguous static-CPTU measurements were performed, and additional four dynamic records were compared with \(f_c\) and DSS datasets derived from gc and cpc samples (see Fig. 3.12, Table 3.5 for location and details). For the dynamic- and static-CPTU tests, one self-designed and two different industrial piezocones were deployed with maximum capacities of 25 to 70 MPa for \(q_c\) and 0.25 to 1 MPa for \(f_s\) and \(u_2\). The minimum accuracy of these three piezocones is better than \(\pm25\text{kPa}\) (see section \textit{Methodology}), which is in accordance with ISSMGE and ISO 22476-1 application/test class 1 required for soft or loose soils (see also Powell and Lunne 2005, Sandven 2010 for details). However, the CPTU
Figure 3.23: Correlation between initial penetration rate ($v_0$) and penetration depth ($z_{depth}$), noting different consolidation states, for all dynamic-CPTU tests in the study area. The average error of ±2% in depth is illustrated for each test. The in situ undrained shear-strength ratio ($s_u/\sigma' V_0$) for normally-consolidated soils are smaller than 0.5, while over-consolidated soils exceed 0.5 (see also Fig. 3.12 for locations and Lunne et al. 1997). The derived empirical equation and basic parameters are presented in Eq. 3.12 and Table 3.7.

Profiles are recorded in the low measuring range of the piezocones, and are quantified with less than 20 kPa for $f_s$, and 0.6 MPa for $q_c$ and $u_2$. This fact reduces probably the resolution of the measured CPTU profiles (e.g. Sandven 2010). In this study, different piezocones with various diameters from 5 to 15 cm$^2$ are used and associated datasets are compared. Titi et al. (2000) compare also CPTU tests with different piezocone diameters, and determine around 10% higher $q_t$ and $f_s$ for a 2 cm$^2$ cone in contrast to a 15 cm$^2$ cone. Moreover, minor variabilities for $q_t$ and less than 4% differences for $f_s$ were ascertained using CPTU measurements collected with similar equipment setups and at the same position, but until now,
the causes are not completely understood (Powell and Lunne 2005, Lunne 2010). Furthermore, CPTU tests are strain-rate dependent measurements resulting in a rise of the dynamic-CPTU records of 35 to 140% for $q_t$ and $f_s$ compared to static-CPTU and laboratory datasets in connection with an increase of the penetration rate, known as strain-rate effect (Figs. 3.17, 3.20 and Stoll et al. 2007, Young et al. 2011). In addition, the $q_{t,ref}$ profile measured with the GOST shows in the first meter up to two times higher values compared to the same $q_{t,q-s}$ section measured with the SWFF-CPTU (Fig. 3.21), most likely caused by the different deployment modes of both systems. The GOST system loads the sub-seafloor soils with more than 6 tons, thus affecting the in situ effective stresses and leading to consolidation of clays as well as compaction of sands (Fig. 3.20, Meunier et al. 2004). In contrast, the uppermost soils are not loaded by the SWFF-CPTU instrument so that the natural soil conditions can be recorded (Fig. 3.21, Stegmann et al. 2006). Consequently, the investigated differences between dynamic-, static-CPTU and laboratory datasets can potentially be the result of: (i) effects related to the comparison of different equipments, (ii) the strain-rate effect, (iii) the spatial variability in the natural geological settings, and (iv) time between the different surveys (i.e. from 1997 until 2010). However, all tests were carried out at nearly the same location (i.e. distance less than 10 m), and the integrated datasets of high resolution seismics, cores and CPTUs have shown that the spatial variability in the natural soils within given areas in the fjord is minimal (see L’Heureux et al. 2012, Steiner et al. 2012, Vardy et al. 2012). Moreover, since no major sedimentary event (e.g. landslide) has occurred in the study area after 1996, the soil conditions in 1997 were most likely similar to those in 2010.

Three state-of-the-art in situ strain-rate correction methods (logarithmic, inverse sin-hyperbolic and power-law) are enhanced by: (i) replacing the $v_{dyn}/v_{ref}$ by the "non-dimensional velocity ratio" (Eq. 3.8, Randolph and Hope 2004, Lehane et al. 2009, Robertson 2010), (ii) assuming that the dynamic- and static-CPTU tests are conducted under fully undrained penetration conditions (Table 3.3), thus comparable with fc and undrained DSS experiments (Vanneste et al. 2013) and (iii) excluding effects related to different piezocone diameters (see also section [Drained, partially drained and undrained penetration]). The in situ strain-rate corrections in clay-rich soils has resulted in an appropriate to good agreement for the three methods tested when using the SSCs illustrated in Table 3.6, Figures 3.18, 3.18h, 3.21 for the $q_t$ and Figures 3.19, 3.19h, 3.19k, Steiner et al. (2012) for the $f_s$. However, using other SSCs, for instance those derived from the soil viscosity correlation proposed by Dayal and Allen (1975), resulted in an underestimation of the natural strength properties (Figs. 3.18, 3.18h, 3.19, 3.19k) or a large scatter in the in situ properties (Fig. 3.18). The other SSCs were derived from experiments on homogeneous, remolded clays (e.g. Dayal and Allen 1975) or soft clays (Mesri and Castro 1987), which have different properties and responses compared to the normally- to over-consolidated clay-dominated soils tested in this study. In
addition, prominent positive and negative anomalies are observed for the logarithmic equation, in particular if \(v_{dyn}\) is below \(v_{ref}\), affecting only 1% of the entire data points (Figs. 3.18c, 3.19b, 3.21a). This inconsistency is potentially based on: (i) the mathematical form of the equation, which goes towards infinity when the \(v_{dyn}\) approaches zero (Randolph 2004), and (ii) the transition from a completely undrained to partially drained penetration (e.g. DeJong et al. 2011).

The inverse sin-hyperbolic equation is well suited to correct in situ dynamic-CPTU records in clays with up to \(v_0=7.8\) m/s due to an existing correlation between \(\mu_{CPTU}\) and \(\mu'_{CPTU}\) (Eq. 3.4). The power-law equation agrees also well with the static datasets, but illustrates unrealistic positive peaks at the last 1% of data points, just as the logarithmic equation (Figs. 3.18h, 3.19e). These findings were confirmed by two-sample Kolmogorov-Smirnov tests (Eadie et al. 1971). These statistical tests were used to assess the "goodness of fit" between the dynamic- and static-CPTU records leading to the highest \(R^2\) values for the inverse sin-hyperbolic and power-law equations and the lowest \(R^2\) values mostly for the logarithmic equation (Table 3.6). The results of this study (Figs. 3.18, 3.19, 3.21, 3.22) are supported by previous studies using laboratory experiments that also employ an inverse sin-hyperbolic equation (Randolph and Hope 2004, Rattley et al. 2008).

Consequently, we suggest that the strain-rate correction method using an inverse sin-hyperbolic equation, adequate SSCs and "non-dimensional velocity ratio" is best suited for dynamic-CPTU tests in clay-rich soils (Eqs. 3.3, 3.4, 3.8 and Table 3.1).

In clayey soils, the significant increase of the \(q_{t,dyn}\) is quantified by SFs less than 1.35 (Figs. 3.18, 3.21b). Similar values have been obtained for various types of penetrometers (e.g. XBP, CPT stinger) and SFs were calculated being up to 1.4 for \(q_t\) and \(s_u\) in soft clay and normally- to slightly over-consolidated soils for \(v_0\) varying between 4 and 9 m/s (Young et al. 2011, Aubeny and Shi 2006). Table 3.8 presents a summary of the maximum SFs calculated in this and other studies including the soil types and output parameters. Dayal et al. (1975) present dynamic field tests on harbor soils in Newfoundland using \(\mu_{CPTU}=0.25\) derived from in situ vane shear tests in conjunction with the soil viscosity correlation of Dayal and Allen (1975). The strain-rate correction results in SFs up to 1.7, consistent with some results of this study (Fig. 3.18h), however, significantly higher than those presented for clayey soils in other studies (e.g. Randolph and Hope 2004, Young et al. 2011). A back-calculation was performed to evaluate this inconsistency using a \(q_{t,dyn}\) of 168 kPa and an assumed \(N_{kt}\) value of 14 (well constrained by CAUC tests, denoted in the symbol list as well as described in detail by Lunne et al. [1997] and Lunne [2010]) in order to determine the \(s_{u qt}\). This calculation shows that the \(\mu_{CPTU}\) of Dayal et al. (1975) leads to an underestimate of the \(s_{u qt}\) (see their example of \(\sim 6.5\) kPa compared to the measured 8.8 kPa). However, when using \(\mu_{CPTU}=0.1\) (see also section [Comparison of dynamic-CPTU with static-CPTU systems], Ran-
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Table 3.8: List of the maximum strain-rate factors derived from this and other studies. (Note: cone penetration resistance ($q_c$), corrected cone penetration resistance ($q_t$), sleeve friction ($f_s$), intact undrained shear-strength ($s_u$) and soil bearing capacity ($q_u$)).

<table>
<thead>
<tr>
<th>soil type</th>
<th>output parameter</th>
<th>maximum strain-rate factor</th>
<th>reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>clay to silty clay</td>
<td>$q_t$</td>
<td>1.25-1.35</td>
<td>this study</td>
</tr>
<tr>
<td>remolded clay</td>
<td>$f_s$</td>
<td>2.10-2.40</td>
<td>Dayal and Allen (1975)</td>
</tr>
<tr>
<td>(Pottery clay)</td>
<td>$q_c$</td>
<td>up to 1.70</td>
<td></td>
</tr>
<tr>
<td>clay</td>
<td>$f_s$</td>
<td>up to 3.30</td>
<td>Aubeny and Shi (2006)</td>
</tr>
<tr>
<td>clay</td>
<td>$s_u$</td>
<td>1.35-1.40</td>
<td>Young et al. (2011)</td>
</tr>
<tr>
<td>clay to silty clay</td>
<td>$q_t$</td>
<td>up to 1.50</td>
<td>Steiner et al. (2012)</td>
</tr>
<tr>
<td>clay</td>
<td>$f_s$</td>
<td>up to 2.30</td>
<td></td>
</tr>
<tr>
<td>water-saturated</td>
<td>$q_c$</td>
<td>2.70-4.40</td>
<td>Stoll et al. (2007)</td>
</tr>
<tr>
<td>sand</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>quartz and</td>
<td>$q_u$</td>
<td>3.50-4.00</td>
<td>Stark et al. (2009)</td>
</tr>
<tr>
<td>carbonate sand</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

dolph and Hope (2004, Young et al. 2011) and following Dayal et al. (1975), SFs are in the range of 1.3 and result in a $s_{uqt}$ of $\sim$8.5 kPa at 1m depth, which is in good agreement with the measured 8.8 kPa. Hence, the tests in this study and the back-calculation using Dayal et al. (1975) example yield basically the same $\mu_{CPTU}$ for the $q_t$ (Figs. 3.18c, 3.21b).

The strain-rate correction for $f_s$ gives SFs up to 2.4 (Figs. 3.19b, 3.19d), while the SFs proposed by Dayal and Allen (1975) are significantly higher (Table 3.8 and Fig. 3.19b). Young et al. (2011) suggest that the $f_{s,dy}$ and $\Delta u_{2,dy}$ records do not need to be corrected for strain-rate effects. However, for $f_s$ these findings disagree with the results of this study (Figs. 3.19b, 3.19d), and a similar disagreement is found for $\Delta u_{2,dy}$ (see results from deep-water dynamic-CPTU tests in the Mediterranean Sea; Kopf et al. 2012).

The penetration process in coarse-grained soils is usually drained and the derived geotechnical parameters are related to drained behavior (e.g. Lunne et al. 1997, Robertson 2009). Here, the non-dimensional velocity is used to evaluate
the drainage behavior during penetration, and is quantified by \( \sim 1930 \) considering the 15 cm\(^2\) dynamic-CPTU piezocone, \( c_{v,DSS} \) value of \( 6.8 \times 10^{-5} \text{ m}^2/\text{s} \) for the sand bed (Table 3.4), and \( v_{dyn} \) of 3 m/s taken from Figure 3.20 at 3.1 m depth. This value is significant higher compared to the undrained penetration criterion of \( \sim 30 \) presented in Table 3.3, Randolph (2004) and Schneider et al. (2007). Consequently, the used undrained strength method with an average \( N_{kt} \) parameter of 14 \( (N_{kt}=10-18; \text{Lunne } 2010) \) is also appropriate for the 0.2 m thick sandy silt to sand bed (Fig. 3.22). However, 30% higher quasi-static CPTU data are observed in contrast to the undrained DSS results (Fig. 3.22), which is potentially attributed to: (i) sample disturbance during preparation in the laboratory and/or (ii) particularly accentuation of the quasi-static CPTU data, when more resistant, coarse-grained material is penetrated (e.g. Stegmann et al. 2006). An accentuated strength behavior has also been found using dynamic penetrometer tests (PRO-BOS and NIMROD) in water-saturated sands (a correction of \( \mu_{CPTU} \) between 0.7 and 1.5 lead to \( SF \)s up to 4.4 at penetration rates of 4-6 m/s; see Tables 3.1, 3.1, and Stoll et al. 2007, Stark et al. 2009).

In this study, we demonstrate that strain-rate correction for dynamic-CPTU records is not universally appropriate and adequate SSCs must be evaluated for the individual CPTU parameters and the specific type of material or setting (e.g. Tables 3.1, 3.2 and Figs. 3.18f, 3.19d, 3.21b, and Stoll et al. 2007, Young et al. 2011).

Correlation of penetration depth to initial penetration rate and consolidation state

The weight of the CPTU device, \( d_{dyn} \), \( v_0 \) and a number of soil properties including consolidation state of the sediment are crucial influencing factors with respect to \( z_{\text{depth}} \). The first two factors do not vary in this study since the same dynamic-CPTU setup was used for all deployments. However, the \( v_0 \) differed in the range of 0.9 to 7.8 m/s between tests (Table 3.5) and the state of consolidation is different, too. Theoretical studies have shown the difficulties in correlating \( v_0 \) with the state of consolidation and \( z_{\text{depth}} \) (e.g. Boguslavskii et al. 1996). Similar to the results from Mulukutla et al. (2011), our data show a direct correlation between \( v_0 \) and \( z_{\text{depth}} \) (Fig. 3.23). This correlation is expressed by a logarithmic relationship for clayey sediments (Eq. 3.12 and Table 3.7), and a similar trend has been found for sand in Boguslavskii et al. (1996). When supplementing the logarithmic correlation by the state of consolidation expressed by \( s_{u,qt}/\sigma'_{V0} \), an ordinary soil classification model for clays is derived (Fig. 3.23). Mulukutla et al. (2011) proposed a more elaborate sediment identification model for coarse silt to medium sand taking into account the normalized \( z_{\text{depth}} \) and firmness factor consisting of the peak acceleration, acceleration due to gravity, total embedment duration, and \( v_0 \). However, both models demonstrate that an increase in \( z_{\text{depth}} \) directly correlates to a reducing state of consolidation or firmness factor (Fig. 3.23).
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Mulukutla et al. (2011).

The free-fall data of this study reached $z_{\text{depth}}$ up to 2.25 times higher than deployments with *winch mode*. Such knowledge is important for planning *in situ* geotechnical characterization of near-surface targets. In *Finneidfjord*, $z_{\text{depth}}$ up to 5 m would be sufficient to assess potential pipeline and cable routes (Lunne et al. 2011) and targets in the surficial sub-seafloor, such as weak layers in landslide areas (L’Heureux et al. 2012).

**Conclusions**

Dynamic cone penetration testing with pore pressure measurements (dynamic-CPTU) provides a valuable and time-efficient method for shallow offshore geotechnical site characterization. The results are affected by rate-dependent deformation and need to be corrected to match conventional CPTU parameters and laboratory results. In this study, three existing methods, relying on logarithmic, inverse sinhyperbolic, power-law equations and soil-specific rate coefficients (SSCs), were assessed for correcting *in situ* dynamic-CPTU tests at penetration rates from 7.8 m/s to zero in Norwegian marine clays. The SSCs best suited for clayey soils were determined using dynamic and conventional CPTU collected at nearly the same location. When using adequate SSCs, all methods yield appropriate to good agreement for tests with high initial penetration rates. However, the inverse sinhyperbolic equation is best suited, because it has the highest coefficient of determination and is the only correction method showing no artifacts at the last 1% of the recorded data points.

Weak layers control the stability of submarine slopes at all scales and in various environments. They often consist of sensitive clay-rich soils embedded within thin beds of sand and silt. In the vicinity of *Finneidfjord* (northern Norway), such weak layers were identified and characterized using the *MARUM* dynamic-CPTU instrument. In summary, the time-efficient *in situ* dynamic method is appropriate to accurately measure soil properties in marine fine-grained soils. Dynamic-CPTU measurements in conjunction with sedimentological and geotechnical results from cores are well suited for detailed area-wide sub-seafloor characterization of importance for adequate design of surficial foundations, cable and pipeline tracks, or slope stability assessments.

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3.2.4 Geological and geotechnical analyses

The primary quasi-static CPTU profiles collected with dynamic penetrometers are equivalent to static-CPTU profiles. Consequently, the primary quasi-static CPTU profiles, in conjunction with existing geological and geotechnical approaches, can be applied to engineering and scientific problems in coastal, nearshore and offshore environments (Fig. 3.24). The required theoretical and empirical correlation factors for the various approaches are taken from previous studies and comparisons between in situ and laboratory tests. In addition, a modified sleeve friction solution and soil classification system are described hereafter and both are important for characterizing and classifying sub-seafloor soils.

In clayey soil, the $s_u$ and in situ remolded undrained shear-strength ($s_{ur}$) are crucial design parameters in offshore engineering and research and both can be derived from in situ CPTU tests (e.g. quasi-static CPTU parameter). The determination of $s_u$ is based on the approaches presented in section 2.1.4. For estimation of $s_{ur}$, three different techniques are appropriate: (i) sleeve friction measurements from dynamic- and static-CPTU cone (Lunne et al. 1997, Robertson 2009b, Farrar 2010), (ii) vane shear, direct simple shear or triaxial experiments performed in the laboratory (Terzaghi et al. 1996, chapter 3), and (iii) cyclic T-bar and Ball full-flow penetration tests (Low et al. 2010, Yafrate and DeJong 2005). The first technique provides a first-order estimate appropriate for reconnaissance and preliminary design, while the latter ones are well-suited for detailed design purposes (e.g. Lunne et al. 2011).

In the following, a modified sleeve friction solution (Eq. 3.13) in combination with values of $s_{ur}$ derived from v-s experiments on core specimens ($s_{ur,v-s}$) are introduced. The modified solution ($mod. f_s \equiv s_{ur,v-s}$) is required due to incomplete
remolding of the sediments when using a conical tip, because a plastic zone is only locally generated around the tip during penetration (Konrad and Law 1987). This is addressed by an empirical sleeve friction factor \( N_{fs} \), which reduces the \( f_s \) as:

\[
\text{mod.} f_s \approx s_{ur,v-s} \approx \frac{f_s}{N_{fs}}.
\]  

(3.13)

The \( s_{ur,v-s} \) serves as a reliable reference dataset from which \( N_{fs} \) is determined from comparisons of \( \text{mod.} f_s \) with \( s_{ur,v-s} \) measured on adjacent gc. Figure 3.25 illustrates comparisons of in situ quasi-static CPTU tests with v-s experiments taken at 10 cm increments of a gc, and static-CPTU tests collected at the western shelf off the international airport of Nice (southeastern France). The \( \text{mod.} f_s \) acts as a standard addendum to the laboratory parameters, but its applicability is limited to soils with soil sensitivity \( (S_t) \) less than 10 (Robertson 2009b). Good agreements are detected for the all three measurements, in particular for the comparison of the \( s_{ur} \) and \( S_t \).

In the 1970s, a new generation of "electric cones" was developed in order to measure \( q_c \) and \( f_s \) simultaneously. These measurements offer the possibility of assessing different soil classes by relating each of these parameters with in situ total and effective stress conditions, called the friction ratio concept (Schmertmann 1969, 1978, Douglas and Olsen 1981). This concept compares the normalized cone
Figure 3.25: Sedimentological and geotechnical characterization for silty clay dominated soils including gravity core description (core-log), intact and remolded undrained shear-strength ($s_u$ and $s_{ur}$), and derived soil sensitivity ($S_t$). For the calculation of the in situ intact undrained shear-strength the $N_{kt}=15-20$ (17) is used (Lunne 2010) and $N_{fs}=1.25$ is applied to derive the $s_{ur}$. Three different dynamic-CPTU tests, a static-CPTU measurement and vane-shear data collected at the western shelf off the Nice international airport (step25_nice, step02_nice, step01_nice, p12-s1, 13946_gx) are compared in order to calculate the soil sensitivity and prove the sleeve friction approach. (Note: modified sleeve friction ($mod.f_s$), intact and remolded vane shear-strength($s_{u,v-s}$ and $s_{ur,v-s}$)
3.2. POST PROCESSING OF DYNAMIC-CPTU RECORDS

penetration resistance \((Q_t)\) with the friction ratio \((R_f)\):

\[
Q_t = \frac{q_t - \sigma_{V0}}{\sigma'_{V0}},
\]

\[
R_f = \frac{f_s}{q_t - \sigma_{V0}}.
\]

Over the last 25 years, several workers have used this concept to estimate soil classification models for fine- to coarse-grained soils (e.g. Robertson and Campanella 1983a, 1983b, Wroth 1988, Ramsey 2002). Ramsey (2002) proposed a friction ratio model, which is based on a large number of CPTU tests and laboratory experiments including a statistical evaluation of the output data (Fig 3.26). This method allows precise distinction between clayey, silty and sandy soils, but it has only been validated for uncemented, non-calcareous North Sea Quaternary soils.

In summary, this post-processing procedure describes a comprehensive and coherent methodology for analyzing dynamic-CPTU tests. These data can be used

Figure 3.26: Novel friction ratio concept, which compares the normalized cone penetration resistance \((Q_t)\) with the friction ratio \((R_f)\) (after Ramsey 2002).
for sub-seafloor modeling, slope stability and risk assessment, as well as other engineering and scientific purposes (e.g. reconnaissance design of cable and pipeline routs). However, full treatment of all objectives is beyond the scope of this doctoral thesis.

3.3 MATLAB routine

A novel and extensive MATLAB routine, called GeoDynamic-CPTU, was developed as part of this research project in order to consider all above presented tasks in a simple and efficient manner (e.g. penetration depth calculation, strain-rate correction, soil classification). In addition, all datasets collected using different MARUM instruments (e.g. shallow-water or deep-water dynamic-CPTU) can be analyzed (i.e. all available dynamic-CPTU cones and transducer configurations are considered). User supporting tools are also implemented, for example, assemblages of several 15 minute ascii-files, offset and calibration tools, automatic start and stop time pickers, and comparison between dynamic- and static-CPTU profiles.

The handling of the GeoDynamic-CPTU routine is user-friendly, i.e. all user input parameters are supplemented by suggestions in the square brackets (Fig. 3.27a). These suggestions are based on literature studies and the findings of this research project and are mainly appropriate for fine-grained soils. The choice of the instrument configuration or additional options, such as with or without inclination correction and type of strain-rate correction, can be selected using individual nomenclature combined with an associated number that has to be entered as a selection (Fig. 3.27b). In addition, the MATLAB routine includes detailed comments and suggestions for the user in order to provide a better understanding of the routine and post-processing of dynamic-CPTU records (green text of Fig. 3.27). The entire MATLAB routine can be found as digital m-file in the Appendix B.
Figure 3.27: (a) Example of an input parameter suggestion, and (b) Example to choose different CPTU instrument configurations or selection options.
Chapter 4

Geotechnical applications

From 2007 until 2012, several scientific cruises with R/Vs *L’Europe*, *Meteor*, *Poseidon* and *Seisma* were carried out in the landslide-prone areas off the: (i) village of Finneidfjord (*Sørfjorden*, northern Norway) and (ii) Nice international airport (southeastern France). A large number of dynamic-CPTU records and gc were retrieved and used to address the following research questions:

- Is the dynamic *in situ* technique an useful addition to static-CPTU and coring methods with regard to the determination of the physical properties of the soils?

- Is it possible to identify and characterize surficial weak layers, or fine-grained and coarse-grained event beds using dynamic-CPTU profiles?

- Are dynamic-CPTU tests suitable to develop geological, geotechnical transects and area-wide sub-seafloor models?

- Do sub-seafloor modeling using a multidisciplinary approach (i.e. combination of bathmetrical, geophysical, geotechnical laboratory and *insitu* data) and 2D numerical calculations result in a better understanding of the natural properties and behavior of the submarine environments?

The following datasets were used to develop three geological and geotechnical manuscripts published in a scientific conference proceedings volume and submitted to a scientific journal. Further project data, with the potential for additional publications, are summarized in section 5.
4.1  *Finneidfjord* landslide

4.1.1  Physical properties of soils

Dynamic-CPTU datasets, collected in the vicinity of the 1996 *Finneidfjord* landslide area (*Sør fjorden*, northern Norway), were compared to static-CPTU results in order to verify and validate the state-of-the-art logarithmic strain-rate correction solution, which is utilized to determine quasi-static CPTU profiles from measured dynamic parameters. The quasi-static CPTU profiles were strengthened by comparisons with \(v-s\) and \(f_c\) laboratory results, taken from cored samples and collected at nearly the same location. The high quality of the quasi-static CPTU profiles enabled to identify and characterize a surficial 45 cm thick weak layer consisting of a soft, sensitive clay layer overlain by a sandy silt to sand bed and another soft, sensitive clay layer. In addition, a geological and geotechnical transect appropriate for slope stability assessments was developed including the weak layer. These findings were presented in the following manuscript accepted for the *Submarine Mass Movement and Their Consequences* conference 2011. The manuscript was honored with the *Best Paper Award* of the conference.
4.1. FINNEIDFJORD LANDSLIDE

An in situ free-fall piezocone penetrometer for characterizing soft and sensitive clays at Finneidfjord (northern Norway)

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Abstract

The identification and characterization of weak layers, potentially acting as detachment planes, are key elements in submarine landslide research. In this study, the MARUM Free-Fall Piezocone Penetrometer (FF-CPTU) was used to characterize soft and sensitive clays at Finneidfjord, a site with historical landslide events. The FF-CPTU measurements are compared with standard, industry Cone Penetration Testing (pushed CPTU) data in order to verify and validate the penetration rate effect by using an empirical closed-form solution to convert dynamic properties to quasi-static ones. The quasi-static properties and sedimentological/laboratory results across the weak layers show significantly lower values for the CPTU parameters ($q_t$ and $f_s$) and undrained shear-strength ($s_u$), build-up of excess pore pressure ($\Delta u$) as well as a normally-consolidated behavior in comparison with the surrounding sediment. These findings allow us to develop a 2D model of the subsurface in which the weak horizon is mapped, and to demonstrate that the light-weight FF-CPTU instrument is a powerful, versatile device.
for the geotechnical evaluation of submarine mass movements and their consequences.

*Key words:* Free-fall cone penetration testing, soft and sensitive clay, weak layer, submarine landslide.

**Introduction**

The presence of weak layers, in this study defined as soft and sensitive clay strata, in conjunction with pre-conditional factors (e.g. rapid sediment accumulation and under-consolidation, gas charging, excess pore-water pressure) has recently been recognized as an important factor for the initiation of slope instabilities and submarine landslides (e.g. Haffidason et al. 2003, Longva et al. 2003). Hence, state-of-the-art *in situ* measurements across such layers are important in order to improve our knowledge of the initiation of submarine landslides and for geohazard assessments.

CPTU is an efficient and widely used *in situ* measurement technique for geotechnical site investigations, both onshore and offshore (Lunne et al. 1997, Osler et al. 2005). It measures three parameters in a semi-continuous way upon penetration, these being cone resistance, sleeve friction and pore pressure (pp) (with possibility for dissipation tests). The combination of these parameters yields vital information on lithology, soil stratification and geotechnical engineering properties. An autonomous, lightweight, modular, FF-CPTU instrument was recently designed at MARUM - Center for Marine Environmental Sciences and Faculty of Geosciences, University of Bremen. This instrument is a straightforward, cost- and time-effective technique to measure de-/accelerations, tilt/temperature and CPTU parameters ($q_c$, $f_s$ and $u_2$) (Stegmann et al. 2006).

In this paper, a set of FF-CPTU measurements collected off the village of Finneidfjord, Northern Norway (Fig. 4.1), is compared with pushed CPTU and laboratory data. The aim is (i) to explain the procedure and handling of the FF-CPTU results with respect to the dynamic, non-linear penetration velocity (Dayal and Allen 1975) in contrast to the fixed penetration velocity (2 cm/s) used in pushed CPTU tests (Lunne et al. 1997), and (ii) to use the data set to identify and geotechnically characterize the slide-prone layers at Finneidfjord.

**Setting**

On June 20th, 1996 a combined submarine/subaerial, retrogressive landslide mobilized approximately $1 \times 10^6$ m$^3$ of sediment and claimed four human lives (Longva
4.1. FINNEIDFJORD LANDSLIDE

Figure 4.1: Morphological analysis according to swath bathymetry data off Finneidfjord presenting the 1996 landslide deposits and landslide scars. The locations of the FF-CPTU measurements, pushed CPTU tests, and Calypso piston and gravity cores are illustrated.

et al. 2003). Ground investigations conducted prior and subsequent to the landslide illustrate that the sediments comprised Holocene silty clay to sandy silt and a pocket of soft, sensitive clay below the shore- and foreshore regions. At the shoreline, the basement lies between 15 and 20 meter below ground level (mbgl) (Janbu 1996, Longva et al. 2003). A schematic illustration of the near-shore setting is presented in L’Heureux et al. (2012).

Material and methods

This study comprises 38 measurements using a FF-CPTU instrument. This instrument relies on a 15 cm² Geomil subtraction piezocone measuring the cone pen-
etraction resistance \( (q_c) \), sleeve friction \( (f_s) \), penetration pp measured behind the tip \( (u_2) \), and tilt/temperature. The probe is extended by metal rods (maximum length up to 8.5 m) and hosts a pressure housing at the top containing microprocessor, tiltmeter and accelerometer (Stegmann et al. 2006). The penetration depth calculation is based on the 1\(^{st}\) and 2\(^{nd}\) integration of the de-/acceleration taking into account a maximum error of \( \leq 2\% \). In our study, the terminal depth varied between 2.0 and 5.3 m below seabed (mbsb) with the vertical resolution of measurements being \( <1 \) cm. The accuracy of the piezcone sensors ranges between 2 and 5 kPa.

Two pushed CPTU tests (205 and 947; Fig. 4.1), performed in 1995 by the Norwegian Road Authorities, were used to compare and validate the FF-CPTU records. A penetration depth of 10 to 20 m mbsb was achieved with a conventional vertical resolution of 2 cm. The 10cm\(^2\) Memocone MK II probe sensors had an accuracy varying between 2 and 20 kPa.

We use also results from 2 short gravity and 2 longer Calypso piston cores collected in 2009 and 2010. Detailed sedimentological description and various geotechnical analyses (e.g. water content, peak and remolded undrained shear-strength using fall cone penetration tests) were performed on the gravity cores. On the Calypso piston cores sedimentological and geotechnical analyses were undertaken including gamma density and magnetic susceptibility measurements using a GEOTEK Multi Sensor Core Logger (MSCL).

Results

Comparison of FF-CPTU and pushed CPTU tests Conventional CPTU equipment is pushed in the soil with a quasi-static penetration velocity of 2 cm/s (Lunne et al. 1997). This is a major difference compared to FF-CPTUs. Due to their non-linear penetration velocity, the dynamics of the probe accentuate the natural sediment parameters (strain-rate rate effect). To correct this effect, a strain-rate rate factor (PRF) represented by an empirical closed-form solution (Eq. 4.1) and developed for clay and sand was introduced (Dayal and Allen 1975). This solution is based on a logarithmic function subject to the ratio of the dynamic \( (v_{dyn}) \) and quasi-static penetration velocity \( (v_{q-s}=2 \text{ cm/s}) \), and the soil viscosity coefficients \( (K_{qt} \text{ and } K_{fs}) \) associated for cone resistance and sleeve friction (Eq. 4.1).

\[
PRF = 1 + K_{qt}K_{fs}\log_{10}\left(\frac{v_{dyn}}{v_{q-s}}\right) \tag{4.1}
\]

The soil viscosity coefficients are determined by back-calculations (Eqs. 4.1 to 4.3) using comparison of FF-CPTU and pushed CPTU tests (Fig. 4.2), and
4.1. FINNEIDFJORD LANDSLIDE

Figure 4.2: Comparison of the dynamic and quasi-static FF-CPTU corrected cone resistance and sleeve friction with the pushed CPTU data. Note that the upper 0.5 m of the pushed CPTU 947 shows inconsistent results (most likely related to internal effects of the equipment). (Note: cone resistance corrected for pp effects ($q_t$), sleeve friction ($f_s$))

valuated by Dayal and Allen 1975. The value for the cone resistance ($K_{qt}$) varies between 0.1 and 0.2. The sleeve friction coefficient ($K_{fs}$) is expected to be between 0.3 and 0.7. The hereafter presented data are based upon $K_{qt}=0.13$ and $K_{fs}=0.45$.

Figure 4.2 illustrates the comparison of the FF-CPTU data with pushed CPTU results at two locations. The first comparison includes the FF-CPTU 26 and 27 and pushed CPTU 947, whereas FF-CPTU 28 and 29 and pushed CPTU 205 are compared at the second location (Fig. 4.1 for location).

The quasi-static properties ($q_{t,q-s}$, $f_{s,q-s}$) are obtained by dividing the dynamic parameters ($q_{t,dyn}$, $f_{s,dyn}$) by the PRF (Eqs. 4.2 and 4.3). PRF, $q_{t,q-s}$ and $f_{s,q-s}$ are calculated for each recorded point.

\[
q_{t,q-s} = \frac{q_{t,dyn}}{PRF} \tag{4.2}
\]

\[
f_{s,q-s} = \frac{f_{s,dyn}}{PRF} \tag{4.3}
\]
The effect of penetration rate on the pp ($u_2$) measurements were also observed and depend on penetration velocity, permeability and consolidation state of the sediments (e.g., Kim and Tumay 2004, Silva 2006). Consequently, the presented solutions are not proper to evaluate the quasi-static pp.

**Laboratory analyses** The lithology in gravity cores 04 and 03 (Fig. 4.1) is dominated by silty clay to clayey silt. It also includes a suspected weak layer with laminated sand (Fig. 4.3). In core 04, this particular layer is found between 1.90 to 2.15 mbsf, whereas in core 03, this layer is found in the uppermost 0.15 m (L’Heureux et al. 2012).

*Calypso* piston cores 02 and 01 (Fig. 4.1 for location) are mainly homogeneous, clayey silt. A weak and laminated clay layer was detected at 2.8 to 3.2 mbsf. The clays have low magnetic susceptibility and MSCL gamma density, whereas the sandy interval within the clay bed shows positive peaks (Fig. 4.4).

The undrained shear-strength ratio ($s_u/\sigma'_{V0}$) generally exceeds 0.3 for the massive clayey silt, which is characteristic for normally- to slightly over-consolidated sediments. For the weak layers, the undrained shear-strength ratio is lower ($0.2<s_u/\sigma'_{V0}<0.3$) and representative of normally-consolidated sediment (Figs. 4.3 and 4.4).

