

A Laboratory Investigation of the Mechanical Properties and Behavior of Glauconite Sand

A thesis submitted by

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I. Abstract

This thesis presents a laboratory investigation into the mechanical behavior of Navesink glauconite sand, a highly evolved glauconite sand sampled from the Navesink Formation in Middletown, New Jersey. Glauconite sand exhibits a unique stress-dependent response, transitioning from granular to cohesive behavior due to particle crushing under load, introducing uncertainty in geotechnical classification and offshore wind monopile design. Piezocone penetration test (CPTu)-based models, such as Alm & Hamre (2001), often underpredict static resistance to driving (SRD) in glauconitic sands, contributing to excessive blow counts and potential pile refusal.

This study characterizes glauconite's crushing, compression, and permeability behavior as fines content increases. Malt mixing and axial compression produced fine-grained glauconite with varied gradations, with malt mixing generating more clay-sized particles. CRS tests showed decreasing permeability and increasing pore pressure beginning around 25–30 MPa. As fines increased, compression behavior shifted from granular to clay-supported, with higher compression indices (C_c , C_r , C_s). Secondary compression, tested at a range of stresses, peaked near 7 MPa for glauconite sand. Resedimented specimens with 25–100% fines showed greater compressibility, lower permeability, and excess pore pressures up to 1.7 MPa. A $CK_{\sigma UC}$ triaxial test on glauconite sand with 75% fine-grained glauconite yielded a friction angle of 30° and K_0 of 0.4.

These results suggest that crushing-induced permeability loss and increased compressibility may lead to undrained conditions during pile driving. Although limited to static 1D loading, the findings provide context field observations of high SRD and pile refusal in glauconite sands.

Future work should include drained and undrained triaxial tests to quantify shear strength and friction angles across fines contents, as well velocity measurements assess shear modulus degradation and damping behavior. Testing less evolved glauconite may reveal stronger crushing and more extreme hydraulic changes, improving drivability predictions for offshore monopile installation.

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VI. Symbols

ADC	Analog-to-Digital Converter
RBBC	Resedimented Boston Blue Clay
CPTu	Piezocone Penetration Test
CRS	Constant Rate of Strain
KSC	Kgf/cm ²
LVDT	Linear Voltage Displacement Transducer
MPa	Megapascals
OCS	Outer Continental Shelf
OSW	Offshore Wind
PVA	Pressure Volume Actuator
RBBC	Resedimented Boston Blue Clay
RGoM-EI	Resedimented Gulf of Mexico Eugene Island Clay
SBT	Soil Behavior Type
SCPTu	Seismic Piezocone Penetration Test
SRD	Soil static Resistance to Driving
TX	Triaxial
C _c	Compression Index
C _r	Recompression index
C _s	Swell Index
CK ₀ UC	K ₀ -Consolidated Undrained Shear Compression Test
e	Void Ratio
K	Lateral Stress Ratio <i>or</i> Permeability
K ₀	Coefficient of Lateral Earth Pressure at Rest
n	Porosity
NC	Normally Consolidated
OC	Overconsolidated

OCR	Overconsolidation Ratio
p'	Average Effective Stress $(s'_1 + s'_3)/2$
q	Shear Stress/Deviatoric Stress $(s'_1 - s'_3)/2$
s_u	Undrained Shear Strength
t	Time
u	Pore Pressure
q_c	Cone Tip Resistance
q_T	Net Cone Tip Resistance
f_s	Cone Sleeve Friction
N_c, N_q, N_y	Bearing Capacity Factors
q_{bc}	Bearing Capacity
τ_{sf}	Shaft Friction
ε_a	Axial Strain
ε_{vol}	Volumetric Strain
φ'	Effective Stress Friction Angle
δ	Interface Friction Angle
γ_t	Total Unit Weight
γ_w	Unit Weight of Water
ρ_t	Bulk Density
σ'_h, σ'_v	Horizontal, Vertical Effective Stress
σ'_p	Effective Preconsolidation Pressure
σ'_m	Mean Effective Stress $(s'_1 + 2s'_3)/3$
τ	Shear Stress

1 Introduction

1.1 Problem Statement

The mechanical properties and behavior of soils differ significantly between granular (cohesionless) and fine-grained (cohesive) materials. Broadly speaking —and ignoring important variables such as stress history, bulk density, and saturation — granular materials tend to be more permeable, less compressible, and stronger, while fine-grained materials tend to be less permeable, more compressible, and weaker. Glauconite sand is a granular material that can transition into a fine-grained material due to its tendency to crush under stress. While most sands are crushable only at very high stress levels, glauconite is especially prone to crushing due to its fissured microstructure and clay-like mineralogy.

The formation of fine-grained glauconite due to crushing poses several challenges for geotechnical engineers. First, glauconite cannot be reliably classified using CPTu-based Soil Behavior Type (SBT) charts (e.g., Robertson, 2016), as it behaves differently from typical sands or clays. Second, CPTu-based models for predicting static soil resistance to driving (SRD), such as Alm & Hamre (2001), tend to underestimate SRD in glauconite sands, which can lead to excessive blow counts or even pile refusal. Third, the mechanical properties of glauconite sand may evolve both during and after pile installation, increasing uncertainty in foundation performance and long-term behavior.

To address these challenges, this study aims to characterize the compression behavior of glauconite sand across a range of fines contents, and to quantify the generation of fines as a function of effective stress and time. However, this introduces a kind of geotechnical Catch-22: the very act of applying stress to measure behavior may itself generate additional fines, altering the system under observation. As a result, it becomes difficult to isolate the effect of initial fines content from the evolving nature of the material, complicating efforts to draw clear cause-and-effect relationships.

1.2 Scope and Objectives

The glauconite sand characterized in this research is a highly evolved sample obtained from the Navesink Formation in Middletown, New Jersey. The glauconite sand fraction was carefully separated from the surrounding fine-grained matrix. The primary goals of this research were to:

1. Characterize the properties of Navesink glauconite sand.
2. Characterize the crushing behavior of this sand and the nature of the fine-grained glauconite produced
3. Measure how increasing fines content affects compression behavior and permeability using CRS testing
4. Explain the relevance of these findings to offshore wind foundation engineering.

1.3 Organization

This thesis is organized into seven chapters, briefly outlined below:

Chapter 2 provides an overview of foundation engineering, with a focus on offshore wind monopiles, as glauconite sand deposits are found in Northeast Atlantic offshore wind lease areas. It discusses drivability and bearing capacity assessments from CPTu data, followed by a review of pile refusal mechanics and the geotechnical characteristics of glauconite sand.

Chapter 3 describes the origins of the Navesink glauconite sand and the sample preparation process. This includes mineralogical and geologic background, separation of the glauconite fraction, and preparation of resedimented specimens for CRS testing.

Chapter 4 overviews the laboratory equipment used in this study, including CRS and triaxial testing setups, transducers, pressure-volume actuators, DC motor controllers, the computer control system, the data acquisition system (DAQ), and the process for converting raw data into engineering parameters.

Chapter 5 presents the research methodology for each test. This includes index property tests comparing coarse- and fine-grained glauconite, a discussion of X-ray Powder Diffraction (XRPD) and scanning electron microscopy (SEM) to investigate mineralogy and microstructure, CRS testing (the core of the research), and triaxial testing (a minor component).

Chapter 6 discusses experimental results. It includes index, XRPD, and SEM results on both coarse and fine glauconite, observations of crushing behavior, and CRS test results on glauconite sand with varying fines contents. A CK₀UC triaxial test is also included. The chapter discusses variation in compression behavior, pore pressure response, permeability, and secondary compression as a function of fines content. Compression parameters (C_c , C_s , C_s , C_{ae}) are derived, and glauconite's properties are compared with in-house materials (e.g., RBBC and RGoM-EI). The transition from granular to cohesive behavior is a key theme.

Chapter 7 summarizes the main findings and their relevance to offshore wind foundation engineering. It highlights new contributions to the literature on glauconite particle crushing, permeability, and compressibility with increasing fines content. It also outlines possible directions for future research, including studies on undrained strength and friction angle as a function of fines content, dynamic properties and damping behavior, expanded CRS and permeability testing, numerical simulation of pile drivability, field comparisons, behavior of less evolved glauconite, and micromechanical modeling of particle-scale crushing.

2 Background

2.1 The Compressive Behavior of Sands

The crushing behavior of sand has long been a key concern in the oil and gas industry, particularly regarding reservoir compaction, permeability changes, and wellbore stability. In the 1990s, deepwater hydrocarbon exploration in regions such as the Shetland Islands (U.K.), the Gulf of Mexico, Angola, and Brazil led to the discovery of oil and gas reservoirs in uncemented sands and weak sandstones. These materials were typically tested under high pressure, as the reservoirs are located several kilometers below the seabed.

The primary concern in these reservoirs was compressibility, as hydrocarbon extraction leads to pore pressure reduction, increasing the effective stress in the sediment. Many of these reservoirs are overpressured, meaning pore pressures exceed hydrostatic levels. Laboratory experiments were conducted to understand how these stress changes affected permeability and compression behavior (Coop and Wilson, 2003).

In 1995, Pestana and Whittle developed a compression model for granular materials subjected to hydrostatic and one-dimensional compression. Their findings demonstrated that sand specimens, regardless of initial density, converge to a unique normal compression line (NCL) or limiting compression curve (LCC). The LCC, which appears linear in a double-logarithmic void ratio-effective stress space, represents irrecoverable plastic strains due to particle sliding and rolling at low stresses and particle crushing at high stresses (Figure 2.1). This concept has since been widely adopted in constitutive models to characterize the compressive behavior of sands.

Most sands, like the silica sands reported by Pestana and Whittle (1995), are composed of quartz and feldspar, which rank 7 and 6, respectively, on the Mohs hardness scale, a metric used to compare mineral hardness on a scale of 1 to 10. This indicates that silica sands have sufficient strength to resist particle crushing under most engineering stresses. However, under extreme loading conditions, silica sands can undergo crushing, reducing them to silt-sized ($<75\mu\text{m}$) or even clay-sized ($<2\mu\text{m}$) particles. At these finer scales, silica particles can develop weak negative surface charges due to the deprotonation of silanol (Si-OH) groups. This increased specific surface area can lead to some degree of cohesion, although silica-derived fines behave very differently from true clay particles due to fundamental mineralogical differences.

Clays, in contrast, develop much stronger negative charges due to isomorphous substitution in their crystal structure. In illites, for example, Al^{3+} replacing Si^{4+} in the tetrahedral sheet creates a net negative charge that must be neutralized by adsorbed K^{+} cations. This leads to the formation of a double-layer of cations, anions, and bound water molecules that prevent particle contact (Figure 2.2). This electrostatic behavior governs the mechanical response of clay-rich soils under low to moderate effective stresses, where repulsive forces and bound water films inhibit direct particle contact, distinguishing them from finely crushed silica sands, where particle interlocking dominates even at low stresses.

2.2 Glauconite Sand: A Unique Granular Material

Glauconite sand exhibits distinct mechanical behavior compared to conventional silica sands. Mineralogically, glauconite is a clay, as it belongs to the illite group (López-Quirós et al., 2020), but texturally, it is often found in sand-sized grains. Unlike typical quartz-dominated sands, glauconite sand has a remarkably low Mohs hardness of 2 to 3 (Deer et al. 1992).

Due to its low hardness, glauconite grains crush at relatively low stresses, releasing silt- and clay-sized particles composed primarily of glauconite minerals. The exact stress levels required for crushing depend on particle age and diagenetic maturity, as older glauconite grains often undergo chemical alteration that influences their strength (Odin & Matter 2003). This high crushability fundamentally alters the compressive behavior of glauconite sands, setting them apart from the silica sands studied by Pestana and Whittle (1995) and Coop and Wilson (2003).

2.2.1 Geotechnical Implications of Glauconite Sands

Glauconite sands are found throughout the world, most notably in New Jersey and Belgium. In geotechnical practice, glauconite sand is strongly associated with pile refusal, a condition where driven piles encounter extreme resistance, preventing further penetration (De Nijs et al., 2015; Perikleous et al. 2023). This behavior poses significant challenges for both onshore and offshore foundation design, as excessive hammering during pile installation can cause pile fatigue or structural failure.

The offshore wind industry, which predominantly relies on driven monopile foundations, has encountered extensive glauconite deposits on the Northwest Atlantic Outer Continental Shelf, where at least 16 major offshore wind farms are under development. Given the mechanical complexity of glauconite sands, understanding their compression behavior, particle breakage characteristics, and implications for pile drivability is essential for mitigating installation risks in offshore wind farm construction.

2.2.2 Research Motivation

Although the oil and gas industry has long recognized the unique properties of glauconite sands, geotechnical research on this material has been historically limited. However, the growing offshore wind sector has renewed interest in glauconite due to its impact on monopile drivability. As a result, multiple research projects are now focused on characterizing glauconite sands and pile-soil interactions.

The Tufts Advanced Geomaterials (TAG) Lab has provided a unique opportunity to investigate these issues. The lab is equipped to test soils under stresses ranging from 0.1 to 100 MPa, making it well-suited for evaluating glauconite's behavior under high-stress conditions relevant to offshore wind foundations. The TAG Lab is also part of the GeoFluids Consortium, a research partnership with the University of Texas at Austin that specializes in experimental studies and poromechanical modeling of stress, deformation, and fluid flow in mudrocks from the Gulf of Mexico. This collaboration, along with the TAG Lab's advanced testing capabilities, have provided a unique

opportunity to conduct geotechnical research on glauconite sands relevant to offshore wind development.

Offshore wind plays a critical role in the energy transition of the Northeastern United States, making the geotechnical behavior of glauconite sands not just a scientific concern but an industry imperative. To contextualize these challenges, the following sections examine the state of the U.S. offshore wind industry, key principles of foundation engineering and drivability analysis, the mechanics of pile refusal, and existing research on the geotechnical properties of glauconite sands.

2.3 U.S. Offshore Wind: State of the Industry

The Northeastern United States—including Massachusetts, Vermont, Maine, Connecticut, Rhode Island, New York, and New Jersey—has committed to legally binding decarbonization goals, aiming to phase out fossil fuel-based electricity generation within varying timelines (Massachusetts EOEEA, 2022; NYSERDA, 2023). Despite differences in policy approaches, these states share a reliance on a diverse electricity mix, which in New England primarily includes natural gas (52%), nuclear (27%), hydropower imports (10%), and solar (5%) (ISO-NE, 2023).

Electricity procurement in the region is managed through long-term power purchase agreements (PPAs) and the wholesale electricity market, overseen by ISO New England, which is responsible for maintaining grid reliability and assessing future energy infrastructure needs (ISO-NE, 2023). Long-term projections by ISO-NE indicate that electricity demand is expected to increase by nearly 75% by 2050, driven by the electrification of heating and transportation (EIA, 2024). To meet this demand while achieving decarbonization targets, offshore wind is projected to supply between 40% and 60% of total electricity needs by mid-century (BOEM, 2023).

The importance of offshore wind in the Northeast's energy transition stems from a combination of geographic constraints, regulatory challenges, and infrastructure limitations. Unlike other regions, the Northeast has negligible fossil fuel resources, high population density, and low solar irradiance, making onshore wind and solar expansion difficult due to land-use conflicts and permitting restrictions (Mai et al., 2022). Additionally, state and federal restrictions on expanding natural gas infrastructure have constrained new pipeline development, contributing to some of the highest electricity prices in the country, ranging from \$80 to \$120/MWh (EIA, 2023; ISO-NE, 2023).

Offshore wind presents a unique advantage—it is located near major coastal population centers yet remains out of sight, minimizing land-use conflicts. Additionally, offshore wind has a higher capacity factor than solar and onshore wind due to more consistent wind speeds, reducing intermittency concerns (NREL, 2023). As a result, state and federal policies increasingly view offshore wind as essential to meeting both decarbonization targets and long-term electricity demand (BOEM, 2023; NYSERDA, 2023).

The offshore wind industry is facing significant challenges, including rising interest rates, supply chain constraints, and an increasingly difficult regulatory environment. A memorandum issued by the White House on January 21, 2025, froze further federal permitting of offshore wind projects,

indefinitely delaying 10 of the 16 projects under development across the North Atlantic Outer Continental Shelf (Figure 2.3). At the state level, Northeastern states remain committed to their legally binding decarbonization goals, yet the future of offshore wind, an essential component of their energy transition strategy, remains uncertain. While the U.S. offshore wind industry faces setbacks, global deployment has continued, driven by technological advancements and supportive policies in Europe and Asia. Realizing the full potential of offshore wind in the U.S. will require long-term policy stability, investments in supply chains, and continued innovation to drive down costs and improve project feasibility.

2.4 Challenges in Offshore Wind Foundation Engineering

Monopiles are the dominant foundation type in the offshore wind industry, accounting for approximately 75% of existing turbines (NREL, 2024). As turbine capacities increase, reaching 15 MW and beyond, the industry has responded by increasing monopile diameters and penetration depths to meet higher load demands while minimizing material use (Alm & Hamre, 2001). These next-generation foundations can reach up to 10 meters in diameter and 90–120 meters in length. They are driven up to 60 meters below the seafloor (Arany et al., 2017). Monopiles are designed to resist complex cyclic loading from wind, waves, and turbine operation, and are typically installed using large hydraulic impact hammers. For example, the Menck MHU 4400 kJ hammer, which stands nearly 30 meters tall, delivers 220-ton ram blows to drive the monopile to its target depth (DNV, 2023). See Figure 2.4 for a depiction of monopile installation.

To reduce costs, monopiles are designed with high diameter-to-thickness (D/t) ratios, sometimes exceeding 120, making them slender and more susceptible to fatigue during installation (DNV, 2020; Acteon, 2023). When excessive soil resistance is encountered during driving, particularly in dense or problematic soils like glauconite sand, the resulting high blow counts can compromise structural integrity and reduce the overall service life of the foundation (BOEM, 2023). As offshore wind development expands into geologically complex seabeds, a thorough understanding of soil–monopile interaction and drivability risks is essential. The following sections explore the determination of pile capacity and the prediction of soil static resistance to driving (SRD), leading into a focused discussion of pile refusal and the challenges posed by glauconite sands.

2.5 Determination of Bearing Capacity of Pile Foundations

The capacity of driven piles depends on the shear strength of the surrounding soil, which varies based on whether the soil behaves under drained or undrained conditions. Because hydraulic hammers used for monopile installation can exceed 120 blows per minute, pile driving in clay typically occurs under undrained loading conditions. In such cases, undrained shear strength is the critical parameter for calculating pile capacity.

Randolph and Gourvenec (2011) describe the pile driving process in clay in four stages: (1) initial undrained loading, (2) development of friction fatigue along the pile shaft, (3) dissipation of excess pore pressures following installation, and (4) foundation loading (Figure 2.5). As the pile is driven into the clay, the soil beneath it is displaced outward, or in the case of an open-ended monopile,

may flow inward to accommodate the pile's advance (Figure 2.6). This results in large shear strains and an incremental change in total stress, $\Delta\sigma_v$:

$$\Delta\sigma_v = \Delta u + \Delta\sigma'_v \quad (2.1)$$

Where Δu is the change in pore pressure due to loading and $\Delta\sigma'_v$ is the change in effective stress due to shearing. In soft, normally consolidated clays, $\Delta\sigma'_v$ is negative, due to contraction, while in stiff over consolidated clays, it is positive due to dilation (Figure 2.7).

As the pile penetrates deeper, a residual shear plane may form adjacent to the shaft due to excess pore pressure buildup. This phenomenon, known as friction fatigue, causes the shaft resistance to decrease with depth as cyclic loading progressively weakens the soil (Heerema, 1978). Once installation is complete, effective stress changes as excess pore pressures dissipate, a process known as pile setup, which occurs at a rate proportional to the clay's permeability.

During pile installation, shaft friction (τ_{sf}) is mobilized along the pile. It is determined from the local horizontal effective stress at failure, σ'_{hf} , the interface friction angle, δ , and the cohesion intercept, c , using the Coulomb failure criterion (Figure 2.8):

$$\tau_{sf} = \sigma'_{hf} \tan\delta + c \quad (2.2)$$

The interface friction angle (δ) depends upon the pile wall's texture and composition and typically ranges from 0.5 to $0.8 \cdot \phi'$, where ϕ' is the soil's internal friction angle. From τ_{sf} , the ultimate shaft resistance, Q_{sf} , is obtained by intergrating the shaft friction over the the pile length:

$$Q_{sf} = \pi D \int_0^L \tau_{sf} dz \quad (2.3)$$

Where D and L are the diameter and length of the pile.

Accurate estimation of τ_{sf} is cricial for drivability and shaft resistance analyses. Two common methods for estimating shaft resistance are the Alpha and Beta methods.

The Alpha Method, used for cohesive materials, correlates shaft friction (τ_{sf}) with undrained shear strength (s_u):

$$\tau_{sf} = \alpha s_u \quad (2.4)$$

The alpha method differentiates between normally consolidated (NC) and over consolidated (OC) clays because their contractive versus dilative behavior affects the shear stress (Rodríguez Piedrabuena, 2024). The alpha values are determined from the strength ratio (s_u / σ'_{v0}) as follows:

For soft NC clays, ($s_u < \sigma'_{v0}$)

$$\alpha = \frac{1}{2} \left(\frac{s_u}{\sigma'_{v0}} \right)^{-1/2} \quad \tau_{sf} = \frac{1}{2} \sqrt{s_u \sigma'_{v0}} \quad (2.5)$$

For stiff OC clays ($s_u > \sigma'_{v0}$)

$$\alpha = \frac{1}{2} \left(\frac{s_u}{\sigma'_{v0}} \right)^{-1/4} \quad \tau_{sf} = \frac{1}{2} s_u^{0.75} \sigma'_{v0} \quad (2.6)$$

The Beta Method, used for cohesionless soils, involves expressing the shaft friction, τ_{sf} , as a function of effective stress:

$$\tau_{sf} = \beta \sigma'_{v0} = K \sigma'_{v0} \tan \delta = \sigma'_{hf} \tan \delta \quad (2.7)$$

Where K is the lateral stress coefficient $\left(\sigma'_h / \sigma'_v \right)$, which varies based on pile type and installation method (see Table 1 for typical values).

In addition to shaft resistance, the pile's base resistance contributes to its overall capacity. In undrained conditions, where the soil's friction angle is assumed to be zero, the base bearing capacity (q_{bf}) is given by:

$$q_{bf} = 9s_u \quad (2.8)$$

Where 9 is the empirical bearing capacity factor, N_c (Skempton, 1951). The ultimate base resistance for a closed-ended pile, Q_{bf} , is then:

$$Q_{bf} = \frac{\pi D^2}{4} q_{bf} \quad (2.9)$$

Pile driving in sand occurs under drained loading conditions (Figure 2.9). The shaft friction is using the beta method (Equation 2.7), while the bearing capacity (q_{bf}) is calculated as:

$$q_{bf} = N_q \sigma'_{v0} \quad (2.10)$$

Where N_q is a dimensionless bearing capacity factor (Skempton, 1951).

Having established the fundamental equations governing the bearing capacity of driven piles in sand and clay, the next section will discuss how these parameters are derived from in situ testing techniques and used to predict soil static resistance to driving (SRD) and assess the risk of pile refusal.

2.6 Prediction of SRD from In Situ Testing Data

Offshore wind site investigation campaigns typically involve extensive geophysical surveying, piezocone penetration testing (CPTu), and the collection of soil samples for laboratory testing. These data are synthesized into a ground model that characterizes soil conditions across the proposed wind farm site. This model enables engineers to assess drivability risks and ensure each foundation meets its design requirements. Standard practice requires CPTu data collection at each turbine location, with regulations such as those set by BOEM mandating site-specific investigations.

CPTu data are commonly correlated with soil parameters such as undrained shear strength, s_u , effective friction angle, ϕ' , or interface friction angle, δ , to estimate SRD for steel monopiles. However, Alm and Hamre (2001) introduced a method to predict SRD directly from CPTu parameters, drawing on data from over 150 pile installations. Their approach correlates key SRD parameters—shaft friction (τ_{sf}), residual shaft friction (τ_{sres}), and bearing capacity (q_{bf})—with CPTu parameters like net cone tip resistance (q_T) and sleeve friction (f_s).

To develop their model, the authors back-calculated SRD from blow count data for each pile installation using GRLWEAP, a wave equation-based program that generates continuous SRD profiles. These profiles were used to calibrate their CPTu-based model.

The equation for side friction (τ_{sf}) along the pile during driving is given by:

$$\tau_{sf} = \tau_{sres} + (f_{si} - \tau_{sres}) \cdot e^{k \cdot (d-p)} \quad (2.11)$$

where d is the depth to the soil layer, and p is the penetration distance through the layer.

For clays, the initial side friction (f_{si}) corresponds to the recorded CPT sleeve friction, while residual friction (τ_{sres}) is related to the normalized net cone tip resistance (q_T) by:

$$\tau_{sres} = 0.004 \cdot q_T (1 - 0.0025 \cdot q_T / p'_0) \quad (2.12)$$

Where p'_0 is the initial vertical effective stress.

For sands, the initial friction (f_{si}) was derived from the static friction equation:

$$f_{si} = K \cdot p'_0 \cdot \tan \delta \quad (2.13)$$

which is identical to Equation 2.2. However, the lateral stress coefficient (K) is linked to (q_T) as follows:

$$K \cdot p'_0 = 0.0132 \cdot q_T \cdot (p'_0/p_a)^{0.13} \quad (2.14)$$

where p_a is the atmospheric pressure. The residual side friction (τ_{sres}) was best approximated as 20% of f_{si} .

The degradation of initial friction in different soils is described by the shape factor (k):

$$k = (q_T/p'_0)^{0.5}/80 \quad (2.15)$$

which determines the rate of rate of friction fatigue and is used in Equation 2.11 (see Figure 2.9).

For pile tip resistance, the bearing capacity (q_{bf}) in clays was found to be approximately 60% of the total cone resistance (q_T), whereas in sands, it follows the following relationship:

$$q_{bf} = 0.15 \cdot q_T \cdot (q_T/p'_0)^{0.2} \quad (2.16)$$

The pile tip resistance typically ranges from 0.35 to 0.55 times q_T depending on whether the sand is loose or dense.

The complete SRD profile for a given pile is calculated by summing the side friction along the pile shaft and the tip resistance at the base. A factor of safety of 1.25 is recommended. It is important to note, however, that the Alm and Hamre (2001) approach does not account for total driving resistance, as it excludes damping resistance, discussed briefly in later sections.

2.7 The Nature of Pile Refusal

Having established the methods for predicting soil resistance to driving (SRD) and bearing capacity, it is necessary to examine conditions that can lead to pile refusal. Driven piles are designed to mobilize side friction at the soil-pile interface (Figure 2.10). As the pile penetrates deeper, side friction increases with horizontal effective stress. Once the pile reaches the target depth, its total capacity, P_n , is calculated as the sum of end-bearing resistance and shaft friction (Coduto, 2016):

$$P_n = q'_{bf}A_t + \sum \tau_{sf}A_s \quad (2.17)$$

Where q'_{bf} is the net end-bearing resistance after accounting for pile weight, A_t is the pile toe area, τ_{sf} is the net side friction, and A_s is the pile shaft surface area.

Pile refusal occurs when soil strength is underestimated, leading to incorrect assumptions of q'_{bf} and τ_{sf} , preventing the pile from reaching the required depth. Three primary mechanisms contributing to pile refusal are considered here: pile setup effects, negative pore pressures, and damping effects.

2.7.1 The Mechanics of Pile Setup

Pile setup refers to the time-dependent increase in pile capacity that occurs after driving ceases. This phenomenon results from three primary mechanisms: consolidation, secondary compression, and thixotropic hardening.

Consolidation behavior is governed by Terzaghi's effective stress principle:

$$\sigma_v' = \sigma_v - u \quad (2.18)$$

Where σ_v' is effective vertical stress, σ_v is total stress, and u is pore water pressure. During pile driving, excess pore pressures develop as load transfers to the incompressible water phase. Dissipation of these pressures over time increases effective stress, enhancing side friction f_s , which is related to effective stress by:

$$f_s = K \sigma_v' \tan \delta \quad (2.19)$$

where K is the lateral stress ratio and δ is the interface friction angle.

Secondary compression refers to gradual deformation under constant effective stress, caused by particle rearrangement and molecular-scale changes in inter-particle forces (Figure 2.11). Thixotropic hardening, in contrast, describes a time-dependent increase in soil strength at constant volume due to the reformation of inter-particle bonds in remolded soil (Figure 2.12). The rate and magnitude of pile setup vary based on soil permeability, drainage conditions, lithology, and stress state. Figure 2.13 presents pile setup data across different soil types, highlighting the complexity of this phenomenon.

2.7.2 Effects of Negative Pore Pressures

Negative pore pressures (suction) are commonly observed during undrained shearing of dense sands and over-consolidated clays. During pile driving, rapid loading forces particles in the shear zone to displace, affecting pore pressure behavior based on soil type (Salgado & Jafari, 2003).

In loose to medium-dense sands, silts, and soft clays, undrained shearing generates positive excess pore pressures, reducing effective stress and lowering resistance to driving. Conversely, in dense sands and over-consolidated clays, interparticle interlocking and bonding contribute to dilation. As the pile advances, void spaces increase without a corresponding rise in pore water volume, generating negative pore pressures that enhance effective stress. This effect is described by:

$$\sigma_v' = \sigma_v - (u + \Delta u) \quad (2.20)$$

where Δu represents additional pore pressure changes. Negative pore pressures increase shaft friction but dissipate over time, making their contribution to SRD temporary.

2.7.3 Damping Along the Pile Shaft

Energy dissipation during pile driving is influenced by two distinct mechanisms: (1) material damping within the soil, and (2) wave transmission efficiency at the pile-soil interface.