**Comparison of in situ and laboratory results** The evaluation of the in situ intact undrained shear-strength is based on the empirical relationship:

$$s_u = \frac{q_t - \sigma_{V0}}{N_{kt}}, \quad (4.4)$$

where $q_t$ is the corrected cone resistance, $\sigma_{V0}$ is the in situ vertical stress and $N_{kt}$ is the empirical cone factor. For the Finneidfjord sediments, the cone factor typically falls in the range from 10 to 18 (used 14) (Lunne 2010). The ratio of in situ undrained shear-strength to sleeve friction ($s_u/f_s$) provides a qualitative indication for the soil sensitivity (Lunne et al. 1997), but is highly influenced by the penetrometer used (Lunne 2010).

Comparison of in situ and laboratory undrained shear-strength values shows a good correlation for sites FF-CPTU 03-06 (Fig. 4.3). The correlation for FF-CPTU 05 and 06 towards the terminal depth is less evident and most likely related to coring effects (e.g., under-sampling) and sediment disturbance (Fig. 4.3b). Both FF-CPTU tests exhibit an over-consolidated characteristic with undrained shear-strength ratio higher than 0.4. The weak layer is classified as normally-consolidated ($0.2<s_u/\sigma'_{V0}<0.3$). Moreover, the sleeve friction compares well with the remolded
4.1. FINNEIDFJORD LANDSLIDE

Figure 4.3: (a) Comparison of FF-CPTU measurements (in situ undrained shear-strength, sleeve friction and in situ sensitivity) with core log, water content and fall-cone tests for locations FF-CPTU 03 and 04 and (b) for locations FF-CPTU 05 and 06. (Note: intact undrained shear-strength ($s_u$), water content ($\omega$), soil sensitivity ($S_t$))
undrained shear-strength ($s_{ur}$) for the upper 1.5 mbsb (FF-CPTU 03 and 04) and the upper 1.0 mbsb (FF-CPTU 05 and 06), whereas the soil sensitivity varies between 2 and 6 (Fig. 4.3).

When comparing the FF-CPTU 14 and 15 data with the Calypso piston core results, we observe a similar trend in the undrained shear-strength data (Fig. 4.4). However, estimates of undrained shear-strength tend to be larger when based upon the FF-CPTU results (i.e. 50-70%). A good correlation of the FF-CPTU data is encountered at depths beyond 2.5 mbsb.

The FF-CPTU measurements reveal significant drops in the undrained shear-strength, associated with a decrease in sleeve friction in the weak layers. The undrained shear-strength is at least 1.8 to 2.2 times lower compared to the surrounding sediments. A similar behavior is observed for the sleeve friction (from 3.5 to <2.0 kPa). The soft and sensitive clay shows an increase in the sensitivity (laboratory) from approximately 4.0 to 7.5. The FF-CPTU measurements further illustrate a pp build-up. In addition to this build-up, a decrease immediately below the soft and sensitive clay occurs (Fig. 4.4). This decrease in pp underneath the weak layer may reflect dilatancy in the potentially gas-rich sediments (L’Heureux et al. 2012).
Discussion and conclusions

Several studies illustrated that strength properties of sediments are highly influenced by the deformation/strain rate (Casagrande and Shannon 1949, Lucius 1971). This so-called strain-rate effect was first described for penetrometers by Dayal and Allen (1975). This effect accommodates for the fact that a dynamic impact serves to accentuate the geotechnical characteristics compared to pushed CPTU systems (Stoll et al. 2007). Dynamic data sets from FF-CPTU tests have non-linear insertion velocity and thus need to be corrected for the strain-rate effect in order to properly compare the results with pushed CPTU data. Several studies show good agreement with pushed deployments (see Comparison of FF-CPTU and pushed CPTU tests, Stoll et al. 2007 and Stegmann 2007). Hence, we chose the PRF to be \( \leq 1.5 \) for cone resistance and \( \leq 2.3 \) for sleeve friction, as is common for soft, marine clay at impact velocities \(<5.0 \text{ m/s}\) (Dayal and Allen 1975). The good agreement between our dynamic CPTU measurements and the earlier pushed CPTU data attest that the approach is valid (Eqs. 4.1 to 4.3).

The soil's sensitivity, i.e. the ratio of undisturbed peak- and remolded shear-strength, is an important parameter for characterizing intrinsic weakness of sediments (e.g. Leroueil 2001). From CPT tests, Robertson and Campanella (1983) and Lunne et al. (1997) have depicted that sleeve friction is a qualitative reference for remolded undrained shear-strength measured in the laboratory, in particular for fine-grained deposits. The comparison of the FF-CPTU data to laboratory measurements on adjacent cores show, in general, similar trends (Fig. 4.3). Some deviation between the two data sets can be explained by (i) the presence of interbedded, thin layers or pockets of soft- or stiff material (thickness <0.2 m) within the surrounding glacial deposits, and (ii) the fact that the test setup (conical cone) is not suitable to obtain full remolding of the sediment. Further discrepancies in the undrained shear-strength are probably related due to the effects of over-sampling in the upper portions of the Calypso piston cores and under-sampling in the gravity cores (see Comparison of in situ and laboratory results, Skinner and McCave 2003). The over-sampling can be seen in the lower undrained shear-strength compared to the FF-CPTU 14 and 15 results assuming a vertical disturbance according to the dilatational effects during deployment for the Calypso piston core (Fig. 4.4). In addition, a slight compaction due to frictional forces between the core barrel and sediment inside results in higher undrained shear-strength compared to the FF-CPTU 03-06 results (Fig. 4.3).

From a geological standpoint, the FF-CPTU data provide important hints for potential slope destabilization as well as the actual location of the failure plane. Corrected cone resistance and sleeve friction values are consistently lower by a factor of 1.5 to 2.0 in those units compared to the surrounding sediments (Figs. 4.3, 4.4 and 4.5). These weak layers are composed of normally-consolidated, soft and sensitive clay with undrained shear-strength ratio \( (s_u / \sigma'_{V_0}) \) between 0.2 and 0.3.
They are surrounded by massive, post-glacial, normally- to over-consolidated silts ($s_u/\sigma'_{V0}>0.3$). Consequently, these horizons are believed to represent the potential failure planes in future mass wasting events. This is of particular importance, considering that the factor of safety (from infinite slope equilibrium) lies close to 1, hinting at potentially unsafe conditions. Layers with similar characteristics were also found to affect the stability of submarine slopes along the Var delta, France (Dan et al. 2007) and in the fjord of Trondheim, Norway (L’Heureux et al. 2010). L’Heureux et al. (2012) interpreted the slide mechanism in Finneidfjord to be translational within the weak layer based on the use of integrated data sets (i.e. including the FF-CPTU data). In addition, the FF-CPTU data presented here

![Figure 4.5](image)

**Figure 4.5:** (a) Analysis of the *in situ* intact undrained shear-strength. (b) Geological & geotechnical sub-seafloor model including cone resistance and sleeve friction profiles.
show a contractive behavior for the mechanically weaker beds (Fig. 4.4). Hence, the failure mechanism could probably involve progressive softening of the sensitive, weak layers as also described for the Storegga slide (e.g. Kvalstad et al. 2005).

In summary, 38 FF-CPTU deployments were successfully carried out within a 3 days survey in Finneidfjord, northern Norway. The large amount of in situ data collected shows the capability of the instrument to characterize complex, shallow depositional environments in a time and cost-effective way. Furthermore, the mechanically weak layers were not only identified, but characterized to be laminated with a thin seam of sand from the high-resolution FF-CPTU record (Fig. 4.4). Therefore, this geotechnical device is well suited for exploring the shallow subsurface of slide-prone areas in shallow water environments (Fig. 4.5) and can be used for slope stability assessments and geohazard identification when combined with multibeam bathymetry data and high-resolution seismic (2D, 3D).

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4.1.2 Role of fluid flow in slope instability

Bathymetrical map, 3D chirp seismic data, in situ quasi-static CPTU profiles, sedimentological and geotechnical results obtained from cored samples were used to investigate the role of weak layer for the stability of submarine soils. The quasi-static CPTU profiles, analyzed and interpreted by myself, and two long French Calypso piston cores were taken for ground truthing purposes. Simple limit equilibrium slope stability assessments were performed using chirp seismic, in situ and coring data. Different factors, such as groundwater flow, free gas growth in the soil, were identified and discussed leading probably to the origin of weak layers and affecting the stability of the foreshore slope off the village of Finneidfjord. These results were represented in the subsequent manuscript accepted for the Submarine Mass Movement and Their Consequences conference 2011.
CHAPTER 4. GEOTECHNICAL APPLICATIONS

Identification of weak layers and their role for the stability of slopes at *Finneidfjord*, northern Norway

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Abstract

The 1996 *Finneidfjord* landslide, which took four human lives in northern Norway, initiated along a weak layer offshore before encroaching the shoreline. The integration of sediment cores, free-fall cone penetrometer tests and high-resolution 3D seismics indicate that the slide-prone layer is a regional event bed likely sourced from clay-slide activity in the catchment of the fjord. Its main influence on slope stability is attributed to their physical and geotechnical properties as they are softer and more sensitive than the surrounding bioturbated, fjord-marine deposit. The geotechnical properties are also likely affected by biogenic gas as found in the stratified event beds. Similar, fine-grained, stratified beds with comparable origin and properties also occur in other Norwegian fjords and are likely present along coastlines of other, previously glaciated margins, where they could be an important factor contributing to mass movements.
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Key words: Landslide, fjord, shoreline, slope stability, weak layer, sensitive clay, gassy sediment.

Introduction

Submarine landslides at all scales are often controlled by mechanically weak layers in the marine deposits. There has been an increased focus in the past decade on mapping and characterizing such weak layers in order to understand their contribution to mass wasting processes and to correctly perform geohazard assessments (e.g. Lastras et al. 2004, Kvalstad et al. 2005, L’Heureux et al. 2010).

In this paper, special attention is given to weak layers found off the village of Finneidfjord, northern Norway, where a catastrophic near-shore landslide occurred in 1996. The landslide mobilized $1\times10^6$ m$^3$ of sediment and, due to its retrogressive behavior, encroached 100-150 m inland. The imminent results were the destruction of the E6 highway and three houses. The landslide claimed four human lives.

Swath bathymetry and high-resolution seismic data show that initial detachment occurred along a specific layer in the Holocene sedimentary succession offshore (Longva et al. 2003). Geotechnical investigations conducted prior and subsequent to the landslide revealed that large volumes of highly sensitive clays (quick clays), below the shore, were mobilized in the later stages of the landslide (Gregersen 1999, Longva et al. 2003). In this study, we combine sedimentological, geotechnical and geophysical data in order to unravel the origin and the role of the weak layers for the stability of subaqueous slopes at Finneidfjord.

Regional setting

Following the last glaciation, the area around Sørfjorden was subject to intense glacio-isostatic rebound and a rapid fall of relative sea-level (the marine limit is at 124 meter above sea level; m.a.s.l.). This resulted in the emergence of glacio-marine and marine deposits, followed by a high rate of river erosion. Today, the lowlands in the study area are almost entirely covered by marine deposits (clay and silt), overlain locally by fluvial deposits. During their emergence in the Holocene, the marine deposits became exposed to groundwater flow and leaching of salts which resulted in the development of quick clays (Rosenquist 1953). Numerous quick clay landslides have occurred along the Rassåga River until the present day (Fig. 4.6).

Olsen et al. (1996) showed that the shoreline south of Finneidfjord, and in the vicinity of the 1996 landslide, is covered by beach deposits (sand and gravel). These coarser sediments overly a thick sequence of clayey silts reposing on bedrock. Pockets of quick clay are also found at several locations along the shoreline and offshore (Gregersen, 1999).
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Figure 4.6: (a) Location of the study area with historical landslide scars in the marine deposits along the \( \text{\textit{Ross\aa}} \) River valley (after Olsen et al. 1996). (b) Morphological interpretation of swath bathymetry data off Finneidfjord showing landslide deposits (black lines) and the location of landslide scars i to v (pink lines). (c) Zoom at the initial sliding area for the 1996 landslide as described by Longva et al. (2003). See text for details.

Data and methods

Bathymetric data from \( \text{\textit{Sorfjorden}} \) were collected in 2003 and again in 2009 using a 250 kHz interferometric sonar system (GeoAcoustics) mounted onboard R/V \( \text{\textit{Seisma}} \). The bathymetric grids have 1 m by 1 m cell size. During the same surveys, a dense network of parametric sub-bottom profiler (TOPAS) was acquired. Also in 2009, 12 gravity cores up to 2.5 m long were retrieved. X-ray imagery, detailed sedimentological description and various geotechnical analyses were performed on the cores.

A very-high-resolution 3D seismic volume (940 m by 175 m) over part of the 1996 landslide area was acquired in 2010. The 3D chirp system provides decimetre-scale horizontal and centimetre-scale vertical resolution sub-surface images (Vardy
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![Diagram of the Finneidfjord landslide](image)

**Figure 4.7:** (a) Fence diagram from the 3D seismic cube showing the 1996 landslide deposits (see Fig. 4.6b for location). (b) Sedimentological and geotechnical details of *Calypso* piston core GS-10-163-02 and results of *in situ* FF-CPTU 14 and 15. *Calypso* piston core GS-10-163-02 is shifted 0.73 m vertically to match with FF-CPTU data. (MS: magnetic susceptibility, \( \rho \): MSCL gamma density log, \( s_u \): undrained shear strength, \( q_t \): corrected cone penetration resistance, \( f_s \): sleeve friction) et al. 2008). Within the 3D survey area are two long *Calypso* piston cores collected in 2010 with R/V *G.O. SARS* (Fig. 4.7b). Multi-scanning core logging (MSCL), XRF core scanning, sedimentological description and geotechnical analyses are being performed on the *Calypso* piston cores.

*In situ* geotechnical data tests were collected conducted in 2010 using *MARUM*’s free-fall piezocone penetrometer (FF-CPTU) from R/V *Seisma*. 38 tests were per-
formed to obtain information about soil type and sediment physical properties in various parts of the landslide as well as adjacent areas. For more information about the FF-CPTU data, we refer to Steiner et al. (2012).

**Results**

**Landslide morphology** Evidence for mass wasting processes is clearly observed at several locations along the Sørfjorden shoreline (Fig. 4.6b). Two distinct end-members exist in terms of seafloor geomorphology: smooth and rough. Smooth seafloor corresponds to evacuated landslide scars devoid of landslide debris (e.g. slides i to v; Fig. 4.6). The slope gradient within the evacuated landslide scars is usually similar to the surrounding, intact slopes and varies between 13 and 21°. In general, the height of the escarpments is of 2-3 m.

In contrast, rough seafloor morphology corresponds to mass wasting deposits up to a few meters thick (Figs. 4.6 to 4.7). The debris deposits occur mainly in the central part of the fjord (Fig. 4.6), and consist of blocks and slabs of compressed sediment (see F; Fig. 4.6). The blocks moved along a high-amplitude seismic reflection (e.g. Longva et al. 2003) (Fig. 4.7a). Bathymetry data also indicate that several landslides retrogressed beyond the shoreline.

**Sediment core analysis** The Calypso piston core GS-10-163-02 was collected outside the 1996 landslide deposits (Fig. 4.6b). The sediment is dominated by a homogenous, brownish, bioturbated, clayey silt with some shell fragments. Physical properties only show subtle variation in MSCL gamma density and magnetic susceptibility (Fig. 4.7). At the depth of 2.8 mbsf, a 0.45 m thick and distinct stratified bed is present. It consist of a sharply based, 15 cm thick, grey clay overlain by 20 cm of coarse sand fining upwards, with another 10 cm of clay on top. Similar beds occur in gravity cores 03 and 04 (Fig. 4.8). These cores were collected, respectively, within and immediately upslope of the ~2 m high headwall created by the initial landslide of 1996 (slide iii; Fig. 4.6c).

In core 04, this particular unit is found at 1.9 mbsf, whereas in core 03, only part of the event bed is found in the uppermost 15 cm of the core (Fig. 4.8b). X-ray analysis of cores 03 and 04 reveal vesicular spots indicative of gas bubbles in the brownish clayey silt immediately below the clay layer. Gas vesicles were also observed within the stratified bed in core 04 (Fig. 4.8b). The beds are marked on the logs of magnetic susceptibility, MSCL gamma density and on the results from the FF-CPTU tests. The clays give low peaks in both magnetic susceptibility and MSCL gamma density, whereas the inverse is observed for the sandy layer (Figs. 4.7 to 4.8). From the FF-CPTU tests, a significant drop in tip cone
resistance accompanied by a decrease in sleeve friction is observed for the clays. Total organic carbon (TOC) is slightly lower in the clays compared to the normal brownish sediments. TOC peaks within the sandy layer (Fig. 4.8).

The depth at which the stratified beds occur matches the level of the failure plane for slide iii (Fig. 4.6c). The failure plane was previously interpreted to correspond to a band of high-amplitude reflections on TOPAS data (Longva et al. 2003). The bed found here confirms this (Fig. 4.7). Importantly, this layer can be mapped throughout the entire fjord basin and this suggests that it corresponds to a regional sedimentological event.
Geotechnical properties  Water contents averages around 35% and vary marginally for the brownish bioturbated sediment for all three cores. In contrast, water content varies significantly in the event beds, varying within the range 45-65%. Intact undrained shear-strength \( (s_u) \), determined from fall cone penetration tests, are typically lower for the event bed with values of 4-8kPa and undrained shear-strength ratio \( (s_u/\sigma'_V) \) in the range 0.2 and 0.3. Strength ratio for the brownish silt generally exceeds 0.3 (Fig. 4.7). Shear strength from in situ FF-CPTU tests compare well with fall cone results (Figs. 4.7 and 4.8) (see also Steiner et al. 2012). Results from both laboratory data and FF-CPTU tests also show that the sensitivity of the event bed is greater than for the normal sediment with values up to 7.5 (Fig. 4.8).

Back-calculation of strength from slope morphologies  Many of the translational slope failures in the fjord have a relatively thin moving mass (around 2-3m) compared to the height of the slope (up to 30m). In several cases, the triggering mechanism for slope failures is related to nearby human activities, e.g. blasting and rapid loading. Such situations are suitable for infinite slope stability slope analyses using undrained conditions, which can be expressed as:

\[
FoS = \frac{s_u}{\gamma'z\sin[\alpha]\cos[\alpha]},
\]

where FoS is the factor of safety, \( s_u \) the intact undrained shear strength at depth \( z \), \( \gamma' \) the submerged unit weight, \( z \) the depth to the failure plane and \( \alpha \) the slope angle. Using Eq. 4.5 one can find combinations of \( s_u \) and \( z \) which would produce failure (i.e. \( FoS=1 \)). For slope angles of 13-21° and slab thicknesses of 2-3m, the required \( s_u \) values needed to produce failure are in the range 4-8kPa (Fig. 4.9). These results are comparable to the measured shear strength data for the stratified event bed in the cores and calculated from in situ FF-CPTU tests. This testifies to a low natural stability of slopes prior to failure.

Discussion and conclusions  The integrated data set presented here illustrates that landslides in Sørfjorden tend to initiate and develop along a regional, weak bed in the stratigraphy (Fig. 4.10). This weaker unit contains alterations of clays and sands and with properties and internal stratification comparable to those described in a fjord system further south (L’Heureux et al. 2010, Hansen et al. 2011). Applying this study as an analogue, the stratified event beds at Finneidfjord were likely deposited rapidly by turbidity currents/flows generated in the fjord following landslides in the emerging marine
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![Table](data:image/png;base64,iVBORw0KGgoAAAANSUhEUgAAAgAAAAAACLAMAAAAF4Aw86AAAAAnBMVEX///8AABwS8AAACXBIWXMAAAsTAAALEwEAmpwYAAAAAAElFTkSuQmCC)

<table>
<thead>
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<th>Slide #</th>
<th>Scarp height (m)</th>
<th>Slope angle (°)</th>
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<tbody>
<tr>
<td>i</td>
<td>2.1-2.7</td>
<td>13-15</td>
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<tr>
<td>ii</td>
<td>2.2</td>
<td>21</td>
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<td>iii</td>
<td>2.5-3.0</td>
<td>17</td>
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<tr>
<td>iv</td>
<td>2.5-3.0</td>
<td>19</td>
</tr>
<tr>
<td>v</td>
<td>3.0</td>
<td>14</td>
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**Figure 4.9:** Combination of slope angle and intact undrained shear strength needed for the failure of sediment slabs. As a comparison, the values of scarp height (z) and slope angle for 5 translational slides in Sørfjorden are given (see Fig. 4.6 for location).

Following landslide activity in the catchment area of the fjord, the sudden and extensive supply of sediment caused an abrupt burial of benthic ecosystems, anoxic conditions and most likely the production of biogenic gas (e.g. Leduc et al. 2002). This would explain the gas vesicles found within and immediately below the event bed in the cores (Fig. 4.7). Biogenic gas is known to significantly alter the sediment’s geotechnical properties, primarily by increasing the compressibility and reducing the undrained shear strength (Sills and Wheeler 1992, Seifert et al. 2008).

The integrated data set suggest that initial failure occurs by translational sliding (basal shear) located in the thin event beds (Fig. 4.10). A slab of sediments starts moving down-slope along these beds, as they are softer and more sensitive
than the surrounding sediments. The regional extent and low-permeability of the beds may also allow for the formation of artesian groundwater pressure at different stratigraphic levels (Fig. 4.10). This causes lowering of effective stresses in the surrounding deposit and undermines slope stability. An excess pore pressure (8 kPa in average) exists at 3 mbsf (see $P$ in Fig. 4.6 for piezometer location). Unfavorable groundwater conditions (e.g. periods of heavy rainfall) as a pre-conditional factor in place at the time of external triggering of the initial landslide (from human activity) lead to a landslide that developed retrogressively from the fjord to the shore. In the case where quick clay pockets are present below the shoreline deposits and on land, these slides can extend fair inland with dramatic consequences such as in the case of the 1996 landslide (Longva et al. 2003).

A prerequisite for the occurrence of the weak event beds, as those found in this study, is the presence of prehistoric clay-slide activity in the catchment and a low to moderately dipping fjord margin on which the event beds are able to accumulate (Hansen et al. 2011). As sensitive, glacio-marine sediments and clay-slides are common in uplifted fjord valleys of Canada and Scandinavia, and to a lesser extent in Alaska, beds with similar origin as those found in this study could play an important role for the stability of other near-shore areas (e.g. Saguenay fjord; Perret et al. 1995). Detecting these slide-prone beds is, therefore, of great importance for geohazards assessment. Such slide-prone beds have successfully been detected in
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Sørøfjorden using high-resolution seismic data in combination with in situ geotechnical testing because of their characteristic acoustic and geotechnical signature.

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4.2 Nice airport landslide

4.2.1 Physical properties of soils and slope stability assessments

In total, ∼150 dynamic-CPTU deployments were collected within four scientific cruises, and I was participant on two of those. The deployments were conducted in the upper slope region, landslide-prone area, shelf and shelf break region off the Nice international airport (southeastern France). The in situ dynamic tests were complemented by several static-CPTU measurements, 2D chirp seismic transects, gravity and French Calypso piston core profiles. This large number of sedimentological, geophysical and geotechnical data allowed to develop an area-wide sub-seafloor model appropriate to use it for slope stability assessments. 2D detailed numerical assessments were carried out in order to evaluate the overall stability of the shallow to medium deep soil successions in the vicinity of the 1979 Nice landslide area. These results were submitted as research article to the Marine Geology Journal.
Sub-seafloor modeling and slope stability assessment in the 1979 Nice landslide area (southeastern France)

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Submitted: Marine Geology Journal

Abstract

In the landslide-prone and tsunamigenic area near the Nice international airport, southeastern France, an interdisciplinary approach is applied to develop realistic lithological/geometrical profiles and geotechnical/strength sub-seafloor models. Such models are indispensable for slope stability assessments using limit equilibrium or finite element methods. Comprehensive regression analyses, based on the undrained shear-strength ($s_u$) of intact gassy sediments are used to generate a sub-seafloor strength model based on 37 short dynamic and eight long static piezocone penetration tests, and laboratory experiments on one Calypso piston and 10 gravity cores. Significant $s_u$ variations were detected when comparing shelf and shelf break measurements and a significant drop up to $5.5\text{ kPa}$ in $s_u$ was interpreted as a weak zone at a depth between 6.5 and 8.5 m. A 10% reduction of the \textit{in situ} total unit weight compared to the surrounding sediments is found and coincides with coarse-grained layers and a weak zone acting as potential detachment plane for former and present-day gravitational and retrogressive slide events, as seen in 2D chirp profiles. The combination of the chirp transects and strength models allows computation of enhanced 2D finite element slope stability analyses with undrained sediment response compared to previous 2D numerical and 3D limit equilibrium assessments. Given that factors of safety are equal or less than 1 when considering
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the effect of free gas, failure geometries with a maximum slide plane depth between 20 and 55 m are determined, and a high risk that small to large landslide events will occur in the (near) future for the shallow to deep sediment complex off the Nice international airport is identified.

Key words: Dynamic penetrometer, sub-seafloor modeling, weak zone, free gas, numerical slope stability.

Introduction

Cone penetration testing with pore pressure recording (CPTU) is a powerful, widely used, cost- and time-efficient in situ technique for sub-seafloor geotechnical investigations (Baligh et al. 1981, Lunne et al. 1997, Meunier et al. 2004, Stoll et al. 2007, Robertson 2009a, Steiner et al. 2012). In marine environments, both dynamic- and static-CPTU devices are applicable, but these devices have significantly different penetration modes and measure different geotechnical properties. The momentum of the dynamic-CPTU device controls the achievable penetration depth with a non-linear penetration rate up to 10 m/s, and the static-CPTU instrument penetrates the sediment with constant penetration rate of 2 cm/s. The measured dynamic-CPTU properties are of a higher order of magnitude compared to the static-CPTU parameters. This distinction is due to the strain-rate effect, and is corrected using the modified inverse sin-hyperbolic equation and associated soil-specific rate coefficients (SSCs) (Randolph and Hope 2004, Steiner et al. [In Review]).

The dynamic-CPTU device can be operated faster, resulting in an increase of the number of in situ sediment physical properties which can be collected in the same time, and are crucial for sub-seafloor modeling. During several field campaigns and scientific cruises from 2007 until 2012, a large number of in situ tests were performed in the vicinity of the Nice international airport, southeastern France, where on October 16th, 1979, a large submarine landslide took place which caused significant material damage and claims several human lives (Gennesseaux et al. 1980, Seed et al. 1988). This tsunamigenic landslide event occurred on the shelf and shelf break off the airport and mobilized approximately 9 million m$^3$ marine clayey to silty sediments as well as engineering fill of the airport/harbor enlargement, which was constructed at that time (Assier-Rzadkiewicz et al. 2000).

Based on 2D numerical slope stability assessments using bathymetry maps before and after the 1979 landslide, a long onshore static-CPTU test, a short gravity core (gc) and a long Calypso piston core (cpc) in conjunction with one-dimensional compression tests, Dan et al. (2007) found out a coincidence of three
pre-conditioning factors and triggering mechanisms leading to the 1979 landslide failure. This study emphasizes the role of: (i) coarse-grained layers and fresh water infiltration causing the origin of weak zones as a result of leaching and the increase of slow deformation processes, (ii) sudden overloading of the sediments induced by anthropogenic fill operations result in a decrease of the intact undrained shear-strength ($s_u$), and (iii) heavy rainfall several days before the landslide resulting in excess pore pressure ($\Delta u$) and a decrease of the effective overburden stresses. Moreover, existing weak zones affected by slow deformation processes were quantified by a factor of safety (FoS) close to unity, and were interpreted as the main causes for the 1979 landslide (Dan et al. 2007). In order to evaluate the present-day stability of the Nice area, in particular, eastern plateau near the airport, nine 28m long offshore static-CPTU tests were used to perform 3D probabilistic slope stability assessments (Leynaud and Sultan 2010). These assessments predict a 5% likelihood that a large landslide (i.e. failure plane at 60 m depth) will occur, and a 50% likelihood that medium landslide (i.e. failure plane at 30 m depth) will occur in the near future. In addition, comparisons of offshore static-CPTU tests with vane shear experiments on cored specimens and core photos indicate free gas growth in the sediments (Sultan et al. 2010), and previous studies revealed sub-hydrostatic excess pore pressures and a 20 to 30% decrease of the $s_u$ in the gassy sediments compared to the undisturbed material (e.g. Nageswaran 1983, Seifert et al. 2008, Sultan et al. 2010). This $s_u$ decrease is related to the compressibility of the gas, structural differences according to the gas concentration (i.e. changes in the void ratio) and increase of the pore pressure during gas accumulation (Nageswaran 1983). However, the impact on the stability of the Nice continental shelf and upper slope has not been analyzed yet.

This manuscript presents an interdisciplinary approach integrating multi-beam swath bathymetry, 2D seismic chirp, coring and in situ data collected in a free gas-affected region off the Nice international airport. An area-wide geotechnical/strength model is developed using detailed regression analyses based on the $s_u$ derived from gc, static and mainly dynamic in situ measurements. In addition, cpc and in situ profiles are used to analyze variations of $s_u$ and $s_u$ gradients with penetration depth in order to detect weak zones. 2D seismic chirp transects are used to map silty to sandy layers and a surficial weak zone including correlations with morphological scars and morphological steps. This interdisciplinary approach is also utilized: (i) to enhance the 3D probabilistic assessments using detailed 2D chirp transects, geotechnical/strength sub-seafloor transects and 2D finite element analyses considering the impact of free gas with undrained sediment behavior, and (ii) to evaluate the FoS as well as the volume of the potential landslide mass. Finally, implications for geohazard and landslide research in the vicinity of the Nice international airport are also discussed.
Regional setting

Geological context

Sedimentology, lithological sequences The French part of the Ligurian margin contains a narrow continental shelf and a steep slope off Nice and is located in the northwestern Mediterranean Sea (Figs. 4.11a, 4.11b). In this region, complex tectonic processes have been occurred due to the collision of the African and Eurasian plates in the Cretaceous and early Tertiary time, including the closure of the Tethyan ocean and orogeny of the Alps (McKenzie 1970, Hsii 1971). The origin of the Ligurian basin is rifting and seafloor stretching during the orogenesis of the western Alps in the late Oligocene-Miocene (Le Pichon et al. 1971, Cherchi and Montadert 1982, Rehault et al. 1984). The basement of this margin is covered by 5-7 km of Miocene to late Neogene-Quaternary sediments consisting of conglomerates, mud and marly ooze affected by frequent erosional events (Fahlquist and Hersey 1969, Auzende et al. 1971, Le Borgne et al. 1971, Auffret et al. 1982, Savoye and Piper 1991). Dubar and Anthony (1995) and Anthony and Julian (1997) carried out detailed sedimentary analyses including three representative stratigraphical sections of the Var, Paillon and Brague river mouths (Fig. 4.11c). These sections mainly consist of a four-layer sedimentological/stratigraphic sequence (Fig. 4.12, see also Guglielmi 1993, Dubar and Anthony 1995, Anthony and Julian 1997, Anthony and Julian 1999 for further details). In 2008, the Var section was extended by a seismic reflection transect (see Fig. 4.11c for location) showing a similar stratigraphic sequence for the Nice continental shelf and upper slope, and potentially the interface between Quaternary and late Neogene deposits (Fig. 4.12, Kopf et al. 2008, 2009). In addition, the Var river mouth and 1979 Nice Airport Landslide area (NAIL) are related to a Gilbert-type delta (Gilbert 1885, Savoye and Piper 1991, Dubar and Anthony 1995). This Gilbert-type delta was formed by the suspended sediments from the western Alpine catchments of the Var river and characterized by high suspended sediment concentration and deltaic sedimentary processes including wave reworking, geostrophic current, turbulent diffusion of seepage water in the basal aquifers and high density suspension flow of the fine-grained sediments (Anthony and Julian 1997).

More than 70 gravity and Calypso piston cores were collected on the Nice upper continental slope, at the Nice shelf and 1979 NAIL area (Kopf et al. 2008, 2009, 2012, Sultan et al. 2008). The sediments were characterized as highly bioturbated, homogeneous, fine-grained hemipelagic deposits consisting of medium plasticity silty clay with scattered coarse beds of silt to sand (Cochonat et al. 1993, Klaucke et al. 2000). Similar sedimentological characteristics were found in the 1979 NAIL area (Dan et al. 2007).
Figure 4.11: (a) Map of France (Europe) shows the city of Nice, which is located in the southeastern part of France. (b) Bathymetry image illustrates the upper slope and shelf morphology of the Ligurian margin. The 1979 Nice landslide area, airport and airport/harbor extension within the study area are also shown. The across-track resolution of the bathymetry image is less than 25 m. (c) Overview map depicts the three main rivers located in the vicinity of Nice and the position of the Var stratigraphic and seismic reflection transects, presented in Figure 4.12

Morphology and bathymetry  The continental shelf of the French Ligurian margin is very narrow at <3 km width which tapers at the Baie des Anges and off the Nice airport to 0-900 m width. The shelf gradients of the last prodeltaic deposits are between 1 and 10% (Cochonat et al. 1993, Mulder et al. 1994). The continental slope is very steep with an average slope angle of 11° over the first 20 km from the coast and down to ~2500 m depth. It is characterized by deeply-incised canyons generated by the Var and Paillon rivers with side-wall gradients up to 27° (Fig. 4.11b, Pautot 1981, Klaucke et al. 2000, Ioualalen et al. 2010), and a large number of smaller steep-sided valleys, eroded surfaces, scars, ridges and gullies (Fig. 4.11b, Migeon et al. 2012). Based on high-resolution hull-mounted and AUV bathymetrical records, ~250 landslide scars were detected at the shelf break and on the upper continental slope between 20 and 1000 m water depth and were categorized as very small, small, medium or large scars. The very small to small scars are semi-circular to ellipsoidal shaped and mostly located at the shelf break as well as along the crest and on the flanks of the interfluvies/ridge. The
Figure 4.12: (a) The stratigraphic transect describes the detailed late Neogene-Quaternary sediment sequence of the Var river mouth modified after Dubar and Anthony (1995), Anthony and Julian (1997). (b) The seismic reflection transect illustrates similar stratigraphic characteristic for the Nice continental shelf and upper slope (modified after Kopf et al. 2008, 2009). The position of both transects is shown in Figure 4.11.

Dimensions vary between 30 and 150 m width and 2 to 30 m height; however, most failure planes or shear bands are less than 10 m deep (Klaucke and Cochonat 1999, Migeon et al. 2012). In addition, back-calculations based on the consolidation state measured on core specimens with a sedimentary hiatus exhibited thicknesses of removed sediments between 4 and 9 m (Cochonat et al. 1993). The medium and large scars are usually located in the deeper regions of the continental slope, except the medium sized scar of the 1979 NAIL event (Figs. 4.11b, 4.13). The dimensions range from 150 to 500 m in width and 30 to 90 m in height (Migeon et al. 2012).
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Figure 4.13: The bathymetry image shows the shelf and shelf break morphology off the Nice international airport, southeastern France including the main part of the 1979 Nice landslide area and scar (dashed black line). The locations of all used dynamic penetrometer tests (circle), static penetrometer tests (box) and Calypso piston or gravity cores (pentagon) are pictured. The black lines indicate the 2D chirp seismic and numerical slope stability transects shown in Figs. 4.22a, 4.22b, 4.25a, 4.25b and 4.25c. The dashed white contour lines depict the morphology with a depth interval of 25 m.

Physical and geotechnical data  

In situ and laboratory datasets were collected using conventional onshore, offshore static-CPTU measurements, Atterberg limits, one-dimensional compression tests, grain size distributions (gsd), vane shear (v-s) and fall cone penetration (fc) experiments performed on cored samples.

In situ measurements  

In 1994, five deep onshore static-CPTU measurements were performed along the shoreline of the Nice airport by Sols Essais reaching up to 42m penetration depth (Sols Essais 1994 [unpublished report], Dan 2007). The sediments along the Nice airport are characterized as normally- to over-consolidated clay (soil class 3, $s_u/\sigma'_{v0} \leq 1$; Ramsey 2002) with embedded thin beds of coarse sediments (soil classes 5-8; Ramsey 2002), except the upper 5 m
of engineering fill (soil classes 4-9; Ramsey 2002). The corrected cone penetration resistance \( (q_t) \) is between 0 and 3 MPa in the hemipelagic deposits; however, prominent peaks in the coarse layers up to 5 MPa were detected. Sultan et al. (2004) and Dan et al. (2007) mention a thick succession of sensitive clay at 30 to 42 m depth and a distinct clayey sand bed at \( \sim 36.5 \) m depth.

In 2008, nine offshore static-CPTU tests were undertaken with a *Penfeld* device (Sultan et al. 2008, 2010). These tests focus on the geotechnical characterization of the prodelta deposits, in particular to identify and characterize existing shear zones potentially consisting of sensitive clays. The identified shear zones were described by significant drops of the \( q_t \) and were detected between 8 and 28 m with an average depth of 22 m. Sultan et al. (2010) remark that the shear zones are more pronounced in the measured sleeve friction \( (f_s) \) measurements (see also Sultan et al. 2007) and can be characterized by a \( f_s \) drop at 3 to 9 m depth.

**Laboratory experiments** Core specimens, collected during *GMO* and *Geoscience* cruises (Sultan et al. 2004, Dan et al. 2007), were used for standard and advanced laboratory experiments with the aim of understanding the strength properties, consolidation state and sensitivity of the surficial sediments. The Multi Sensor Core Logging (MSCL) gamma density \( (\rho) \) is 1.6-2.1 g/cm\(^3\) for the upper continental slope (Cochonat et al. 1993, Sultan et al. 2004) and show higher values up to 2.2 g/cm\(^3\) in the 1979 NAIL area (Dan et al. 2007). The compressional wave velocity \( (v_p) \) is in the range of 1500 to 1700 m/s (Sultan et al. 2004, Dan et al. 2007). The clay content is quantified as being in the range of 16 to 30% with an average grain size of 20 to 30 \( \mu \)m (Cochonat et al. 1993, Klaucke et al. 2000, Sultan et al. 2004). The proportion of sandy silt to sand in the coarse interbeds is 20 to 90%. The soil's sensitivity, described by the ratio of the intact and remolded undrained shear-strength (e.g. Locat and Lefebvre 1985, Leroueil 2001), was determined to be less than 5 considering \( s_u \) of 3 to 8 kPa in the upper slope and up to 34 kPa in the 1979 NAIL area (Dan et al. 2007).

In summary, 14 static-CPTU tests and more than 70 cores attest that the vicinity of the Nice international airport is one of the best explored regions in the Mediterranean Sea, and is best suited as natural laboratory to investigate present-day slope stability processes.

**Materials**

**In situ piezocone testing** From 2007 until 2012, \( \sim 150 \) long static-CPTU and mainly short dynamic-CPTU tests were collected on the Nice upper continental slope, at the Nice shelf and at the 1979 NAIL area (i.e. PRISME: Sultan et al.
In this study, eight deep offshore static-CPTU tests collected with the *Ifremer Penfeld* penetrometer (Fig. 4.14) are used to recover *in situ* records from up to 28 m penetration depth (see also Sultan et al. 2010). This device pushes an oil-balanced 10 cm$^2$ multisensor static-CPTU cone into the sediments and measures cone penetration resistance ($q_c$), $f_s$ and $\Delta u$ using a differential pore pressure configuration. The geometry and accuracy class 1 of the piezocene follows ISO 22476-1 (2012). More details of this offshore static-CPTU instrument are described in Meunier et al. (2000) and (2004).

Complementarily, the *MARUM* shallow-water dynamic penetrometer rely on a 15 cm$^2$ *Geomil* piezocone recording dynamic $q_c$, dynamic $f_s$, dynamic pore pressure behind the tip ($u_2$) and tilt/temperature (Fig. 4.15). In this study, 37 short dynamic-CPTU measurements up to 5.4 m penetration depth are performed (Kopf et al. 2008, 2009, 2012 and N. Sultan, unpublished data). The dynamic-CPTU cone is extended by 1 m long metal rods up to a maximum length of 8.5 m and hosts a pressure-tight housing containing a microprocessor, standard secure digital memory card, tiltmeter, accelerometer, power supply and data interface module. The

![Figure 4.14: The *Ifremer Penfeld* penetrometer is shown including a rough description of the main components of this seabed tool (see also Meunier et al. 2000, 2004 for details). A general drawing of the hydrostatic pressure compensated multisensor piezocone is also illustrated (copyright *Ifremer*).](image)
4.2. **NICE AIRPORT LANDSLIDE**

**Figure 4.15:** The MARUM shallow-water dynamic penetrometer is presented including a picture of the main electronic (see also Stegmann et al. 2006, Steiner et al. 2012 for details). The measured cone penetration resistance ($q_c$), measured sleeve friction ($f_s$) and measured pore pressure behind the tip ($u_2$) are dynamic parameters according to the strain-rate effect (see section [Strain-rate correction]) (copyright MARUM).

Geometry and accuracy class 1 of the piezocone follows ISO 22476-1 (2012). Details regarding all mechanical and electronic components are presented in Stegmann et al. (2006), Steiner et al. (2012) and Steiner et al. [In Review].

**Coring and sampling** In 2007, four up to 17 m long cpc were collected at the shelf break using the R/V Marion Dufresne (Dannielou 2002, Sultan et al. 2004). This coring technique is highly effective in marine environments for recovering long sediment cores with minimum disturbance. The corer weighs 10 tons and consists of a 64 m long metal tube with a 14 cm diameter containing a high-pressure PVC liner with a 10 cm diameter. A mechanical trigger and a piston ensure uniform sampling of the sediment within the liner during the final free-fall, from about 1 m above the seabed (http://www.eurofleets.eu/np4/72.html).

In 2007, 2009 and again in 2012, ~50 short gc with up to 5.3 m core recovery were collected at the shelf break and 1979 NAIL area using the R/Vs Meteor and Poseidon (Kopf et al. 2008, 2009, 2012). The corer weighs 1.5-2.0 tons and con-
sists of a 2 or 6 m long metal tube containing an internal PVC liner with a 10 cm diameter (Emery and Dietz 1941). The cores were cut into sections of 1 m length, and were stored in a refrigerated container.

Geophysical data

**Bathymetry** During two geophysical explorations in 2000 and 2006, the Nice shelf and upper continental slope were mapped using a hull-mounted Simrad EM300 sonar system. The swath bathymetric records were compiled to a Digital Terrain Model. The across-track resolution varies between 20 and 25 m according to the water depth (Fig. 4.11b, 4.13, Dan et al. 2007, Sultan et al. 2010).

**2D chirp transects** A chirp sub-bottom profiler is an efficient and widely used geophysical technique to explore the shallow sub-seafloor sediments with a high resolution. In 2009, ~100 2D transects were recorded (Henry and Migeon 2009). The profiler uses primarily frequencies operating at 1.8 to 5.3 kHz, appropriate for use in shallow-water environments at decimeter scale vertical resolution. 4 hydrophones record the reflected waves.

**Methods**

*In situ piezocone penetrometer tests*

**Strain-rate correction** Numerous studies have determined that both physical and hydraulic properties of clays, silts and sands depend non-linearly on the strain-rate (e.g. Eyring 1936, Casagrande and Shannon 1949, Suklje 1957). Dynamic-CPTU tests, with high penetration rates up to 10 m/s, show higher properties compared to the static-CPTU parameters, and thus have to be corrected for this so-called strain-rate effect (e.g. Dayal et al. 1975, Stoll et al. 2007, Steiner et al. 2012). Previous studies illustrate that the modified inverse sinh-hyperbolic equation, comprising the ratio of the dynamic and static penetration rate (\(v_{dyn}\) and \(v_{ref}\)) as well as SSC, is best suited for the strain-rate correction of clays (e.g. Randolph 2004, Steiner et al. [In Review]), described as follows for the three CPTU parameters:

\[
q_{t,q-s} = \frac{q_{t,dyn}}{1 + \left(\frac{\mu_{CPTU,qt}}{\ln(10)}\right) \text{arcsinh} \left(\frac{v_{dyndyn}}{v_{refdref}}\right)},
\]

\[
f_{s,q-s} = \frac{f_{s,dyn}}{1 + \left(\frac{\mu_{CPTU,f}}{\ln(10)}\right) \text{arcsinh} \left(\frac{v_{dyndyn}}{v_{refdref}}\right)},
\]
\[ \Delta u_{2,q-s} = \frac{\Delta u_{2,dyn}}{1 + (\frac{\mu_{CPTU,\Delta u}}{m'|10'}) \arcsinh \left( \frac{v_{dyn,dyn}}{v_{ref,dyn}} \right)}. \]  

(4.8)

where \( q_{t,dyn} \) and \( q_{t,q-s} \) are the dynamic and quasi-static corrected cone penetration resistances (Eq. 4.6), \( f_{s,dyn} \) and \( f_{s,q-s} \) are the dynamic and quasi-static measured sleeve friction (Eq. 4.7), \( \Delta u_{2,dyn} \) and \( \Delta u_{2,q-s} \) are the dynamic and quasi-static excess pore pressures measured behind the tip (Eq. 4.8), and \( d_{dyn} \) and \( d_{ref} \) are the diameters of the dynamic- and static-CPTU piezo cones (see also Randolph 2004, Mitchell and Soga 2005, Steiner et al. [In Review]). Finally, if using strain-rate corrected (quasi-static) CPTU parameters, all existing geotechnical solutions can be directly used.

**In situ intact undrained shear-strength** The determination of the in situ intact undrained shear-strength (\( s_{u,qt} \)) is based on both theoretical solutions (i.e. classical bearing capacity theory, Terzaghi 1943; cavity expansion theory, Skempton 1951; strain path theory, Baligh 1985) and empirical correlations (Aas et al. 1986). According to simplifications in the theoretical solutions with respect to the soil behavior, failure mechanism and boundary condition, the empirical correlation is preferred in geotechnical practice and scientific research, and is described as follows:

\[ s_{u,qt} = \frac{q_t - \sigma_{V0}}{N_{kt}}, \]  

(4.9)

where \( \sigma_{V0} \) is the in situ total overburden stress, \( N_{kt} \) is the empirical cone penetration resistance factor, and \( q_{t,q-s} \) and \( q_{t,ref} \) are the corrected cone penetration resistance (Lunne et al. 1997, Low et al. 2010). The additional subscripts for \( q_t \) reflect the type of instrument ("q-s" for the dynamic-CPTU instrument and "ref" for the static-CPTU device).

**In situ total unit weight** The correct determination of the in situ total unit weight (\( \gamma_t = \rho_t + g \)) is required for the assessment of the in situ total overburden stresses, and is imperative for many CPTU correlations (e.g. Lunne et al. 1997). The \( \rho_t \) is the in situ bulk density in g/cm\(^3\), and \( g \) is the gravity (9.81 m/s\(^2\)). An empirical equation, developed by Mayne et al. (2010), is used to directly determine \( \gamma_t \) from the penetration depth (\( z \)), \( q_t \) and \( f_s \), expressed as:

\[ \gamma_t = 11.46 + 0.33 \log[z] + 3.10 \log[f_s] + 0.70 \log[q_t], \]  

(4.10)

where the in situ total unit weight is here given in kN/m\(^3\) or kPa/m. Mayne et al. (2010) performed multiple regression analyses comprising 44 sites, where unit weight tests, shear wave velocity measurements and CPTU datasets were collected. The equation is valid for clayey to sandy deposits, except diatomaceous clays and highly calcareous sediments (Mayne et al. 2010).
Laboratory experiments

Index properties and grain size distribution  Atterberg limits, i.e. the liquid limit ($\omega_L$) and plastic limit ($\omega_P$), were analyzed using a Casagrande-percussion and an electronically driven apparatus to evaluate $\omega_L$ and $\omega_P$. Both limits are used to determine the plasticity ($I_P$) and liquidity index ($I_L$), which together with $\omega_L$ and moisture content ($\omega$) provide the basis for soil classification and correlation of sediment properties (BS 1377-2 1990, BS 5930 1999).

Grain size distribution was analyzed using a Coulter Counter LS-13320 laser particle size analyzer. The Coulter Counter LS-13320 measures grain size in 117 classes ranging from 0.04 to 2000 $\mu$m as a volume percent (Syvitski et al. 1991). The associated soil classification is based on the ISO 14688-1 (2002).

Shear experiments  Standard vane shear experiments were performed to determine the $s_{u,v}$. A four-bladed vane with a length / diameter of 12.5 mm, height of 6.25 mm and a constant rotation rate of 90°/min are used (Blum 1997). A fall cone device was utilized with a defined weight of 80.51 g and 30° cone geometry in order to determine the intact fall cone penetration shear-strength ($s_{u,fc}$) (Hansbo 1957). All cores of this study were tested using both techniques at a sampling interval of 0.1 to 0.15 m.

One-dimensional compression and permeameter tests  The coefficient volume compressibility ($m_v$) and Young’s modulus ($E$) were evaluated using on-dimensional compression tests (CTs). CT consists of a metal ring retaining the sediment specimen and preventing lateral expansion of the material. The diameter and height of the metal ring are 3.6 cm and 1.5 cm. The samples were incrementally loaded up to 6.3 MPa axial stress (i.e. effective overburden stress with zero excess pore pressure) and associated settlements due to drainage were recorded resulting in a non-linear relationship between the settlements and axial stresses. The $m_v$ and $E$ are calculated using this non-linear relationship, defined as:

$$m_v = \frac{\Delta H_{inc}}{H_{ini}} \frac{1}{\Delta \sigma'_{inc}},$$  \hspace{1cm} (4.11)

$$E = \frac{1}{m_v} \frac{(1 + \nu)(1 - 2\nu)}{(1 - \nu)},$$  \hspace{1cm} (4.12)

where $\Delta H_{inc}$ is the difference between the height of the specimen at the start of a loading increment ($H_{ini}$) and the height at the end of that increment, $\Delta \sigma'_{inc}$ is the effective stress difference related to several loading stages, and $\nu$ is the Poisson ratio assumed at about 0.3 (see BS 1377-5 1990 for details). In addition, regression
analyses using all load increments up to 800 kPa (i.e. \( \sim 115 \) m depth with respect to the numerical slope stability models) are carried out to evaluate the average \( E \) for the seabed \( (E_{\text{ref}}) \) and the complemented gradient with \( z \) \( (E_{\text{inc}}) \).

The coefficient of permeability \( (k_f, \text{here given in m/s}) \) was measured using a Permeameter triaxial cell and core specimens with a diameter and height of 2.5 cm. Specimens at different depth were loaded by their in situ effective overburden stress \( (\sigma'_{V0}) \) and \( k_f \) are calculated. A temperature correction was performed using a correction factor of 0.771 in order to convert the measurements at 20°C to parameters at 10°C (see BS 1377-6 1990 for details). In addition, the coefficient of consolidation \( (c_v, \text{here given in m}^2/\text{s}) \) is based on the relationship between \( k_f \) and \( m_v \), expressed as:

\[
    c_v = \frac{k_f}{m_v \gamma_w}, \quad \text{(4.13)}
\]

where the \( \gamma_w \) is the unit weight of water, here given in kN/m\(^3\) or kPa/m (Terzaghi et al. 1996). Based on \( c_v \), the "non-dimensional velocity" is determined, which is a measure for the degree of consolidation during penetration (Finnie and Randolph 1994, Steiner et al. [In Review]). The non-dimensional dynamic velocity \( (V_{\text{dyn}}) \) and non-dimensional static velocity \( (V_{\text{ref}}) \) are expressed as:

\[
    V_{\text{dyn}} = \frac{v_{\text{dyn}} d_{\text{dyn}}}{c_v}, \quad \text{(4.14)}
\]
\[
    V_{\text{ref}} = \frac{v_{\text{ref}} d_{\text{ref}}}{c_v}. \quad \text{(4.15)}
\]

Geophysical data A constant velocity of 1500 m/s is assumed for the depth migration of the seismic data due to the lack of an appropriate velocity model (Yilmaz 2001). The seabed reflector of the depth migrated profiles is compared with the associated bathymetrical transects for corroboration. The strength and lithological profiles derived from in situ CPTU tests and laboratory experiments are utilized for ground truthing purposes.

Slope stability analyses 2D numerical models were developed using the Plaxis finite element software (Vermeer 1979) to take into consideration realistic natural boundary conditions. The geometrical constrains are derived from the 2D chirp transects. The linear elastic perfectly plastic Mohr Coulomb (MC) constitutive law is used for undrained effective stress analyses. These analyses utilize the \( s_u \) at seabed and linear increase of the \( s_u \) with \( z \) as direct input parameters. FoS is defined as the ratio of the actual \( s_u \) compared to the minimum \( s_u \) required for equilibrium utilizing an incremental reduction of \( s_u \).
Results

Sediment properties  A sedimentological description of the shelf and shelf break off the Nice international airport was carried out using one long Calypso core and 10 short gravity cores (see Fig. 4.13 for location). The visual core log of 919-gc serves as an example describing the lithology of the upper 4 m of sediments (Fig. 4.16a). It shows predominantly clays with thin to thick intermediate layers of silty clay and irregularly distributed silty to sandy beds. The silt beds are up to 20 cm thick and the sand beds are less than 5 cm thick.

The fc and v-s experiments performed every 5 cm were used to verify and validate the calculated $s_{u,qt}$ taken from two strain-rate corrected (quasi-static) CPTU tests (Eq. 4.9), in particular, to confirm the applied empirical $N_{kt}$ factor (Fig. 4.16b, see also section [In situ intact undrained shear-strength]). Consequently, $s_{u,qt}$ profiles of 954-3 and 13-st are compared to $s_{u,fc}$ and $s_{u,v-s}$ measured on core 919-gc. An $N_{kt}$ of 20 is employed due to the potential existence of free gas. Good agreement is found between both in situ records, fc and v-s experiments, except two 20 cm thick coarse-grained beds between 1 and 2 m depth, where an unrealistically low $s_{u,fc}$ is observed. From 3.3 to 4.0 m depth, $s_{u,fc}$ shows several positive peaks up to 52 kPa and a slightly higher trend compared to the $s_{u,qt}$, potentially related to shell fragments encountered in the homogeneous clay section or due to compaction of the sediments during core recovery. Good agreement can also be seen comparing $s_{u,qt}$ with $s_{u,v-s}$ of 2470-cpc (Sultan et al. 2004), which exhibits a broader range due to uncertainties related to the lower sampling rate of 15 cm (Fig. 4.16b). From a geotechnical standpoint, $s_u$ varies between 2 and 5 kPa at and close to the seabed, and an average strength increase with $z$ of 0.8 kPa/m. Several peaks up to 12 kPa are detected, usually related to coarse-grained layers (Table 4.1 and Figs. 4.16a, 4.16b).

The $\Delta u_{2,q-s}$ profiles of 954-3, 13-st and 14-st serve as examples to illustrate the difference between free gas-affected and -unaffected sediments (Fig. 4.16c and Fig. 4.113 for location). The first two profiles show sub-hydrostatic conditions, potentially related to free gas accumulation. The significant difference between the profiles is likely due to the different test locations with a distance of $\sim$125 m and potentially illustrates the high variability of the gas content. Conversely, the latter was collected in an eroded area on the upper continental slope (see Fig. 4.13 for location) and shows a constant increase with $\sigma_{V0}'$, but may be unaffected by the presence of free gas. The $\Delta u_{2,q-s}$ parameters are as low as -25 kPa for the free gas-affected tests, while the unaffected profiles show values less than 20 kPa for the first 4 m of sediments (Fig. 4.16c).

The grain size distribution was evaluated for the 926-gc and 952-gc using a sampling interval of 10 cm, and was supplemented by additional samples extracted from 935-gc and 936-gc. The clayey to sandy sediments are characterized by up
4.2. **NICE AIRPORT LANDSLIDE**

**Figure 4.16:** Sedimentological and geotechnical profile of the gravity core 919-gc collected at the shelf break off the *Nice* international airport (southeastern France). (a) The visual core description is shown. (b) The laboratory geotechnical profiles comprise the intact vane shear-strength ($s_{u,v-s}$), intact fall cone penetration strength ($s_{u,fc}$) measured on 919-gc (box and circle) and the first 4 m $s_{u,v-s}$ of the *Calypso* piston core 2470-cpc (grey corridor; Sultan et al. 2004). In addition to the laboratory datasets, two adjacent dynamic-CPTU profiles, labeled by 13-st and 954-3, are depicted using an empirical cone penetration resistance factor ($N_{kt}$) of 20 to derive the *in situ* intact undrained shear-strength ($s_{u,qt}$) from the corrected cone penetration resistance ($q_t$). (c) The quasi-static excess pore pressure profiles ($\Delta u_{2,qt-s}$) of 13-st, 954-3 and 14-st are illustrated. The first two are potentially influenced by free gas growth, while the latter is unaffected and located in an eroded area (see Fig. 4.13 for location).
Table 4.1: Soil properties of clay to silty clay with embedded layers of silt and sand derived from in situ CPTU tests, sedimentological and geotechnical experiments on cored samples taken from Calypso piston / gravity cores.

<table>
<thead>
<tr>
<th>sediment properties</th>
<th>clay to silty clay with embedded layers of silt and sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>in situ total unit weight, $\gamma_t$</td>
<td>14-18 (kN/m$^3$)</td>
</tr>
<tr>
<td>moisture content, $\omega$ (%)</td>
<td>32-45</td>
</tr>
<tr>
<td>liquid limit, $\omega_L$ (%)</td>
<td>30-40</td>
</tr>
<tr>
<td>plastic limit, $\omega_P$ (%)</td>
<td>15-20</td>
</tr>
<tr>
<td>plasticity index, $I_P$ (%)</td>
<td>10-25</td>
</tr>
<tr>
<td>liquidity index, $I_L$ (%)</td>
<td>125-135</td>
</tr>
<tr>
<td>sand proportion &gt;0.063 mm (%)</td>
<td>2-30</td>
</tr>
<tr>
<td>silt proportion &gt;0.002 to 0.063 mm (%)</td>
<td>40-87</td>
</tr>
<tr>
<td>clay content &lt;0.002 mm (%)</td>
<td>10-30</td>
</tr>
<tr>
<td>intact fall cone penetration shear-strength, $s_{u,fc}$</td>
<td>2.5-12.0 (kPa)</td>
</tr>
<tr>
<td>intact vane shear-strength, $s_{u,v-s}$</td>
<td>2.5-7.5</td>
</tr>
<tr>
<td>in situ intact undrained shear-strength, $s_{u,qt}$</td>
<td>2.5-8.0</td>
</tr>
<tr>
<td>in situ intact undrained shear-strength gradient with depth (kPa/m)</td>
<td>$\geq$0.78</td>
</tr>
<tr>
<td>reference Young’s modulus for normally consolidated sediments, $E_{ref1}$ (MPa)</td>
<td>3-7</td>
</tr>
<tr>
<td>incremental Young’s modulus for normally consolidated sediments, $E_{inc1}$ (MPa/m)</td>
<td>0.97-1.23</td>
</tr>
<tr>
<td>reference Young’s modulus for sediments affected by slide processes, $E_{ref2}$ (MPa)</td>
<td>11-15</td>
</tr>
<tr>
<td>incremental Young’s modulus for sediments affected by slide processes, $E_{inc2}$ (MPa/m)</td>
<td>1.43-1.50</td>
</tr>
<tr>
<td>vertical coefficient of permeability, $k_f$ (m/s)</td>
<td>$1.2<em>10^{-9}$-$1.9</em>10^{-9}$</td>
</tr>
<tr>
<td>coefficient of consolidation, $c_v$ ($m^2/s$)</td>
<td>$2.7<em>10^{-6}$-$3.0</em>10^{-6}$</td>
</tr>
</tbody>
</table>

Note: 1 derived in situ parameters based on Mayne et al. 2010,
2 $fc = $ fall cone penetration experiment, post cruise,
3 $v-s = $ vane shear experiment, post cruise, and
4 in situ intact undrained shear-strength derived from the corrected cone penetration resistance.

to 30% clay content, up to 87% silt proportion and less than 30% sand proportion (Table 4.1). The high sampling rate enables a detailed analysis, when linking the gsd with the visual core logs. Figure 4.17 illustrates the cumulative distribution (ISO 14688-1 2002) including three narrow bands of gsd for the clay to silty clay, silt and sandy silt to sand. The clay content for the clay to silty clay, defined as fine-grained sediments hereafter, varies between 20 and 30%. The clay content
4.2. NICE AIRPORT LANDSLIDE

Figure 4.17: The cumulative grain size distribution (gsd) is shown according to ISO 14688-1 (2002). The gsd is subdivided in three sections representing the main sediment classes of the study area, described as clay to silty clay, silt and silty sand to sand.

...for the silt and sandy silt to sand, defined as coarse-grained sediments hereafter, show values in the range of 15 to 21% and 10 to 18%. An increased amount of the sand proportion is observed comparing fine-grained and coarse-grained sediments (Fig. 4.17). In addition, the index properties $\omega$, $\omega_L$ and $\omega_P$ were determined on representative gc samples of the 935-gc and 936-gc (see Fig. 4.13 for location). The $\omega$ varies between 32 and 45% weight fraction of the wet mass. The $\omega_L$ are up to 40% and $\omega_P$ are less than 20%. Both limits result in an $I_P$ between 10 and 25%. The combination of $I_P$ and $\omega_L$ describes a low to medium plastic clay, low plastic silt and minor cohesive sand using the Casagrande "A-line" soil classification chart (BS 5930 1999).

...The $m_v$ and $E$ parameters were assessed using CT on surficial sediments taken from 925-gc and 939-gc recovered in the 1979 NAIL area. Four tests were performed on samples deposited prior to and after the 1979 NAIL event. The $m_v$ was ascertained to be between $2 \times 10^{-5}$ and $1 \times 10^{-3}$ m$^2$/kN for up to 35 m depth. In addition, $E$ is determined for sediments unaffected by mass movements, such as normally consolidated deposits, and for sediments influenced by slide events or erosional processes. Regression analyses result in $E$ up to 7 MPa for the seabed ($E_{ref1}$) and the $E$ gradient with $z$ ($E_{inc1}$) varies between 1.0 and 1.3 MPa/m for the unaffected sediments. The affected sediments reflect 3.5 times higher values...
at the seabed ($E_{ref2}$) and the $E_{inc2}$ is up to 1.4 times higher. The vertical and horizontal $k_f$ values were measured on three core specimens of 958-gc and 957-gc in order to evaluate the hydraulic anisotropy. The vertical $k_f$ are between $1.2*10^{-9}$ and $1.9*10^{-9}$ m/s, while the horizontal $k_f$ are up to 1.3 times higher resulting in a favored horizontal flow direction. The $m_v$ supplemented by the $k_f$ were used to determine the $c_v$, which is higher than $2.7*10^{-6}$ m$^2$/s (see also Table 4.1 for details). The "non-dimensional velocities" (Eqs. 4.14, 4.15) exceed a value of 30; thus, all static- and dynamic-CPTU tests were performed under fully undrained conditions (Finnie and Randolph 1994, Steiner et al. [In Review]).

Sub-seafloor modeling

Geotechnical/strength modeling  Detailed regression analyses were performed to determine the in situ undrained shear-strength ($s_{u,reg}$), described by the $s_{u,reg}$ at the seabed/surface ($d_{1-3}$) and $s_{u,reg}$ gradient ($k_{1-3}$) with $z$. The subscript defines three different layers relevant for the 2D numerical transects. As an example, Figure 4.18 presents the $s_{u,qt}$ profile of the dynamic-CPTU test 23-st describing all results of the regression analysis. The $d_1$ is 3.92 kPa and $k_1$ is 0.98 kPa/m. In the regression analysis, prominent soft, stiff and probably coarse-grained beds are excluded using a robust regression solution (Holland and Welsch 1977) and visual engineering judgment (Fig. 4.18).

Almost all in situ and coring data (see Fig. 4.13 for location) are used to construct an area-wide contour map of $d_1$ with an interval of 0.5 kPa (Fig. 4.19). This map is incorporated with the bathymetry to obtain a cross reference with the morphological features. For instance, low $d_1$ in the range of 2.5 kPa are linked to several morphological scars and the increase of the slope angle ($\alpha$), which can be seen in the southern part of the map. Figure 4.19 also illustrates five dynamic-CPTU benchmarks, denoted as 086-3, 954-3, 23-st, 539 and 954-13-3, in order to obtain a better understanding of the relationship between the map and the associated $d_1$ presented in Table 4.2. The first two tests describe the lowest $d_1$ of 2.5 kPa located at the southern region, and the latter tests show distinctive higher $d_1$ up to 44 kPa compared to the surrounding tests. Consequently, the latter tests are probably located within an anthropogenic fill area and thus, are excluded because of the low amount of measurements to attain a realistic $s_{u,reg}$ contour map. The highest $d_1$ up to 11.7 kPa are located in the center part of the shelf (dynamic-CPTU test 541) and $d_1$ varies between 4 and 6 kPa at the eastern shelf break (Fig. 4.19).

The seabed map is supplemented by an area-wide contour map describing $k_1$ (Fig. 4.20). The minimum spacing is 0.1 kPa/m and the map is also linked to the bathymetry. The dynamic-CPTU test 541 shows the maximum $k_1$ of 2.52 kPa/m. The minimum $k_1$ is 0.78 kPa/m observed at the dynamic-CPTU site 954-3. The
4.2. NICE AIRPORT LANDSLIDE

**Figure 4.18:** Example of a dynamic-CPTU profile (23-st) showing the *in situ* intact undrained shear-strength \( (s_u,qt) \) derived from the corrected cone penetration resistance \( (q_t) \) and the associated regression *in situ* intact undrained shear-strength \( (s_u,reg) \). The empirical cone penetration resistance factor \( (N_{kt}) \) is 20. The regression equation, \( s_u,reg \) at the seabed \( (d_1) \) and \( s_u,reg \) gradient \( (k_1) \) with penetration depth \( (z) \) are illustrated and quantified. The soft, stiff and probably coarse-grained beds are marked.

dynamic-CPTU tests at the potential anthropogenic fill area (539, 954-13-3) show prominent peaks up to 220 kPa/m (see also Table 4.2) and very low penetration depths less than 0.2 m, and were excluded in the sub-seafloor modeling. Moreover, due to different penetration depths of the dynamic-CPTU tests, the \( k_1 \) contour map is validated for the upper 2 m of sediments at the center part of the shelf, and between 6.5 and 8.5 m at the shelf edge and shelf break (Figs. 4.20).

A relationship between \( \alpha \), \( k_1 \) and \( d_1 \) is detected for the surficial sediments and is represented by a decrease of both \( s_u,reg \) parameters with increasing \( \alpha \) (i.e. going from the center part to the southern and eastern shelf break). The \( s_u,reg/\alpha \) relationship is also shown by comparing static-CPTU tests (11-s3, 11-s4, 11-s5, 11-s6, 12-s2, 12-s3) closely located to the shelf edge (see also Sultan et al. 2010, Leynaud and Sultan 2010) and v-s experiments on 2470-cpc situated at the shelf break (see also Sultan et al. 2004). Figure 4.21 presents the \( s_u,qt \) of the static-CPTU test 12-s3 using an \( N_{kt} \) of 20, and all other static profiles are illustrated as an \( s_u,reg \) range, except 11-s1 and 11-s2, because both tests are located in the center part of the shelf (see Fig. 4.13 for locations). The first 6.5 m long \( s_u,v−s \) profile (2470-cpc) is 1.4 to 2.3 times lower compared to the \( s_u,reg \) of the static-CPTU profiles, potentially related to slow deformation processes of a distinct, surficial sediment mass. In this upper sediment succession, \( d_1 \) exhibits values up to 7 kPa and \( k_1 \) is less
than 1.2 kPa/m. At 6.5 m depth for the static-CPTU tests and at 8.5 m depth for the v-s experiments, a significant drop in \( s_{u,reg} \) and \( s_{u,v-s} \) is detected to between 1.0 and 5.5 kPa. The regression range for the shallow sediments up to 28 m depth is quantified by \( d_{2-3} \) up to 3.9 kPa and \( k_{2-3} \) varies between 0.84 and 0.95 kPa/m (Fig. 4.21). Table 4.3 summarizes the \( d_{2-3} \) and \( k_{2-3} \) parameters for 6.5 to 28 m depth of the static-CPTU tests and v-s experiments on 2470-cpc. In summary, the variability in the \( k_1 \) and \( d_1 \) contour maps as well as the static-CPTU tests reveal two distinct sediment units when comparing the inner and outer shelf properties.

**Lithological/geometrical modeling** Two chirp seismic transects are chosen for the lithological/geometrical modeling of the sub-seafloor sediments and to identify coarse-grained and potential weak layers. The first transect (ha 100) is located south of the center shelf (Fig. 4.22a) and the second one (ha 102) is located at the western part of the shelf (Fig. 4.22b and Fig. 4.13 for location).
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Table 4.2: List of selected surficial regression \textit{in situ} intact undrained shear-strength ($s_{u,\text{reg}}$) at the seabed ($d_1$) and $s_{u,\text{reg}}$ gradient ($k_1$) with penetration depth.

<table>
<thead>
<tr>
<th>in situ and coring data</th>
<th>surficial regression \textit{in situ} intact undrained shear-strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$k_1$ (kPa/m)</td>
</tr>
<tr>
<td>086-3</td>
<td>0.80</td>
</tr>
<tr>
<td>11-s1</td>
<td>2.45</td>
</tr>
<tr>
<td>11-s2</td>
<td>2.08</td>
</tr>
<tr>
<td>11-s6</td>
<td>0.92</td>
</tr>
<tr>
<td>12-s3</td>
<td>1.14</td>
</tr>
<tr>
<td>23-st</td>
<td>0.98</td>
</tr>
<tr>
<td>24-st</td>
<td>1.03</td>
</tr>
<tr>
<td>539</td>
<td>173.10</td>
</tr>
<tr>
<td>540</td>
<td>1.87</td>
</tr>
<tr>
<td>541</td>
<td>2.52</td>
</tr>
<tr>
<td>905-6</td>
<td>1.22</td>
</tr>
<tr>
<td>919-gc</td>
<td>0.78</td>
</tr>
<tr>
<td>926-gc</td>
<td>0.89</td>
</tr>
<tr>
<td>952-gc</td>
<td>0.87</td>
</tr>
<tr>
<td>954-3</td>
<td>0.78</td>
</tr>
<tr>
<td>954-13-3</td>
<td>219.90</td>
</tr>
<tr>
<td>954-14</td>
<td>0.87</td>
</tr>
<tr>
<td>954-15</td>
<td>1.37</td>
</tr>
</tbody>
</table>

Table 4.3: List of the shallow regression \textit{in situ} intact undrained shear-strength ($s_{u,\text{reg}}$) at the seabed ($d_{2-3}$) and $s_{u,\text{reg}}$ gradient ($k_{2-3}$) with penetration depth.