In wave equation models, material damping is captured by a dashpot coefficient, C , defined as:

$$C = \alpha V_s \quad (2.21)$$

Where V_s is shear wave velocity and α is an empirical coefficient dependent on soil type. The forces acting on the pile shaft and at the toe are expressed as:

$$F_s = R_s + CV_{shaft} \quad (2.22)$$

$$F_t = R_t + CV_{toe} \quad (2.23)$$

where F_s and F_t are forces along the shaft and at the pile toe, R_s and R_t are static resistances, and V_{shaft} and V_{toe} are the local pile velocities.

Soils with lower wave velocities, particularly fine-grained soils like clays, often exhibit greater hysteretic and viscous damping due to their higher plasticity and viscous behavior. As a result, more of the hammer's energy can be dissipated along the shaft as velocity-dependent resistance, reducing the stress magnitude that reaches the pile toe.

Separately, wave transmission efficiency is affected by the acoustic impedance mismatch between the pile and the surrounding soil. Acoustic impedance is the product of mass density and wave velocity. A greater mismatch leads to higher energy reflection at the pile-soil interface, regardless of material damping. While softer materials like water or soft clay may have lower impedances, the reflection of stress waves depends on the relative impedance difference, not absolute values. Stiffer soils (e.g., dense sands or rocks) better match the impedance of the pile, allowing more efficient transmission of energy, though they can resist penetration due to higher static resistance.

In monopile installations in the North Sea and Mid-Atlantic regions, clay-rich soils have been observed to cause increased blow counts and reduced driving efficiency, attributed to both higher material damping and impedance mismatches (Anusic & Jardin, 2016). Similar mechanisms may occur in glauconite sands, which, under driving stresses, tend to crush into fine-grained material, potentially increasing both the soil damping and the static resistance (Westgate et al., 2024).

The mechanics of particle crushing during pile driving in silica and calcareous sands have been studied by Mao et al. (2020) and Peng et al. (2020). During pile driving, a one-dimensional compression zone forms beneath the pile toe. Beneath the compression zone, soil undergoing intense shearing and lateral displacement to accommodate penetration (Figure 2.14). The highest degree of particle crushing occurs within the shear zone, where extreme shear strains cause particles to grind and fracture. The crushed material flows around the pile, forming a band of fragmented particles along the pile shaft.

Westgate et al. (2024) conducted field-scale pile driving tests in glauconite sand deposits in New Jersey, finding that the crushed material band along the pile sidewall ranged from 0.5 to 0.8 times the pile wall thickness. Given the unique crushability and fine-grained nature of glauconite sand, further investigation into its damping and transmission characteristics is essential for predicting energy dissipation during installation and optimizing driving parameters.

2.8 Consequences of Pile Refusal

Offshore wind monopiles are designed to withstand cyclic loading from wind, waves, and rotor blade rotations while minimizing steel usage. However, excessive blow counts during installation can have a detrimental impact on their structural integrity. This section outlines the implications of pile refusal on monopile performance.

Offshore wind structures are generally designed with a *soft-stiff* approach, where a stiff foundation supports a flexible tower. This design ensures that the structure avoids resonance with key dynamic loads, including wind, wave, and the frequencies associated with rotor (1P) and blade passing (3P) motion. The natural frequency of a soft-stiff monopile typically falls between the 1P and 3P frequencies, as shown in Figure 2.15.

Under these cyclic loads, alternating tensile and compressive stresses are induced in the steel, leading to progressive fatigue damage over time. This process is accelerated by the harsh offshore environment, where chemical corrosion, especially in areas exposed to wave breaking and sea spray, further weakens the structure by concentrating stresses in vulnerable regions. To mitigate these effects, cathodic protection systems and corrosion-resistant materials are often implemented to extend the monopile's service life.

Minimizing fatigue damage is crucial for ensuring the long-term performance of offshore wind monopiles.

The impact of pile refusal on structural capacity is best evaluated through fatigue analysis. Steel fatigue is typically characterized using S-N curves, which relate the number of cycles to failure (N) at a given stress level (S). Figure 2.16 illustrates a representative S-N curve.

Offshore wind monopiles are commonly driven using hydraulic hammers, typically with a 4,400 kJ capacity, and 6,000 kJ hammers are currently under development. A simple method to approximate the force exerted on the steel during a refusal scenario involves dividing the maximum hammer energy by the average penetration per blow. To convert this force into stress, it is divided by the monopile's cross-sectional area. For a typical monopile with a 10-meter diameter and a 10 cm wall thickness, the number of hammer blows experienced during refusal can be estimated by dividing a typical penetration depth of 50 meters by the average penetration per blow, which in refusal conditions might range from 0.5 to 2 cm.

The percentage of steel fatigue capacity consumed during a refusal scenario can be calculated using the following equation:

$$\text{Accumulated Fatigue} = \frac{N_b}{N_f} \times 100\% \quad (2.24)$$

Where:

N_b is the number of hammer blows at the given average penetration depth,

N_f is the number of cycles to failure at the corresponding stress level, as defined by the S-N curve.

Figure 2.17 quantifies the effects of refusal on monopile fatigue.

In the case shown in Figure 2.17, where excessive SRD leads to 9% fatigue damage, one can estimate the impact on the turbine's service life. Assuming a service life of 30 years (the typical lease length provided by BOEM), 9% fatigue damage corresponds to a 2.7-year reduction in service life. This reduction can lead to a significant loss in future revenue.

For a 15 MW turbine, the revenue loss can be estimated as follows:

$$\text{Revenue Loss} = \text{Rated Capacity} \times \text{Annual Operating Hours} \times \text{Capacity Factor} \times \text{Price of Electricity} \quad (2.25)$$

Given:

Rated capacity = 15 MW,

Number of hours per year = 8,760,

Capacity factor = 50% (approx.)

Price of electricity = \$140/MWh

A 2.7-year reduction in service life corresponds to a loss of approximately \$24.8 million in future revenue. For comparison, a 1% fatigue reduction would lead to a \$2.75 million loss.

Although these calculations are based on undiscounted cash flows, they clearly demonstrate the financial implications of encountering multiple refusal or SRD cases in an offshore wind farm.

Understanding the relationship between pile refusal and fatigue is critical for optimizing monopile design and installation practices. Excessive blow counts during installation can significantly reduce the fatigue life of a monopile, highlighting the importance of balancing installation efficiency with long-term structural integrity to minimize future financial losses.

2.9 Mechanical Properties and Behavior of Glauconite Sands

Having discussed the nature of pile capacity, SRD prediction, pile refusal, and the relationship between pile refusal and structural fatigue, this section returns to the issue of glauconite sands. It is important to note that refusal is not a problem specific to glauconite sand. Piles driven through dense soil layers, glacial till, boulders, and stiff over-consolidated clay can also experience refusal. The primary causes of refusal include inadequate site investigation, improper pile-hammer configuration, and a lack of understanding of soil mechanics.

Given these considerations, a thorough review of the literature on glauconite sand and glauconite-soil-pile interaction is essential. Understanding how and when glauconite transitions from a granular to a cohesive material is critical in assessing its influence on SRD predictions and dynamic resistance to driving. This knowledge can provide valuable insights into the unique challenges posed by glauconite sands in offshore foundation engineering.

2.9.1 Crushing Behavior of Glauconite Sand

Glauconite sand is distinct due to its crushing behavior and mineralogy. Under stress, glauconite particles fracture, releasing silt- and clay-sized particles composed of glauconite mineral. The stress at which glauconite particles crush varies based on their strength, which is influenced by their age (Westgate et al., 2023). Over timescales ranging from 100,000 to 1 million years, glauconite undergoes diagenetic alteration, during which the original source material—likely marine fecal pellets—hardens as it becomes enriched with potassium and iron oxide (Obasi et al., 2011) (Figure 2.18). Consequently, older glauconite deposits tend to be stronger and less susceptible to crushing (Westgate et al., 2023).

Mass transport processes that sort particles by size and density can also influence glauconite deposit composition. Authigenic (in situ) glauconite deposits typically contain weaker sand particles and a higher proportion of interparticle fines, while allogenic (re-deposited) glauconite deposits tend to have stronger, more uniform particles (Westgate et al., 2023).

Laboratory testing of glauconite crushing behavior has primarily been conducted using malt-mixer machines. Originally designed for making milkshakes, these machines shear particles in a water slurry by agitating them with a rotating metal blade, rupturing some of the weaker grains. Westgate et al. (2023) subjected glauconite sands to different durations of malt-mixing and measured the resulting particle size distributions (Figure 2.19). The study found that significant crushing and clay generation occurred within the first 1 to 30 minutes of malt-mixing. Notably, glauconite from the Hornerstown Formation, which is less diagenetically evolved, generated up about 65% clay after 30 minutes, whereas the stronger, more evolved Navesink Formation produced about 5% clay after 30 minutes. These results underscore the role of diagenetic alteration in glauconite sand crushing behavior.

Although the malt-mixing method does not establish a direct relationship between compressive or shear stress and particle crushing, it provides a useful means of relative comparison. A field-scale study by Westgate et al. (2024) investigated the feasibility of piling in glauconite sand and developed a methodology for sampling crushed glauconite material adhered to the pile wall. Grain-size analysis of the adhered material was compared to core samples that had been subjected to different durations of malt-mixing. The findings indicated that the crushed material closely resembled samples malt-mixed for 1 to 10 minutes (Figure 2.20). The study also observed that fines content increased along the shaft as the pile was driven, suggesting a relationship between shear strain and particle crushing. Additionally, a thinner, more degraded zone was noted immediately adjacent to the pile, highlighting the challenge of directly measuring glauconite crushing.

2.9.2 Index Tests, Mineralogy, and Microscopy

Index tests were performed to compare the properties of glauconite sand with those of other geomaterials. These tests included specific gravity, minimum and maximum void ratio, and Atterberg limits, which assess the plasticity of fine particles. This section also presents the mineralogical composition and microstructure of glauconite sand as determined by scanning electron microscopy (SEM). The discussion centers on two glauconite sands examined by Westgate et al. (2023): Crosswicks Hornerstown sand and Poricy Navesink sand. The authors

used a magnetic separation and sieving procedure to separate the glauconite particles from the non-glauconitic material, and a glauconite content of 95% was obtained. The Crosswicks Hornerstown and Poricy Navesink sands are slightly evolved and highly evolved glauconite sands, respectively, allowing for comparisons based on the degree of diagenetic alteration. Additional data for other sands are provided in tables at the end of the chapter.

Glauconite sand exhibits a particularly high specific gravity due to its elevated iron oxide content, ranging from 2.97 for Crosswicks Hornerstown samples to 3.04 for the more evolved Poricy Navesink samples. Minimum and maximum void ratios range from 0.85 to 1.21 for Poricy Navesink and from 1.23 to 2.18 for Crosswicks Hornerstown (Table 2).

Atterberg limit tests were conducted on both Crosswicks Hornerstown and Poricy Navesink sands after artificial degradation using malt mixing or wet ball milling. Following one hour of malt mixing and washing through a No. 40 (425 μm) sieve, the liquid limit and plasticity index for Crosswicks Hornerstown sand were 90 and 56, respectively. For Poricy Navesink sand, the corresponding values were 26 and 8 (Table 3). According to the Casagrande Plasticity Chart, these values classify the degraded Crosswicks Hornerstown fines as high-plasticity clay (CH) and the Poricy Navesink fines as low-plasticity silt (ML) (DeGroot et al., 2023). Similarly, the Sewell Hornerstown sand, a slightly evolved to evolved glauconite sand, was observed to transition from silty sand (SM) to high-plasticity clay (CH) after one hour of malt mixing (Westgate et al., 2023b). These findings illustrate the strong dependency of plasticity on particle degradation, highlighting the challenges of classifying glauconite sands using the Unified Soil Classification System (USCS).

X-ray fluorescence (XRF) analysis was used to determine the chemical composition of glauconite sands in terms of oxide weight percentages. Crosswicks Hornerstown sand was found to contain 21.9% iron oxide (Fe_2O_3) and 5.7% potassium oxide (K_2O), while Poricy Navesink sand contained 26.9% iron oxide and 7.0% potassium oxide. These results are consistent with the findings of Obasi et al. (2011), which indicate that glauconite sands become increasingly enriched in Fe_2O_3 and K_2O with age. A complete breakdown of oxide concentrations is presented in Table 4.

Representative images of Crosswicks Hornerstown and Poricy Navesink sands within the No. 20 (850 μm) and No. 40 (425 μm) sieve ranges are shown in Figure 2.21. The Crosswicks Hornerstown sand, representing a less evolved material, displays a rough, fissured surface texture, while the highly evolved Poricy Navesink sand appears smoother and more intact. However, SEM imaging is limited in that it does not reveal the internal microstructure of the grains or reveal the specific mechanisms by which glauconitic clay is released during particle crushing.

2.9.3 Shear Strength of Glauconite Sand

Thixotropic hardening tests, which measure time-dependent increases in undrained shear strength at constant effective stress, offer insights into pile setup effects observed in the field. DeGroot et al. (2023) prepared three specimens of slightly to moderately evolved Sewell Hornerstown glauconite sand. One specimen was malt-mixed for 10 minutes, while the other two

underwent 1 hour of malt mixing. The 10-minute specimen was prepared at a water content equal to the liquid limit (42%). The 1-hour specimens were prepared at water contents equal to the liquid limit (75%) and at 75% of the liquid limit (56%).

The rate of thixotropic hardening observed was significant (Figure 2.22) and was comparable to, or greater than, the hardening rates of Boston Blue Clay and Prestige Kaolin (Figure 2.23). The underlying physico-chemical mechanisms driving this behavior remain unclear. Translating these laboratory results to field conditions is challenging, as the tested water contents were much higher than the in situ water content of the natural soil (approximately 16%). The broader implications for pile setup in glauconite sands will be discussed in later sections.

Westgate et al. (2023) investigated the effect of particle crushing and fines content on the drained shear strength of evolved Sewell Hornerstown glauconite sand using direct shear and interface direct shear tests in accordance with ASTM D3080. While direct shear tests have known limitations—including stress nonuniformities, inadequate boundary conditions, and limited ability to measure residual strength (Potts et al., 1987; Bishop et al., 1971)—they can still offer useful relative comparisons of shear strength.

Specimens were malt-mixed for 10 minutes, 30 minutes, 1 hour, and 8 hours, then normally consolidated to 150 kPa and sheared under drained conditions. Shear strength was observed to decrease with increased particle crushing (Figure 2.24). The residual stress ratio (τ/σ'_{v0}) declined from 0.6 for the natural specimen—a typical value for sands and slightly higher than the 0.53 ratio observed in Ottawa 20–30 silica sand—to 0.32 for both the 1-hour and 8-hour malt-mixed specimens, a representative value clays.

All glauconite specimens exhibited contractive behavior during shearing, in contrast to the dilative response and peak shear strength observed in Ottawa 20–30 sand. More extensively crushed glauconite specimens also showed increasing strain softening. Interface direct shear tests performed against a sand-blasted steel surface yielded similar trends: increasing fines content led to increased strain softening. The peak stress ratio declined from 0.45 in the natural specimen to 0.34 after 1 hour of malt mixing, compared to 0.49 for Ottawa sand. Residual stress ratios for the natural glauconite sample dropped from 0.38 to 0.21 with crushing. These values corresponded to a peak internal friction angle (ϕ) of 31° and an interface friction angle (δ) of 24° for the non-degraded Sewell Hornerstown sand. The residual interface friction angle ranged from 13° to 21° , depending on the degree of degradation. Full details of the test conditions and results are presented in Tables 5 and 6.

The interface friction angles reported by Westgate et al. (2023) are consistent with findings from Quinteros et al. (2023), who conducted a parametric study on 13 glauconite sands using a Bishop-NGI ring shear apparatus. These sands, with glauconite contents ranging from 34% to 93%, spanned a wide range of fines contents. While the study did not explicitly characterize the degree of diagenetic alteration (e.g., slightly evolved to highly evolved), the reported particle colors, from green to black, suggest a broad maturity range. The full database of interface friction angles and corresponding test specifications is provided in Table 7.

DeGroot et al. (2023) also compared the undrained shear behavior of reconstituted samples of authigenic Sewell Hornerstown glauconite sand and allogenic Poricy Navesink sand. The highly evolved Poricy Navesink particles, being allogenic, are both stronger and more uniform in size than the Sewell Hornerstown particles. Specimens were prepared at 95% relative density, saturated, consolidated, and in some cases unloaded to an overconsolidation ratio (OCR) of 4 before being sheared undrained using a direct simple shear (DSS) device.

The two sands exhibited notably different behaviors. Normally consolidated (NC) Sewell Hornerstown specimens showed contractive behavior, while NC Poricy Navesink specimens transitioned to dilative behavior. The Poricy Navesink samples converged to a shear stress ratio (τ_h/σ'_v) of 0.54, while the Sewell Hornerstown samples converged to 0.50. The cause of this difference is not yet fully understood but may relate to differences in gradation, particle roundness, or iron content. Over consolidated (OC) specimens of both sands exhibited dilative behavior (Figure 2.25).

2.9.4 Stress Dependency of Glauconite Crushing Behavior

As discussed in Section 2.1, measuring the compression behavior of granular materials is essential for evaluating the stress dependency of particle crushing. The crushability of granular materials depends on particle strength, size, uniformity, and angularity. Emidio et al. (2009) performed constant rate of strain (CRS) oedometer tests on Belgian glauconite sand up to 30 MPa. They found that glauconite sand was more compressible than quartz sand but less compressible than carbonate sand. The material's compressibility was directly related to the degree of crushing, as indicated by the crushing coefficient B_t (Figure 2.26). B_t is calculated as follows:

$$B_t = \sum \Delta F \quad (2.26)$$

Where ΔF is the difference between the initial and final percentage passing for each sieve size on a particle size distribution (PSD) curve after breakage.

Because it is difficult to isolate factors such as particle strength, size, and angularity, the authors also conducted tests on pasta. They observed that particle strength increased, and crushing decreased, with smaller and more uniform particles (Figure 2.27).

Beringen et al. (1982) measured the volumetric strains of Belgian glauconite sand under isotropic compression at stresses up to 30–40 MPa. In these tests, the specimens exhibited volumetric strains and crushing coefficients that were twice as high as those of calcareous sand deposits from India and Australia. Joustra and De Gijt (1982) suggested that the observed differences in compression behavior, as seen in the work of Emidio et al. (2009) and Beringen et al. (1982), could be due to variations in gradation.

More recently, Westgate et al. (2022) conducted oedometer tests on poorly graded Poricy Navesink glauconite sand. The specimens were prepared at two concentrations (54% and 100%). Note that the Poricy Navesink sample with 54% glauconite content contained other minerals like quartz, plagioclase, K-feldspar, calcite, dolomite, siderite, and chlorite. The specimens were

compressed up to 4 MPa. Grain-size analyses before and after testing revealed negligible changes in fines content (Figure 2.28). Notably, the Navesink sand is an evolved glauconite sand, meaning that 4 MPa may be lower than the particle tensile strength, which governs particle crushing behavior.

As mentioned earlier, measuring fines generation during shear-induced crushing of glauconite sand is challenging due to the shear band being localized along the failure plane. However, significant evidence suggests that particle crushing is more pronounced under shear than under compression. Van Alboom et al. (2012) conducted torsional shear testing on glauconite sand and observed greater crushing and an increase in fines content (up to 29%) as a function of stress level. Similar increases in fines content were observed in soil samples taken from an auger following tunnel boring operations (Westgate et al. 2022). De Nijs et al. (2015) measured an increase in fines content from 13% in natural soil to 27% in the soil adhering to the side of an extracted sheet pile during a retaining wall installation through glauconite sands in Antwerp, Belgium. Van Raak (2009) noted that the fines content in the shear plane increased from 5% to 20% during direct shear testing of Belgian glauconite sand under normal stresses of 200 kPa.

2.9.5 In Situ Testing and Piling in Glauconite Sand

Glauconite crushing during cone penetration testing (CPT) significantly affects the CPTu parameters. Offshore glauconite deposits are characterized by high cone tip resistance (q_t) and high sleeve friction ratios (F_r), where F_r is defined as $[f_s/(q_t - \sigma_v)] \times 100\%$ (De Beer 1974; Van Raak 2009; Van Abloom et al. 2012; Rogiers et al. 2017; Long et al. 2019). The cone tip resistance can vary greatly, from less than 10 MPa in loose sands to over 50 MPa in dense sand. Friction ratios typically range from 3 to 7%, but they can be as high as 10% or more (Long et al. 2019; Westgate et al. 2024). This combination of high cone tip resistance (typical of sands) and high friction ratios (typical of clays) is increasingly used to identify glauconite sands, which cannot be accurately classified using typical soil behavior type charts (e.g., Robertson 2016).

Alba Rodríguez (2024) collected CPTu data from laboratory tests on Belgian glauconite sand. The samples were prepared at a relative density of 60-80% in a large chamber 450 mm deep. Based on the concentration of K₂O, the only reliable indicator of glauconite maturity, most glauconite particles were in the nascent stage of maturity. However, cone tip resistances over 30 MPa and excess pore pressures exceeding 35 kPa, about ten times the hydrostatic pressure, were encountered. These results suggest significant particle crushing and a transition from granular to cohesive (undrained) behavior.

During cyclic CPTu tests, negative pore pressures were observed. Once cyclic loading stopped, these negative pore pressures were quickly dissipated, indicating silty clay behavior. A zone of high particle breakage was observed under the cone tip and along the shaft, where fines increased by 20-40%.

Westgate et al. (2024) conducted CPTu tests through New Jersey glauconite sand, encountering resistances as high as 200 kN, leading to refusal. The authors successfully used cyclic water jetting to reduce resistance during CPTu data collection.

Turning to the issue of piling in glauconite sand, Westgate et al. (2024) installed nine different piles through Hornerstown and Navesink glauconite sands. They compared the resistance of open-ended and closed-ended piles, as well as piles with and without shoes, in an effort to reduce shaft friction. The authors conducted a preliminary pile driving analysis (PDA) using the GRLWEAP software, which suggested that pile driving would be possible with the selected hammers (9 tons for the 0.32-meter diameter piles and 20 tons for the 1.5-meter diameter piles).

However, several of the 0.32-meter piles encountered refusal and were compromised due to extremely high blow counts (exceeding 100 blows per quarter meter). Pile buckling occurred in some cases where pile setup and re-strike was allowed. The authors observed negligible pile setup after one hour and significant pile setup after two days, ranging from 30-100% for the 0.32-meter diameter piles with multiple external shoes and no external shoes. The 1.5-meter diameter pile showed resistance, increasing by 60-70% after a 3-4 day setup period. This amount of pile setup is consistent with the degree of thixotropic hardening observed in the laboratory.

The authors used the Case method, a continuous method for computing static and dynamic resistance during driving, comparing it to predictions made using the Alm and Hamre (2001) model based on CPTu data (Figure 2.29). The results show that modeling glauconite as a sand significantly underpredicts both the static and dynamic resistance to driving. The clay model, based on CPTu parameters, provides a better match to the calibrated Case model (the measured data). A hybrid model using both clay and sand models for shaft friction and base resistance, respectively, showed similar agreement to the clay model for both shaft friction and base resistance.

This is consistent with the observations of Alba Rodríguez (2024), who noted similar or greater fines generation at the pile toe compared to the pile shaft.

Perikleous et al. (2023) studied the installation of offshore wind monopiles in glauconite sand. The authors analyzed driving resistance for piles driven through North Sea soils with glauconite contents ranging from 5-20%. Modeling the material as a silty sand, as it would be classified from a grain size analysis, led to significant underprediction of driving resistance due to the higher shaft friction caused by glauconite crushing. In some layers, recorded blow counts were more than double the predicted values, underscoring the importance of even small amounts (5-20%) of glauconite in affecting driving resistance.

Additionally, the maturity of glauconite particles was found to be a significant factor. Deposits with less evolved, greener particles led to underpredictions in total resistance, while deposits with more evolved, dark particles behaved as expected. The authors recommend modeling driving resistance using the Alm and Hamre (2001) clay model, which directly incorporates the CPT shaft friction value (f_s) into the calculation. This approach provided more accurate predictions of total driving resistance (Figure 2.30).

Table 2.1: Typical values for the lateral earth pressure coefficient, K , for different types of piles (Source: Dr. Jen's CEE146 Course Notes).

Mansur and Hunter (1970) (pile tests)			
H piles		$K_E =$	1.4 to 1.9
Pipe piles			1.2 to 1.3
Precast square concrete piles			1.45 to 1.6
Timber (1 test)			1.25
Tension (8 tests)			0.4 to 0.9
Tavenas (1971)			
H piles		$K =$	0.5
Precast concrete			0.7
Tapered timber			1.25
Tension			0.6
Lambe and Whitman (1969) - Table 33.1			
Summarizes tests and theory for horizontal stress on piles driven in sand. $K = 0.3$ to 3 .			
Kulhawy Notes for ADSC (1987)			
Jettied pile		$K/K_o =$	$1/2$ to $2/3$
Drilled shaft, cast-in-place			$2/3$ to 1
Driven pile, small disp.			$3/4$ to $1\frac{1}{2}$
Driven pile, large disp.			1 to 2

Table 2.2: Index properties of New Jersey glauconite sands (source: Westgate et al. 2023).

Soil unit	Glauconite content (%)	Specific gravity		Minimum dry density $\rho_{d, \min}$ (g/cm ³)	Maximum void ratio ^a $e_{\max}$	Maximum dry density $\rho_{d, \max}$ (g/cm ³)	Minimum void ratio ^a $e_{\min}$
		Oven-dried (110 degC)	As-received moist				
Crosswicks	95.5 ^b	2.97	2.96	0.93	2.18	1.33	1.23
Hornerstown							
Sewell	98.1 ^b	3.01	3.02	1.14	1.65	1.49	1.02
Hornerstown							
Crosswicks	92.6 ^b	2.94	2.94	0.93	2.15	1.36	1.17
Navesink							
Poricy Navesink	~95 ^c	3.04	NP	1.37	1.21	1.64	0.85

Notes: NP = not possible due to initially dry as-sampled state; ^a oven-dried specific gravity values were used to calculate $e_{\min}$ and $e_{\max}$; ^b performed on coarse-grained fraction of air-dried core samples taken at depth; ^c performed on coarse-grained fraction of natural outcrop sample.

Table 2.3: Atterberg limits and clay activity of crushed glauconite sand (source: Westgate et al. 2023).

Soil unit	Mix duration	Liquid limit (%)	Plastic limit (%)	Plasticity index (%)	Clay fraction ^d (%)	Clay activity
Crosswicks Hornerstown	–	43	29	14	21	0.67
	1 hr ^c	90	34	56	54	1.2
Sewell Hornerstown	–	34	25	9	13	0.69
	10 min	51	25	26	46	0.57
	30 min	69	33	36	56	0.64
	1 hr ^c	78	36	42	62	0.68
	8 hr ^a	84	40	44	81	0.54
Crosswicks Navesink	–	32	24	8	3	2.7
	1 h	66	25	41	6	6.8
Poricy Navesink	–	NP	NP	NP	<1	NP
	1 hr ^c	26	18	8	8	1.0
	8 hr ^b	43	30	13	27	0.48

Notes: NP = non-plastic; NA = not available; ^a 8 h dispersion; ^b 8 h wet ball milling; ^c after passing No. 40 sieve; ^d air-dried values.

Table 2.4: Chemical composition (% oxides by weight) of New Jersey glauconite sands (Westgate et al. 2023)

Formation	SiO	Al ₂ O ₃	Fe ₂ O ₃	SO ₃	MgO	K ₂ O	CaO	P ₂ O ₅
Crosswicks Hornerstown	54.3	14.1	21.9	0.33	2.46	5.7	ND	0.54
Sewell Hornerstown	48.4	8.6	24.5	1.42	3.03	8.4	2.61	2.60
Crosswicks Navesink	50.6	14.9	19.0	6.72	2.11	5.2	0.11	0.71
Poricy Navesink	50.0	8.9	26.9	0.50	3.04	7.0	1.87	1.29

Notes: ND = not detected.