<table>
<thead>
<tr>
<th>in situ and coring data</th>
<th>shallow regression \textit{in situ} intact undrained shear-strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$k_{2-3}$ (kPa/m)</td>
</tr>
<tr>
<td>11-s3</td>
<td>0.89</td>
</tr>
<tr>
<td>11-s4</td>
<td>0.84</td>
</tr>
<tr>
<td>11-s5</td>
<td>0.87</td>
</tr>
<tr>
<td>11-s6</td>
<td>0.87</td>
</tr>
<tr>
<td>12-s2</td>
<td>0.89</td>
</tr>
<tr>
<td>12-s3</td>
<td>0.90</td>
</tr>
<tr>
<td>2470-cpc</td>
<td>0.95</td>
</tr>
</tbody>
</table>
Figure 4.20: Area-wide contour map of the regression *in situ* intact undrained shear-strength \( (s_{u,reg}) \) gradient with penetration depth \( (k_1) \), bathymetrical picture and contour map with a depth interval of 25 m (dashed white lines), and several dynamic-CPTU benchmarks and related values are shown. The dashed black line depicts the 1979 Nice landslide scar.
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Figure 4.21: The in situ intact undrained shear-strength ($s_{u,qt}$) of the static-CPTU test 12-s3 (modified after Leynaud and Sultan 2010) and the intact vane shear-strength of the 2470-cpc (after Sultan et al. 2004) are illustrated considering an empirical cone penetration resistance factor ($N_{kt}$) of 20. The regression range for the static-CPTU tests (grey corridor) and regression line for 2470-cpc (dashed black line) are shown and supplemented by the surficial ($k_1$) and shallow ($k_2$) average regression parameters. The $s_{u,qt}$ drop and $s_{u,qt}$ minimum related to the weak zone are also depicted and quantified.
Figure 4.22: (a) 2D chirp seismic transect (ha 100) located south to the center part of the shelf, and (b) the second transect (ha 102) situated at the western part of the shelf and shelf break are shown (see Fig. 4.13 for locations). Both transects contain the positions of the in situ and coring profiles, and illustrate the coarse-grained beds (dashed black line) and weak zone (shaded area). The morphological scars, landslide eroded areas and morphological steps are also marked. (c, d) The in situ intact undrained shear-strength ($s_{u,qt}$) profiles of crucial tests are illustrated using an empirical cone penetration resistance factor ($N_{kt}$) of 20. The $s_{u,qt}$ drop and $s_{u,qt}$ minimum related to the weak zone can be seen.
For ground truthing purposes, the lithological modeling is supplemented by core logs (see as example Fig. 4.16), $s_{u,v-s}$ and $s_{u,qt}$ profiles (Table 4.2, Figs. 4.22, 4.23). This combination enabled identification of three different coarse-grained layers. The first one is characterized by 15 cm thick silt and 5 cm thick sandy silt to sand beds at 1 to 2 m depth. The $s_u$ is 1.5 to 2.0 times higher compared to the surrounding sediments. At 2.5 to 3.0 m depth, the second layer shows an alternating sequence of clay, silt, sandy silt to sand with a thickness of 50 cm including a $s_u$ increase of 1.5 times. The third layer, at a depth of 4.0 to 5.5 m, is a 10 cm thick sandy silt to sand bed with significant $s_u$ peaks up to 30 kPa (Figs. 4.22, 4.23). Figure 4.23 depicts also decreased $s_u$ behavior, when going from the shelf to the shelf break (see also Fig. 4.21).

Beyond 6.5 m depth, a weak zone with a maximum thickness of 8.5 m is detected in the static-CPTU tests and 2470-cpc (see also Sultan et al. 2010), and is mapped in the chirp transects (Figs. 4.22, 4.23). The exact thickness can only be validated in the static-CPTU tests (Fig. 4.21), but not in the chirp transects due to gas blanking or limited penetration depth. The weak zone consists of soft clays with a minimum $s_u$ of 6.5 kPa and embedded silt, sandy silt to sand with $s_u$ up to 26 kPa (Figs. 4.21, 4.23). A direct correlation between the surface of the weak zone and the eroded area beneath the morphological scar is found. Morphological steps at the shelf area are connected to the coarse-grained beds and the surface of the weak zone (Figs. 4.22, 4.23).

**Slope stability assessment**

**In situ total unit weight** The $\gamma_t$ is assessed using three static-CPTU tests and two adjacent dynamic-CPTU records, indicated as 11-s6, 12-s2, 12-s3, 23-st and 24-st, which are located at the shelf edge (see Fig. 4.13 for location). Figure 4.23 illustrates, for reasons of clarity, only the CPTU tests 11-s6, 12-s3 and 23-st; however, 11-s6 represents the maximum $\gamma_t$ compared to the other CPTU tests. Good agreement can be seen between the static- and dynamic-CPTU tests, except the first 2 m, which is probably attributed to the $f_s$ records of the static device, but remain unclear. A non-linear increase with $z$ down to 28 m is observed and quantified up to an average $\gamma_t$ of 17 kN/m$^3$, except the drop to an average $\gamma_t$ of 15 kN/m$^3$ detected in the weak zone between 6.5 and 15 m. Dan et al. (2007) present consolidation tests on gc, which are used to calculate the total and submerged unit weights, and the tests are connected to the lithological profile of onshore CPTU measurements. The test results show very good agreement with $\gamma_t$ except the first 4 m, where the total unit weight is 10% higher. At 20.5 m depth, a slightly increased behavior less than 5% are detected in contrast to the offshore CPTU records (Fig. 4.23).
Figure 4.23: *In situ* total unit weight ($\gamma_t$) profiles, derived from dynamic- and static-CPTU tests using the empirical solution proposed by Mayne et al. (2010), are shown (see also Eq. 4.10). In addition, laboratory total unit weight records based on consolidation tests (after Dan et al. 2007) and MCSL logging (see also Leynaud and Sultan 2010) are illustrated.
Leynaud and Sultan (2010) present MSCL total unit weight ($\gamma$) data with values between 18 and 20 kN/m$^3$ collected at the shelf and 1979 NAIL area, and similar $\gamma$ are derived from 926-gc and 2470-cpc. The MSCL data exhibit at least 1.3 times higher values compared to the in situ and consolidation tests. However, a convergent behavior of the 2470-cpc datasets is observed at depths greater than 15 m.

**Geometry and intact undrained shear-strength distribution** The geometrical boundaries of the 2D numerical models are derived from the 2D chirp transects (see also Fig. 4.22 for details). A vertical prolongation down to 200 m and a horizontal prolongation of 75 m beyond the SE end of profile ha100 (Fig. 4.24a) were applied to avoid edge effects. The maximum slope angle of 15.5° is observed at ha102 (Fig. 4.24b). Both models are subdivided in vertical segments with a width less than 25 m, and a tripartite layering system (L1, L2 and L3) are implemented to take into account the $\gamma_t$ and $s_u$ distributions. The $\gamma_t$ of the surficial sediments (L1) and the weak zone (L2) is 15 kN/m$^3$, and the layer thickness is equal or less than 8.5 m. The shallow to deep sediment succession, indicated by L3, is described by $\gamma_t=17\text{kN/m}^3$ (see also Fig. 4.23).

The $s_u$ distribution is quantified by $k_{1-3}$ and $d_{1-3}$ considering a linear increase with $z$ (see also Figs. 4.18-4.21 for details). The $k_1$ varies between 0.78 and 1.05 kPa/m and $d_1$ shows values up to 3.8 kPa (see also Figs. 4.19-4.20). For L2, the values of 2470-cpc and the static-CPTU records are used (Table 4.3), except for the vertical segments of the landslide eroded areas. Consequently, $k_{2-3}$ varies between 0.88 and 0.95 kPa/m, and the $s_u,\text{reg}$ at the seabed is less than 1.95 kPa (i.e. mean value of the static-CPTU tests; see Table 4.3). Both parameters are applied to calculate $d_2$, which is between 7.9 and 9.4 kPa depending on the thickness of L1. A similar procedure is used to evaluate the $k_3$ and $d_3$; thus, $k_3$ is equivalent to the $k_2$, and $d_3$ varies between 15.4 and 16.9 kPa depending on the thicknesses of L1 and L2 (i.e. $d_3$ is the regression parameter at the surface of L3). The 2D numerical models require also $d_{2-3}$ and $k_{2-3}$ for the landslide eroded areas (Figs. 4.24a, 4.24b). These parameters are derived from dynamic-CPTU tests performed in failed and landslide-prone areas, such as 036-1, 036-2, 14-st and 958-gc (see also Fig. 4.13 for location). Table 4.4 summarizes these CPTU tests, and the average $d_2$ is 6.1 kPa with a $k_2$ of 1.16 kPa/m. A similar procedure as presented above is applied to determine the parameters for L3. It is worth noting that the left segments of the 2D numerical model utilize elastic properties in order to avoid unrealistic stress conditions and geometrical boundary effects.

**Results of the stability analyses** Three 2D finite element analyses were performed to: (i) evaluate the undrained stability of the shelf and shelf break near
CHAPTER 4. GEOTECHNICAL APPLICATIONS

Figure 4.24: Geometrical boundaries, \textit{in situ} total unit weight ($\gamma$) and regression \textit{in situ} intact undained shear-strength ($d_{1-3}$, $k_{1-3}$) distributions of two 2D finite element models are illustrated. The 2D finite element models are located at the chirp transects ha 100 (a) and ha 102 (b). A sketch of the implemented strength profile with penetration depth is also depicted including the reference to the tripartite layering system (L1, L2, L3).

the \textit{Nice} international airport, and (ii) estimate different maximum shear failure geometries and volumes of potential landslides. Figure 4.25 illustrates a concave-shaped progressive failure with a FoS equal 1, which is located south of the shelf center area. The failure length is $\sim$275 m, depth is 55 m and the sediment volume having the potential to fail is up to 10 million m$^3$ assuming a failure width between 700 and 800 m. The assumed failure width considers that the progressive failure affects the entire eastern plateau (see Fig. 4.13).

Two similar analyses were carried out at the western part of the shelf and shelf break. Figure 4.25 depicts the initial shear failure geometry with a length of 75 m and a depth of 20 m, but a main failure geometry with $\sim$230 m length and 43 m
Table 4.4: List of the regression in situ intact undrained shear-strength ($s_{u,reg}$) at the seabed ($d_2$) and $s_{u,reg}$ gradient ($k_2$) with penetration depth collected at landslide eroded areas in the vicinity of the 1979 NAIL area.

<table>
<thead>
<tr>
<th>in situ and coring data</th>
<th>regression in situ intact undrained shear-strength for the landslide eroded areas</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$k_2$ (kPa/m)</td>
</tr>
<tr>
<td>036-1</td>
<td>1.13</td>
</tr>
<tr>
<td>036-2</td>
<td>1.20</td>
</tr>
<tr>
<td>14-st</td>
<td>1.15</td>
</tr>
<tr>
<td>958-gc</td>
<td>1.17</td>
</tr>
</tbody>
</table>

Figure 4.25: Results of the 2D numerical slope stability analyses presenting the initial and overall shear failure geometries and failure depths for the transects ha100 (a) and ha102 (b, c).
depth are observed when integrating large deformations of the 2D finite element mesh and a FoS \( \leq 1 \) (Fig. 4.25b). The volume of a potential landslide is up to 6 million m\(^3\) assuming a failure width of 700 to 800 m (i.e. collapse of the entire eastern plateau).

Discussion

**Dynamic-CPTU device and sediment properties** Comprehensive dynamic-CPTU datasets (~140 tests near the *Nice* international airport) reveal the ability of the *MARUM* dynamic-CPTU instrument to characterize complex surficial depositional environments in a time- and cost-effective way (Fig. 4.13, Kopf et al. 2008, 2009, 2012 and N. Sultan, unpublished data). Similar successful applications of the dynamic-CPTU instrument were performed in northern Norway (Steiner et al. 2012, Steiner et al. [In Review]) and Chile (G. Wiemer, pers. comm., 2013), where penetration depths up to 8.3 m were reached in soft to normally-consolidated clays. The dynamic nature of such an instrument causes higher \( q_t \), \( f_s \) and \( \Delta u_2 \) values compared to the static-CPTU parameters, and thus have to be corrected using a modified inverse sinhypberbolic equation, best suited for the strain-rate correction in clays (Eqs. 4.6, 4.7, 4.8, Randolph and Hope 2004, Steiner et al. [In Review]). Dynamic-CPTU tests in sediments with a clay content up to 30% (Table 4.1) require SSCs for \( q_t \) and \( \Delta u_2 \) (\( \mu_{CPTU,q_t} \) and \( \mu_{CPTU,\Delta u} \)) between 0.1 and 0.15, and SSC for \( f_s \) (\( \mu_{CPTU,f_s} \)) of less than 1.4. Using these SSCs, good agreement between the static \( s_{u,qt} \) of 12-s3 and quasi-static \( s_{u,qt} \) of 24-st (Fig. 4.22b) and between quasi-static \( s_{u,fc} \), \( s_{u,v} \) and \( s_{u,v-s} \) profiles (Fig. 4.16b) were found. Other studies in similar sediments confirm these findings (Randolph and Hope 2004, Steiner et al. 2012, Kopf et al. 2012, Steiner et al. [In Review]).

In previous studies, \( N_{kt} \) was quantified between 8 and 20 for clays depending on the \( I_P \) and overconsolidation ratio (e.g. Aas et al. 1986, Powell and Quarterman 1988, Robertson 2009). Low et al. (2010) combined a worldwide high-quality database for the assessment of \( s_{u,qt} \) and remolded undrained shear-strength, and recommend \( N_{kt} \) in the range of 11.5 to 15.5 based on the average undrained shear-strength (\( s_{u,ave} \)) measured in direct simple shear, triaxial compression and extension experiments (see also Lunne et al. 2011). At the *Nice* study area, assumed average \( N_{kt} \) of 12 and 15 according to Lunne et al. (1997) were used to calculate \( s_{u,qt} \) for numerical 2D and probabilistic 3D slope stability assessments (Dan et al. 2007); however, in the probabilistic assessment, \( s_{u,qt} \) uncertainties were considered by \( N_{kt} \) of 10 to 20 correspond to three standard deviations (Leynaud and Sultan 2010). Sultan et al. (2010) use comparisons of static-CPTU tests with v-s experiments on cores to determine \( N_{kt} \) resulting in values of 14 for intact saturated sediments and 20 for sediments affected by dissolved and free gas. A similar decrease of \( s_u \) between 20 and 25% was detected in two independent laboratory studies using v-s tests on fine-grained intact samples and sediments containing large
gas bubbles (Nageswaran 1983, Sham 1989). In this study, comparisons of $s_{u,fc}$, $s_{u,v,s}$ taken from laboratory experiments on short, long cores and $s_{u,qt}$ obtained from contiguous CPTU tests show very good agreement when using an $N_{kt}$ of 20 (Fig. 4.16b), and is therefore appropriate for intact gassy sediments.

The associated $\Delta u_{2,q,s}$ shows a negative response for deployments at the shelf and shelf break (i.e. sub-hydrostatic excess pore pressures), which are not ascertained in the landslide-prone or eroded regions (Fig. 4.16c and Fig. 4.13 for location). Seifert et al. (2008) describe such a negative response in $\Delta u_{2,q,s}$ by the occurrence of free gas in the sediments. Two possibilities were discussed: (i) free gas accumulation due to decay of organic material resulting in variations of shear-strength, in particular, an increase of the gas content reducing the intact undrained shear-strength by about 20% (e.g. Wheeler 1986, Brandes 1999), and (ii) buoyancy of free gas in fissures generated during penetration and along the rods of the dynamic-CPTU instrument causing sub-hydrostatic $\Delta u_{2,q,s}$ (e.g. Seifert and Kopf 2012). In addition, X-ray scans and visual observations on 957-gc and 926-gc illustrate sub-vertical conduits (piping) with a diameter less than 10 mm, which connect two silt layers. These conduits may be related to overpressure release and fluidization (Kopf et al. 2009, Stegmann et al. 2011). In low-permeability, fine-grained sediments with $k_f$ lower than 1.9*10$^{-9}$ m/s (Table 4.1), disruption of the sediment structure may be caused when pore pressure, as a result of fluid flow or gas accumulation, exceeds the minimum effective stresses and tensile strength of the material, known as hydro-fracturing (Anderson et al. 1994). Consequently, the observations of $\Delta u_{2,q,s}$ and comparisons of $s_u$ near the Nice international airport reveal that this area is potentially affected by free gas having significant impact on the sediment properties (Figs. 4.16b, 4.16c, 4.19, 4.20, Sultan et al. 2010).

The in situ and laboratory $s_u$ profiles yield to values ranging between 2.5 and 12 kPa over the first 4 m of sediments (Table 4.1 and Fig. 4.16b). This $s_u$ range is confirmed by an upper limit of 10 kPa for normally-consolidated sediments and by a lower limit of 3 kPa representing also the boundary between under-consolidated and normally-consolidated sediments derived from v-s experiments at the Nice upper continental slope (Cochonat et al. 1993) and similar limits were detected at the shelf break (Sultan et al. 2004). Dan et al. (2007) present average $s_{u,v,s}$ varying between 59.2 and 67.28 kPa for the first 4.1 m below the seafloor within the 1979 landslide area. However, this $s_{u,v,s}$ range differs significantly from the average $s_u$ of 13 kPa found in the first 10 m of sediment at the eastern plateau area (Leynaud and Sultan 2010) and the results of this study (Table 4.1 and Fig. 4.16b).

Sub-seafloor modeling Data from 45 offshore CPTU tests and 11 cores collected at the eastern plateau near the Nice international airport are used to: (i) develop a detailed area-wide sub-seafloor model, and (ii) perform 2D numerical
stability analyses (Figs. 4.19, 4.20, 4.21, 4.22, 4.25), enhancing the existing 2D and 3D slope stability assessments (Dan et al. 2007, Leynaud and Sultan 2010). The sub-seafloor model and stability assessments take into account $s_u$ variability at a shallow level, in particular, the increase of the $s_u$ with depth using linear regressions defined in cells of up to 25 m horizontal length (Fig. 4.24). So, this model images more accurately the natural $s_u$ distribution than models using slope parallel layers with thicknesses up to 13 m and average $s_u$ parameters (Dan et al. 2007, Leynaud and Sultan 2010).

A comparison of the total unit weight obtained from CPTU tests (Mayne et al. 2010), MSCL logging and consolidation tests (i.e. moisture and density (MAD)) is used to evaluate the ($\textit{in situ}$) total unit weight variability with respect to different measurement techniques (Fig. 4.23). The most common technique is using MAD tests and MSCL logging on core samples in the laboratory. Both experiments are highly sensitive to the quality of the cores (e.g. deformed samples; Skinner and McCave 2003), evaporation/shrinking processes during core preparation and storage (Gerland and Villinger 1995), and presence of free gas in the sediments (e.g. Nageswaran 1983). These processes can lead to an overestimation of the $\gamma_t$ up to 30%, in particular, for the surficial sediments (Fig. 4.23). The total unit weight derived from the CPTU records (Eq. 4.10) is not a direct measurement, but is a proxy for the $\gamma_t$, which were developed based on a multiple regression analysis using 215 datasets collected worldwide (Mayne et al. 2010). This proxy underestimates the $\gamma_t$ at the seafloor (up to 2.5 m depth); however, very good agreement with the consolidation tests is observed at a shallow level (Fig. 4.23). Consequently, we favor the use of CPTU records to assess $\gamma_t$ for slope stability assessments (Fig. 4.24, 4.25). Moreover, the herein presented sub-seafloor model considers also the influence of free gas disrupting the sediment structure that reduces $s_u$ by up to 30% (Fig. 4.16b, Nageswaran 1983, Sham 1989, Sultan et al. 2010). This $s_u$ decrease and the $s_u$ drop (up to 2.3 times), when going from shelf to shelf break (Figs. 4.21, 4.22b), correlates well with the findings mentioned by Dan et al. (2007) that a $\geq 15\%$ $s_u,qt$ reduction is required to generate medium to large shear bands, and gravitational and progressive failure (Fig. 4.25 Leynaud and Sultan 2010).

The existence of a potentially up to 8.5 m thick shallow weak zone, labeled as a possible shear zone by Sultan et al. (2010), is confirmed by two chirp transects (Figs. 4.22a, 4.22b). At the shelf region, the surface of the weak zone and surficial coarse-grained layers are connected to morphological steps or rupture-surface propagation, probably indicating the development of progressive failure (Figs. 4.22a, 4.22b). Petley et al. (2005) propose a four-step conceptual model for the development of progressive failure in cohesive soils. The first step describes the development of microcracks, and is followed by the interaction and coalescence of these microcracks allowing the formation of a shear surface. The formation of a shear surface reduces the $s_u$ of the sediments, and can be seen by the $s_u$ drop
4.2. NICE AIRPORT LANDSLIDE

of up to 5.5 kPa in the *insitu* and coring data (Fig. 4.21). Pore pressure rise in the sandy silt to sand beds favors also the formation of a shear surface, and may be confirmed by the sub-vertical conduits in the less permeable clays connecting two silt layers (Fig. 4.16, Kopf et al. 2009, Stegmann et al. 2011). Such coarse-grained sediments are also found in the 2470-cpc at depth between 15 and 17 m (Sultan et al. 2004, Dan 2007). Consequently, deeper coarse-grained beds may act as fresh-water aquifers, and leaching processes may develop weak zones acting potentially as detachment planes for larger landslides (Dan et al. 2007, Sultan et al. 2010). The moderate hydraulic anisotropy measured on samples, with up to 1.3 times higher horizontal *k*_f values, is insufficient to channel fluid horizontally. Channeling may only occur along laterally extensive permeable sandy layers, which remain hypothetical (see section [Sediment properties]). The third step describes stress concentrations at the shear surface resulting in slow deformation processes, which are displayed by the morphological steps and a connection of the weak zone surface to the landslide eroded area is ascertained (Figs. 4.22, 4.22, Sultan et al. 2010). A fully developed shear surface and FoS of equal or less than unity cause initial failure, resulting in a small rotational slide with a failure height up to 20 m, but the potentially existing weak layer has not much influence on the geometry of the rupture surface (Fig. 4.25c). Similar landslides were identified on the *Nice* upper continental slope using hull-mounted and AUV bathymetrical mapping (Migeon et al. 2012).

**Slope stability assessments** Slow deformation processes are evaluated by partially drained to drained slope stability analyses, where the generated Δ*u* has time to dissipate. In contrast, earthquake loading and rapid pore pressure increase due to high precipitation events in conjunction with low permeability sediments require undrained slope stability analyses, where the Δ*u* not has time to fully drain. Long-term pore pressure records at the 1979 NAIL area reveal a significant correlation between precipitation events and Δ*u*, associated with the beginning of the snowmelt and heavily rainfall events over a period of 3 months (Stegmann et al. 2011). In addition, a seismological network was installed covering the region between *Nice* and the Italian border (Courboulex et al. 2007). More than 500 microearthquakes recorded between 2000 and 2001 reveal a NNE alignment on the so-called *Blausacs* fault, which is probably the "hidden root of the Peille-Laghet fault". Simulations to assess the peak ground accelerations (PGAs) in the city of *Nice* were performed assuming a magnitude 5.7 earthquake if the entire 8 km portion of the *Blausacs* fault fails (Courboulex et al. 2007). The average PGAs were quantified between 0.3 to 1.5 m/s² and potentially strong enough to trigger small to large landslides (Fig. 4.25). Cyclic loading tests assuming a PGA of 1.5 m/s² show FoS close to 1 for a sediment succession with a thickness less than 10 m (Sultan et al. 2004).
In this study, the 2D numerical calculations show FoS equal to or less than 1 considering undrained behavior, free gas accumulation, large deformations of the 2D finite element mesh, and a surficial weak zone resulting in a shear failure depth between 20 and 43 m (Figs. 4.25b, 4.25c). The 1979 NAIL event took place in a depth range of 25 to 40 m (Mulder et al. 1997), and slope stability assessments considering a weak layer (i.e. soft, sensitive clays and embedded sandy silt to sand beds) and creeping result in a FoS equal to unity (Dan et al. 2007). However, a larger landslide with a shear band depth of up to 60 m can also occur at the eastern plateau near the Nice airport (Fig. 4.25a, Leynaud and Sultan 2010) mobilizing up to 10 million m$^3$ of sediments and potentially acting as an initial landslide for a very large event (e.g. Storegga landslide; Bryn et al. 2005). Deep seismic reflection transects reveal a strong, predominantly smooth reflector in a depth of 60 to 100 m (Fig. 4.12b, Kopf et al. 2008, 2009), and is potentially the interface between Quaternary gravels and late Neogene conglomerates (Fig. 4.12, Dubar and Anthony 1995). So, this transition zone probably acts as fresh-water aquifer in which excess pressures can be generated, resulting in a higher potential for liquefaction in combination with the decrease of the effective overburden stresses due to the initial landslide (Fig. 4.25a, Leynaud and Sultan 2010, Anthony and Julian 1997). A detailed characterization of the transition zone requires longer core drillings supplemented by sedimentological and geotechnical analyses on drilled specimens taken above and below this zone. This procedure is indispensable for obtaining a better understanding of the potential likelihood that a very large submarine landslide will occur. Moreover, such a large landslide has the potential to trigger a huge tsunami affecting the coastline of the French Riviera and have a catastrophic impact on social and economic life of this region.

Conclusion

An interdisciplinary approach is commonly used in deep-water environments (e.g. Storegga submarine landslide; Bryn et al. 2005), but is currently applied in coastal and nearshore environments (e.g. Nidelva fjord delta; L’Heureux et al. 2010, Finneidfjord; Vanneste et al. 2013). Such an approach is required for area-wide sub-seafloor modeling including sedimentological and geotechnical characterization of the sediments, identification of coarse-grained / weak layers and evaluation of influencing factors on the sediment physical properties as a result of free gas growth. This modeling is crucial for: (i) performing realistic numerical slope stability assessments, and (ii) obtaining a better knowledge of the initiation and development of submarine landslides.

Numerical slope stability assessments need a thorough assessment of geotechnical properties that, for instance, where determined combining coring with static- and dynamic-CPTU techniques. The dynamic system is operated faster due to its dynamic nature and the static system can reach deeper sediment targets. Con-
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subsequently, the combination of both systems is a powerful, time- and cost-effective way to explore the surficial and shallow sub-surface sediments in shallow- and deep-water environments in up to 4000 m water depth.

Free gas growth in the sediment disrupts the sediment structure, and reduces the strength properties by up to 30% compared to the intact properties. At the shelf and shelf break area near the Nice international airport, a gas weakening of the intact undrained shear-strength ($s_u$) was detected using comparisons of static and dynamic CPTU tests with laboratory experiments on cored samples. These lower $s_u$ values reduce the factor of safety in slope stability assessments at the Nice eastern plateau, and potentially result in a higher likelihood that a tsunamigenic submarine landslide event with a failure depth of up to 100 m will occur in the (near) future. Such a landslide would have catastrophic social and economic consequences for the French Rivera.

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Chapter 5

Ongoing projects

Several exploration datasets, collected in this doctoral project, have not been presented in a manuscript/paper and have not been submitted to a journal or conference yet. These datasets, preliminary results and findings are shown hereafter.

5.1 Landslides

5.1.1 Gela basin landslide

Abstract

In summer 2010 (June 5th, 2010 to July 04th, 2010), the third leg of the R/V Maria S. Merian cruise MSM15 "Biogeochemistry and methane hydrates of the Black Sea; Slides, deep water formation and seismicity of the Mediterranean" was carried out. This research expedition aimed to explore two distinct landslide-prone areas in the vicinity of the Gela basin (southern Sicily, Italy) using multibeam echosounder, sub-bottom profiler, gravity corer, the MARUM sea floor drill rig (MeBo) and the MARUM deep-water dynamic-CPTU instrument. Three sites were drilled with MeBo, and seven gravity cores and 13 deep-water dynamic-CPTU (DWFF-CPTU) profiles were retrieved additionally. The work by the MARUM Marine Geotechnics team during MSM15/3 resulted in a chapter for the cruise report (see hereafter). The following bullet points summarize the main findings obtained with the DWFF-CPTU device:

- The new design of the DWFF-CPTU instrument was tested for WD up to 600 m in the Gela basin landslide area, and reached penetration depth of $\leq 4$ m in soft to normally-consolidated clays.
• The \textit{in situ} intact undrained shear-strength profiles derived from the excess pore pressure and corrected using logarithmic, inverse \textit{sin-hyperbolic} and \textit{velocity ratio strain-rate} solutions result in good agreement with the fall cone penetration and vane shear-strength profiles.

• Morphological steps, seafloor gullies, seepage forces and embedded layers, consisting of stiff fine-grained or coarse-grained soils, are potentially indications for surficial slow deformation processes and gas accumulation in the soil.

\textit{Key words:} Deep-water dynamic penetrometer, excess pore pressure, sub-seafloor modeling, northern and southern \textit{Twin} slides.
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Geotechnical Subchapter of the Cruise Report SACRE
R/V Maria S. Merian MSM15/3 (Elefsina/Greece - Valletta/Malta)

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Part of the cruise report: K. Huhn, and cruise participants (unpublished data)

Introduction

Submarine landslides occur at coastal, nearshore and offshore environments everywhere on earth (e.g. Hampton et al. 1996, Locat and Lee 2000, Kvalstad et al. 2005, Dan et al. 2007). The stability of submarine slope soils is controlled by the preconditioning factors (e.g. excess pore pressure, sedimentation rates) and soil physical properties (e.g. undrained and drained strength parameters). In landslide research, the \textit{in situ} intact undrained shear-strength ($s_u$) and excess pore pressure ($\Delta u$) are two crucial properties, and both can be measured using laboratory experiments on cored/drilled samples and \textit{in situ} CPTU tests (e.g. Sultan et al. 2004, Lunne et al. 1997, Lunne et al. 2011).

Cone penetration testing with pore pressure measurements (CPTU) are a widely used method for \textit{in situ} soil characterization in onshore and offshore settings (e.g. Dayal et al. 1975, Lunne et al. 1997, Sultan et al. 2010, Steiner et al. 2012). Two different systems are in use for the exploration of submarine landslides: (i) the \textit{seabed mode} system, which parks a heavy rig on the seabed and push the CPTU cone with an engine into the sub-seafloor, and (ii) the \textit{winch or free-fall mode} system, which lowers the dynamic-CPTU instrument via a winch or in free-fall through the water column and the instrument penetrates the sub-seafloor by its own momentum. At MARUM - Center for Marine Environmental Sciences, University of Bremen, a \textit{winch or free-fall mode} system was developed in 2006. This autonomous deep-water dynamic-CPTU instrument, named \textit{DWFF-CPTU}, is a straightforward cost- and time-effective system measuring the cone penetration resistance ($q_c$), sleeve friction ($f_s$) and $\Delta u$ (Stegmann and Kopf 2007).
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During the MSM15/3, DWFF-CPTU tests, shipboard sedimentological and geotechnical laboratory experiments (e.g. index properties, moisture and density [MAD], vane shear [v-s] and fall cone penetration [fc] tests) on cored/drilled samples were performed. In this report, the preliminary results and findings are presented reflecting the surficial soil characteristic found in the vicinity of the northern (NTS) and southern Twin slides (STS), which are located on the northeastern slope of the Gela basin (southern Sicily, Italy). The \textit{in situ} intact undrained shear-strength profiles derived from the excess pore pressure ($s_{u,\Delta u}$) are compared with intact strength parameters deduced from v-s and fc experiments ($s_{u,v-s}$ and $s_{u,fc}$) in order to demonstrate the good agreement between \textit{in situ} and laboratory datasets. In addition, the sedimentological and geotechnical data and a previous published sub-bottom profiler transect (Minisini et al. 2007, Minisini and Trincardi 2009) are utilized to develop a simple sub-seafloor model, appropriate for further slope stability and risk assessments.

**Regional setting**

The \textit{Gela} basin is a Neogene-Quaternary foredeep basin of the \textit{Maghrebian} fold-and-thrust belt and is located within the strait of Sicily, central Mediterranean Sea. The basement is covered by a 2.5 km thick complex of shallowing-upward marine soils, mostly fine-grained deposits (Argnani 1988, Trincardi and Argnani 1990).

Morphological investigations present evidence of a giant, old, buried landslide scar, the so-called \textit{father} slide ($\sim$20 ka B.P.), and recently exposed landslide scars, defined as the NTS (Fig. 5.1) and STS (Fig. 5.2), occurred in the Holocene ($\sim$8000 years B.P.) (e.g. Minisini et al. 2007, Kuhlmann et al. 2014). The mass transport deposits (MTD) of both slides consist of debris avalanche deposits; however, the NTS deposits were overlaid by slump soils from a rotational landslide (Minisini and Trincardi 2009).

**Materials and methods**

**In situ CPTU measurement** The \textit{MARUM} deep-water dynamic-CPTU instrument has been deployed (Fig. 3.7 Stegmann and Kopf 2007). A detailed description of the instrument is given in section 3.1.2.

**Instrument setup** The instrument was used with a self-developed piezocone/adapter equipped with pore pressure ports at two locations (i.e. $\Delta u_1$ - at the tip and $\Delta u_3$ - 0.75 m behind the tip), but no $q_c$ and $f_s$ were measured. The deployments were carried out using a 4.1 m long instrument setup (i.e. CPTU cone/adapter + 1 rod + pressure-tight housing).
Figure 5.1: Morphological map of the northern *Twin* slide complex (southern Sicily, Italy). The landslide scar, *DWFF-CPTU* tests GeoB 14403-09 to 14413-01 and GeoB 14420-01 as well as gravity core GeoB 14403-01 are illustrated. The locations are represented with the code digits of the GeoB nomenclature. In addition, prior collected piston cores and seismic profiles are shown (Minisini et al. 2007). The across-track resolution of the map is better than 50 m.

**Deployment mode** The deployments aims at the: (i) high-resolution vertical record of the $\Delta u_1$ and $\Delta u_3$ using 1 kHz logging frequency (i.e. vertical profiling) and (ii) recording of the $\Delta u$ evolution once the *DWFF-CPTU* instrument sticks in the soil (i.e. dissipation test). Figure 5.3 illustrates the $\Delta u_1$, $\Delta u_3$ and equilibrium pore pressure ($u_0$) during vertical profiling and dissipation in a schematic illustration. Pore pressure dissipation is usually recorded for a 20 to 30 min period.
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Figure 5.2: Morphological map of the southern Twin slide complex (southern Sicily, Italy). The landslide scar, *DWFF-CPTU* tests GeoB 14416-01 to 14419-01 and gravity cores GeoB 14404-01 and 14414-02. The locations are represented with the code digits of the GeoB nomenclature. In addition, a prior collected seismic transect is shown (Minisini et al. 2007). The across-track resolution of the map is better than 50m.