Table 2.5: Direct shear test conditions and results (Westgate et al. 2023)

Soil unit	Mix duration	Dry density (kg/m ³)	Normal stress (kPa)	Shearing rate (mm/min)	Peak stress ratio (–)	Peak friction angle (deg)	Residual stress ratio (–)	Residual friction angle (deg)
Sewell Hornerstown	–	1489	150	0.1	0.60	31	0.60	31
	10 min	1520	150	0.005	0.55	30	0.52	28
	30 min	1309	150	0.005	0.47	25	0.37	20
	1 h	1154	150	0.005	0.41	22	0.32	18
	8 h	1083	150	0.005	0.43	23	0.32	18
Ottawa 20–30 sand	–	1713	150	0.1	0.73	36	0.53	28

Table 2.6: Interface direct shear test conditions and results (Westgate et al. 2023)

Soil unit	Mix duration	Dry density (kg/m ³)	Normal stress (kPa)	Shearing rate (mm/min)	Peak interface stress ratio (–)	Peak interface friction angle (deg)	Residual interface stress ratio (–)	Residual interface friction angle (deg)
Sewell Hornerstown	–	1467	150	0.1	0.45	24	0.38	21
	10 min	1515	150	0.005	0.38	21	0.27	15
	30 min	1296	150	0.005	0.37	20	0.23	13
	1 h	1184	150	0.005	0.34	19	0.21	12
	8 h	1098	150	0.005	0.41	22	0.22	13
Ottawa 20–30 sand	–	1756	150	0.1	0.49	26	0.45	24

Table 2.7: Interface friction angles of glauconite sand of vary maturity, glauconite and fines content (Source: Quinteros et al. 2023)

Soil ID	Soil				Colour	Interface roughness (R_{CLA})		Fast pre-shearing		Slow shearing		Interface friction angle	
	D_{50} (mm)	C_c (-)	FC* (%)	GC† (%)		R_{CLA} before (μm)	R_{CLA} after (μm)	σ'_v (kPa)	Shear rate (mm/min)	σ'_v (kPa)	Shear rate (mm/min)	δ_p (°)	δ_{res} (°)
G1	0.10	N/A	36	85	greyish green	10.2	4.7	900/400	500	400	0.05	21.1	21.2
G2	0.25	3.4	29	85	light green and very dark greyish green	8.9	4.9	900/397	500	397	0.05	20.6	20.2
G3	0.25	9.3	23	77	light green to dark green	9.9	5.0	900	500	400	0.05	14.0	13.5‡
G4	0.32	2.5	23	93	dark green	9.9	5.0	900	500	400	0.05	17.2	16.7‡
G5	0.20	10.2	20	11	very dark green to greenish	9.4	6.8	900	500	199	0.05	28.8	28.4
G6	0.13	2.7	28	47	dark green	10.4	4.8	900	500	200	0.05	26.7	27.0
G7	0.34	6.3	15	N/A	very dark green	9.3	5.3	98	500	109	0.5	29.7	27.9
G8	0.21	0.4	21	N/A	very dark greenish grey / greyish green	9.9	8.0	82	500	82	0.5	22.8	22.8
G9	0.25	2.5	22	N/A	very dark greenish grey / greyish green	11.0	7.3	86	500	86	0.5	27.5	25.2
G10	0.25	1.4	17	N/A	very dark grey / greyish grey	10.0	8.2	110	500	110	0.5	25.6	25.6
G11	N/A	N/A	N/A	58	greenish	9.0	5.9	141	500	153	0.5	31.4	25.2
G12	N/A	N/A	N/A	46	greenish	10.0	6.5	116	500	97	0.5	27.9	25.5
G13	N/A	N/A	N/A	34	dark black	9.7	3.8	539	500	566	0.5	31.8	27.5

* FC = Fines content, 63 μm after ISO, † GC = Glauconite content, ‡ Pre-crushed soil

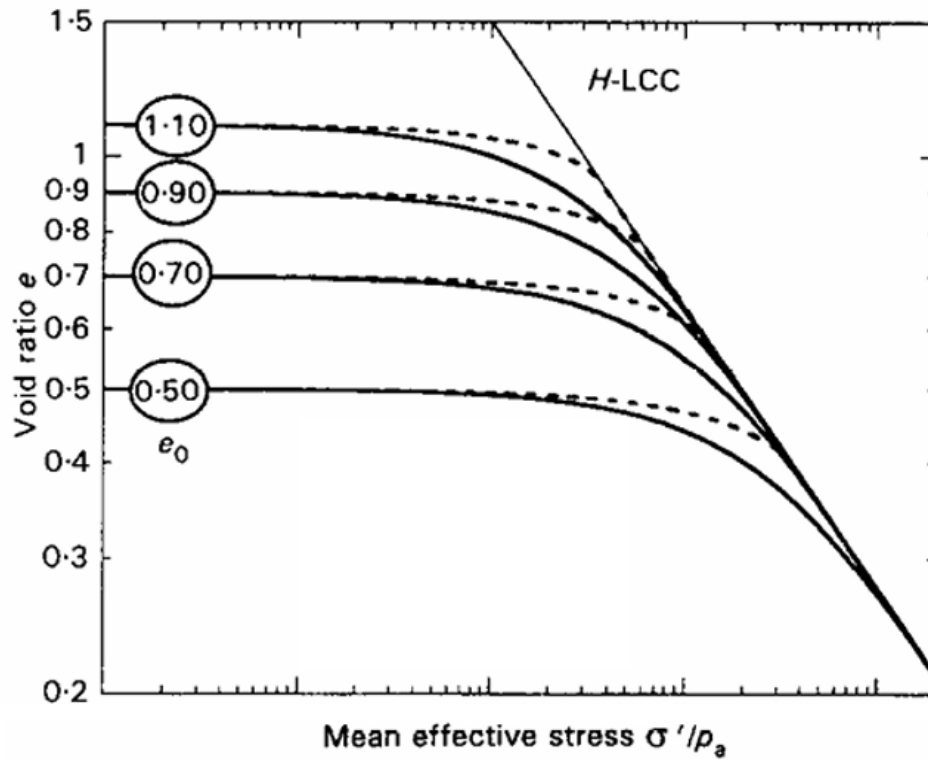


Figure 2.1: A plot of the compression behavior of silica sands prepared at different starting void ratios, e , in double-logarithmic void ratio mean effective stress space. Regardless of their starting void ratio, these sands converge upon a limiting compression curve that is used in constitutive models (Figure adapted from Pestana & Whittle, 1995).

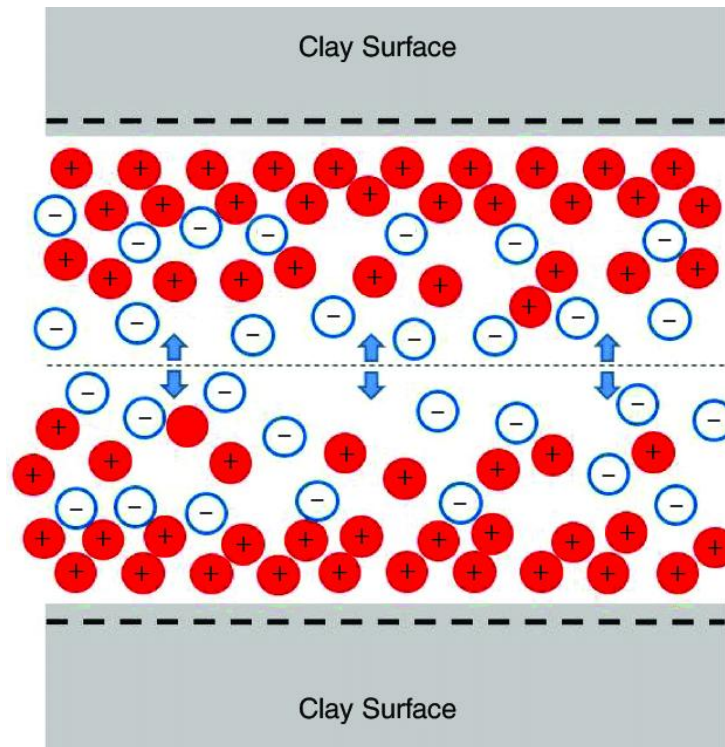


Figure 2.2: A depiction of the diffuse double layer between clay particles. Note that negatively charged clay particles attract cations which attract polar water molecules, creating a diffuse double layer that prevents the clay particles from directly touching.

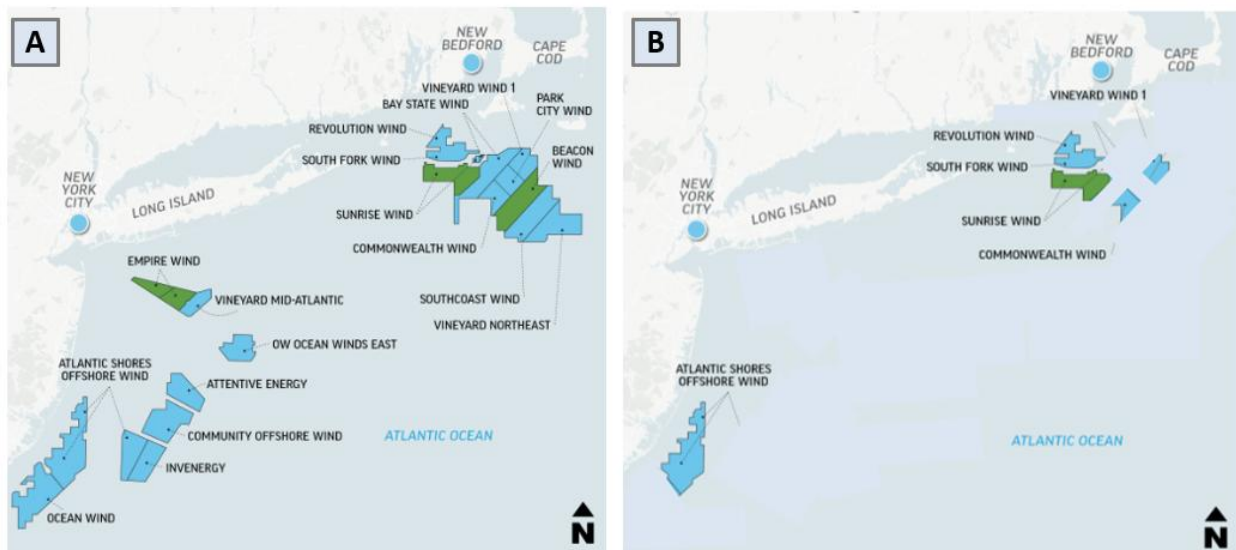


Figure 2.3: (A) Map of 18 offshore wind projects under development in the Northeastern United States. (B) Map of offshore wind projects with federal permitting. Lease areas with confirmed glauconite deposits are colored green (Modified from the New Bedford Light).

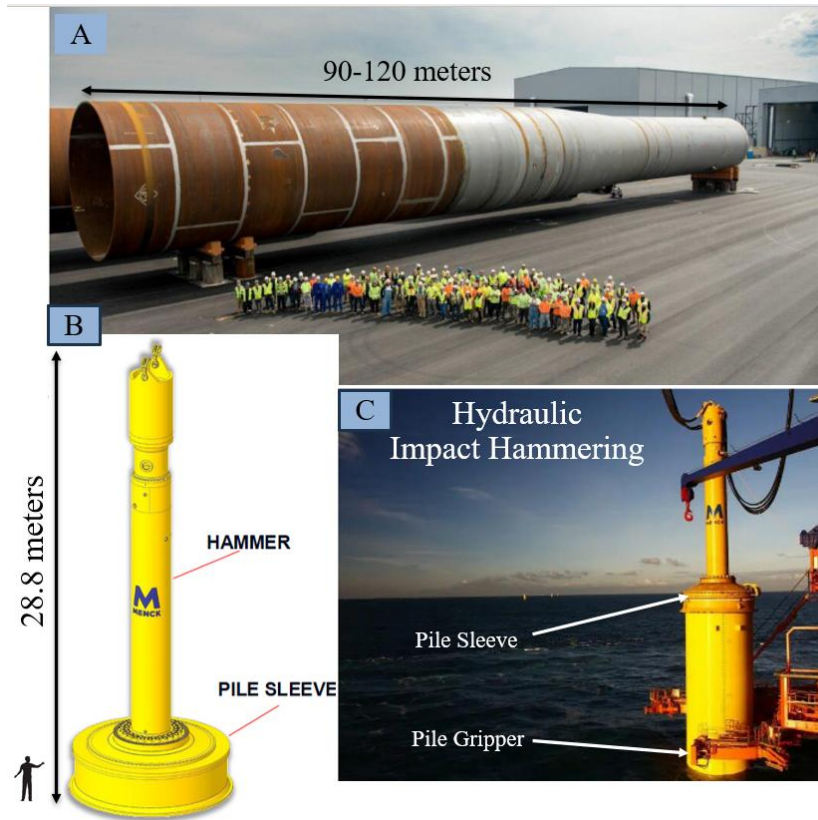


Figure 2.4: A photo of an offshore wind monopile (A) and an illustration of a Menck 4400 kJ installation hammer (B). In C a pile gripper on an offshore heavy-lift vessel holds a monopile in place as it's hammered to its required depth. (Images adapted from American Offshore Wind Academy)

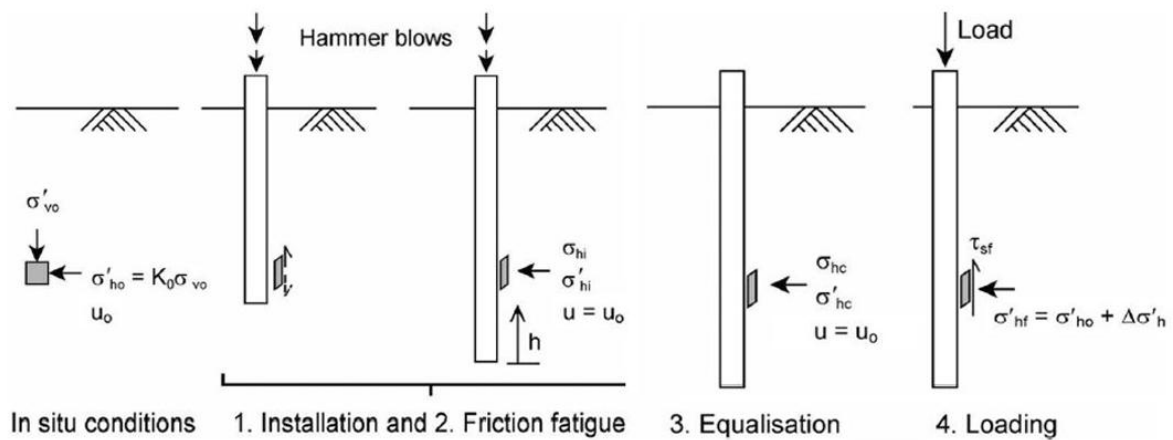


Figure 2.5: A depiction of the installation and loading of a driven pile in clay (Source: Randolph & Gourvenec 2011).

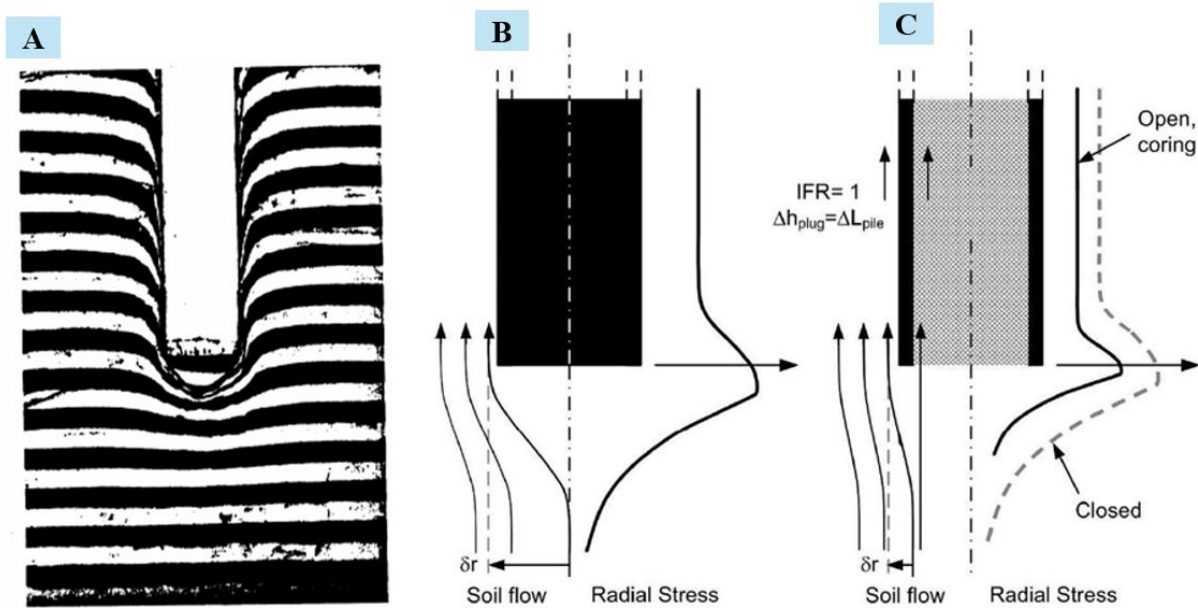


Figure 2.6: (A) Failure pattern under a closed-ended pile in soft clay (B) Schematic of soil flow under a closed-ended pile and through (C) an open-ended pile (Sources: Dr. Jen's CEE 146 Course Notes; White, 2005).

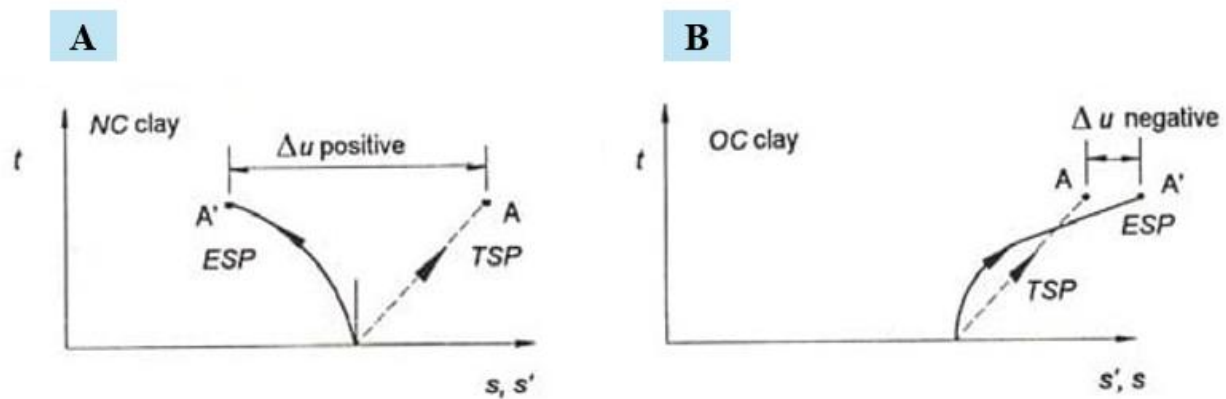


Figure 2.7: A plot of the stress paths of (a) normally consolidated clay and (b) over consolidated clay during undrained compressive shear, where s is the mean stress and t is the deviator stress. Note that normally consolidated clay develops positive pore pressure, decreasing effective stress, while over consolidated clays develop negative pore pressure, increasing effective stress. As a result, NC clay approaches the failure envelope (not depicted) at lower shear stress than OC clay (Source: Bishop et al. 1965).

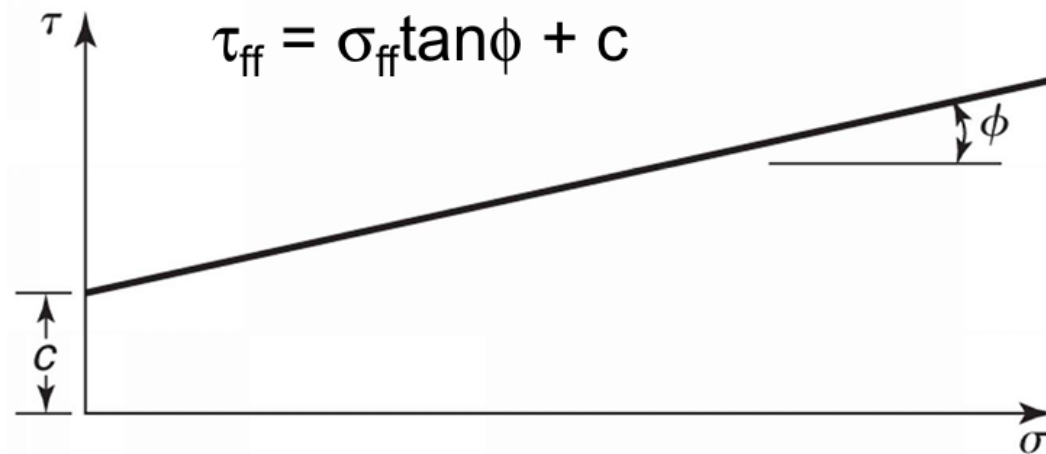


Figure 2.8: A plot of the Coulomb strength equation which models the shear stress on the failure plane at failure, τ_{ff} , as a function of the normal stress on that plane, σ_{ff} , and the angle of internal friction, ϕ , and the cohesion, c , of the material (Source: Dr. Jen's CEE 042 Course Notes).

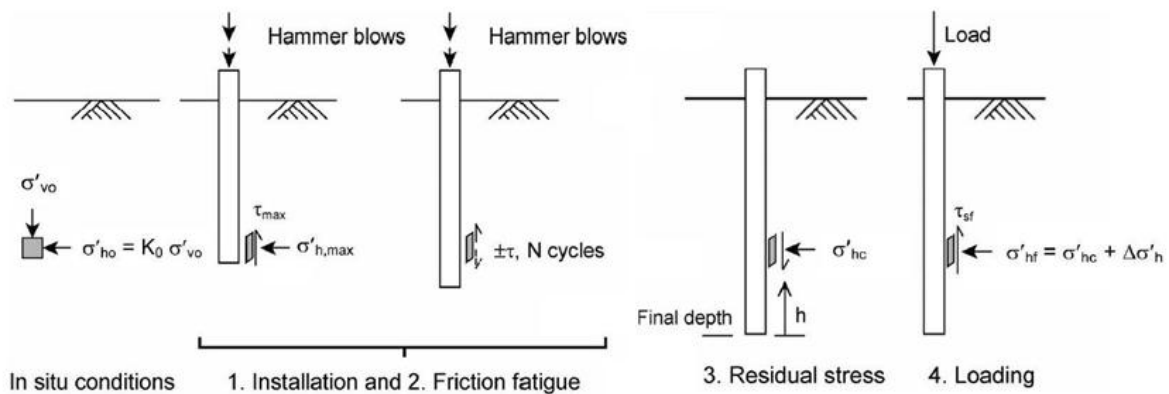


Figure 2.9: A depiction of the installation and loading of a driven pile in sand. Note that the absence of a pore pressure equalization stage like in clay (Source: Rodríguez Piedrabuena, 2024).

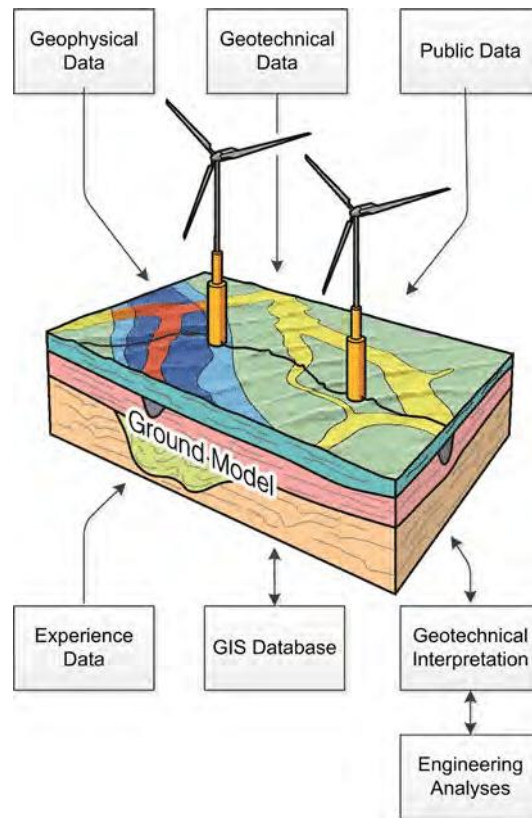


Figure 2.8: A schematic depiction of a ground model for an offshore wind farm. Laboratory and in situ testing data and geophysical logs are the principal inputs to the model that is used to assess pile drivability and foundation performance (Source: Rattley et al. 2017).

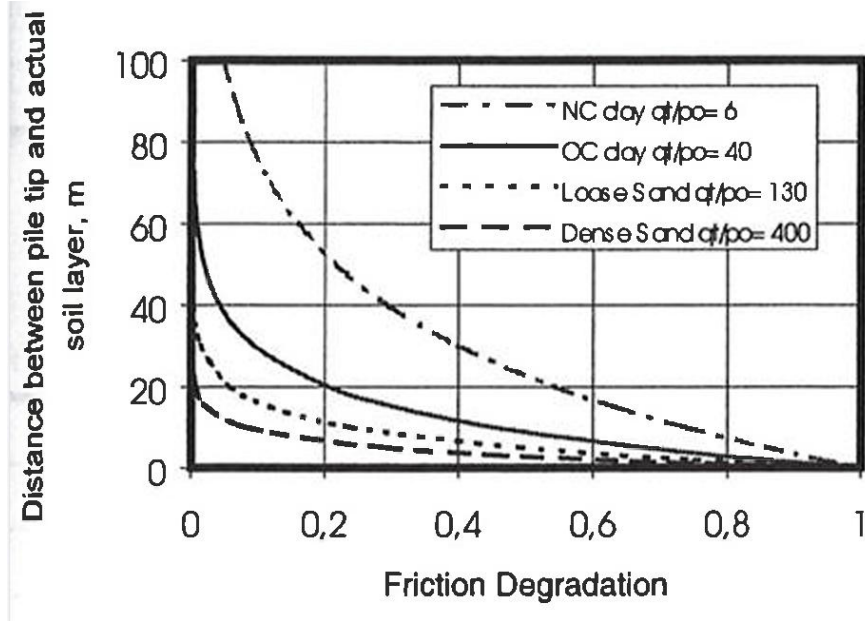


Figure 2.9: A plot of how the initial skin friction along the pile shaft degrades as the pile penetrates NC clay, OC clay, loose sand, and dense sand. Note that friction degrades most rapidly in dense sand and least rapidly in NC clay (Source: Alm & Hamre, 2001).

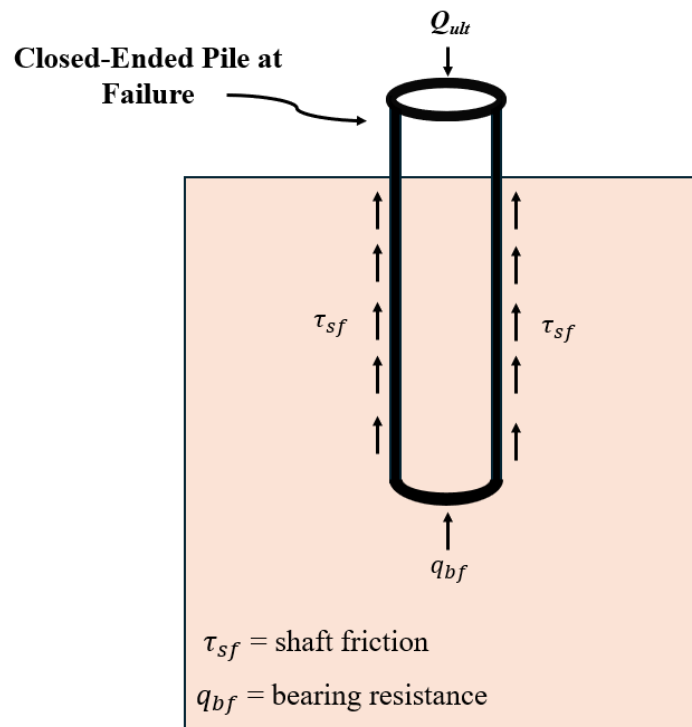


Figure 2.10: An illustration of static soil resistance to driving. Accurate prediction of these forces is essential for successful installation.

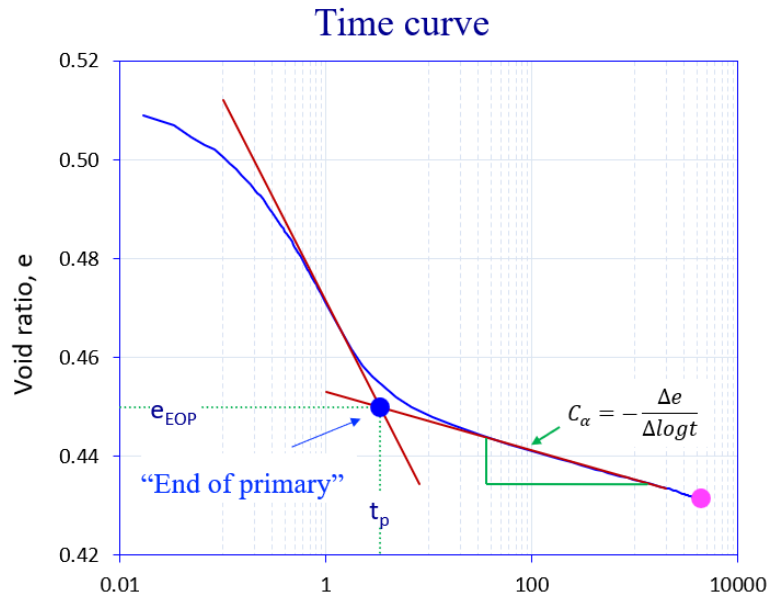


Figure 2.11: A graph of void ratio versus time from a 1-dimensional oedometer test on Intact BBC. Secondary compression theoretically begins at the end of primary consolidation (Figure from Uyeturk & Germaine, 2024).