*DWFF-CPTU* instrument was veered at 1.2 m/s winch speed to a level 30-50 m above the seafloor, then the winch speed was varied between 0.5-1.2 m/s until the *DWFF-CPTU* cone hits the seabed and dynamically decelerated until its penetration depth of several meters. The instrument is recovered after the dissipation test.
Strain-rate correction  Based on the non-linearly decreasing penetration rate of dynamic-CPTU tests, higher soil physical properties compared to static-CPTU systems are measured (e.g. Young et al. 2011, Steiner et al. 2012, Steiner et al. [In Review]). This so-called strain-rate effect was first described for dynamic penetrometers by Dayal and Allen (1975). To correct this effect, two different state-of-the-art strain-rate correction solutions were introduced (e.g. Dayal and Allen, 1975, Randolph 2004, Mitchell and Soga 2005). These solutions are based on the logarithmic and inverse sinh-hyperbolic equation depending on the ratio of the dynamic ($v_{dyn}$) and reference penetration rates ($v_{ref} = 2 \text{ cm/s}$), and the soil-specific rate coefficients ($\mu_{CPTU}$ and $\mu_{CPTU}'$) associated to the dynamic excess pore pressure ($\Delta u_{dyn}$). In addition, a newly developed strain-rate correction solution, called velocity ratio solution, is presented and expressed as:

$$SF = 1 + \left[ \mu_{CPTU} \left( 1 - \frac{v_{dyn}}{v_{ref}} \right) \right], \quad (5.1)$$

where $SF_{velo}$ is the strain-rate correction factor of the velocity ratio solution and $\mu_{CPTU}^*$ is the soil-specific rate coefficient of this solution.

The assessment of the $s_{u,\Delta u}$ considers the empirical geotechnical equation presented in section 2.1.4. For this study, the empirical excess pore pressure factor ($N_{\Delta u}$) typically falls into the range of 3 to 9 (used 4 for very soft to soft clay) (see Table 2.3 Robertson 2009a, Low et al. 2010 for details).

Physical properties  Shipboard geotechnical tests were performed using v-s and fc experiments on the working half of the collected cores. The geotechnical properties were determined according to the British Standard Institution
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(BS 1377 1975). Multi Sensor Core Logger (MSCL) measurements on the undisturbed archive half were conducted immediately after the cruise at MARUM (Bremen, Germany). A detailed description of the equipments is given hereafter and in several cruise reports, for example Kopf et al. (2008, 2009a, 2012a, 2012b).

Fall cone penetration experiment  A Wykeham-Farrance fall cone penetrometer WF 21600 was utilized for the first-order estimate of the soil resistance (Fig. 5.4a). The interval of fc experiments is ∼10 cm. A metal cone/tip was brought to a point exactly on the surface of the working half (Wood 1985). A manual displacement transducer was applied to measure the distance prior to and after release of the cone (i.e. penetration depth after free-fall process). The accuracy of the penetration depth record is 0.1 mm and penetration depth can be correlated with the soil shear-strength (Hansbo 1957). A fall cone with a defined weight (80.51 g) and geometry (30° cone) was deployed. The intact fall cone penetration strength \( s_{u,fc} \) can be calculated using the geotechnical equation as follows:

\[
s_{u,fc} = \frac{kmg}{x^2}
\]

where \( k \) is the empirical cone factor, \( m \) is the mass of the cone, \( g \) is the gravity (9.81 m/s\(^2\)), and \( x \) is the measured penetration depth. Wood (1985) calculated an average empirical cone factor of 0.85 for a 30° cone comparing fc and miniature v-s experiments.

Vane shear experiment  A GSC Atlantic double vane shear apparatus was used to obtain the soil resistance and residual shear-strength (Fig. 5.4b). The distance between the two v-s experiments is less than 50 cm. A four-bladed vane, with a length \( l \) of 12.5 mm and diameter \( d \) of 12.5 mm, was pushed into the undisturbed core surface of the working half and turned at a fix rate of 90°/min. A spring transfers this rotation to the four-bladed vane. The torque was recorded until full shearing of the soil along the vertical and horizontal edges of the vane occurs. The intact vane shear-strength \( s_{u,v} \) is defined by the torque \( T \) divided by the empirical vane shear constant \( K \), expressed as:

\[
T = B_1\omega + B_2
\]

\[
K = \pi d^2 \left[ \left( \frac{l}{2} \right) + \left( \frac{d}{6} \right) \right]
\]

where \( B_1 \) and \( B_2 \) are spring constants, and \( \omega \) is the maximum torque angle at failure (Blum 1997).

Shore-based laboratory testing were conducted including moisture and density (MAD) tests, direct shear measurements, ring shear experiments, one-dimensional
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Figure 5.4: (a) Wykeham-Farrance fall cone penetrometer WF 21600 and (b) GSC Atlantic vane shear apparatus were used on the working half of the cores to measure the intact undrained shear strength (after Kopf et al. 2008, 2009a, 2012a, 2012b).

compression and static/dynamic triaxial shear tests to obtain the liquefaction potential, strain-rate dependent frictional properties, stiffness, peak and residual shear-strength of the soils. These data are presented and discussed in other or future manuscripts (e.g. Ai et al. 2014).

Multi Sensor Core Logger  The GEOTEK MSCL combines three sensors on an automated track. A schematic illustration of this equipment can be seen in Figure 5.5. The magnetic susceptibility (MS), MSCL gamma density ($\rho$) and P-wave velocity (PWV) were recorded. In addition, RGB color images were created using a full color digital line scan imaging device. A brief overview of the three MSCL sensors is presented hereafter.

A Bartington point sensor MS2 was used to record MS profiles using a non-saturating, alternating magnetic field of 2kHz and a low intensity of 80A/m (0.1mT). The sensitivity setting was 1.0Hz, and sampling period and interval were defined to be 2.0sec and 4.0cm. The average parameter is automatically determined and saved on a PC. It is worth noting that the MS profiles are relative
measurements, thus the difference between gc (80 mm) and coil diameters (88 mm) was not taken into account (http://www.geotek.co.uk).

A GRA bulk densitometer was utilized to measure the $\rho$ using a narrow beam of gamma rays from a Caesium-137 source with energies $\sim 0.662$ MeV. The photons cross the archive half of the gc and are recorded by a detector on the other side. The electrons in the gc scatter the photons resulting in a partial energy loss. The $\rho$ can directly be correlated to the number of unattenuated photons pass through the gc (http://www.geotek.co.uk).

Two P-wave transducers were used to obtain the transverse PWV using a 320 kHz compressional wave pulse passes through the gc horizontally. A transmitter and receiver are aligned perpendicular to the gc while a pair of displacement sensors monitors the distance between transmitter and receiver. The quality of the gc is a crucial factor affecting the accuracy of the measurements (e.g. liner is not fully filled with soil).

Preliminary results

In situ CPTU measurement Nine DWFF-CPTU measurements were performed in the vicinity of the NTS (see Fig. 5.1 for location) and four DWFF-CPTU tests were carried out at the STS area (see Fig. 5.2 for location). Tables 5.1 and 5.2 summaries all data vital for the analysis of the CPTU tests, for example, test location, WD, instrument setup, start velocity, time of dissipation and penetration depth. The WD varies between 150 and 550 mbsl (meter below sea-level), and penetration depth ranges between 2 and 4 m. Due to performance and electronic problems, two DWFF-CPTU tests failed (GeoB14412-01 and 14413-01). The other measurements present good results concerning the characteristic of the sub-seafloor soils (GeoB 14403-09, 14407-01 to 14411-01 and 14416-01 to 14420-01).
### Table 5.1: List of DWFF-CPTU deployments carried out in the vicinity of the northern Twin slide (southern Sicily, Italy).

<table>
<thead>
<tr>
<th>type of investigation (AG KOPF)</th>
<th>GeoB No.</th>
<th>UTM N, 33; WGS 1984 x-re.</th>
<th>LAT (north)</th>
<th>LONG (east)</th>
<th>water depth</th>
<th>DWFF-CPTU length</th>
<th>start velocity</th>
<th>duration of measurement</th>
<th>picture no.</th>
<th>visual penetration depth</th>
<th>penetration depth</th>
<th>geological assessment</th>
</tr>
</thead>
<tbody>
<tr>
<td>DWFF-CPTU xx</td>
<td>144 02-09</td>
<td>431523.00 4079264.00 36°51.41' 14°13.91'</td>
<td>181.98</td>
<td>4.10</td>
<td>1.30</td>
<td>22</td>
<td>n/a</td>
<td>[ ]</td>
<td>3.90</td>
<td>4.10</td>
<td>G1.7 = 4.01</td>
<td>very soft clay</td>
</tr>
<tr>
<td>DWFF-CPTU 02</td>
<td>144 07-01</td>
<td>431462.00 4078355.00 36°50.9172' 14°13.8744'</td>
<td>192.62</td>
<td>4.10</td>
<td>0.88</td>
<td>22</td>
<td>001</td>
<td>3.30</td>
<td>3.40</td>
<td>G1.7 = 3.31</td>
<td>very soft clay</td>
<td></td>
</tr>
<tr>
<td>DWFF-CPTU 03</td>
<td>144 08-01</td>
<td>431356.00 4078229.00 36°50.8482' 14°13.803'</td>
<td>198.48</td>
<td>4.10</td>
<td>0.70</td>
<td>28</td>
<td>002</td>
<td>3.10</td>
<td>3.30</td>
<td>G1.7 = 3.29</td>
<td>very soft clay</td>
<td></td>
</tr>
<tr>
<td>DWFF-CPTU 04</td>
<td>144 09-01</td>
<td>431263.00 4078113.00 36°50.7852' 14°13.7412'</td>
<td>203.97</td>
<td>4.10</td>
<td>0.60</td>
<td>21</td>
<td>003</td>
<td>3.40</td>
<td>3.60</td>
<td>G1.7 = 3.15</td>
<td>very soft clay</td>
<td></td>
</tr>
<tr>
<td>DWFF-CPTU 05</td>
<td>144 10-01</td>
<td>431162.00 4077969.00 36°50.7072' 14°13.674'</td>
<td>207.69</td>
<td>4.10</td>
<td>0.57</td>
<td>16</td>
<td>004</td>
<td>3.30</td>
<td>3.50</td>
<td>G1.7 = 3.15</td>
<td>very soft clay</td>
<td></td>
</tr>
<tr>
<td>DWFF-CPTU 06</td>
<td>144 11-01</td>
<td>431065.00 4077837.00 36°50.6346' 14°13.6098'</td>
<td>214.15</td>
<td>4.10</td>
<td>0.48</td>
<td>24</td>
<td>005</td>
<td>3.50</td>
<td>3.70</td>
<td>G1.7 = 3.35</td>
<td>very soft clay</td>
<td></td>
</tr>
<tr>
<td>DWFF-CPTU 07</td>
<td>144 12-01</td>
<td>430844.00 4077429.00 36°50.414' 14°13.463'</td>
<td>290.00</td>
<td>4.00</td>
<td>4.10</td>
<td>006</td>
<td>n/a</td>
<td>n/a</td>
<td>4.00</td>
<td>4.10</td>
<td>G1.7 = 3.57</td>
<td>very soft clay</td>
</tr>
<tr>
<td>DWFF-CPTU 09</td>
<td>144 14-01</td>
<td>430182.00 4076848.00 36°46.340' 14°13.620'</td>
<td>277.00</td>
<td>4.00</td>
<td>4.10</td>
<td>006</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
<td>n/a</td>
<td>G1.7 = 3.61</td>
<td>very soft clay</td>
</tr>
<tr>
<td>DWFF-CPTU 13</td>
<td>144 20-01</td>
<td>428434.00 4075153.00 36°49.372' 14°11.854'</td>
<td>545.16</td>
<td>4.10</td>
<td>0.60</td>
<td>22</td>
<td>0012</td>
<td>3.80</td>
<td>4.00</td>
<td>G1.7 = 3.92</td>
<td>very soft clay</td>
<td></td>
</tr>
</tbody>
</table>

**Note:**
- a: g sensor result in improper data (no penetration depth available)
- b: g sensor is used for the analysis
- c: calculated penetration rate fits not with the winch velocity
- d: DWFF-CPTU test fails (no data are available)
Table 5.2: List of *DWFF-CPTU* deployments conducted in the vicinity of the southern *Twin* slide (southern Sicily, Italy).

<table>
<thead>
<tr>
<th>type of investigation</th>
<th>GeoB No.</th>
<th>UTM N, 33; WGS 1984 x-re.</th>
<th>LAF (north)</th>
<th>LONG (east)</th>
<th>water depth</th>
<th>DWFF-CPTU length</th>
<th>start velocity</th>
<th>duration of measurement</th>
<th>picture no.</th>
<th>visual penetration depth</th>
<th>penetration depth</th>
<th>geological assessment</th>
</tr>
</thead>
<tbody>
<tr>
<td>DWFF-CPTU 09</td>
<td>144 16-01</td>
<td>437075.00 4072549.00</td>
<td>36°47.800'</td>
<td>14°17.680'</td>
<td>154.90</td>
<td></td>
<td>4.10</td>
<td>0.61</td>
<td>17</td>
<td>008</td>
<td>2.40 - 2.60</td>
<td>G1.7 = 2.42</td>
</tr>
<tr>
<td></td>
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<td></td>
<td></td>
<td></td>
<td>G18 = 2.40</td>
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<td></td>
<td>G70 = n/a</td>
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<td>G1.7 = n/a</td>
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<td>G1.7 = n/a</td>
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<td></td>
<td></td>
<td></td>
<td>G70 = n/a</td>
</tr>
<tr>
<td>DWFF-CPTU 10</td>
<td>144 17-01</td>
<td>436935.00 4072231.00</td>
<td>36°47.628'</td>
<td>14°17.587'</td>
<td>167.65</td>
<td></td>
<td>4.10</td>
<td>0.66¹</td>
<td>16</td>
<td>009</td>
<td>2.10 - 2.30</td>
<td>very soft clay</td>
</tr>
<tr>
<td>DWFF-CPTU 11</td>
<td>144 18-01</td>
<td>436746.00 4071872.00</td>
<td>36°47.433'</td>
<td>14°17.462'</td>
<td>182.16</td>
<td></td>
<td>4.10</td>
<td>0.62</td>
<td>22</td>
<td>0010</td>
<td>2.40 - 2.60</td>
<td>very soft clay</td>
</tr>
<tr>
<td>DWFF-CPTU 12</td>
<td>144 19-01</td>
<td>436504.00 4071935.00</td>
<td>36°47.466'</td>
<td>14°17.299'</td>
<td>191.03</td>
<td></td>
<td>4.10</td>
<td>0.58</td>
<td>17</td>
<td>0011</td>
<td>3.00 - 3.30</td>
<td>very soft clay</td>
</tr>
</tbody>
</table>

*Note:* ¹ g sensor result in improper data (no penetration depth available)
² g sensor is used for the analysis
³ calculated penetration rate fits not with the winch velocity
⁴ *DWFF-CPTU* test fails (no data are available)
5.1. LANDSLIDES

This report presents one example of a *DWFF-CPTU* deployment (GeoB 14418-01) showing the measured dynamic-CPTU parameters (Fig. 5.6), dynamic Δu evolutions (Fig. 5.7) and excess pore pressure dissipation curves (Fig. 5.8). This test can be seen as good example for all other measurements in order to illustrate the characteristic of deep-water dynamic tests and how they have to be handled. The penetration depth is ~2.55 m and the depth was confirmed by the visual penetration depth (i.e. amount of clay sticking on the metal rod; see also Fig. 5.7).

The Δu$_{1,\text{dyn}}$ and Δu$_{3,\text{dyn}}$ vary between 15 and 50 kPa for the NTS and range between 18 and 55 kPa for the STS (e.g. Fig. 5.7). The Δu dissipation outside of the landslide complex shows values up to 30 kPa (e.g. Fig. 5.8). In some *DWFF-

![Figure 5.6](image.png)

**Figure 5.6:** (a) Measured deceleration, (b) derived dynamic penetration rate, (c) measured dynamic excess pore pressure at the tip (Δu$_{1,\text{dyn}}$) and (d) measured dynamic excess pore pressure 0.75 m behind the tip (Δu$_{3,\text{dyn}}$) of the *DWFF-CPTU* test GeoB 14418-01.
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Figure 5.7: Dynamic excess pore pressure of the DWFF-CPTU measurement GeoB 14418-01. The black line represents the measured $\Delta u_{1,\text{dyn}}$. The dashed/dotted line shows the offset corrected $\Delta u_{3,\text{dyn}}$. In addition, a picture of the visual penetration depth is presented showing approximately 2.4 to 2.6 m visual depth.

CPTU tests, negative $\Delta u$ values were observed. These negative values are probably characteristic for free gas growth in the soil, and/or the soils are affected by morphological steps detected in the bathymetry (Figs. 5.1, 5.1). The morphological steps may be described as slow deformation processes, such as creeping (see also section 2.2.1).

Physical properties The differences in the soil physical properties, between the NTS and STS, were analyzed by subdividing the core data collected: (i) in relatively undisturbed area (i.e. north to the landslide scars) and (ii) in mass transport deposit (MTD) area. The first area comprises the cores GeoB 14403-01, 02, 08 at the NTS and GeoB 14404-01, 14414-01 at the STS. The latter contains the cores GeoB 14401-02, 03, 05, 14405-01, 14406-01 at the NTS, but no cores exist
at the STS. Detailed core and physical properties are presented in the Appendix C.

Northern Twin slide  In the undisturbed area, the \( s_{u,fc} \) and \( s_{u,v-s} \) increase linearly from 2 to 10 kPa within the upper most 5 m. At 5 m, a significant drop of \( \sim 5 \) kPa is observed, followed by a linear rise of the \( s_u \) down to a depth of 54 m. The \( s_{u,fc} \) and \( s_{u,v-s} \) reaches maximum values up to 40 kPa at the final depth. Consequently, the soil is considered to be normally consolidated.

In the MTD area, all cores show a similar behavior regarding the MS, \( \rho \) and \( s_u \). At the eastern flank of the MTD (GeoB 14401-02, 03, 05), the \( s_{u,fc} \) and \( s_{u,v-s} \) increases linearly up to 5 kPa within the first 2.6 m. At 2.6 m the \( s_{u,fc} \) gradient increases suddenly to a value of \( \sim 50 \) kPa. MS follows this trend and increases from 20 to \( 40 \times 10^{-6} \) SI between 2.6 and 4.0 m depth. From \( \sim 2.6 \) to 3.8 m, the \( \rho \) increases from 1.5 to 1.7 g/cm\(^3\). The latter value does not differ significantly until to the final depth of 36 m, except two positive peaks at 15 and 28 m (up to 2.2 g/cm\(^3\)).

Similar soil properties compared to the eastern flank are detected in the central part and the foot of the MTD (GeoB 14405-01 and 14406-01), except the MS, which is 5 times higher (see Appendix C for details).

Southern Twin slide  Only cores in the undisturbed area were collected for the STS. The soil shows a similar MS, \( \rho \) and \( s_u \) characteristic as presented for the undisturbed area of the NTS. In the first 3 m, the \( s_{u,fc} \) and \( s_{u,v-s} \) are in the range of 4 to 6 kPa and a significant rise up to 40 kPa is detected at 3.5 m depth.

---

**Figure 5.8:** Excess pore pressure dissipation curves of the \textit{DWFF-CPTU} measurement GeoB 14418-01. The line with the circle represents the \( \Delta u_1 \) dissipation curve. The line with the box shows the \( \Delta u_3 \) dissipation curve.
From 0 to 3 m, the $\rho$ varies from 1.5 to 1.6 g/cm$^3$ and increases to 1.8 g/cm$^3$ at the final depth of the GeoB14404-01. The MS is nearly constant with a value of $100*10^{-6}$ SI. However, the MS and $s_{u,fc}$ show a distinct peak at 3.9 m depth. At the depth between 5 and 29 m, the $s_{u,fc}$ and $s_{u,v-s}$ increases linearly with depth characterizing a normally- to over-consolidation soil succession. MS and $\rho$ illustrate similar parameters as mentioned before.

**Comparison of *in situ* and laboratory tests**  
Four dynamic-CPTU tests, three for the NTS and one for the STS, are analyzed in conjunction with previously conducted biostratigraphical analyses on cores (Minisini et al. 2007), sedimentological and geotechnical laboratory experiments taken from gc collected nearly at the same location. These analyses shed light on the behavior and performance of the deep-water *in situ* instrument, correlations between *in situ* tests and biostratigraphical features, and are used to obtain a better understanding of the strain-rate effect for the $\Delta u_1$ and $\Delta u_3$ (see also section 3.2.3 for details).

For the NTS, the *in situ* undrained shear-strength derived from the $\Delta u_1$ and $\Delta u_3$ ($s_{u,\Delta u_1}$ and $s_{u,\Delta u_3}$) and taken from the measurement GeoB14403-09 are compared with the fc and v-s experiments performed on the core GeoB14403-01 (Fig. 5.9). This comparison shows the difference between the dynamic-CPTU parameters in contrast to the fc and v-s data. The three strain-rate correction solutions in combination with the associated SSCs related to the different pore pressure ports correct this difference and result in a SF between 1.7 and 2.0. The used SSCs are presented in Table 5.3, however, these values are only confirmed for very soft to soft clay (probably sensitive soils) and for maximum initial penetration rates less than 1.5 m/s. Good agreement between quasi-static CPTU parameters and laboratory data are found for all tested strain-rate correction solutions. Based on MS, *in situ* and laboratory $s_u$ records, two interesting layers were detected consisting of an alternative sequence of clayey to sandy soils (Fig. 5.10). Moreover, for the upper 5 m of soil, the *in situ* and laboratory $s_u$ varies between 4 and 12 kPa and this soil succession is characterized as normally- to slightly over-consolidated with an undrained shear-strength ratio ($s_u/\sigma_{V0}'$) of 0.2 to 0.5. Minisini et al. (2007) used several gc’s for biostratigraphical analyses and two of those (P8 and P7), combined with MS, $s_{u,\Delta u_1}$ and $s_{u,\Delta u_3}$ profiles collected at the same location (GeoB14411-01 and 14420-01), are shown in Figure 5.11. The location of the first comparison is above the NTS scar, while the second one is positioned in the MTDs of the NTS. It is worth noting that an good correlation between age dating on core P8 and $s_{u,\Delta u_1}$ of GeoB14411-01 are imminent at a depth of 3.2 m (i.e. change in the sedimentation rate leading to a higher consolidation state).

For the STS, the *in situ* measurement GeoB14418-01 is linked to the sedimentological and geotechnical laboratory records taken from GeoB 14404-01 (Figs. 5.12.
5.1. LANDSLIDES

Figure 5.9: Measured and corrected excess pore pressure at the tip ($\Delta u_1$) and 0.75 m behind the tip ($\Delta u_3$) of the DWFF-CPTU measurement GeoB 14403-09. Additionally, the intact undrained shear-strength profiles derived from the excess pore pressure ($s_{u,\Delta u_1}$ and $s_{u,\Delta u_3}$) are compared with the intact fall cone penetration strength ($s_{u,fc}$) and intact vane shear-strength ($s_{u,v-s}$) taken from core GeoB 14403-01. The green line represents the measured dynamic-CPTU parameters. The red, blue and orange lines show corrected quasi-static CPTU parameters based on state-of-the-art and newly developed strain-rate correction solutions (e.g. Dayal and Allen 1975, Mitchell and Soga 2005, A. Steiner unpublished data).
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Figure 5.10: Sedimentological and geotechnical profile for gravity core GeoB 14403-01 and DWFF-CPTU test GeoB 14403-09 collected above the northern Twin slide scar. The core description (core log), MAD bulk density ($\rho_{\text{bulk}}$), water content ($\omega$), magnetic susceptibility (MS) and a detailed comparison of the quasi-static CPTU ($s_u, \Delta u_1$ and $s_u, \Delta u_3$) and laboratory intact undrained shear-strength ($s_{u,fc}$ and $s_{u,v-s}$) are illustrated. The correction of the in situ data is based on a novel strain-rate correction solution developed by A. Steiner (unpublished data). The consolidation setting is given by the undrained shear-strength ratio ($s_u/\sigma'_V$).
Table 5.3: Soil-specific rate coefficients ($\mu_{CPTU}^*, \mu_C^{CPTU}, \mu_C^{CPTU}$) for the three strain-rate correction solutions and different pore pressure port locations.

<table>
<thead>
<tr>
<th>soil-specific rate coefficient</th>
<th>strain-rate correction solution</th>
<th>$\Delta u_1$</th>
<th>$\Delta u_3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>velocity ratio, $\mu_{CPTU}^*$</td>
<td>0.015-0.017</td>
<td>0.003-0.018</td>
<td></td>
</tr>
<tr>
<td>inverse sin-hyperbolic, $\mu_C^{CPTU}$</td>
<td>0.22-0.26</td>
<td>0.04-0.24</td>
<td></td>
</tr>
<tr>
<td>logarithmic, $\mu_C^{CPTU}$</td>
<td>0.5-0.6</td>
<td>0.08-0.55</td>
<td></td>
</tr>
</tbody>
</table>

and 5.13. The strain-rate correction and SSCs presented in Table 5.3 result in good accordance between in situ and laboratory datasets for all correction solutions. However, it seems as if the velocity ratio solution illustrates for both test sites the best match (Figs. 5.9 and 5.12). In addition, for the first 3m depth, the $s_u, \Delta u$, $s_{u,fc}$ and $s_{u,v-a}$ varies between 4 and 10kPa and the $s_u/\sigma_{V0}'$ is in the range of 0.3 to 0.7, which is characteristic for slightly over- to over-consolidated soils (Fig. 5.13).

Figure 5.11: Comparison of quasi-static CPTU measurements ($s_u, \Delta u_1$ and $s_u, \Delta u_3$) with biostratigraphical analyses on cored samples above the northern Twin slide scar and at the mass transport deposits (MTD). In addition, the magnetic susceptibility (MS) and undrained shear-strength ratio limits ($s_u/\sigma_{V0}'$) are shown. (a) The first comparison contains the GeoB 14411-01 in situ test and previous published data taken from core P8 (Minisini et al. 2007). (b) The second one consists the GeoB 14420-01 in situ test and previous presented results collected from core P7 (Minisini et al. 2007).
CHAPTER 5. ONGOING PROJECTS

Figure 5.12: Measured and corrected excess pore pressure at the tip ($\Delta u_1$) and 0.75 m behind the tip ($\Delta u_3$) of the DWFF-CPTU measurement GeoB 14418-01. Additionally, the undrained shear-strength derived from the excess pore pressure ($s_{u,\Delta u_1}$ and $s_{u,\Delta u_3}$) is compared with the intact fall cone penetration strength ($s_{u,fc}$) and intact vane shear-strength ($s_{u,v}$) taken from GeoB 14404-01 core. The green line represents the measured dynamic-CPTU parameters. The red, blue and orange lines show corrected quasi-static CPTU parameters based on state-of-the-art and newly developed strain-rate correction solutions (e.g. Dayal and Allen 1975, Mitchell and Soga 2005, A. Steiner unpublished data).

2D sub-seafloor modeling The collected $s_{u,\Delta u_1}$ and $s_{u,\Delta u_3}$ profiles are used to develop a preliminary 2D strength transect of the surficial soils across the upper, undisturbed area of the NTS (Fig. 5.14a). The combination of this transect with a sub-bottom profiler section (HE 112) published earlier (Minisini et al. 2007) allows to develop an ordinary 2D sub-seafloor model, which is adequate for slope stability and risk assessments (Fig. 5.14b). In this model, two surficial, distinct layers are mapped consisting of stiff, fine-grained or coarse-grained soils. Further sedimentological and geotechnical experiments on cored/drilled soils as well as more comparisons with in situ data are required in order to characterize these layers in detail. This will be performed in the future.
5.1. LANDSLIDES

Figure 5.13: Sedimentological and geotechnical profile for gravity core GeoB 14404-01 and DWFF-CPTU measurement GeoB 14418-01 collected above the southern Twin slide scar. The core description (core log), MAD bulk density ($\rho_{\text{bulk}}$), water content ($\omega$), magnetic susceptibility (MS) and a detailed comparison of the quasi-static CPTU ($s_{u,\Delta u_1}$ and $s_{u,\Delta u_3}$) and intact laboratory undrained shear-strength ($s_{u,fc}$ and $s_{u,v-s}$) are illustrated. The correction of the in situ data is based on a novel strain-rate correction solution developed by A. Steiner (unpublished data). The consolidation setting is given by the undrained shear-strength ratio ($s_u/\sigma'_{V_0}$).
Figure 5.14: (a) Analyses of the \textit{in situ} and laboratory undrained shear-strength profiles ($s_{u1}$, $s_{u3}$ and $s_{\text{lab}}$). (b) The 2D sub-seafloor model includes the intact undrained shear-strength profiles, sub-bottom profiler transect (HE 112; Minisini et al. 2007), preliminary stratigraphical interpretations and identification of morphological features.
5.1. LANDSLIDES

Conclusions and outlook

Previous studies emphasize that the stability of submarine slopes are highly affected by the excess pore pressure evolution, intact and remolded drained/undrained shear-strength properties (e.g. Locat and Lee 2000, Sultan et al. 2004, Vanneste et al. 2013). A time- and cost-effective in situ dynamic-CPTU instrument, ship-board and post-cruise laboratory records obtained from cored/drilled samples were used to collect these properties in the landslide-prone area of Gela basin (southern Sicily, Italy). The following conclusions can be drawn:

1. The DWFF-CPTU instrument is well suited to explore the surficial soils down to a penetration depth of less than 7 m, and is approved for WDs up to 4000 m (Stegmann and Kopf 2007).

2. The state-of-the-art (e.g. Dayal and Allen 1975, Mitchell and Soga 2005) and the newly developed strain-rate correction solutions are appropriate to consider the strain-rate effect. The $s_u,\Delta u_1$ and $s_u,\Delta u_3$ profiles corrected with the latter solution shows best match with the $s_{ufc}$, $s_{uv-s}$ profiles and prior published bio-stratigraphical analyses (Minisini et al. 2007).

3. The comparison of the undrained shear-strength ratio for the northern and southern Twin slides (NTS and STS) illustrates different order of magnitude varying between 0.2 and 0.5 for the NTS and between 0.3 and 0.7 for the STS.

4. The morphological steps and gullies are probably an indication for slow deformation processes and seepage forces, and/or evidence for gas growth in the surficial soils (Max et al. 1993, Minisini et al. 2007, 2009).

5. Two embedded layers, consisting of stiff fine-grained or coarse-grained soils, are identified and characterized by the in situ dynamic-CPTU tests and are mapped in the sub-bottom profiler transect (Minisini et al. 2007).

The above presented findings can be supplemented and improved by laboratory experiments, for instance torshear tests with high rotating speed up to 30cm/min in order to verify the newly developed strain-rate correction solution. In addition, detailed comparisons with high-resolution parasound transects and advanced laboratory experiments on the embedded layers (e.g. direct shear, torshear, triaxial compression and extension tests) are recommended to obtain a better view of the sedimentological and geotechnical soil characteristics.
CHAPTER 5. ONGOING PROJECTS

5.2 Mud volcanoes

5.2.1 Mediterranean Ridge accretionary complex

Abstract

In spring 2011, the R/V Poseidon cruise P 410 "MUDFLOW - Mud volcanism, faulting and fluid flow on the Mediterranean Ridge accretionary complex" and in spring 2012, the R/V Poseidon cruise P 429 "MEDFLUIDS - Slope stability, mud volcanism, faulting and fluid flow in the eastern Mediterranean Sea (Cretan Sea, Mediterranean Ridge) and Ligurian margin (Nice slope)" were conducted. Both research expeditions aimed to explore: (i) mud volcanoes (MV) in the Olimpi field, located at the outer wedge of the Mediterranean Ridge accretionary complex (MedRidge), (ii) the inner deformation front and ridge of the MedRidge, and (iii) the deformable backstop in the vicinity of the Cretan margin (southern Crete, Greece) using multibeam echosounder, sub-bottom profiler, gravity corer, temperature lance, chemical and aqueous transport (CAT) meter and the MARUM in situ dynamic-CPTU instrument. A total of 12 deployments with the DWFF-CPTU were conducted, and main findings can be summarized as follows:

- The DWFF-CPTU instrument is used to characterize the superficial soils (less than 3 m) in and around MVs within deep-water environments up to a WD of 4000 m.

- The velocity ratio and modified inverse sin-hyperbolic solutions are best suited to obtain quasi-static excess pore pressures from dynamic excess pore pressures. The associated soil-specific coefficients vary of 0.003 to 0.017 for the velocity ratio solution, and of 0.04 to 0.65 for the modified inverse sin-hyperbolic solution.

- The undrained shear-strength ratio \( \frac{s_u}{\sigma_V^I} \) is used to assess the activity of the MVs. Active MVs have \( s_u/\sigma_V^I \) values less than 0.5, while the \( s_u/\sigma_V^I \) values for inactive MVs are around 1.5.

Key words: Deep-water dynamic penetrometer, excess pore pressure, mud volcanoes, Mediterranean Ridge accretionary complex.

Introduction

The MedRidge is one of the fastest growing accretionary prisms on earth (e.g. Kas tens 1991, Kopf et al. 2003). In such collisional tectonic settings, mud volcanoes
5.2. **MUD VOLCANOES**

(MVs), diatremes and mud diapirs are well-known geological phenomena occurring in a wide variety of sizes, shapes and configurations (e.g. Yassir 1989, Kopf 1999).

In this geotechnical study, deep-water dynamic-CPTU (DWFF-CPTU) tests were carried out, shipboard and post-cruise laboratory experiments (e.g. core logs, MSCL measurements, v-s and fc tests) on working halves of recovered cores were analyzed in order to investigate the correlation between DWFF-CPTU and laboratory datasets. The surficial soil properties in and around of different MVs are illustrated showing core logs, core photos, MSCL gamma density ($\rho$), magnetic susceptibility (MS), P-wave velocity (PWV) and intact undrained shear-strength measured in situ ($s_u,\Delta u_1$ - pressure port at the tip and $s_u,\Delta u_3$ - pressure port behind the sleeve friction), with a vane shear device ($s_u,\Delta v_1$) and with a fall cone penetration apparatus ($s_u,fc$). These datasets, and in particular, the undrained shear-strength ratio ($s_u/\sigma_V^{\prime}$) are used to investigate the activity of different MVs.

**Regional setting**

The MedRidge, located in the south of Crete (Greece), is part of the Hellenic subduction zone, where the African plate subducts beneath the Eurasian plate. This complex is more than 300 km wide in NS direction and strikes $\sim$2000 km in WE direction (e.g. Kastens 1991, Kopf et al. 2003). Such a subduction zone is subject to several tectonical processes, for instance, extensional deformations in the inner part of the forearc, compressional deformations and conjugated faulting in the outer wedge of the MedRidge (e.g. Le Pichon et al. 1982). In the Olimpi field, seven MVs were mapped in an area of $\sim$100 km$^2$. The diameters of the MVs vary between 1.5 and 4.0 km and the heights are less than 200 m, resulting in a slope angle up to 16$^\circ$ (Camerlenghi et al. 1992). Figure 5.15 illustrates five MVs and the southwestern edge of the inner deformation front including the locations of the gravity cores (gc) and DWFF-CPTU tests.

**Materials and methods**

**In situ CPTU measurement** The DWFF-CPTU instrument was utilized during both scientific expeditions and a detailed description of this device is given in section 3.1.2. The instrument setup, deployment mode and strain-rate correction was similar to that shown in section 5.1.1 (see Materials and methods). However, during P429, the location of the $\Delta u_3$ port was 0.35 m behind the tip in contrast to the 0.75 m used during MSM 15/3 and P410.