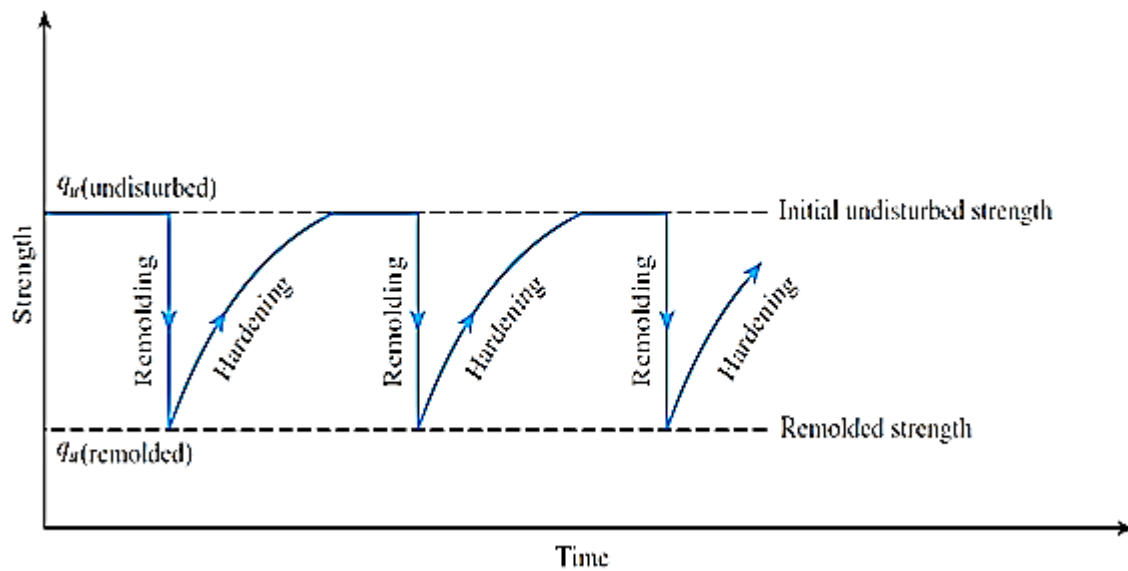


Figure 2.12: A graphical representation of thixotropic hardening, where the undrained shear strength q_u increases with time due to hardening and bonding effects (Source: Al-Rubaiee & Al Salami, 2020)

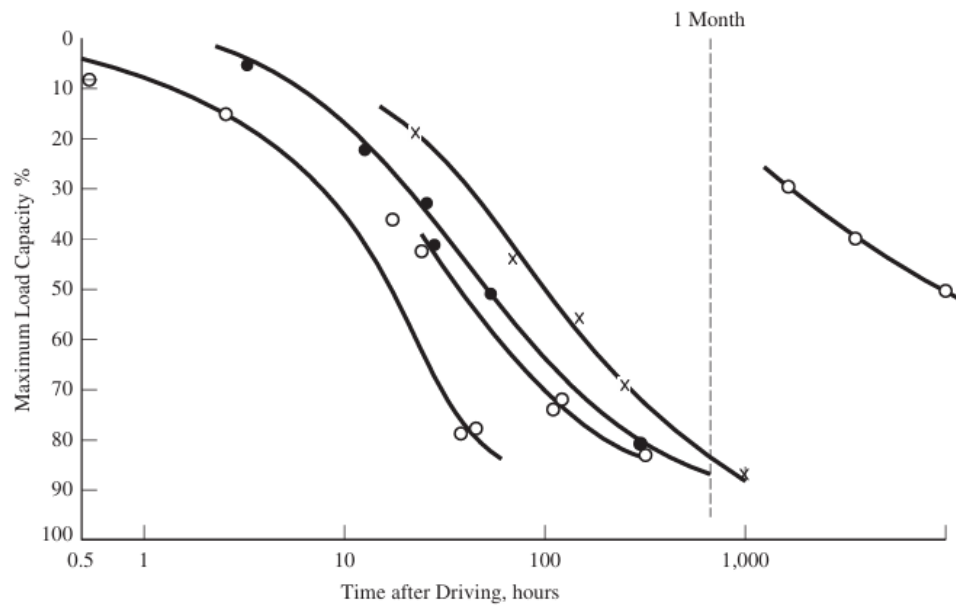


Figure 2.13: A graph depicting pile setup effects in various cohesive soils. Setup effect can vary significantly and are therefore difficult to predict (Source: Codutu, 2016).

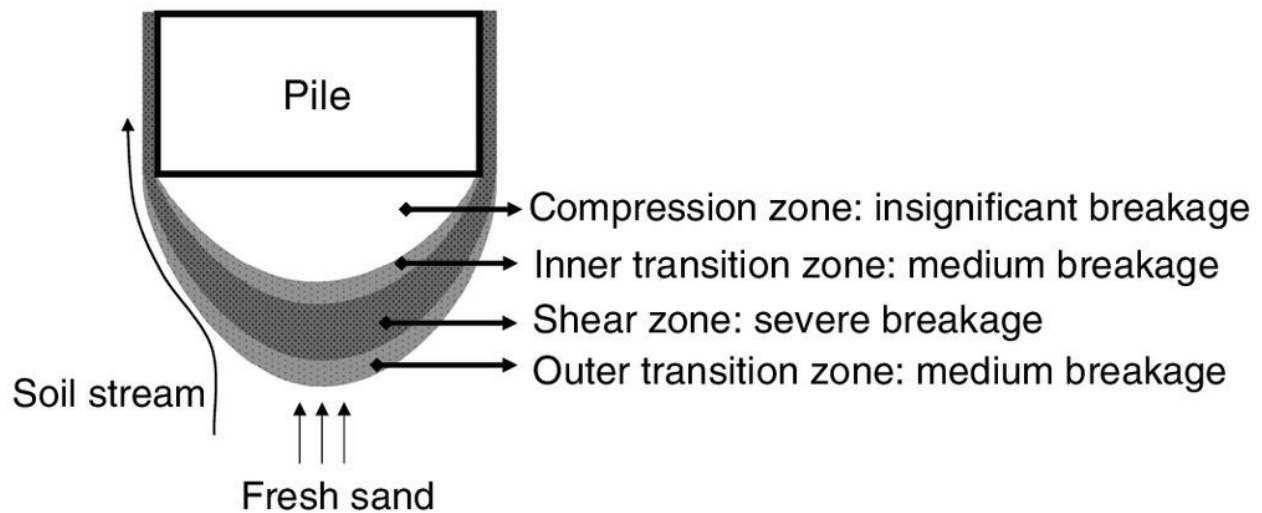


Figure 2.14: A schematic depiction of where particle breakage occurs at the pile toe, during pile driving through granular material (Source: Mao et al., 2020).

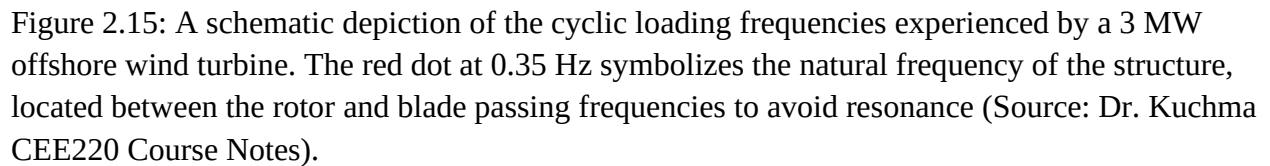


Figure 2.16: An S-N curve which depicts the number of cycles to failure at various normal stresses for single side steel weld. The failure line (denoted as the D Curve) is drawn below the data points where the steel failed (Source: Dr. Kuchma's CEE220 Course Notes).

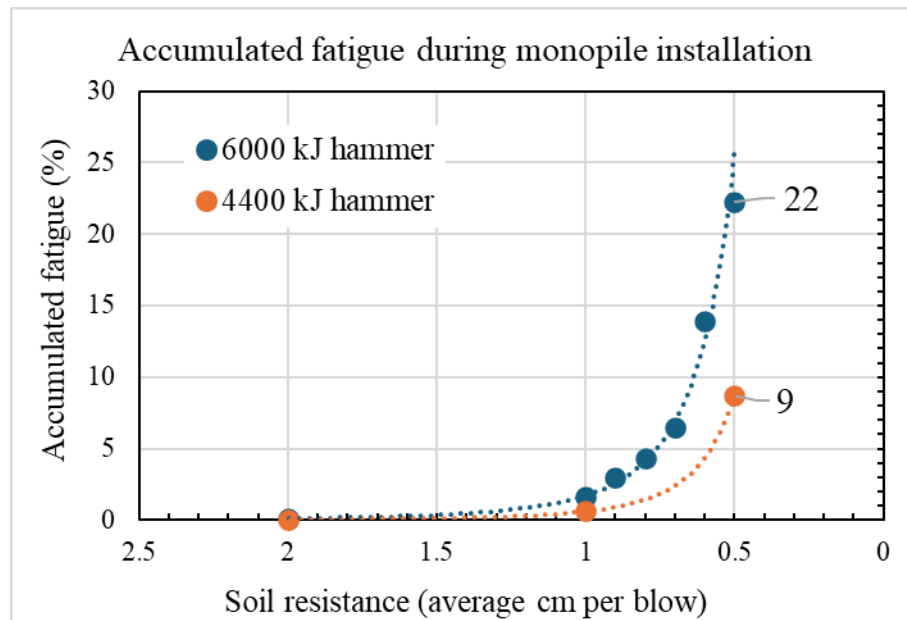


Figure 2.17: A plot depicting the consequences of high SRD and blow counts on accumulated steel fatigue during monopile installation. Fatigue accumulates with increasing SRD and blow counts.

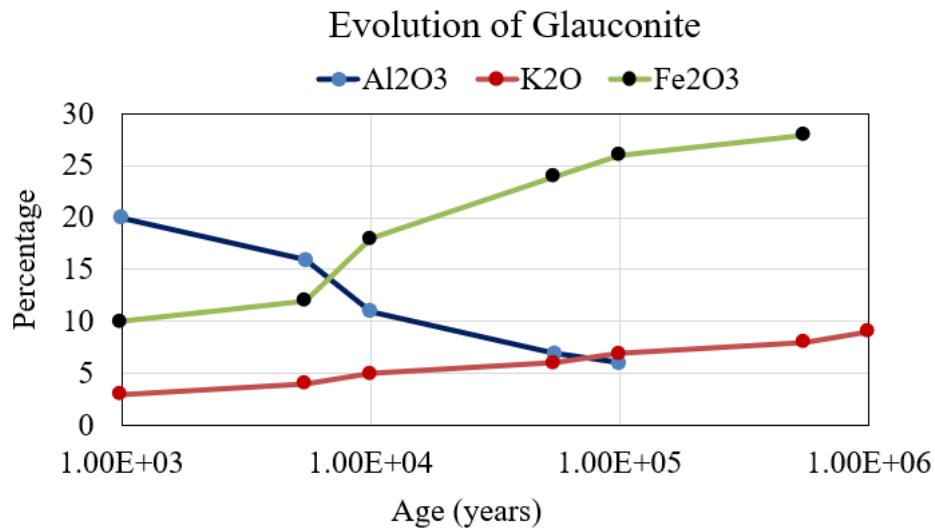
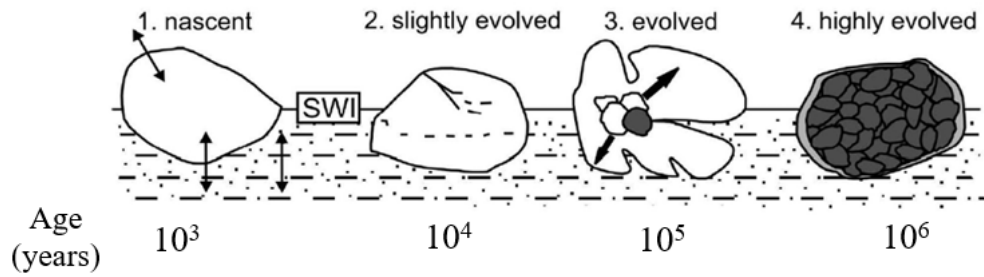


Figure 2.18: A plot depicting the diagenetic alteration of glauconite over 100 ka to 1 Ma timescales. Particles grow stronger as their potassium and iron oxide content is enriched (Modified from Obasi et al. 2011).

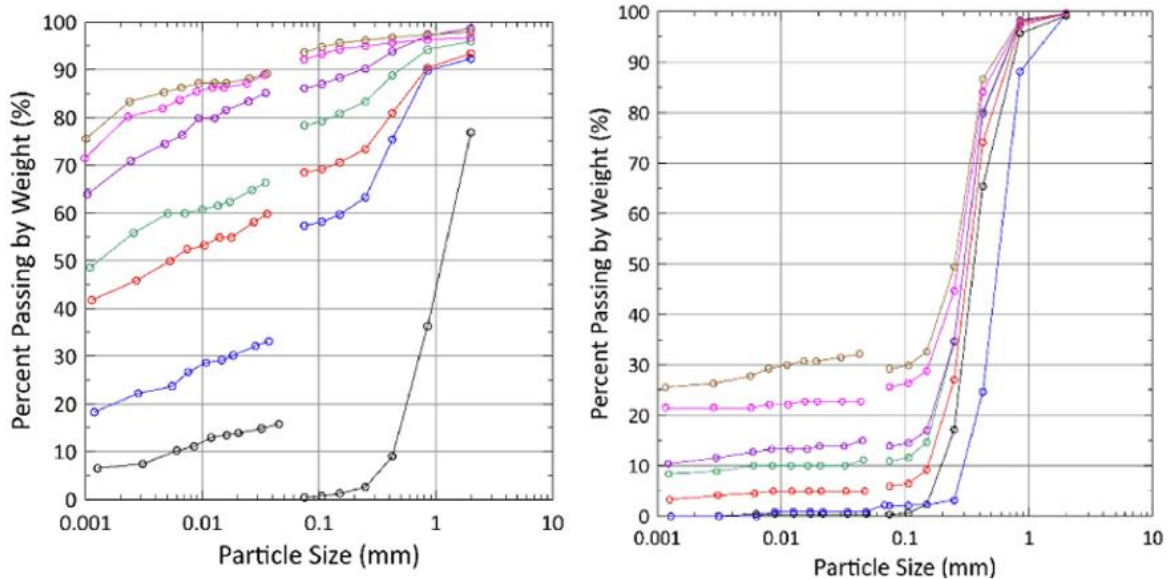


Figure 2.19: Grain-size distribution plots showing the difference between slightly evolved (left) and highly evolved glauconite sand. The black line is the original grain size distribution, the blue line is after 1 minute of malt-mixing, the red line is after 30 minutes (Source: Westgate et al. 2023).

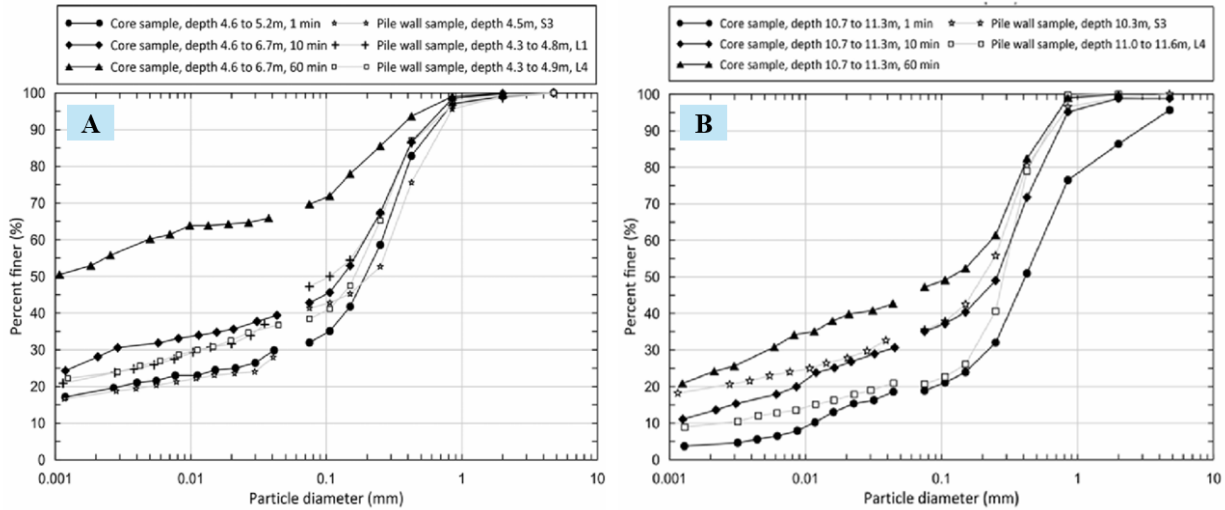


Figure 2.20: Grain-size distribution plots comparing crushed glauconite sand collected from the pile wall to borehole samples that were crushed in the malt mixer. Note that material collected from the Upper Hornerstown layer and Lower Navesink layer plots between borehole samples that were malt-mixed between 1 to 10 minutes (Source: Westgate et al. 2024).