**Physical properties** Shipboard characterizations (e.g. visual core description), fc and v-s experiments were performed in order to obtain sedimentological
Figure 5.15: Morphological map shows the locations of the *DWFF-CPTU* deployments and gravity cores collected during R/V *Poseidon* cruises P 410 and P 429. The CPTU deployments and cores are represented with the last two digits of the GeoB nomenclature. The across-track resolution of the map is better than 50 m.

and geotechnical properties of the soil. Post-cruise MSCL logging and imaging (core photos) were also conducted at *MARUM* (Bremen, Germany). Detailed descriptions of the used apparatuses and devices are given in section 5.1.1 (see *Materials and methods*).
Geotechnical Subchapter of the Cruise Report

*MUDFLOW* R/V *Poseidon* P 410 (Heraklion/Greece - Taranto/Italy)

*Alois Steiner\(^a\), Gauvain Wiemer\(^a\), Achim J. Kopf\(^a\)

\(^a\) Marine Geotechnics, MARUM - Center for Marine Environmental Sciences and Faculty of Geosciences, University of Bremen, Bremen, Germany


**Preliminary results**

In total, 10 DWFF-CPTU measurements were performed in 2011 (see Table 5.4 and Fig. 5.15 for location). The WD varies between 1900 and 1950 mbsl (meter below sea-level) and penetration depth ranges between 1.5 and 3.0 m. Due to performance and electronic problems the first six tests failed (GeoB 15304-01-02, 15315-01, 15322-01-03); however, the last four dynamic-CPTU measurements illustrate good results of the surficial soil properties (GeoB 15364-01, 15365-01, 15371-01, 15372-01). These measurements are located at the MVs *Leipzig, Bergamo, Napoli* and *Maidstone*. The gc deployments (GeoB15326, 15363, 15332, 15370 and 15314) are quite close to the *in situ* measurements (see Fig. 5.15 for location).
### Table 5.4: List of DWFF-CPTU deployments performed during R/V Poseidon cruise P.410 in the vicinity of the Olimpi field (southern Crete, Greece).

<table>
<thead>
<tr>
<th>GeoB</th>
<th>LAT (north) [°]</th>
<th>LONG (east) [°]</th>
<th>date dd.mm.yy</th>
<th>WD [m]</th>
<th>start velocity [m/s]</th>
<th>duration of measurement [min]</th>
<th>penetration depth [m]</th>
<th>probe setup, comments</th>
<th>MV</th>
</tr>
</thead>
<tbody>
<tr>
<td>153xx-yy</td>
<td>04-01 33°51.83' 24°34.25'</td>
<td>14.03.11</td>
<td>1925</td>
<td>1.2</td>
<td>20</td>
<td>-</td>
<td>test failed</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td>04-02 33°51.74' 24°34.32'</td>
<td>14.03.11</td>
<td>1919</td>
<td>1.2</td>
<td>21</td>
<td>-</td>
<td>test failed</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td>15-01 33°43.50' 24°41.01'</td>
<td>17.03.11</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>defect of the instrument</td>
<td>Napoli</td>
<td></td>
</tr>
<tr>
<td></td>
<td>22-01 33°43.62' 24°40.84'</td>
<td>18.03.11</td>
<td>1910</td>
<td>1.0</td>
<td>15</td>
<td>-</td>
<td>test failed</td>
<td>Napoli</td>
<td></td>
</tr>
<tr>
<td></td>
<td>22-02 33°43.62' 24°40.84'</td>
<td>18.03.11</td>
<td>1916</td>
<td>1.2</td>
<td>10</td>
<td>-</td>
<td>test failed</td>
<td>Napoli</td>
<td></td>
</tr>
<tr>
<td></td>
<td>22-03 33°43.53' 24°40.81'</td>
<td>18.03.11</td>
<td>1911</td>
<td>1.2</td>
<td>7</td>
<td>-</td>
<td>test failed</td>
<td>Napoli</td>
<td></td>
</tr>
<tr>
<td></td>
<td>64-01 33°44.12' 24°44.87'</td>
<td>27.03.11</td>
<td>1945</td>
<td>1.2</td>
<td>32</td>
<td>2.12</td>
<td>DWFF-CPTU, length 4.1 m, Δu₁ and Δu₃</td>
<td>Bergamo</td>
<td></td>
</tr>
<tr>
<td></td>
<td>65-01 33°47.02' 24°39.14'</td>
<td>27.03.11</td>
<td>1886</td>
<td>1.2</td>
<td>37</td>
<td>1.96</td>
<td>DWFF-CPTU, length 4.1 m, Δu₁ and Δu₃</td>
<td>Leipzig</td>
<td></td>
</tr>
<tr>
<td></td>
<td>71-01 33°37.23' 24°40.33'</td>
<td>28.03.11</td>
<td>1933</td>
<td>1.2</td>
<td>40</td>
<td>1.86</td>
<td>DWFF-CPTU, length 4.1 m, Δu₁ and Δu₃</td>
<td>Maidstone</td>
<td></td>
</tr>
<tr>
<td></td>
<td>72-01 33°43.52' 24°41.01'</td>
<td>28.03.11</td>
<td>1904</td>
<td>1.2</td>
<td>37</td>
<td>2.58</td>
<td>DWFF-CPTU, length 4.1 m, Δu₁ and Δu₃</td>
<td>Napoli</td>
<td></td>
</tr>
</tbody>
</table>

**Note:** LAT and LONG are the point locations in the geographical coordinate system. MV is the mud volcano. WD is the water depth.
5.2. MUD VOLCANOES

Table 5.5: Soil-specific rate coefficients ($\mu^*_{\text{CPTU}}$, $\mu'_{\text{CPTU}}$, $\mu_{\text{CPTU}}$) for the three strain-rate correction solutions and different pore pressure port locations.

<table>
<thead>
<tr>
<th>strain-rate correction solution</th>
<th>soil-specific rate coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>velocity ratio, $\mu^*_{\text{CPTU}}$</td>
<td>$\Delta u_1$</td>
</tr>
<tr>
<td>inverse sin-hyperbolic, $\mu'_{\text{CPTU}}$</td>
<td>$\Delta u_1$</td>
</tr>
<tr>
<td>logarithmic, $\mu_{\text{CPTU}}$</td>
<td>$\Delta u_3$</td>
</tr>
<tr>
<td></td>
<td>0.014-0.017</td>
</tr>
<tr>
<td></td>
<td>0.003-0.005</td>
</tr>
<tr>
<td></td>
<td>0.50-0.65</td>
</tr>
<tr>
<td></td>
<td>0.03-0.04</td>
</tr>
<tr>
<td></td>
<td>1.2-1.5</td>
</tr>
<tr>
<td></td>
<td>0.08-0.12</td>
</tr>
</tbody>
</table>

All excess pore pressure profiles ($\Delta u_1$ and $\Delta u_3$) were corrected using three different strain-rate correction solutions and associated soil-specific rate coefficients (SSCs) related to the different pore pressure ports (see also section 3.2.3 and 5.1.1). Figure 5.16 shows the dynamic-CPTU test GeoB 15372 and geotechnical data of GeoB 15314 as an example, and the other tests are presented in the Appendix C. In addition, the $s_u, \Delta u_1$ and $s_u, \Delta u_3$ are derived from the $\Delta u_1$ and $\Delta u_3$, and these datasets are compared to the $s_{ufc}$ and $s_{u,v-s}$ profiles in order to verify and validate the strain-rate correction solutions and to confirm the used SSCs (Table 5.5). All correction solutions and SSCs follow the general trend, which can be seen in the $fc$ and $v-s$ results; however, the velocity ratio solution illustrates the best match between in situ and laboratory data (Fig. 5.16 and Appendix C). The strain-rate correction factors ($SF$s) are between 1.5 and 3.0.

From a sedimentological and geotechnical perspective, the core photos, visual core description (core log), MSCL gamma density, magnetic susceptibility, p-wave velocity, in situ and laboratory $s_u$ are correlated to investigate the consolidation state of the surficial soils ($s_u/\sigma'_{V_0}$) and the activity of different MVs. The in situ and laboratory $s_u$ profiles for the MVs Bergamo (Fig. 5.17 and Fig. 5.18), Leipzig (Fig. 5.19) and Maidstone (Fig. 5.20) vary between 5 and 26 kPa with a linearly increasing behavior according to the effective overburden stresses. The soils are characterized as slightly over- to highly over-consolidated with a $s_u/\sigma'_{V_0}$ in the range of 0.5 to 1.5 (e.g. Figs. 5.17 and 5.20). Consequently, these results indicate that the above mentioned MVs may be inactive. In contrast, the measurements for the MV Napoli show in situ and laboratory $s_u$ profiles between 2 and 6 kPa (Fig. 5.21). The soils are normally- to slightly over-consolidated with a $s_u/\sigma'_{V_0}$ of 0.2 to 0.5. Consequently, this MV is probably an active one. In all cores (see core photos and core logs of GeoB 15326, 15363, 15332, 15370 and 15314), shells/shell fragments, fluid/gas structures, coarse-grained soils and angular to well rounded clasts are scattered along the first 3.5 m. These features are identified by significant drops in the $\Delta u$ (see Appendix C) and an increase of the $s_{ufc}$ and $s_{u,v-s}$ profiles (e.g. Fig. 5.19).
CHAPTER 5. ONGOING PROJECTS

Figure 5.16: Measured and corrected excess pore pressure at the tip ($\Delta u_1$) and 0.75 m behind the tip ($\Delta u_3$) for the mud volcano (MV) Napoli. Additionally, the derived intact undrained shear-strength of the $DWFF-CPTU$ measurement ($s_u,\Delta u_1$ and $s_u,\Delta u_3$) (GeoB 15372-01) compared with the fall cone penetration and vane shear tests ($s_{u,fc}$ and $s_{u,v-s}$) performed aboard (GeoB 15314) are illustrated. The green line represents the measured dynamic-CPTU parameters. The red, blue and orange lines show quasi-static CPTU parameters taking into account the three empirical strain-rate correction solutions (e.g. Dayal and Allen 1975, Mitchell and Soga 2005, A. Steiner [unpublished data]).
Figure 5.17: Sedimentological and geotechnical profile of gravity core GeoB 15326 and DWFF-CPTU measurement GeoB 15364-01 collected at the MV Bergamo. The core photo and description (core log), MSCL gamma density ($\rho$), magnetic susceptibility (MS), p-wave velocity (PWV) and a detailed comparison of the quasi-static CPTU ($s_u, \Delta u_1$ and $s_u, \Delta u_3$) and laboratory intact undrained shear-strength properties ($s_{u,f,c}$ and $s_{u,v-s}$) are illustrated. The correction of the in situ data is based on a novel strain-rate correction solution developed by A. Steiner (unpublished data). The consolidation setting is given by the undrained shear-strength ratio ($s_u/\sigma'_{V0}$).
Figure 5.18: Sedimentological and geotechnical profile of gravity core GeoB 15363 and DWFF-CPTU measurement GeoB 15364-01 collected at the MV Bergamo. The core photo and description (core log), MSCL gamma density ($\rho$), magnetic susceptibility (MS), p-wave velocity (PWV) and a detailed comparison of the quasi-static CPTU ($s_{u,\Delta u1}$ and $s_{u,\Delta u3}$) and laboratory intact undrained shear-strength properties ($s_{u,fc}$ and $s_{u,v-s}$) are illustrated. The correction of the in situ data is based on a novel strain-rate correction solution developed by A. Steiner (unpublished data). The consolidation setting is given by the undrained shear-strength ratio ($s_{u}/\sigma'_{V0}$).
Figure 5.19: Sedimentological and geotechnical profile of gravity core GeoB 15332 and DWFF-CPTU measurement GeoB 15365-01 collected at the MV Leipzig. The core photo and description (core log), MSCL gamma density ($\rho$), magnetic susceptibility (MS), p-wave velocity (PWV) and a detailed comparison of the quasi-static CPTU ($s_u, \Delta u_1$ and $s_u, \Delta u_3$) and laboratory intact undrained shear-strength properties ($s_u, f_{uc}$) are illustrated. The correction of the in situ data is based on a novel strain-rate correction solution developed by A. Steiner (unpublished data). The consolidation setting is given by the undrained shear-strength ratio ($s_u/\sigma^V_0$).
Figure 5.20: Sedimentological and geotechnical profile of gravity core GeoB 15370 and DWFF-CPTU measurement GeoB15371-01 collected at the MV Maidstone. The core photo and description (core log), MSCL gamma density ($\rho$), magnetic susceptibility (MS), p-wave velocity (PWV) and a detailed comparison of the quasi-static CPTU ($s_u, \Delta u_1$ and $s_u, \Delta u_3$) and laboratory intact undrained shear-strength properties ($s_{u,fr}$) are illustrated. The correction of the in situ data is based on a novel strain-rate correction solution developed by A. Steiner (unpublished data). The consolidation setting is given by the undrained shear-strength ratio ($s_u/\sigma'_{V0}$).
Figure 5.21: Sedimentological and geotechnical profile of gravity core GeoB 15314 and DWFF-CPTU measurement GeoB15372-01 collected at the MV Napoli. The core photo and description (core log), MSCL gamma density ($\rho$), magnetic susceptibility (MS), p-wave velocity (PWV) and a detailed comparison of the quasi-static CPTU ($s_u, \Delta u_1$ and $s_u, \Delta u_3$) and laboratory intact undrained shear-strength properties ($s_{u, fc}$ and $s_{u, v-s}$) are illustrated. The correction of the in situ data is based on a novel strain-rate correction solution developed by A. Steiner (unpublished data). The consolidation setting is given by the undrained shear-strength ratio ($s_u/\sigma_V^0$).
CHAPTER 5. ONGOING PROJECTS

Geotechnical Subchapter of the Cruise Report

*MEDFLUIDS* R/V *Poseidon* P 429 (Heraklion/Greece - La Seyne sur Mer/France)

*Alois Steiner*, Gauvain Wiemer*, Achim J. Kopf*

*a* Marine Geotechnics, *MARUM - Center for Marine Environmental Sciences* and Faculty of Geosciences, University of Bremen, Bremen, Germany


**Preliminary results**

Two *DWFF-CPTU* tests were conducted in 2012 (see Table 5.6 and Fig. 5.15 for location). The first one was undertaken at the MV *Milano* (GeoB 16522) and the second one was carried out at MV *Napoli* (GeoB 16524). Two adjacent gravity cores (GeoB 15312 and 15362) were collected in 2011. The WD varies between 1900 and 1950 mbsl (meter below sea-level) and the penetration depth is up to 2.5 m.
Table 5.6: List of *DWFF-CPTU* deployments performed during R/V *Poseidon* cruise P429 in the vicinity of the *Olimpi* field (southern Crete, Greece).

<table>
<thead>
<tr>
<th>GeoB</th>
<th>LAT (north)</th>
<th>LONG (east)</th>
<th>date</th>
<th>WD</th>
<th>start velocity</th>
<th>duration of measurement</th>
<th>penetration depth</th>
<th>probe setup, comments</th>
<th>MV</th>
</tr>
</thead>
<tbody>
<tr>
<td>165xx-yy</td>
<td>[°]</td>
<td>[°]</td>
<td>dd.mm.yy</td>
<td>[m]</td>
<td>[m/s]</td>
<td>[min]</td>
<td>[m]</td>
<td>[-]</td>
<td>[-]</td>
</tr>
<tr>
<td>22-01</td>
<td>33°44.02'</td>
<td>24°46.57'</td>
<td>29.03.12</td>
<td>1923</td>
<td>1.5</td>
<td>5</td>
<td>2.35</td>
<td><em>DWFF-CPTU</em>, length 4.1 m, $\Delta u_1$ and $\Delta u_3$</td>
<td><em>Milano</em></td>
</tr>
<tr>
<td>24-01</td>
<td>33°43.64'</td>
<td>24°40.86'</td>
<td>29.03.12</td>
<td>1909</td>
<td>1.5</td>
<td>7</td>
<td>2.10</td>
<td><em>DWFF-CPTU</em>, length 4.1 m, $\Delta u_1$ and $\Delta u_3$</td>
<td><em>Napoli</em></td>
</tr>
</tbody>
</table>

Note: LAT and LONG are the point locations in the geographical coordinate system. MV is the mud volcano. WD is the water depth.
**Table 5.7**: Soil-specific rate coefficients ($\mu_{CPTU}^*, \mu_{CPTU}', \mu_{CPTU}$) for the three strain-rate correction solutions and different pore pressure ports.

<table>
<thead>
<tr>
<th>strain-rate correction solution</th>
<th>soil-specific rate coefficient</th>
<th>$\Delta u_1$</th>
<th>$\Delta u_3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>velocity ratio, $\mu_{CPTU}^*$</td>
<td>0.013-0.014</td>
<td>0.003-0.004</td>
<td></td>
</tr>
<tr>
<td>inverse sin-hyperbolic, $\mu_{CPTU}'$</td>
<td>0.5-0.65</td>
<td>0.03-0.05</td>
<td></td>
</tr>
<tr>
<td>logarithmic, $\mu_{CPTU}$</td>
<td>1.2-1.5</td>
<td>0.08-0.12</td>
<td></td>
</tr>
</tbody>
</table>

All $\Delta u_1$ and $\Delta u_3$ profiles were corrected using the three different strain-rate correction solutions and associated SSCs related to the different pore pressure ports (see also sections 3.2.3 and 5.1.1). The results are summarized in the Appendix C. The $s_u, \Delta u_1$ and $s_u, \Delta u_3$ profiles are compared with the $s_{u,fc}$ and $s_{u,v-s}$ profiles in order to confirm the strain-rate correction solutions and associated SSCs (Table 5.7). The velocity ratio solution indicates the best match between *in situ* and laboratory data (Appendix C). The $SF$s are similar to those mentioned above.

From a sedimentological and geotechnical point of view, the *in situ* and laboratory $s_u$ profiles for MV *Milano* varies between 5 and 10 kPa (Fig. 5.22). The soil succession is characterized as normally- to slightly-over-consolidated with a $s_u/\sigma'_{V0}$ in the range of 0.2 to 0.7. Consequently, this MV is probably active, because a potential high amount of mud emission causes the formation of soft soils (e.g. Kopf and Deyhle 2002). The measurements for the MV *Napoli* show *in situ* and laboratory $s_u$ profiles between 5 and 15 kPa (Fig. 5.23). Characteristic for these soils is the slightly- to highly over-consolidated behavior with a $s_u/\sigma'_{V0}$ of 0.5 to 1.0. Therefore, this deployment may be outside of the active region (i.e. test at the shoulder area of the MV). In all tests, fluid/gas structures, coarser sediments and angular to well rounded clasts are scattered along the sediment succession.
Figure 5.22: Sedimentological and geotechnical profile of gravity core GeoB 15362 (P 410) and DWFF-CPTU measurement GeoB16522-01 (P 429) collected at the MV Milano. The core photo and description (core log), MSCL gamma density ($\rho$), magnetic susceptibility (MS), p-wave velocity (PWV) and a detailed comparison of the quasi-static CPTU ($s_{u,\Delta u1}$ and $s_{u,\Delta u3}$) and laboratory intact undrained shear-strength properties ($s_{u,fc}$ and $s_{u,v-s}$) are illustrated. The correction of the in situ data is based on a novel strain-rate correction solution developed by A. Steiner (unpublished data). The consolidation setting is given by the undrained shear-strength ratio ($s_u/\sigma'_V$).
Figure 5.23: Sedimentological and geotechnical profile of gravity core GeoB 15312 (P 410) and DWFF-CPTU measurement GeoB16524-01 (P 429) collected at the MV Napoli. The core photo and description (core log), MSCL gamma density ($\rho$), magnetic susceptibility (MS), p-wave velocity (PWV) and a detailed comparison of the quasi-static CPTU ($s_u, \Delta u_1$ and $s_u, \Delta u_3$) and laboratory intact undrained shear-strength properties ($s_u, f_c$ and $s_u, v_s$) are illustrated. The correction of the in situ data is based on a novel strain-rate correction solution developed by A. Steiner (unpublished data). The consolidation setting is given by the undrained shear-strength ratio ($s_u/\sigma_V^{'0}$).
5.2. MUD VOLCANOES

5.2.2 Kumano basin

Abstract

In summer 2012, the R/V Sonne cruise SO 222 "MEMO - MeBo drilling and in situ Monitoring offshore Japan" was carried out. The main objectives are to obtain a better understanding of: (i) deep-seated tectonic and fluid processes, and (ii) the connection between seismicity and mud volcanism in the Kumano basin, which is located along the Nankai Trough accretionary complex (southwestern Japan). This research expedition consisted of two legs (SO 222a and SO 222b). Leg SO 222a focused on coring, drilling and in situ testing using gravity corer, heat flow, CAT meter, the MARUM MeBo drilling rig and MARUM dynamic-CPTU instrument. Leg SO 222b aimed on monitoring and long-term in situ testing using the MARUM QUEST remotely operated vehicle (ROV) and newly developed long-term borehole observatories. The surficial soils of three mud volcanoes (MVs) were investigated using the DWFF-CPTU instrument, and the following findings can be drawn:

- The DWFF-CPTU instrument is used to characterize the surficial soils (less than 2.5 m) in and around MVs within deep-water environments up to a WD of 4000 m.
- The modified inverse sinhyperbolic solution is best suited for the strain-rate correction of the excess pore pressures when using soil-specific coefficients of 0.04 to 0.065 for $\Delta u_2$ and $\Delta u_3$.
- The active MVs have undrained shear-strength ratios ($s_u/\sigma'_{V0}$) less than 0.6; however, MV #4 show a sharp step at 0.6 m depth and $s_u/\sigma'_{V0}$ values up to 1.3 below this step.

Key words: Deep-water dynamic penetrometer, excess pore pressure, mud volcanoes, Nankai Trough accretionary complex.
Geotechnical Subchapter of the Cruise Report \textit{MEMO}
\textit{R/V Sonne} SO 222
(Hongkong/China - Pusan/South Korea)

\textit{Alois Steiner}$^a$, Matt Ikari$^a$, Achim J. Kopf$^a$, Matthias Lange$^a$, Timo Fleischmann$^a$

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\section*{Introduction}

The \textit{Nankai} Trough accretionary complex originates from the subduction processes of the Philippine Sea Plate beneath the Eurasia Plate (southwestern Japan). In the \textit{Kumano} basin, thrust activities in the \textit{Nankai} plate-boundary interface caused the origin of offshore mud volcanoes (MV's), which carry gaseous, fluid and solid mixtures from several kilometers depth to the seafloor (e.g. Kuramoto et al. 1998, 2001, Morita et al. 2004). Hence, the exploration of such MV's is indispensable to obtain a better understanding of the geological and mechanical processes occurring along the plate-boundary interface.

This geotechnical study presents \textit{in situ} dynamic-CPTU records, core logs, core images and fall cone penetration (fc) experiments in order to characterize the superficial soils in and around three different MV's. Comparisons of the intact undrained shear-strength measured \textit{in situ} ($s_{u,\Delta u2}$ - pressure port behind the tip and $s_{u,\Delta u3}$ - pressure port behind the sleeve friction) and with a fc device ($s_{u,fc}$) demonstrate the good correlation between \textit{in situ} and laboratory techniques. In addition, these $s_u$ profiles and in particular, the undrained shear-strength ratio ($s_u/\sigma_{V0}'$) are utilized to characterize the soils and to investigate the activity of these three MV's.

\section*{Regional setting}

The \textit{Kumano} basin is located on the eastern \textit{Nankai} Trough accretionary complex off the southwestern region of Japan. It is the largest forearc basin of the \textit{Nankai}
subduction zone extending over 7000 km$^2$ with an almost flat seafloor at around 2000 m WD (Fig. 1.5, Morita et al. 2004). Previous morphological and sub-seafloor studies illustrate a large sedimentary basin including three anticlines in the northern zone and a substantial uplift of the inner wedge at the southern edge of the Kumano basin (outer arc high) (e.g. Park et al. 2002, Moore et al. 2009). Several MVs and mud diapirs were identified with a maximum diameter of 2 km and up to 160 m height, and all are connected to the anticlines (e.g. Fig. 5.24, Kuramoto et al. 2001, Sawada et al. 2002).

Materials and methods

**In situ CPTU measurements** The DWFF-CPTU instrument was utilized and a detailed summary of this device is given in section 3.1.2. The instrument setup, deployment mode and strain-rate correction were similar to that shown in section 5.1.1 (see Materials and methods); however, the location of the $\Delta u_3$ port was 0.35 m behind the tip (see also P429 cruise). All dynamic-CPTU tests were corrected for the strain-rate effect using the modified inverse sinh-hyperbolic correction solution presented in section 3.2.3. The SSCs vary between 0.04 and 0.05 for
\[ \Delta u_2, \text{ and between 0.04 and 0.065 for } \Delta u_3. \]

**Physical properties**  Shipboard characterizations (e.g. core photos and logs) and fc experiments were performed. Detailed specifications of the apparatuses are given in section 5.1.1 (see Materials and methods).

**Preliminary results**

A total of 10 *DWFF-CPTU* measurements were carried out at three different MVs, known as KK#2, 3 and 4 (*Kumano Knoll*), (see Fig. 5.24 for location). The measurements are located in and around of these MVs. The WD varies between 1950 and 2050 mbls (meter below sea-level). The penetration rate ranges between 0.9 and 1.5 m/s resulting in penetration depths of 1.1 to 2.9 m (Table 5.8).
### Table 5.8: Protocol of **DWFF-CPTU** deployments performed during R/V *Sonne* cruise SO 222 in the *Kumano* basin (southern Japan).

<table>
<thead>
<tr>
<th>GeoB 167-xx-yy</th>
<th>LAT (north) [°]</th>
<th>LONG (east) [°]</th>
<th>date dd.mm.yy</th>
<th>WD start velocity [m/s]</th>
<th>penetration depth [m]</th>
<th>probe setup, comments</th>
<th>MV No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>10-01</td>
<td>33°37.97'</td>
<td>136°40.20'</td>
<td>17.06.12</td>
<td>1.27</td>
<td>1.1</td>
<td><strong>DWFF-CPTU</strong>&lt;sup&gt;, length 4.1 m, ∆u&lt;sub&gt;2&lt;/sub&gt; and ∆u&lt;sub&gt;3&lt;/sub&gt;&lt;/sup&gt;</td>
<td>3</td>
</tr>
<tr>
<td>10-02</td>
<td>33°37.95'</td>
<td>136°40.15'</td>
<td>17.06.12</td>
<td>1.24</td>
<td>2.4</td>
<td><strong>DWFF-CPTU</strong>&lt;sup&gt;, length 4.1 m, ∆u&lt;sub&gt;2&lt;/sub&gt; and ∆u&lt;sub&gt;3&lt;/sub&gt;&lt;/sup&gt;</td>
<td>3</td>
</tr>
<tr>
<td>10-03</td>
<td>33°37.93'</td>
<td>136°40.10'</td>
<td>17.06.12</td>
<td>-</td>
<td>-</td>
<td><strong>DWFF-CPTU</strong>, length 4.1 m, ∆u&lt;sub&gt;2&lt;/sub&gt; and ∆u&lt;sub&gt;3&lt;/sub&gt;, test failed</td>
<td>3</td>
</tr>
<tr>
<td>10-04</td>
<td>33°37.90'</td>
<td>136°40.00'</td>
<td>17.06.12</td>
<td>1.26</td>
<td>2.2</td>
<td><strong>DWFF-CPTU</strong>&lt;sup&gt;, length 4.1 m, ∆u&lt;sub&gt;2&lt;/sub&gt; and ∆u&lt;sub&gt;3&lt;/sub&gt;&lt;/sup&gt;</td>
<td>3</td>
</tr>
<tr>
<td>30-01</td>
<td>33°39.38'</td>
<td>136°38.01'</td>
<td>24.06.12</td>
<td>0.93</td>
<td>2.1</td>
<td><strong>DWFF-CPTU</strong>&lt;sup&gt;, length 4.1 m, ∆u&lt;sub&gt;2&lt;/sub&gt; and ∆u&lt;sub&gt;3&lt;/sub&gt;&lt;/sup&gt;</td>
<td>4</td>
</tr>
<tr>
<td>30-02</td>
<td>33°39.44'</td>
<td>136°38.03'</td>
<td>24.06.12</td>
<td>1.46</td>
<td>1.5</td>
<td><strong>DWFF-CPTU</strong>&lt;sup&gt;, length 4.1 m, ∆u&lt;sub&gt;2&lt;/sub&gt; and ∆u&lt;sub&gt;3&lt;/sub&gt;&lt;/sup&gt;</td>
<td>4</td>
</tr>
<tr>
<td>30-03</td>
<td>33°39.49'</td>
<td>136°38.04'</td>
<td>24.06.12</td>
<td>1.29</td>
<td>2.9</td>
<td><strong>DWFF-CPTU</strong>&lt;sup&gt;, length 4.1 m, ∆u&lt;sub&gt;2&lt;/sub&gt; and ∆u&lt;sub&gt;3&lt;/sub&gt;&lt;/sup&gt;</td>
<td>4</td>
</tr>
</tbody>
</table>

Continued on next page...
Table 5.8 - Continued

<table>
<thead>
<tr>
<th>GeoB</th>
<th>LAT (north)</th>
<th>LONG (east)</th>
<th>date</th>
<th>WD</th>
<th>start velocity</th>
<th>penetration depth</th>
<th>probe setup, comments</th>
<th>MV No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>73-01</td>
<td>33°40.65'</td>
<td>136°55.20'</td>
<td>09.07.12</td>
<td>2004</td>
<td>-</td>
<td>-</td>
<td>DWFF-CPTU, length 4.1 m, $\Delta u_2$ and $\Delta u_3$, unprocessed</td>
<td>2</td>
</tr>
<tr>
<td>73-02</td>
<td>33°40.60'</td>
<td>136°55.25'</td>
<td>09.07.12</td>
<td>2003</td>
<td>-</td>
<td>-</td>
<td>DWFF-CPTU, length 4.1 m, $\Delta u_2$ and $\Delta u_3$, unprocessed</td>
<td>2</td>
</tr>
<tr>
<td>77-01</td>
<td>33°40.64'</td>
<td>136°55.19'</td>
<td>10.07.12</td>
<td>2014</td>
<td>-</td>
<td>-</td>
<td>DWFF-CPTU, length 4.1 m, $\Delta u_2$ and $\Delta u_3$, unprocessed</td>
<td>2</td>
</tr>
</tbody>
</table>

**Note:** LAT and LONG are the point locations in the geographical coordinate system. MV is the mud volcano. WD is the water depth.
5.2. MUD VOLCANOES

**Kumano Knoll #3** The first four DWFF-CPTU deployments were undertaken on top of the KK#3. One of the dynamic-CPTU tests (GeoB 16710-03) fails due to technical problems (i.e. failure during data logging). Figure 5.25 shows three quasi-static CPTU profiles represented by the excess pore pressure ($\Delta u$) evolution during failure measured at two different pore pressure ports ($\Delta u_2$ and $\Delta u_3$). Based on these datasets, the intact undrained shear-strength profiles ($s_u$) are derived using the empirical excess pore pressure factor ($N_{\Delta u}$; see section 2.1.4). In addition, the lithological core descriptions, core photos and $s_{u,fc}$ profiles of two adjacent gc’s are illustrated (GeoB 16712 and 16735). The lithology is characterized by mud breccia with clay to silty clay matrix and irregular distributed clasts (clay-to sandstone). Single sharp lithological features are detected in the pore pressure signal. The $\Delta u_2$ and $\Delta u_3$ vary between 5 and 20 kPa, except the first meter of the CPTU records. In this first section, positive peaks up to 60 kPa are encountered. The $s_u$ profiles show values less than 20 kPa. A very good agreement between the fc and in situ datasets can be seen. The slight discrepancies and peaks may be explained by the occurrence of gas/fluid escape structures and gas hydrate as well as differences in the core and in situ test locations due to vessel movements. The consolidation state is described as normally- to slightly over-consolidated with $s_u/\sigma'_V$ of 0.2 to 0.6.

**Kumano Knoll #4** In total, three DWFF-CPTU tests were carried out in and around of the KK #4. Figure 5.26 illustrates quasi-static CPTU profiles represented by the $\Delta u$ evolution during failure measured at two different ports ($\Delta u_2$ and $\Delta u_3$), and associated $s_{u,\Delta u_2}$ and $s_{u,\Delta u_3}$. The similar geotechnical solution is used to determine the $s_u$ from the $\Delta u$ (see section 2.1.4). The lithological core descriptions, core photos and $s_{u,fc}$ profiles of two adjacent gc’s are also depicted (GeoB 16725 and 16736). The stratigraphical sequence is characterized by mud breccia with clay to silty clay matrix and irregular distributed clasts (clay- to sandstone). The $s_{u,\Delta u_2}$ and $s_{u,\Delta u_3}$ profiles (GeoB 16730-01 and 16730-02) show a sharp step between soft and stiff soils at 0.6 m depth, which can also be seen in the $s_{u,fc}$. The $\Delta u_2$ and $\Delta u_3$ of the soils vary between 5 and 20 kPa for the first 0.6 m and increase up to 60 kPa in the deeper sections. In GeoB 16730-03, this sharp increase was not encountered due to the fact that this test is located at the shoulder area of the MV #4, which is characterized by homogeneous slope soils. The $s_u$ varies from 5 to 10 kPa with an increase to >15 kPa at >0.6 mbsf for GeoB 16730-01 and 16730-02. The dominated soils are normally-consolidated to slightly over-consolidated according to a $s_u/\sigma'_V$ of 0.2 to 0.5. The consolidation state of the increased section is described by $s_u/\sigma'_V$ values up to 1.3 characterizing highly over-consolidated soils. In all tests, fluid/gas structures, coarser sediments and angular to well rounded clasts are scattered along the soil succession.
Figure 5.25: Corrected excess pore pressure behind the tip ($\Delta u_2$) and 0.35 m behind the tip ($\Delta u_3$) for the MV #3 (GeoB 16710-01, 16710-02 and 16710-04). Additionally, the derived undrained shear-strength ($s_u$) of the DWFF-CPTU measurements compared with the fc data (GeoB 16712 and 16735) are illustrated. All dynamic-CPTU data sets are strain-rate corrected using the state-of-the-art inverse sinh-hyperbolic equation described in Mitchell and Soga (2005). Selected details of the dynamic-CPTU measurements can be seen in Table 5.8.
Figure 5.26: Corrected excess pore pressure behind the tip ($\Delta u_2$) and 0.35 m behind the tip ($\Delta u_3$) for the MV #4 (GeoB 16730-01 to 16730-03). Additionally, the derived undrained shear-strength ($s_u$) of the DWFF-CPTU measurements compared with the fc data (GeoB 16725 and 16736) are illustrated. All dynamic-CPTU data sets are strain-rate corrected using the state-of-the-art inverse sinh-hyperbolic equation described in Mitchell and Soga (2005). Selected details of the dynamic-CPTU measurements can be seen in Table 5.8.
Kumano Knoll #2  The dynamic-CPTU tests for the KK #2 are not processed at the moment, but will be incorporated in further studies.

Conclusions and outlook

Submarine mud volcanoes are predominant in convergent tectonic settings (e.g., Mediterranean Ridge accretionary complex and Nankai Trough accretionary complex) and provide a window to deep-seated processes due to transport of gaseous, fluid and solid composites to the seafloor (e.g. Robertson 1996, Kopf 1999, 2002). Dynamic-CPTU tests and laboratory experiments on cored samples are well suited to analyze the soil properties of these composites resulting to the following conclusions:

1. The DWFF-CPTU instrument is well suited to characterize surficial soils in and around MVs down to a penetration depth of less than 7 m, and is approved for a WD up to 4000 m (Stegmann and Kopf 2007).

2. For dynamic-CPTU tests, the most appropriate strain-rate correction solutions are based on the velocity ratio and modified inverse sin-hyperbolic equations. It is worth noting that all presented soil-specific rate coefficients are only valid for clays to silty clays with a normal- to over-consolidation state and an initial penetration rate of less than 2 m/s.

3. The soil-specific rate coefficients are in the range of 0.003 to 0.017 for the velocity ratio solution and of 0.04 to 0.65 for the modified inverse sin-hyperbolic solution.

4. The surficial soils of active and inactive MVs were characterized using geotechnical laboratory and in situ tests. The consolidation settings for the active ones illustrate \( s_u/\sigma'_{V0} \) usually less than 0.5, while the inactive MVs have very high consolidation states up to 1.5.

Advanced geotechnical laboratory experiments (e.g. direct shear, triaxial compression tests), and long-term in situ measurements with excess pore pressure records and fluid extraction for isotope analyses (e.g. boron-isotopes) are recommended. Such investigations deliver a better view of the geological, geochemical and geotechnical processes in the Earth’s crust.
5.3 Pockmarks

5.3.1 Patras harbor

Abstract

In spring 2012, a close cooperation between the University of Patras (Patras, Greece) and MARUM (Bremen, Germany) took place in order to geotechnically explore the pockmark field in and nearby the new harbor of Patras (western Greece). This area has been described as temporarily active, and seepage of gas has been associated with local seismic activity. A geotechnical survey was carried out using a local \( \sim 10 \) m long vessel from which the MARUM shallow-water dynamic-CPTU instrument (SWFF-CPTU) was deployed. Initial findings are summarized as follows:

- The \textit{in situ} intact undrained shear-strength derived from the excess pore pressure measured at the tip \((s_{u,\Delta u1})\) illustrates higher resolution compared to the strength properties derived from the corrected cone penetration resistance \((s_{u,qt})\).

- The intact undrained shear-strength \((s_u)\) properties of soils with free or dissolved gas are up to 30\% lower compared to \(s_u\) of undisturbed soils.

- The \(s_u\) profiles of active pockmark soils are 5 times lower than the strength properties of inactive pockmarks, and sub-hydrostatic pore pressures are detected for active ones.

\textit{Key words:} Shallow-water dynamic penetrometer, intact undrained shear-strength, pockmarks, new harbor of Patras.
Geotechnical Report of the *In Situ* Dynamic-CPTU Tests in the Pockmark Field off the Harbor of *Patras* (Patras/western Greece)

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Internal geotechnical cruise report: *MARUM - Center for Marine Environmental Sciences and Faculty of Geosciences, University of Bremen, Bremen, Germany.*

**Introduction**

Pockmarks are fluid and gas escape structures that appear as craters on the seafloor and occurring in a wide variety of sizes, shapes and configurations (e.g. Hovland et al. 2002). Many visual observations suggest that pockmarks are activated prior and during earthquakes, characterized by an increase of the rates of fluid flow and/or gas venting (e.g. Dando et al. 1995, Hasiotis et al. 1996). Such a phenomenon was observed at the pockmark field nearby the new harbor of *Patras* (western Greece) in conjunction with a major 5.4 earthquake in 1993 (Hasiotis et al. 1996).

This geotechnical study sheds light on the geotechnical soil properties inside of several pockmarks and outside of the pockmark field using *in situ* dynamic-CPTU records, and physical and geotechnical laboratory datasets (e.g. grain size tests, moisture and density [MAD] and vane shear [v-s] experiments), which were collected prior to the construction of the new harbor. Consequently, the connection between the pockmarks and deep-seated tectonic features, and activity of individual pockmarks are investigated.

**Regional setting**

The late Neogene-Quaternary graben system of the Gulf of *Patras* is controlled by several active WNW-ESE trending faults (western Greece). The maximum WD of the Gulf of *Patras* is 130 m. The south-western margin of the Gulf is covered by soft-layered Holocene clay-rich soils (Ferentinos et al. 1985, Chronis et al. 1991,
5.3. POCKMARKS

Hasiotis et al. 1996).

An active pockmark field was found in and off the new city harbor of Patras (south-east of the Gulf). This field consists of 72 pockmarks and covers an area of 1.7 km² in water depths of 40 m or less. Single and composites of pockmarks are evident from bathymetrical mapping (Fig. 5.27). The diameters of these depressions vary between 25 and 250 m, and the depth is up to 15 m for the larger ones with a mean value of 9 m (Hasiotis et al. 1996, Christodoulou et al. 2003).

Materials and methods

**In situ CPTU measurements** The SWFF-CPTU was used during this geotechnical cruise and a detailed summary of this device is given in section 3.1.1. The instrument configuration consists of 4 weights (i.e. 60 kg in total), main body, three 1 m long rods and dynamic-CPTU cone. A CPTU cone with a maximum capacity of 25 m MPa for $q_t$, 0.25 MPa for $f_s$ and 2 MPa for $u_1$ (i.e. pore pressure at the tip) was utilized. All presented dynamic-CPTU data were corrected for the strain-rate effect using the modified inverse sinhyperbolic correction solution presented in section 3.2.3. The soil-specific rate coefficients (SSCs) vary between 0.04 and 0.05 for $q_t$, and between 0.08 and 0.10 for $\Delta u_1$. Both CPTU parameters are used to determine the in situ intact undrained shear-strength ($s_u$) using the empirical correlations presented in section 2.1.4. The empirical cone penetration resistance factor ($N_{kt}$) are defined in the order of 15 to 25 (used 20) characterizing values for gassy soils (e.g. Sultan et al. 2010 and section 4.2). Moreover, the empirical excess pore pressure factor ($N_{\Delta u_1}$) is set between 3 and 11 (used 6.5), which is in full compliance with the data sets presented by Low et al. 2010 (see also Table 2.3).

**Physical properties** Grain size distribution, water content, MAD bulk density, specific gravity, Atterberg limits and intact vane shear-strength ($s_{u,v}$) were determined during a previous study (Papatheodorou et al. [unpublished report]). All laboratory experiments were carried out in accordance to the British Standards Institution (BSI 1377; 1975, see also section 5.1.1 [see Materials and methods]).

**Preliminary results**

In total, 11 SWFF-CPTU tests were performed (see Fig. 5.27 for location). The WD ranges between 26 and 42 mbsl (meter below sea-level). The penetration rate varies between 1.0 and 7.9 m/s resulting in penetration depths in the order of 0.65 to 4.15 m (Table 5.9).
Figure 5.27: Morphological map shows the locations of the SWFF-CPTU deployments (PP or PRef) collected during the geotechnical survey in 2012. All PP deployments are located inside of the pockmarks, and the PRef test defines a reference location outside of the pockmarks. The gravity cores (GC) were recovered prior to the construction of the new harbor of Patras (i.e. GC3, GC8, GC10 and GC12 are located direct in the construction areas). The contourlines with a spacing of 5 m describe the water depth (modified after Christodoulou et al. (2003), Marinaro et al. (2006)).
Table 5.9: List of *SWFF-CPTU* deployments performed during the geotechnical cruise in the vicinity of the city harbor of Patras (western Greece).

<table>
<thead>
<tr>
<th>Point No.</th>
<th>LAT (north)</th>
<th>LONG (east)</th>
<th>date</th>
<th>WD</th>
<th>start velocity</th>
<th>duration of measurement</th>
<th>penetration depth</th>
<th>probe setup, comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>PP4</td>
<td>38°12'42.43&quot;</td>
<td>21°42'33.30&quot;</td>
<td>16.03.12</td>
<td>26</td>
<td>3.3</td>
<td>10</td>
<td>3.75</td>
<td><em>SWFF-CPTU</em>, 3 rods + housing, 4 weights, ( u_1 )</td>
</tr>
<tr>
<td>PP11</td>
<td>38°12'56.48&quot;</td>
<td>21°42'41.59&quot;</td>
<td>16.03.12</td>
<td>33</td>
<td>2.2</td>
<td>5</td>
<td>3.65</td>
<td><em>SWFF-CPTU</em>, 3 rods + housing, 4 weights, ( u_1 )</td>
</tr>
<tr>
<td>PP75</td>
<td>38°13'02.01&quot;</td>
<td>21°42'43.45&quot;</td>
<td>16.03.12</td>
<td>37</td>
<td>1.0</td>
<td>4</td>
<td>1.45</td>
<td><em>SWFF-CPTU</em>, 3 rods + housing, 4 weights, ( u_1 )</td>
</tr>
<tr>
<td>PP1</td>
<td>38°13'21.98&quot;</td>
<td>21°42'53.32&quot;</td>
<td>16.03.12</td>
<td>30</td>
<td>5.7</td>
<td>12</td>
<td>2.85</td>
<td><em>SWFF-CPTU</em>, 3 rods + housing, 4 weights, ( u_1 )</td>
</tr>
<tr>
<td>PP8-1</td>
<td>38°13'43.11&quot;</td>
<td>21°42'48.02&quot;</td>
<td>17.03.12</td>
<td>39</td>
<td>3.4</td>
<td>10</td>
<td>0.65</td>
<td><em>SWFF-CPTU</em>, 3 rods + housing, 4 weights, ( u_1 )</td>
</tr>
<tr>
<td>PP8-2</td>
<td>38°13'43.11&quot;</td>
<td>21°42'48.02&quot;</td>
<td>17.03.12</td>
<td>40</td>
<td>6.5</td>
<td>10</td>
<td>1.05</td>
<td><em>SWFF-CPTU</em>, 3 rods + housing, 4 weights, ( u_1 )</td>
</tr>
<tr>
<td>PP16</td>
<td>38°13'10.28&quot;</td>
<td>21°42'36.27&quot;</td>
<td>17.03.12</td>
<td>42</td>
<td>6.3</td>
<td>13</td>
<td>1.95</td>
<td><em>SWFF-CPTU</em>, 3 rods + housing, 4 weights, ( u_1 )</td>
</tr>
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</table>

Continued on next page...
<table>
<thead>
<tr>
<th>Point No.</th>
<th>LAT (north)</th>
<th>LONG (east)</th>
<th>date</th>
<th>WD</th>
<th>start velocity</th>
<th>duration of measurement</th>
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<td>42</td>
<td>7.9</td>
<td>14</td>
<td>3.25</td>
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<td>PP2</td>
<td>38°13'18.66&quot;</td>
<td>21°42'42.05&quot;</td>
<td>17.03.12</td>
<td>37</td>
<td>7.7</td>
<td>12</td>
<td>3.95</td>
<td>SWFF-CPTU, 3 rods + housing, 4 weights, ( u_1 )</td>
</tr>
<tr>
<td>PRef1</td>
<td>38°13'17.93&quot;</td>
<td>21°42'47.20&quot;</td>
<td>17.03.12</td>
<td>29</td>
<td>7.2</td>
<td>14</td>
<td>2.60</td>
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<tr>
<td>PP38</td>
<td>38°13'29.29&quot;</td>
<td>21°42'53.46&quot;</td>
<td>17.03.12</td>
<td>31</td>
<td>7.7</td>
<td>11</td>
<td>4.15</td>
<td>SWFF-CPTU, 3 rods + housing, 4 weights, ( u_1 )</td>
</tr>
</tbody>
</table>

**Note:** LAT and LONG are the point locations in the geographical coordinate system.

MV is the mud volcano.

WD is the water depth.
In the course of the new harbor construction, 14 gravity cores were collected in the area of the pockmark field. Figure 5.28 presents sedimentological and geotechnical datasets of four cores located in the center part of the field (GC8, GC9, GC12 and GC13) in order to characterize the surficial soils. In addition, a comparison between $s_{u,v-s}$ and in situ intact undrained shear-strength derived from the $q_t (s_{u,qt})$ are illustrated to confirm that: (i) strain-rate correction function and SSCs are appropriate and (ii) $N_{kt}$ and $N_{k\Delta u}$ are adequate for this exploration site. The surficial soil succession up to 2 m is very heterogeneous. The dominated soils are clay to silty clay. At $\sim 1$ m depth, irregularly distributed coarse-grained layers of clayey silt to silty sand are detected. The dominated soil has a clay content of up to 50% and the sand proportion is negligible. The coarse-grained layers illustrate a clay content of less than 10% with a sand amount of up to 15% (Fig. 5.28a). MAD bulk density ($\rho_{\text{bulk}}$) measurements document values in the order of magnitude of 1.65 to 1.85 g/cm$^3$ (Fig. 5.28b). Three $s_{u,qt}$ profiles (PP1, PP16 and PP75) were compared to the $s_{u,v-s}$ data obtained from the four cores (Fig. 5.28c). Both datasets show good agreement and thus, the above mentioned strain-rate and geotechnical factors are appropriate for this study.

Figure 5.28: Sedimentological and geotechnical analyses using four gravity core data sets (PP8, PP9, PP12 and PP13) and three in situ CPTU measurements (PP1, PP16 and PP75). (a) presents an ordinary core log of the first 2 m and grain-size distribution of the cores, (b) illustrates the MAD bulk density ($\rho_{\text{bulk}}$) and (c) depicts intact undrained shear-strength ($s_u$) derived from the gravity cores using vane shear experiments and in situ measurements.
In this manner, all other SWFF-CPTU tests are analyzed to investigate the geotechnical properties of the soil succession within or nearby to the different pockmarks (see Fig. 5.27 for location). Figure 5.29 illustrates the $s_{uqt}$ profiles of all 11 SWFF-CPTU tests including the total and effective overburden stress conditions ($\sigma_{V0}$ and $\sigma'_{V0}$). The profiles can be categorized as follows:

1. profiles contain single or several massive peaks related to coarse-grained layers (i.e. fluvioglacial deposits from the Glafkos river or probably engineering fill from the harbor construction (PRefl, PP8-1, PP8-2 and PP72)),

2. profiles show normally or slightly over-consolidated state with undrained shear-strength ratio ($s_u/\sigma'_{V0}$) between 0.2 and 0.5 (PP1, PP2, PP4, PP16 and PP38) and

3. profiles with $s_u/\sigma'_{V0}$ less than 0.2 characterizing under-consolidated soils, which are probably affected by gas or fluid discharge (PP11 and PP75).
Figure 5.29: *In situ* intact undrained shear-strength profiles derived from the corrected cone penetration resistance ($s_{u,qt}$) collected in the pockmark field off the harbor of *Patras* (western Greece). The total and effective overburden stress conditions are also shown ($\sigma_{V0}$ and $\sigma'_{V0}$).
From our limited number of tests, the pockmarks of the third category seem to be active while the ones from category 2 may be inactive. Moreover, the excess pore pressure measurements are also used to evaluate the \textit{in situ} intact undrained shear-strength (Fig. 5.30). Five \textit{SWFF-CPTU} profiles are depicted, as an example (PP2, PP4, PP11, PP16 and PP75). A significant difference between the \textit{in situ} CPTU tests in under- and normally consolidated soils are observed. The first three tests (PP2, PP4 and PP16) show good agreement between $s_{u,qt}$ and $s_{u,\Delta u1}$ (Fig. 5.30). In contrast, the remaining two (PP11 and PP75) illustrate $\Delta u$ anomalies (i.e. sub-hydrostatic excess pore pressures) resulting in negative $s_{u,\Delta u1}$ (Fig. 5.30). Seifert et al. (2008) describe such a negative response in $\Delta u$ by the occurrence of dissolved or free gas in the soils. Two potential possibilities were discussed: (i) free gas accumulation due to decay of organic material resulting in variations of shear-strength, in particular, an increase of the gas concentration reduces the intact undrained shear-strength around 20\% (e.g. Wheeler 1986, Brandes 1999), and (ii) buoyancy of free gas in fissures generated during penetration and along the rods of the dynamic-CPTU instrument (e.g. Seifert and Kopf 2012). This underlines the assumption that both CPTU deployments (PP11 and PP75) were performed inside of active pockmarks.

Conclusions and outlook

Pockmarks on continental slopes are usually coupled to gas hydrates, deep-seated fluid pressure build-up and landslide activities (e.g. Hovland 2002). Prior and after earthquakes, higher rates of fluid flow and/or gas venting in pockmark fields were observed at many locations (e.g. off California [Field and Jennings 1987], \textit{Patras} [Hasiotis et al. 1996]). The geotechnical study in the \textit{Patras} pockmark field results in the following conclusions:

1. \textit{In situ} dynamic-CPTU tests are best suited to explore nearshore pockmarks in a time- and cost-efficient manner (i.e. 11 dynamic-CPTU tests were performed in less than 6 hours).

2. High-resolution \textit{in situ} records compared to laboratory experiments on cored samples indicate free gas growth in the soils, which have up to 30\% lower intact undrained shear-strength properties.

3. Active pockmarks are identified and characterized by 5 times lower intact undrained shear-strength properties and sub-hydrostatic pore pressures.

At the \textit{Patras} pockmark field, periodical measurements using \textit{in situ} instruments and/or the installation of long-term \textit{in situ} observatories measuring the excess pore pressure and collecting fluids for analyses of volatile elements are recommended in order to obtain new findings regarding to: (i) the link between pockmarks and deep-seated processes, and (ii) the prediction of earthquakes.
Figure 5.30: Comparison of the in situ intact undrained shear-strength derived from the corrected cone penetration resistance \( s_{u,qt} \) and excess pore pressure at the tip \( s_{u,\Delta u1} \). The effective and total overburden stress conditions are also shown \( (\sigma'_{V0} \text{ and } \sigma_{V0}) \).
CHAPTER 5. ONGOING PROJECTS
Chapter 6
Conclusions

Six multidisciplinary studies, two condensed into peer-reviewed manuscripts (sections 3.2.3, 4.1, 4.2) and four published as work in progress in section 5, were performed within this doctoral project using geophysical transects, and a large number of *in situ* and laboratory datasets collected with two different dynamic-CPTU instruments, static-CPTU devices, *Calypso* piston and gravity corer. The following conclusions can be drawn:

- Post-processing of dynamic-CPTU measurements, in particular with very high initial penetration rates of up to 10 m/s, is more demanding compared to analyses of static-CPTU tests due to the dynamic nature of the measurements and the strain-rate effect (section 3.2).

- The development of a uniform strain-rate correction solution is difficult to develop on a theoretical basis due to the variability and non-linear behavior of the soil. Comparison of dynamic-CPTU measurements with nearby static-CPTU tests and laboratory experiments, such as fall cone penetration (fc), vane shear (v-s), direct simple shear (DSS), anisotropically-consolidated undrained compression and extension triaxial tests on samples taken by *Calypso* piston and gravity coring are key ways of finding the best strain-rate correction solution and appropriate soil-specific rate coefficients (SSCs) (section 3.2.3).

- The comparison of several dynamic-CPTU measurements with static-CPTU, DSS, fc and v-s experiments clearly shows that the modified inverse sinhypberbolic strain-rate correction solution is best suited to correct dynamic-CPTU measurements in fine-grained soils, combined with SSCs in the range of 0.04 to 0.065 for the corrected cone penetration resistance, 0.2 to 0.6 for the sleeve friction and 0.04 to 0.65 for the excess pore pressure, depending on the location of the different pressure ports (sections 3.2.3, 5).

- The penetration depth depends on the soil resistance/strength, consolidation state and composition as well as deployment mode (i.e. *winch* or *free-fall*).
Penetration depths of $\sim 3$ m were achieved using *winch mode* and $6$ m using a *free-fall drop* in normally consolidated soils. However, a penetration depth of up to $8.5$ m was reached in soft soils (south-central Chile, South America; G. Wiemer, pers. comm., 2013) (sections 3.2.3, 4.2).

- The dynamic-CPTU tests are appropriate to identify and characterize thin potentially weak layers with a thickness of $\sim 0.45$ m containing $0.1$ to $0.15$ m thick soft and sensitive clay-rich beds and a $0.2$ m thick embedded sandy silt to sand bed (section 4.1).

- Dynamic-CPTU measurements can be collected very quickly (i.e. $10$ to $20$ deployments including dissipation tests within $8$ h in water depths $<4000$ m) resulting in a large number of dynamic tests for each exploration site. Such large datasets can be used: (i) to develop sub-seafloor models (i.e. lithological and geotechnical 3D models) of wide spatial coverage, and (ii) to perform slope stability and risk assessments in case the sub-seafloor model is supplemented by geophysical, coring/sampling and other *in situ* datasets (section 4.2).

- The state-of-the-art geotechnical hypothesis, deriving the intact undrained shear-strength from the excess pore pressure, is tested for both *MARUM* dynamic-CPTU instruments, and shows good agreement with intact vane shear and intact fall cone penetration data obtained from laboratory experiments on contiguous gravity cores (section 5).

- Offshore mud volcanoes were investigated using coring/sampling and dynamic-CPTU measurements, resulting in: (i) a geotechnical characterization of the surficial soils and (ii) clear indications concerning the activity of the different mud volcanoes in combination with the soil consolidation state (section 5.2).

- Dynamic-CPTU tests can easily be used to examine the activity of pockmarks using pore pressure and intact undrained shear-strength profiles, in order to distinguish between active and inactive pockmarks (section 5.3).

In summary, dynamic-CPTU instruments are powerful *in situ* tools for exploring the sub-seafloor soil succession up to $8.5$ m depth in soft to normally consolidated clays. Findings attained with these instruments have the potential to improve the understanding of natural slope stability processes, and phenomena related to mud volcanoes and pockmarks (e.g. the link with seismicity), which is of social and economical relevance for coastal communities and offshore structures (e.g. geohazards related to coastal landslides and tsunamis generated by offshore earthquakes and submarine landslides). However, mud volcanoes and pockmarks have to be investigated in more comprehensive studies and require further research efforts taking into account more detailed geophysical, geochemical, sedimentological and geotechnical datasets.
Chapter 7

Outlook

The findings of this doctoral thesis can be seen as fundamental work regarding data processing and developing a strain-rate correction for dynamic-CPTU measurements, and application of these in situ tests for geological and geotechnical purposes. The main objectives chosen at the beginning were achieved; however, existing uncertainties, limitations and new questions concerning the used solutions and approaches may be addressed in the future.

The main focus of this thesis was the comparison of dynamic-CPTU and static-CPTU tests under in situ conditions. However, this approach required several simplifications and uncertainties with respect to the soil anisotropy, consolidation state, stress conditions and site specific factors, which are difficult to measure in situ. An alternative method is the use of calibration chamber and soil target tests (e.g. tank or bucket experiments) in order to reduce these uncertainties and limitations. In the laboratory or in a test facility, such controlled measurements can be utilized: (i) to compare different soil composites (e.g. clay to sand) prepared under different boundary conditions, and (ii) to link datasets collected with different dynamic penetrometers (e.g. Nimrod [Stark et al. 2009], SWFF-CPTU) or static-CPTU instruments (e.g. GOST) considering several penetration rates and piezocone diameters and geometries. Consequently, the strain-rate correction and associated soil-specific rate coefficients can be evaluated under controlled conditions for the three CPTU parameters (cone penetration resistance, sleeve friction, pore pressure). Based on the findings from this thesis, the following controlled tests are recommended:

- Variation of the initial penetration rates from 0.02 to 10 m/s;
- Variation of the cone diameter/geometry (e.g. disk, conical, ball and T-Bar piezocones) and pore pressure port location;
- Variation of the suspension concentration in fluid sands to analyze the link between the rheological soil parameters and nautical depth;
• Variation of grain size distribution, in particular for sandy soils;
• Incorporation of embedded layers with variable texture and density.

These experiments are highly complex and require a meticulous preparation; however, the new findings would provide a solid basis for the strain-rate correction and applicability of dynamic-CPTU measurements for different geological settings.

Artesian and excess pore pressures in the soil have recently been determined to be crucial factors for the stability of slopes in coastal environments (e.g. Nice airport [Stegmann et al. 2011], Trondheim harbor [L’Heureux et al. 2010]). In the study area off the village of Finneidjord (northern Norway), a large number of dynamic-CPTU tests were collected and almost half of the tests comprise pore pressure dissipation records. These datasets may be used to develop a hydrological model similar to those presented by Stegmann et al. (2011). The conduction of an additional scientific field campaign in the study area increases the accuracy of this model. The focus of this campaign should be placed on coring, including shipboard and post-cruise laboratory experiments (e.g. fc, v-s and permeameter tests) and in situ dynamic- and static-CPTU measurements.

In the vicinity of the Nice international airport (southeastern France), seismic reflection surveys were carried out in 2007 (Kopf et al. 2008). In a depth of ∼100 m, a strong, predominantly smooth reflector was mapped and interpreted as the interface between the Holocene mud wedge and Quaternary gravel (Dubar and Anthony 1995). This transition zone may be acting as a ground water aquifer along this artesian, where excess fluid pressures can be generated resulting in a higher potential for liquefaction in combination with the decrease of the effective vertical stresses (Anthony and Julian 1997). A detailed characterization of the transition zone is necessary; however this requires long core samples supplemented by lithological and geotechnical analyses of drilled specimens taken above and below of this zone. In addition, the new geotechnical properties and numerical slope stability assessments may be used to ascertain the potential likelihood that a very large submarine landslide could occur near the Nice international airport and may negatively influence the coastline of the French Riviera.

In the Nice airport (southeastern France) and Twin slide (southern Sicily, Italy) areas, a down-slope thickening of the shallow mud complex was observed (e.g. Dubar and Anthony 1995, Minisini et al. 2007). The sedimentation rates within the same time interval must have increased down-slope, which may have resulted in a rise of pore pressures (Gibson 1958). This hypothesis can be utilized to investigate the correlation between the architecture of the slope, sedimentation rate and pore pressure evolution with respect to the stability of both landslide-prone areas.
The analyses of surficial gas, fluid and solid composites and gas venting in and around mud volcanoes and pockmarks provide insight regarding: (i) the link between mud volcanoes/pockmarks and deep-seated processes, (ii) the prediction of earthquakes, and (iii) the better understanding of the geological, geochemical and geotechnical processes in the Earth’s crust. Periodical measurements using in situ instruments (e.g. SWFF- and DWFF-CPTU) and the installation of long-term in situ observatories measuring excess pore pressure and collecting fluids for analyses of volatile elements (e.g. oxygen, lithium and boron isotopes) may be the key to further understanding.
Chapter 8

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This doctoral thesis could not have been realized without the help of the following people. I also want to thank all the persons, who helped me with small things over the last three and a half years and who are not mentioned hereafter.

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Bremen, November 1\textsuperscript{st}, 2013

(Alois Steiner)
CHAPTER 8. ACKNOWLEDGMENTS
Chapter 9

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# Chapter 10

## Symbol list

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
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<tbody>
<tr>
<td>$a$</td>
<td>net area ratio</td>
</tr>
<tr>
<td>AGO</td>
<td>A.G.O. Environmental Electronics Ltd.</td>
</tr>
<tr>
<td>$B_q$</td>
<td>pore pressure ratio</td>
</tr>
<tr>
<td>$B_{1, 2}$</td>
<td>spring constants (vane shear test)</td>
</tr>
<tr>
<td>B.P.</td>
<td>before past</td>
</tr>
<tr>
<td>$B_q$</td>
<td>pore pressure ratio ($= \Delta u/(q_t-\sigma V_0)$)</td>
</tr>
<tr>
<td>$c'$</td>
<td>effective cohesion</td>
</tr>
<tr>
<td>CAT</td>
<td>chemical and aqueous transport</td>
</tr>
<tr>
<td>CAUE</td>
<td>anisotropically-consolidated undrained extension triaxial</td>
</tr>
<tr>
<td>CAUC</td>
<td>anisotropically-consolidated undrained compression triaxial</td>
</tr>
<tr>
<td>core log</td>
<td>visual core description</td>
</tr>
<tr>
<td>cpc</td>
<td><em>Calypso</em> piston core</td>
</tr>
<tr>
<td>CPT</td>
<td>cone penetration test</td>
</tr>
<tr>
<td>CPTU</td>
<td>cone penetration test with pore pressure measurement</td>
</tr>
<tr>
<td>CT</td>
<td>one-dimensional compression test</td>
</tr>
<tr>
<td>$c_v$</td>
<td>coefficient of consolidation</td>
</tr>
<tr>
<td>$c_{v,DSS}$</td>
<td>coefficient of consolidation derived from the DSS</td>
</tr>
<tr>
<td>$d$</td>
<td>diameter of the vane (vane shear test)</td>
</tr>
<tr>
<td>$d_{1-3}$</td>
<td>regression <em>in situ</em> intact undrained shear strength at the seabed/surface</td>
</tr>
<tr>
<td>Symbol</td>
<td>Definition</td>
</tr>
<tr>
<td>--------</td>
<td>------------</td>
</tr>
<tr>
<td>$D$</td>
<td>constitutive law matrix</td>
</tr>
<tr>
<td>$d_{dyn}$</td>
<td>piezocone diameter of the dynamic-CPTU device</td>
</tr>
<tr>
<td>$DND$</td>
<td>Department of National Defense (Canada)</td>
</tr>
<tr>
<td>$d_{ref}$</td>
<td>piezocone diameter of the static-CPTU equipment</td>
</tr>
<tr>
<td>DSS</td>
<td>direct simple shear</td>
</tr>
<tr>
<td>$DWFF\text{-}CPTU$</td>
<td>deep-water dynamic piezocone penetrometer ($\leq 4000$ m)</td>
</tr>
<tr>
<td>$E$</td>
<td>Young’s modulus</td>
</tr>
<tr>
<td>$E_{ref}$</td>
<td>Young’s modulus at seabed</td>
</tr>
<tr>
<td>$E_{inc}$</td>
<td>Young’s modulus increase with depth</td>
</tr>
<tr>
<td>$fc$</td>
<td>fall cone penetration</td>
</tr>
<tr>
<td>$FF\text{-}CPTU$</td>
<td>Free-Fall Piezocone Penetrometer</td>
</tr>
<tr>
<td>FoS</td>
<td>factor of safety</td>
</tr>
<tr>
<td>$f_s$</td>
<td>sleeve friction</td>
</tr>
<tr>
<td>$f_{s, dyn}$</td>
<td>dynamic sleeve friction</td>
</tr>
<tr>
<td>$f_{s,q-s}$</td>
<td>SF-corrected (quasi-static) sleeve friction</td>
</tr>
<tr>
<td>$f_{s, ref}$</td>
<td>static sleeve friction recorded at the reference penetration rate i.e. 2 cm/s</td>
</tr>
<tr>
<td>$g$</td>
<td>gravity ($9.81$ m/s$^2$)</td>
</tr>
<tr>
<td>$G$</td>
<td>undrained shear modulus ($=E/(2(1+\nu))$)</td>
</tr>
<tr>
<td>$gc$</td>
<td>gravity core</td>
</tr>
<tr>
<td>$GEO$</td>
<td>Danish Geotechnical Institute</td>
</tr>
<tr>
<td>$GOST$</td>
<td>Geotechnical Offshore Seabed Tool</td>
</tr>
<tr>
<td>$gs$</td>
<td>grain size</td>
</tr>
<tr>
<td>$gsd$</td>
<td>grain size distribution</td>
</tr>
<tr>
<td>$H_{ini}$</td>
<td>initial height of a specimen at the start of a loading increment</td>
</tr>
<tr>
<td>$ICG$</td>
<td>International Centre for Geohazards</td>
</tr>
<tr>
<td>Ifremer</td>
<td>Institut français de recherche pour l’exploitation de la mer (French Research Institute for Exploitation of the Sea)</td>
</tr>
<tr>
<td>$I_L$</td>
<td>liquidity index ($=(\omega_{-p}/(\omega_{L}-\omega_{p}))$)</td>
</tr>
</tbody>
</table>
$I_P$ plasticity index ($=\omega_L-\omega_P$)

ISSMGE International Society for Soil Mechanics and Geotechnical Engineering

$k, d$ basic parameters for the interaction equation between $v_0$ and $z_{\text{depth}}$

$k$ empirical cone factor (fall cone penetration test)

$K$ empirical vane shear constant (vane shear test)

$k_f$ coefficient of permeability

KK Kumano Knoll

$k_{1-3}$ regression $insitu$ intact undrained shear-strength gradient with depth

$l$ length of the vane (vane shear test)

$L$ differential operator

$L^T$ transpose of the differential operator

$L1, L2, L3$ tripartite layering system

$m$ mass of the cone (fall cone penetration test)

MAD moisture and density

mag.sus., MS magnetic susceptibility

MARUM Zentrum für Marine Umweltwissenschaften (Center for Marine Environmental Sciences)

MC Mohr Coulomb

MeBo MARUM sea floor drill rig

MedRidge Mediterranean Ridge accretionary complex

$mod.f_s$ modified sleeve friction solution

MSCL Multi Sensor Core Logging

MTD mass transport deposit

$m_v$ coefficient volume compressibility

MV mud volcanoes

NAIL 1979 Nice airport landslide

NGI Norwegian Geotechnical Institute
### CHAPTER 10. SYMBOL LIST

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
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<tbody>
<tr>
<td><strong>NGU</strong></td>
<td>Norges geologiske undersøkelse (Geological Survey of Norway)</td>
</tr>
<tr>
<td>$N_c$</td>
<td>theoretical dimensionless bearing capacity factor ((-q_{\text{net}}/s_u))</td>
</tr>
<tr>
<td>$N_{fs}$</td>
<td>empirical sleeve friction factor ((=f_s/s_{ur}))</td>
</tr>
<tr>
<td>$N_{ke}$</td>
<td>effective empirical cone penetration resistance factor ((=q_e/s_u))</td>
</tr>
<tr>
<td>$N_{kt}$</td>
<td>empirical cone penetration resistance factor ((=q_{\text{net}}/s_{a,qt}))</td>
</tr>
<tr>
<td>$N_{kt,s_u,ave}$</td>
<td>empirical cone penetration resistance factor reflecting the average of the CAUE, DSS and CAUC strength ((-q_{\text{net}}/s_{u,ave}))</td>
</tr>
<tr>
<td>$N_{kt,s_u,c}$</td>
<td>empirical cone penetration resistance factor reflecting the measure of the CAUC strength ((-q_{\text{net}}/s_{u,c}))</td>
</tr>
<tr>
<td>$N_{kt,s_u,v-s}$</td>
<td>empirical cone penetration resistance factor reflecting the measure of the \textit{in situ} v-s strength ((-q_{\text{net}}/s_{u,v-s}))</td>
</tr>
<tr>
<td>$N_{\Delta u}$</td>
<td>empirical excess pore pressure factor ((=\Delta u/s_u))</td>
</tr>
<tr>
<td>$N_{\Delta u,s_u,ave}$</td>
<td>empirical excess pore pressure factor reflecting the average of the CAUE, DSS and CAUC strength ((-\Delta u/s_{u,ave}))</td>
</tr>
<tr>
<td>$N_{\Delta u,s_u,c}$</td>
<td>empirical excess pore pressure factor reflecting the measure of the CAUC strength ((-\Delta u/s_{u,c}))</td>
</tr>
<tr>
<td>$N_{\Delta u,s_u,v-s}$</td>
<td>empirical excess pore pressure factor reflecting the measure of the \textit{in situ} v-s strength ((-\Delta u/s_{u,v-s}))</td>
</tr>
<tr>
<td><strong>NTS</strong></td>
<td>northern Twin slide</td>
</tr>
<tr>
<td><strong>p</strong></td>
<td>external load</td>
</tr>
<tr>
<td><strong>PC</strong></td>
<td>personal computer</td>
</tr>
<tr>
<td><strong>PDIM</strong></td>
<td>power and data interface module</td>
</tr>
<tr>
<td><strong>PGA</strong></td>
<td>peak ground acceleration</td>
</tr>
<tr>
<td><strong>pp</strong></td>
<td>pore pressure</td>
</tr>
<tr>
<td><strong>pushed CPTU</strong></td>
<td>standard, industry Cone Penetration Testing</td>
</tr>
<tr>
<td>$q_c$</td>
<td>cone penetration resistance</td>
</tr>
<tr>
<td>$q_{c,\text{net}}$</td>
<td>net cone penetration resistance ((=q_c-\sigma_{V0}))</td>
</tr>
<tr>
<td>$q_{c,\text{dyn}}$</td>
<td>dynamic cone penetration resistance</td>
</tr>
<tr>
<td>$q_e$</td>
<td>effective cone penetration resistance ((=q_e-u_2))</td>
</tr>
</tbody>
</table>
$q_{\text{net}}$  
net corrected cone penetration resistance ($=q-\sigma V_0$)

$q_t$  
cone penetration resistance corrected for pore pressure effects ($=q_c+u(1-a)$)

$q_{t,\text{dyn}}$  
dynamic cone penetration resistance corrected for pore pressure effects ($=q_{c,\text{dyn}}+u_{\text{dyn}}(1-a)$)

$q_{t,q-s}$  
SF-corrected (quasi-static) corrected cone penetration resistance

$q_{t,\text{ref}}$  
static corrected cone penetration resistance recorded at the reference penetration rate i.e. 2 cm/s

$q_u$  
soil bearing capacity

$Q_t$  
normalized cone penetration resistance

$R_f$  
friction ratio

ROV  
remotely operated vehicle

SD  
standard secure digital memory card

$SF, PRF$  
strain-rate correction factor

$SF_{\log}$  
strain-rate factor of the logarithmic equation

$SF_{\text{arcsinh}}$  
strain-rate factor of the inverse sinh-hyperbolic equation

$SF_{\text{power}}$  
strain-rate factor of the power-law equation

$SF_{\text{velo}}$  
strain-rate factor of the velocity ratio equation

SSC  
soil-specific rate coefficient

$S_t$  
soil sensitivity

STS  
southern Twin slide

$s_u$  
intact undrained shear-strength

$s_u/\sigma'_{V0}$  
undrained shear-strength ratio

$s_u,\text{ave}$  
average of triaxial and simple shear intact undrained shear-strength

$s_u,DSS$  
direct simple shear-strength

$s_u,DSS/\sigma'_{ac,DSS}$  
undrained shear-strength ratio derived from the DSS

$s_u,\text{dyn}$  
dynamic in situ intact undrained shear-strength

$s_u,fc$  
intact fall cone penetration strength

$s_u,qt$  
in situ intact undrained shear-strength derived from the corrected cone penetration resistance
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$s_{u,qt}/\sigma'_{V0}$</td>
<td>in situ undrained shear-strength ratio</td>
</tr>
<tr>
<td>$s_{ur}$</td>
<td>in situ remolded undrained shear-strength</td>
</tr>
<tr>
<td>$s_{u,reg}$</td>
<td>regression in situ undrained shear-strength</td>
</tr>
<tr>
<td>$s_{ur,v-s}$</td>
<td>remolded vane shear-strength</td>
</tr>
<tr>
<td>$s_{u,v-s}$</td>
<td>intact vane shear-strength</td>
</tr>
<tr>
<td>$s_{u,\Delta u}$</td>
<td>in situ intact undrained shear-strength derived from the excess pore pressure</td>
</tr>
<tr>
<td>$s_{u,\Delta u1}$, $s_{u1}$</td>
<td>in situ intact undrained shear-strength derived from the excess pore pressure and measured at the tip</td>
</tr>
<tr>
<td>$s_{u,\Delta u2}$, $s_{u2}$</td>
<td>in situ intact undrained shear-strength derived from the excess pore pressure and measured behind the tip</td>
</tr>
<tr>
<td>$s_{u,\Delta u3}$, $s_{u3}$</td>
<td>in situ intact undrained shear-strength derived from the excess pore pressure and measured behind the sleeve</td>
</tr>
<tr>
<td>$SWFF-CPTU$</td>
<td>shallow-water dynamic piezocone penetrometer ($\leq 500$ m)</td>
</tr>
<tr>
<td>$T$</td>
<td>torque (vane shear experiment)</td>
</tr>
<tr>
<td>TOC</td>
<td>total organic carbon</td>
</tr>
<tr>
<td>$u$</td>
<td>pore pressure</td>
</tr>
<tr>
<td>$u_{\text{disp}}$</td>
<td>external displacement</td>
</tr>
<tr>
<td>$u_{\text{dyn}}$</td>
<td>dynamic pore pressure</td>
</tr>
<tr>
<td>$u_0$</td>
<td>hydrostatic water pressure</td>
</tr>
<tr>
<td>$u_1$</td>
<td>pore pressure measured at the tip</td>
</tr>
<tr>
<td>$u_2$</td>
<td>pore pressure measured behind the tip</td>
</tr>
<tr>
<td>$u_3$</td>
<td>pore pressure measured behind the sleeve</td>
</tr>
<tr>
<td>$v_{\text{dyn}}$</td>
<td>dynamic penetration rate</td>
</tr>
<tr>
<td>$V$</td>
<td>non-dimensional velocity</td>
</tr>
<tr>
<td>$V_{\text{dyn}}$</td>
<td>non-dimensional dynamic velocity</td>
</tr>
<tr>
<td>$V_{\text{dyn}}/V_{\text{ref}}$</td>
<td>non-dimensional velocity ratio</td>
</tr>
<tr>
<td>$V_{\text{ref}}$</td>
<td>non-dimensional static velocity</td>
</tr>
<tr>
<td>$v_p$</td>
<td>compressional wave velocity</td>
</tr>
</tbody>
</table>
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\( v_{\text{ref}}, v_{q-s} \)  
static penetration rate, i.e. 2 cm/s

\( v-s \)  
vane shear

\( v_0 \)  
dynamic penetration rate at which the strain-rate effect start to decay toward zero (initial penetration rate)

\( w \)  
maximum torque angle at failure (vane shear test)

WD  
water depth

\( x, z, z_{\text{depth}} \)  
penetration depth or depth below seabed

XBP  
eXpendable Bottom Penetrometer

\( \alpha \)  
slope angle

\( \beta \)  
soil-specific rate exponent for the power-law equation

\( \gamma \)  
MSCL total unit weight of the soil

\( \gamma' \)  
submerged unit weight of the soil

\( \gamma_{\text{balk}} \)  
MAD total unit weight of the soil

\( \gamma_{\text{it}} \)  
in situ total unit weight of the soil

\( \gamma_{\text{w}} \)  
unit weight of the water

\( \delta \)  
semipapex angle of the cone

\( \Delta H_{\text{inc}} \)  
height difference of a specimen between start and end of a loading increment

\( \Delta u \)  
excess pore pressure (=\( u-u_0 \))

\( \Delta u_1 \)  
excess pore pressure measured at the tip (=\( u_1-u_0 \))

\( \Delta u_2 \)  
excess pore pressure measured behind the tip (=\( u_2-u_0 \))

\( \Delta u_3 \)  
excess pore pressure measured behind the sleeve (=\( u_3-u_0 \))

\( \Delta u_{2,\text{dyn}} \)  
dynamic excess pore pressure measured behind the tip

\( \Delta u_{2,q-s} \)  
strain-rate corrected (quasi-static) excess pore pressure measured behind the tip

\( \Delta u_{2,\text{ref}} \)  
static excess pore pressure measured behind the tip and recorded at the reference penetration rate i.e. 2 cm/s

\( \Delta u_{\text{dyn}} \)  
dynamic excess pore pressure (=\( u_{\text{dyn}}-u_0 \))

\( \Delta u_{\text{ref}} \)  
static excess pore pressure recorded at the reference penetration rate i.e. 2 cm/s

\( \Delta \sigma_{\text{inc}}' \)  
effective stress difference related to the loading stage
\( \epsilon \) particular strain-rate
\( \epsilon_{ref} \) reference strain-rate
\( \Lambda \) reciprocal of the landslide displacement rate
\( \mu_{CPTU} \) soil-specific rate coefficient for the logarithmic equation
\( \mu'_{CPTU} \) soil-specific rate coefficient for the inverse sinh-hyperbolic equation
\( \mu^*_{CPTU} \) soil-specific rate coefficient for the velocity ratio equation
\( \mu_{CPTU,fs}, K_{fs} \) soil-specific rate coefficient for the sleeve friction
\( \mu_{CPTU,qc} \) soil-specific rate coefficient for the cone penetration resistance
\( \mu_{CPTU,qt}, K_{qt} \) soil-specific rate coefficient for the corrected cone penetration resistance
\( \mu_{CPTU,\Delta u} \) soil-specific rate coefficient for the excess pore pressure
\( \nu \) Poisson’s ratio
\( \omega \) moisture content or water content
\( \omega_L \) liquid limit
\( \omega_P \) plastic limit
\( \rho \) MSCL gamma density
\( \rho_{bulk} \) MAD bulk density
\( \rho_t \) in situ bulk density
\( \sigma \) internal stress
\( \sigma_{H0} \) total horizontal stress
\( \sigma_{OCT} \) total octahedral stress
\( \sigma'_P \) preconsolidation pressure
\( \sigma_{V0} \) total overburden stress
\( \sigma'_V \) effective overburden stress
\( \varepsilon \) internal strain
\( \varphi' \) effective friction angle
Appendix A

List of reports and publications

The following list summarizes all reports and publications, that I have prepared during the time of this doctoral project. This list includes all internal and external reports, which were used for collaborations between MARUM and other organizations (e.g. NGI, NGU, Ifremer), as well as manuscripts, which were submitted and/or published in conference proceedings book or in a scientific journal.
Table A.1: List of reports and publications during the doctoral project.

<table>
<thead>
<tr>
<th>Date</th>
<th>Type</th>
<th>Authors</th>
<th>Title and Remarks</th>
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</thead>
<tbody>
<tr>
<td>2010</td>
<td>external report</td>
<td>Steiner, A., Lange, M., Kopf, A.</td>
<td>Preliminary FF-CPTU results and analyses of R/V Seisma cruise at Finneidfjord; collaboration between MARUM and NGU, NGI.</td>
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<tr>
<td>2010</td>
<td>external report</td>
<td>Steiner, A., Lange, M., Kopf, A.</td>
<td>Preliminary FF-CPTU results and analyses of R/V Seisma cruise at Hommelvika; collaboration between MARUM and NGU, NGI.</td>
</tr>
<tr>
<td>2011</td>
<td>external report</td>
<td>Steiner, A., Lange, M., Kopf, A.</td>
<td>Preliminary FF-CPTU results and analyses of R/V Seisma cruise at Trondheim harbor; collaboration between MARUM and NGU, NGI.</td>
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<th>title and remarks</th>
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<td>paper</td>
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<td>report</td>
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Table A.1 – Continued

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<tr>
<td></td>
<td></td>
<td>Ferentinos, G., Fleischmann, T.,</td>
<td>Stability, Mud volcanism, Faulting and Fluid Flow in the Eastern <em>Mediterranean</em></td>
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<tr>
<td></td>
<td></td>
<td>Geraga, M., Kufner, S., Schlenzek, S.,</td>
<td>Sea (Cretan Sea, <em>Mediterranean</em> Ridge) and <em>Ligurian</em> Margin (Nice slope),</td>
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<tr>
<td></td>
<td></td>
<td>Steiner, A., Tryon, M., Wiemer, G.</td>
<td>Heraklion / Greece, 22.03.2012 - La Seyne sur Mer / France, 06.04.2012. Berichte</td>
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<td></td>
<td></td>
<td></td>
<td>aus dem Fachbereich Geowissenschaften der Universität Bremen, 286, 80 pp.</td>
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<tr>
<td>2012</td>
<td>internal report</td>
<td>K. Huhn, and cruise participants [Steiner, A.,</td>
<td>RV <em>MARIA S. MERIAN</em> Cruise MSM15. Biogeochemistry and methane hydrates of the <em>Black</em></td>
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<tr>
<td></td>
<td>for the cruise</td>
<td>Wiemer, G., Kopf, A.J., Zoellner, C., Ai, F.</td>
<td>Sea; Slides, deep water formation and seismicity of the <em>Mediterranean</em>, Limassol -</td>
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<tr>
<td></td>
<td>report MSM 15/3</td>
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<td>Istanbul - Piraeus - Valletta - Rostock, 07.04.2010 - 29.07.2010. Berichte aus dem</td>
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<td>Fachbereich Geowissenschaften der Universität Bremen.</td>
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<tr>
<td>2013</td>
<td>conference paper</td>
<td>Vanneste, M., Longva, O.,</td>
<td><em>Finneidfjord</em>, a field laboratory for integrated submarine slope stability</td>
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<td></td>
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<td>L'Heureux, J.-S., Vardy, M.E., Morgan, E.,</td>
<td>assessments and characterization of landslide-prone sediments: a review. In Offshore</td>
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<td></td>
<td></td>
<td>Forsberg, C.F., Kvalstad, T.J., Strout,</td>
<td>Technology Conference, Houston, Texas, OTC 130TC-P-686-OTC.</td>
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<td></td>
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<td>J.M., Brendryen, J., Hafidlasson, H.,</td>
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<td></td>
<td></td>
<td>Lecomte, I., Steiner, A., Kopf, A.J., Mörz,</td>
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<td></td>
<td></td>
<td>T., Kreiter, S.,</td>
<td></td>
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<td></td>
<td></td>
<td>Kreiter, S., Stegmann, S., Hafidlasson, H.,</td>
<td>reappraisal of strain-rate corrections. Canadian Geotechnical Journal, [<em>In Review</em>].</td>
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<td></td>
<td>Moerz, T.,</td>
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<th>type</th>
<th>authors</th>
<th>title and remarks</th>
</tr>
</thead>
</table>
Appendix B

*MATLAB* routine (digital)

The source code of the *MATLAB* routine *GeoDynamic-CPTU* is presented and saved on the doctoral thesis CD as digital file.
Appendix C

More data

Additional datasets, measured with the SWFF-CPTU and DWFF-CPTU instruments and not presented within a manuscript or in the ongoing project chapters, are illustrated hereafter. The datasets have to be regarded as raw data; hence, no detailed analyses and interpretations were performed.

C.1 Hommelvika

This report presents the preliminary results and findings of the in situ measurements conducted during the R/V Seisma cruise (Hommelvika, northern Norway) at July 05\textsuperscript{th}, 2010.

In total, seven in situ measurements were conducted using a self-contained, modular FF-CPTU instrument (Stegmann et al., 2006) developed at MARUM - Center for Marine Environmental Sciences, University of Bremen (Fig. C.1 and Table C.1).

The entire report is presented and saved on the doctoral thesis CD as digital file.
Figure C.1: Map of the Hommelvika study area (northern Norway) including the locations of the FF-CPTU measurements 01 to 07 (numbering is FF-CPTU xx).
Table C.1: List of FF-CPTU deployments carried out in the Hommelvika study area (northern Norway) at July 05th, 2010.

<table>
<thead>
<tr>
<th>Type of investigation (FFU)</th>
<th>LAT</th>
<th>LON</th>
<th>Water Depth</th>
<th>Rod Length</th>
<th>Start Velocity</th>
<th>Duration of measurement</th>
<th>Picture No.</th>
<th>Visual Penetration Depth</th>
<th>Permeability Depth</th>
<th>Geological assessment</th>
<th>Comments MM: D-Mag</th>
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<tbody>
<tr>
<td>FF-CPTU 01</td>
<td>70</td>
<td>61</td>
<td>35.55</td>
<td>22</td>
<td>0.58</td>
<td>0.55</td>
<td>4.0 - 4.5</td>
<td>2.7 - 2.25</td>
<td>n/v</td>
<td>6.0</td>
<td>1006001</td>
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<tr>
<td>FF-CPTU 02</td>
<td>70</td>
<td>61</td>
<td>35.55</td>
<td>22</td>
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<td>4.0 - 4.5</td>
<td>2.7 - 2.25</td>
<td>n/v</td>
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<tr>
<td>FF-CPTU 03</td>
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<td>61</td>
<td>35.55</td>
<td>22</td>
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<td>0.55</td>
<td>4.0 - 4.5</td>
<td>2.7 - 2.25</td>
<td>n/v</td>
<td>6.0</td>
<td>1006001</td>
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<tr>
<td>FF-CPTU 04</td>
<td>70</td>
<td>61</td>
<td>35.55</td>
<td>22</td>
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<td>0.55</td>
<td>4.0 - 4.5</td>
<td>2.7 - 2.25</td>
<td>n/v</td>
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<td>FF-CPTU 05</td>
<td>70</td>
<td>61</td>
<td>35.55</td>
<td>22</td>
<td>0.58</td>
<td>0.55</td>
<td>4.0 - 4.5</td>
<td>2.7 - 2.25</td>
<td>n/v</td>
<td>6.0</td>
<td>1006001</td>
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<tr>
<td>FF-CPTU 06</td>
<td>70</td>
<td>61</td>
<td>35.55</td>
<td>22</td>
<td>0.58</td>
<td>0.55</td>
<td>4.0 - 4.5</td>
<td>2.7 - 2.25</td>
<td>n/v</td>
<td>6.0</td>
<td>1006001</td>
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<tr>
<td>FF-CPTU 07</td>
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<td>61</td>
<td>35.55</td>
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<td>0.55</td>
<td>4.0 - 4.5</td>
<td>2.7 - 2.25</td>
<td>n/v</td>
<td>6.0</td>
<td>1006001</td>
</tr>
</tbody>
</table>

Notes:
- the deceleration sensor reaches the maximum limit of its range (e.g., a > 1.7g)
- a g-sensor is used for the analysis
- visual penetration depth file not with the calculated penetration depth (cause is not clear)
- no material is on the rods

C.2 Trondheim harbor

This report presents the preliminary results and findings of the in situ measurements carried out during the R/V Seisma cruise (Trondheim harbor, northern Norway) from July 06th to 07th, 2010.

In total, 28 in situ measurements were conducted using a self-contained, modular FF-CPTU instrument (Stegmann et al., 2006) developed at MARUM - Center for Marine Environmental Sciences, University of Bremen (Fig. C.2 and Table C.2).

The entire report is presented and saved on the doctoral thesis CD as digital file.
Figure C.2: Map of the Trondheim harbor study area ( northern Norway) including the locations of the FF-CPTU measurements 01 to 28 (numbering is FF-CPTU xx).
### Table C.2: List of FF-CPTU deployments carried out in the Trondheim harbor study area (northern Norway) at July 6th, 2010.

<table>
<thead>
<tr>
<th>Type of investigation</th>
<th>UTM / WGS ZONE</th>
<th>X / Y-bound</th>
<th>LAT</th>
<th>LONG</th>
<th>water depth</th>
<th>Rod Length</th>
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<th>Duration of measurement</th>
<th>Picture No.</th>
<th>IMGA-ax</th>
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<tr>
<td>FF-CPTU 01</td>
<td>575106.00</td>
<td>709811.00</td>
<td>67°27’23”</td>
<td>N 3°25’12”</td>
<td>E 68</td>
<td>4.00</td>
<td>3.76</td>
<td>20</td>
<td>371</td>
<td>2.5-2.5</td>
<td>sand/shale = 1.20 clay/silt/sand</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td></td>
<td>575106.00</td>
<td>709811.00</td>
<td>67°27’23”</td>
<td>N 3°25’12”</td>
<td>E</td>
<td>68</td>
<td>3.76</td>
<td>1</td>
<td>r/c</td>
<td>r/c</td>
<td>sand/shale = 1.20 clay/silt/sand</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>575106.00</td>
<td>709811.00</td>
<td>67°27’23”</td>
<td>N 3°25’12”</td>
<td>E 55</td>
<td>4.00</td>
<td>3.04</td>
<td>20</td>
<td>r/c</td>
<td>r/c</td>
<td>sand/shale = 1.20 clay/silt/sand</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>575106.00</td>
<td>709811.00</td>
<td>67°27’23”</td>
<td>N 3°25’12”</td>
<td>E</td>
<td>55</td>
<td>3.71</td>
<td>1</td>
<td>r/c</td>
<td>r/c</td>
<td>sand/shale = 1.20 clay/silt/sand</td>
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<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>575106.00</td>
<td>709811.00</td>
<td>67°27’23”</td>
<td>N 3°25’12”</td>
<td>E 36</td>
<td>4.00</td>
<td>4.11</td>
<td>20</td>
<td>r/c</td>
<td>r/c</td>
<td>sand/shale = 1.20 clay/silt/sand</td>
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<td></td>
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<tr>
<td></td>
<td>575106.00</td>
<td>709811.00</td>
<td>67°27’23”</td>
<td>N 3°25’12”</td>
<td>E 36</td>
<td>4.00</td>
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<td>N 3°25’12”</td>
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<td>r/c</td>
<td>r/c</td>
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<td>709811.00</td>
<td>67°27’23”</td>
<td>N 3°25’12”</td>
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<td>20</td>
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<td>sand/shale = 1.20 clay/silt/sand</td>
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<tr>
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<td>E 70</td>
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<td>sand/shale = 1.20 clay/silt/sand</td>
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**Notes:**
* The deceleration sensor reaches the maximum limit of its range (e.g., a > 1.7g)
* This g-sensor is used for the analysis
* Visual penetration depth file not with the calculated penetration depth (cause is not clear)
* No material is on the rods
### Table C.3: List of FF-CPTU deployments carried out in the Trondheim harbor study area (northern Norway) at July 7th, 2010.

<table>
<thead>
<tr>
<th>Type of investigation</th>
<th>UTM WGS 84 NAD 83</th>
<th>x-band</th>
<th>y-band</th>
<th>LAT</th>
<th>LONGS</th>
<th>water depth (m)</th>
<th>Rod Length</th>
<th>Start Velocity</th>
<th>Duration of measurement</th>
<th>Picture No</th>
<th>Visual penetration Depth</th>
<th>Termination depth</th>
<th>geological assessment</th>
<th>Comments NGU-S-log</th>
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<td>n/a</td>
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<td>3.9 3.5</td>
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<td>sand on top, clay</td>
<td>sand on top, clay</td>
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<td>3.9 3.5</td>
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<td>sand on top, clay</td>
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<td>4.0 4.5</td>
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<td>-21.11</td>
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<td>60</td>
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<td>1.5 2.0</td>
<td>1.7 × 2.56</td>
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<td>4.0 4.5</td>
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<td>60</td>
<td>4.00 5.06 20</td>
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<td>4.0 4.5</td>
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<td>sand on top, clay</td>
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<td>4.00 5.06 20</td>
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<td>n/a</td>
<td>1.7 × 2.15</td>
<td>1 0195-0196</td>
<td>4.0 4.5</td>
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<tr>
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<td>1.7 × 2.15</td>
<td>1 0200-204</td>
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<td>sand on clay</td>
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</table>

Note: 

a the deceleration sensor reaches the maximum limit of its range (e.g. a > 1.7g)

b this g-sensor is used for the analysis
c visual penetration depth file not with the calculated penetration depth (cause is not clear)
d no material is on the rods
C.3  Gela basin

The detailed core analyses, including Multi Sensor Core Logging (MSCL), moisture and density (MAD) and geotechnical measurements, are presented hereafter for all collected cores in the Gela basin area (southern Sicily, Italy) (see also section 5.1 for details and Ai et al. 2014).
**Figure C.3:** Detailed laboratory analyses of the *MARUM MeBo* core GeoB14403-1, 2, 8 based on Multi Sensor Core Logging (MSCL), moisture and density (MAD), fall cone penetration (fc) and vane shear (v-s) experiments.
Figure C.4: Detailed laboratory analyses of the gravity core GeoB 14404-1 based on Multi Sensor Core Logging (MSCL), moisture and density (MAD), fall cone penetration (fc) and vane shear (v-s) experiments.
Figure C.5: Detailed laboratory analyses of the MARUM MeBo core GeoB 14414-1 based on Multi Sensor Core Logging (MSCL), moisture and density (MAD), fall cone penetration (fc) and vane shear (vs) experiments.
Figure C.6: Detailed laboratory analyses of the MeBo core GeoB 14401-2, 3, 5 based on Multi Sensor Core Logging (MSCL), moisture and density (MAD), fall cone penetration (fc), and vane shear (v-s) experiments.
Figure C.7: Detailed laboratory analyses of the gravity core GeoB 14405-1 based on Multi Sensor Core Logging (MSCL), moisture and density (MAD), fall cone penetration (fc) and vane shear (v-s) experiments.
Figure C.8: Detailed laboratory analyses of the gravity core GeoB 14406-1 based on Multi Sensor Core Logging (MSCL), moisture and density (MAD), fall cone penetration (fc) and vane shear (v-s) experiments.
C.4 *Mediterranean* Ridge accretionary complex

Analyses and strain-rate corrections of the surficial *DWFF-CPTU* records collected in the vicinity of the *Mediterranean* Ridge accretionary complex (*Olimpi* field; southern Crete, Greece) (see also section 5.2 for details).
Figure C.9: Measured and corrected excess pore pressure at the tip ($\Delta u_1$) and 0.75 m behind the tip ($\Delta u_3$) for the mud volcano (MV) Ber gamo. Additionally, the derived intact undrained shear-strength of the $DWFF\text{-}CPTU$ measurement ($s_{u,\Delta u_1}$ and $s_{u,\Delta u_3}$) (GeoB15364-01) compared with the fall cone penetration and vane shear tests ($s_{u,fc}$ and $s_{u,v-s}$) performed aboard (GeoB15326) are illustrated. The green line represents the measured dynamic-CPTU parameters. The red, blue and orange lines show quasi-static CPTU parameters taking into account the empirical strain-rate correction solutions (e.g. Dayal and Allen 1975, Mitchell and Soga 2005, A. Steiner [unpublished data]).
Figure C.10: Measured and corrected excess pore pressure at the tip (Δυ₁) and 0.75 m behind the tip (Δυ₃) for the MV Bergamo. Additionally, the derived intact undrained shear-strength of the DWFF-CPTU measurement (sᵤ,Δυ₁ and sᵤ,Δυ₃) (GeoB 15364-01) compared with the fall cone penetration and vane shear tests (sᵤ,fc and sᵤ,v−s) performed aboard (GeoB 15363) are illustrated. The green line represents the measured dynamic-CPTU parameters. The red, blue and orange lines show quasi-static CPTU parameters taking into account the empirical strain-rate correction solutions (e.g. Dayal and Allen 1975, Mitchell and Soga 2005, A. Steiner [unpublished data]).
Figure C.11: Measured and corrected excess pore pressure at the tip ($\Delta u_1$) and 0.75 m behind the tip ($\Delta u_3$) for the MV Leipzig. Additionally, the derived intact undrained shear-strength of the DWFF-CPTU measurement ($s_{u,\Delta u_1}$ and $s_{u,\Delta u_3}$) (GeoB 15365-01) compared with the fall cone penetration and vane shear tests ($s_{u,fc}$ and $s_{u,v}$) performed aboard (GeoB 15332) are illustrated. The green line represents the measured dynamic-CPTU parameters. The red, blue and orange lines show quasi-static CPTU parameters taking into account the empirical strain-rate correction solutions (e.g. Dayal and Allen 1975, Mitchell and Soga 2005, A. Steiner [unpublished data]).
Figure C.12: Measured and corrected excess pore pressure at the tip ($\Delta u_1$) and 0.75 m behind the tip ($\Delta u_3$) for the MV Maidstone. Additionally, the derived intact undrained shear-strength of the DWFF-CPTU measurement ($s_u,\Delta u_1$ and $s_u,\Delta u_3$) (GeoB 15371-01) compared with the fall cone penetration and vane shear tests ($s_{u,fc}$ and $s_{u,v}-s$) performed aboard (GeoB 15370) are illustrated. The green line represents the measured dynamic-CPTU parameters. The red, blue and orange lines show quasi-static CPTU parameters taking into account the empirical strain-rate correction solutions (e.g. Dayal and Allen 1975, Mitchell and Soga 2005, A. Steiner [unpublished data]).
Figure C.13: Measured and corrected excess pore pressure at the tip ($\Delta u_1$) and 0.35 m behind the tip ($\Delta u_3$) for the MV Milano. Additionally, the derived intact undrained shear-strength of the DWFF-CPTU measurement ($s_u, \Delta u_1$ and $s_u, \Delta u_3$) (P 429: GeoB 16522-01) compared with the fall cone penetration and vane shear tests ($s_{u, fc}$ and $s_{u,v} - s$) performed aboard (P 410: GeoB 15362) are illustrated. The green line represents the measured dynamic-CPTU parameters. The red, blue and orange lines show quasi-static CPTU parameters taking into account the empirical strain-rate correction solutions (e.g. Dayal and Allen 1975, Mitchell and Soga 2005, A. Steiner [unpublished data]).
Figure C.14: Measured and corrected excess pore pressure at the tip ($\Delta u_1$) and 0.35 m behind the tip ($\Delta u_3$) for the MV Napoli. Additionally, the derived intact undrained shear-strength of the DWFF-CPTU measurement ($s_{u,\Delta u1}$ and $s_{u,\Delta u3}$) (P 429: GeoB 16524-01) compared with the fall cone penetration and vane shear tests ($s_{u,fc}$ and $s_{u,v-s}$) performed aboard (P 410: GeoB 15312) are illustrated. The green line represents the measured dynamic-CPTU parameters. The red, blue and orange lines show quasi-static CPTU parameters taking into account the empirical strain-rate correction solutions (e.g. Dayal and Allen 1975, Mitchell and Soga 2005, A. Steiner [unpublished data]).
C.5 Cretan Sea landslide east of Spatha Ridge

This geotechnical sub-chapter presents the preliminary results and findings of the in situ measurements conducted during the R/V Poseidon cruise P429 (northwestern Crete, Greece) and is part of the cruise report: Kopf, A.J. and cruise participants 2012. Report and preliminary results of RV Poseidon Cruise P429: MEDFLUIDS: Slope Stability, Mud volcanism, Faulting and Fluid Flow in the Eastern Mediterranean Sea (Cretan Sea, Mediterranean Ridge) and Ligurian Margin (Nice slope), Heraklion / Greece, 22.03.2012 - La Seyne sur Mer / France, 06.04.2012. Berichte aus dem Fachbereich Geowissenschaften der Universität Bremen, 286. Department of Geosciences, Bremen University.

A total of 26 shallow-water dynamic-CPTU (SWFF-CPTU) drops were carried out over the complete study area (Fig. C.1) in water depths between 150 and 440 m (Table C.4).

The SWFF-CPTU measurements are addressed to the following scientific goals:


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<th>GeoB18Sex</th>
<th>PositionLat</th>
<th>PositionLon</th>
<th>date</th>
<th>WD [m]</th>
<th>equipment</th>
<th>probe</th>
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Figure C.15: Map showing all SWFF-CPTU deployments during the R/V Poseidon cruise P429. The deployments are represented with the last two/four digits of the GeoB nomenclature.
C.5. CRETAN SEA LANDSLIDE EAST OF SPATHA RIDGE

Figure C.16: Seismic profile 55/56 along which the SWFF-CPTU equipment was used (Table C.4).

- *in situ* characterization of the surficial sub-seafloor sediments (sediment physical- and hydrological properties),
- comparison with seismic data and ground truthing (Fig. C.16; Ferentinos and Papatheodorou 1989),
- identification and characterization of surficial slide plans (Fig. C.16), and
- comparison of the SWFF-CPTU tests with gravity core data (vane shear [v-s] and fall cone penetration [fc] experiments) in respect of the strain-rate correction of the *in situ* datasets (Fig. C.17).

Most of the tests are deployed at the area where the slumped and slid material are located (western slope). In this area, only penetration depths of 0.5 to 1.5 m were achieved due to very high compaction/consolidation of the sediments. The cause is not clearly understood, right now. One reason is probably the high-rate of seismicity in the northern part of Crete, Greece (Papazachos and Papazachou 1977).

In this cruise report, we present three characteristic results along the seismic profile (Figs. C.18 to C.20). The first test is above the slumping/sliding area, the second one is inside of the slumping area and the tired test is below the slumping area (runout region).
When running down the western slope (Fig. C.16) an increase of the penetration depth was detected. This is directly correlated with the decrease of the $s_u$ of the sediments (Fig. C.18 to C.20). The $s_u$ varies between 20 and 45 kPa in the upper slope area and between 10 and 30 kPa in the lower slope region. All tests show a very high consolidation state with an undrained shear-strength ratio ($s_u/\sigma'_V$) higher than 2-3 (highly over-consolidated).

The comparison of the in situ measurements with the gravity core data illustrates similar trends in the sediment physical properties. But, the compar-
ison is very difficult due to disturbance of the upper portion of the gravity core sediments (coring process) and heterogeneity of the material (e.g. Fig. C.20). More data will be acquired during post-cruise geological/geotechnical laboratory measurements (standard- and advanced tests). The comparison with the seismic data will also be carried out in this phase (ground truthing).
Figure C.18: Derived in situ intact undrained shear-strength ($s_u$) calculated from the corrected cone resistance ($q_t$; upper plot) and calculated from the excess pore pressure ($\Delta u_1$; lower plot) for the test GeoB 16517-04. Additionally, the laboratory tests ($s_{u,fc}$ and $s_{u,v-s}$) are also illustrated (GeoB 16513). (Note: empirical cone penetration factor ($N_{kt}$) and empirical excess pore pressure factor ($N_{\Delta u}$). Both were used to calculate the $s_{u,qt}$ and $s_{u,\Delta u}$.)
Figure C.19: Derived in situ intact undrained shear-strength \(s_u\) calculated from the corrected cone resistance \(q_t\) (upper plot) and calculated from the excess pore pressure \(\Delta u\) (lower plot) for the test GeoB16517-15. Additionally, the laboratory tests \(s_{u,fc}\) and \(s_{u,v-s}\) are also illustrated (GeoB16516). (Note: empirical cone penetration factor \(N_{kt}\) and empirical excess pore pressure factor \(N_{\Delta u}\). Both were used to calculate the \(s_{u,qt}\) and \(s_{u,\Delta u}\).)
Figure C.20: Derived in situ intact undrained shear-strength ($s_u$) calculated from the corrected cone resistance ($q_c$; upper plot) and calculated from the excess pore pressure ($\Delta u_1$; lower plot) for the test GeoB 16517-22. Additionally, the laboratory tests ($s_{u,fc}$ and $s_{u,v-s}$) are also illustrated (GeoB 16505-2). **Note:** empirical cone penetration factor ($N_{kt}$) and empirical excess pore pressure factor ($N_{\Delta u}$). Both were used to calculate the $s_{u,qt}$ and $s_{u,\Delta u}$. 

![Image of Figure C.20](image-url)
Appendix D

Dynamic- and static-CPTU raw data

The dynamic- and static-CPTU raw data collected and used within this doctoral project are in preparation for upload to *Pangaea - Data Publisher for Earth & Environmental Science* (www.pangaea.de).
Appendix E

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 und

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

Bremen, den 07.11.2013

(Alois Steiner)