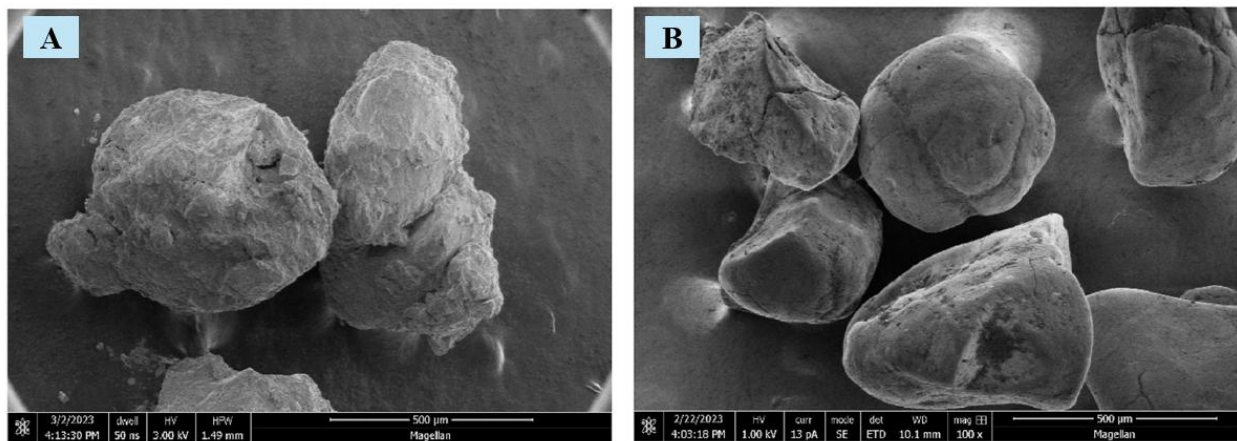


Figure 2.21: SEM images of (A) slightly evolved Crosswicks Hornerstown and highly evolved (B) Poricy Navesink glauconite sand (Source: Westgate et al. 2023).

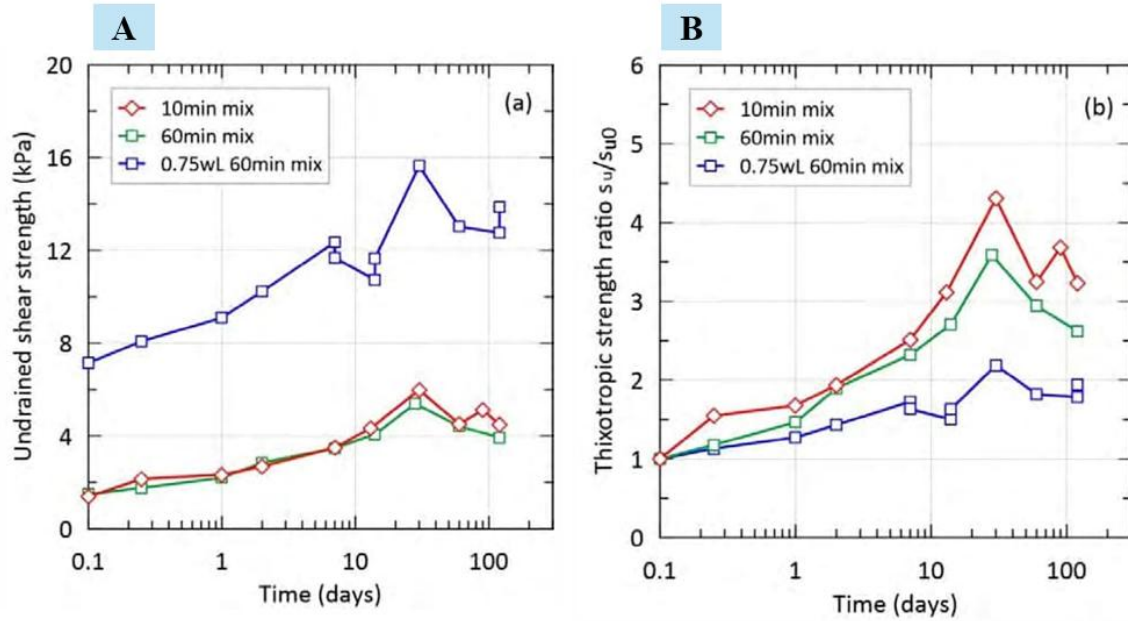


Figure 2.22: A plot detailing thixotropic hardening of malt-mixed glauconite sand prepared at various water contents. The plots show the undrained shear strength (A) shows the thixotropic strength ratio (B) as a function of time (Source: DeGroot et al. 2023).

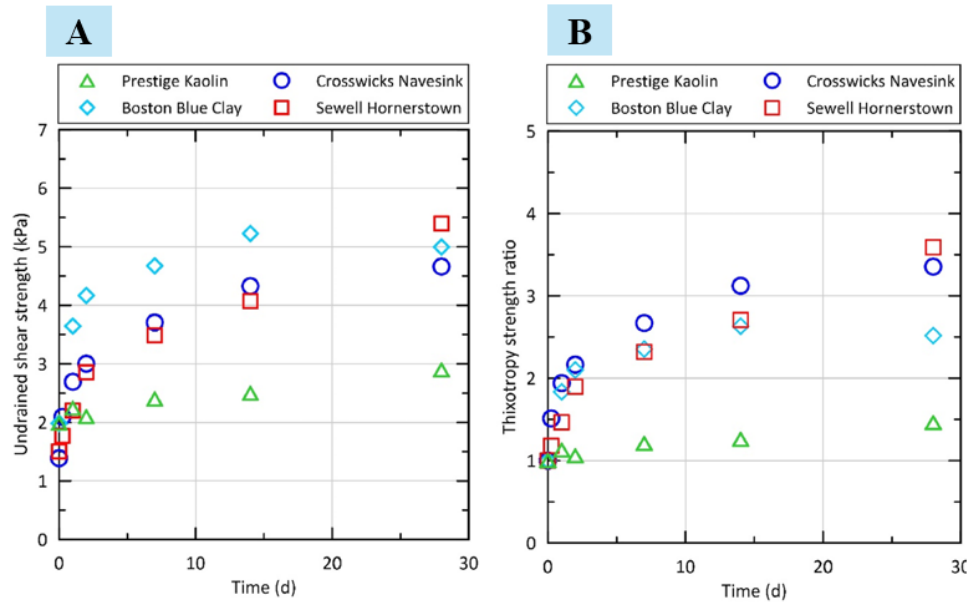


Figure 2.23: Two plots plot detailing thixotropic hardening of malt-mixed glauconite sand as compared to other marine clays. The plots show the undrained shear strength (A) shows the thixotropic strength ratio (B) as a function of time (Source: Westgate et al. 2023).

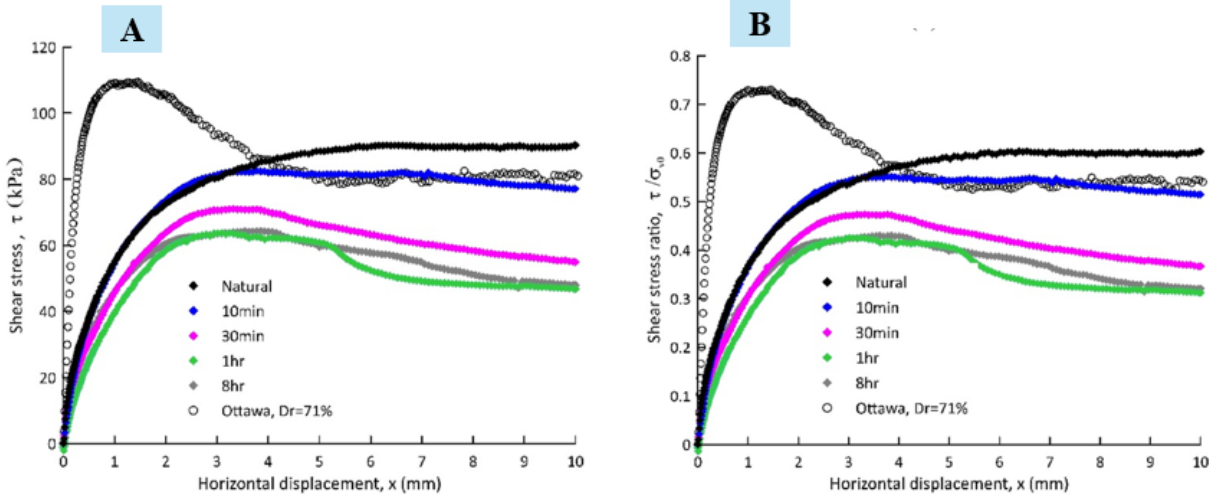


Figure 2.24: Two plots comparing results from direct shear tests at 150 kPa normal stress on Ottawa sand (silica sand) and Hornerstown sand (evolved glauconite sand) that was degraded using a malt-mixer. The plots show the drained shear strength (A) and the shear stress ratio (B) as a function of horizontal displacement (Source: Westgate et al. 2023).

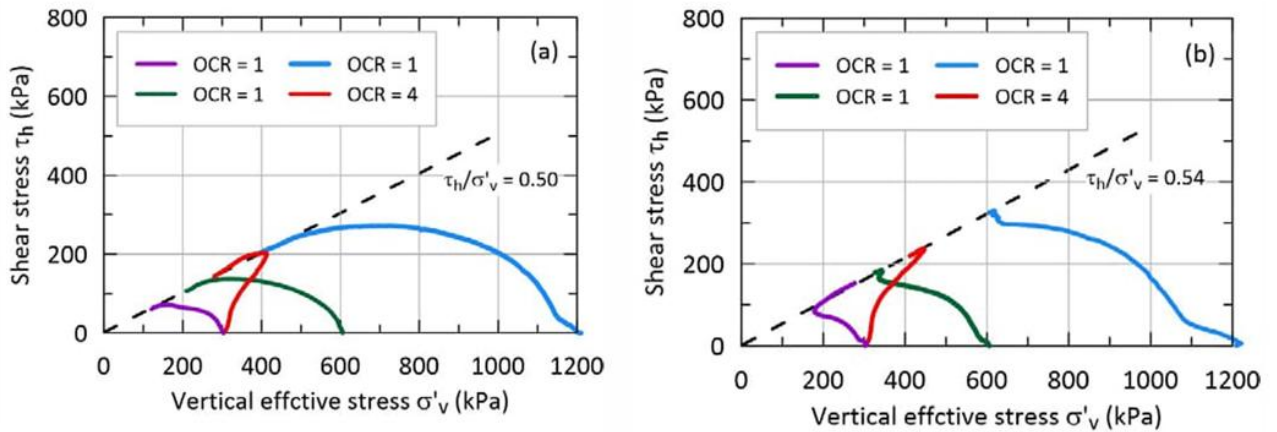


Figure 2.25: Two plots comparing results from undrained direct simple shear tests on (A) Sewell Hornerstown sand (evolved glauconite sand) and (B) Poricy Navesink sand (highly evolved glauconite sand). Note the higher strength of the Poricy Navesink sand (Source: DeGroot et al. 2023).

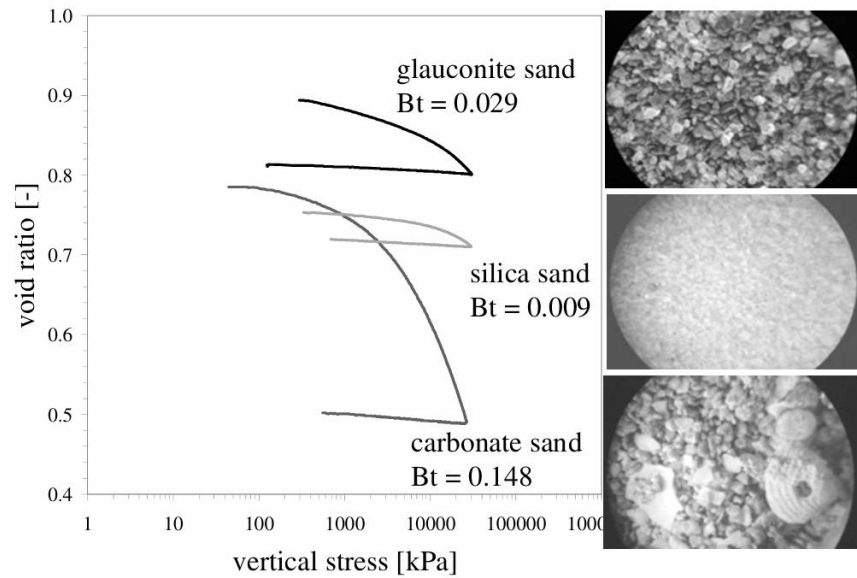


Figure 2.26: A series of compression curves comparing the compressibility of Belgian glauconite, silica, and carbonate sand. Note that the amount of compressibility or change in void ratio, is proportional to the particle breakage coefficient (B_t) (Emidio et al. 2009).

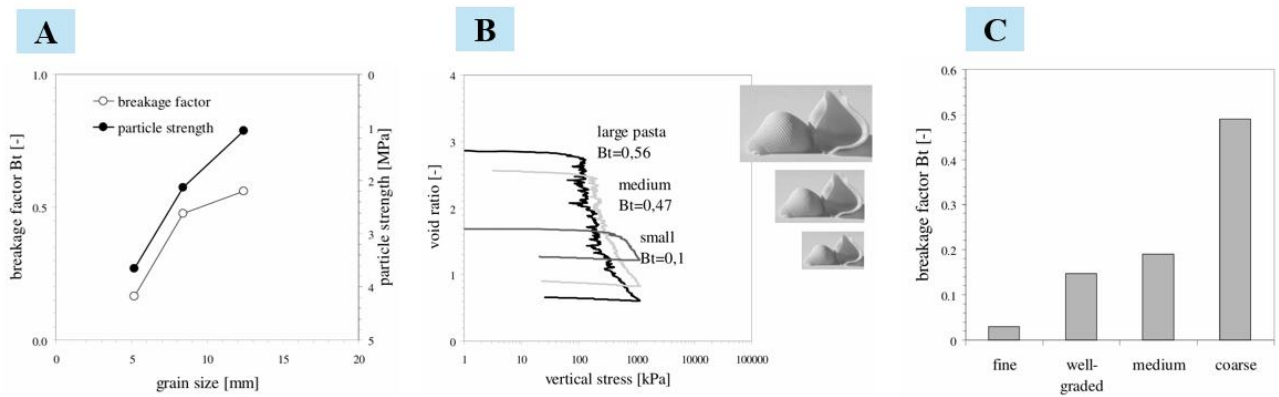


Figure 2.27: A series of plots from CRS tests on pasta showing: (A) increased particle size results in increased breakage, as determined by the breakage coefficient B_t , and decreased tensile strength, (B) decreased particle size decreases compressibility, and (C) increasing uniformity decreases breakage (Emidio et al. 2009).

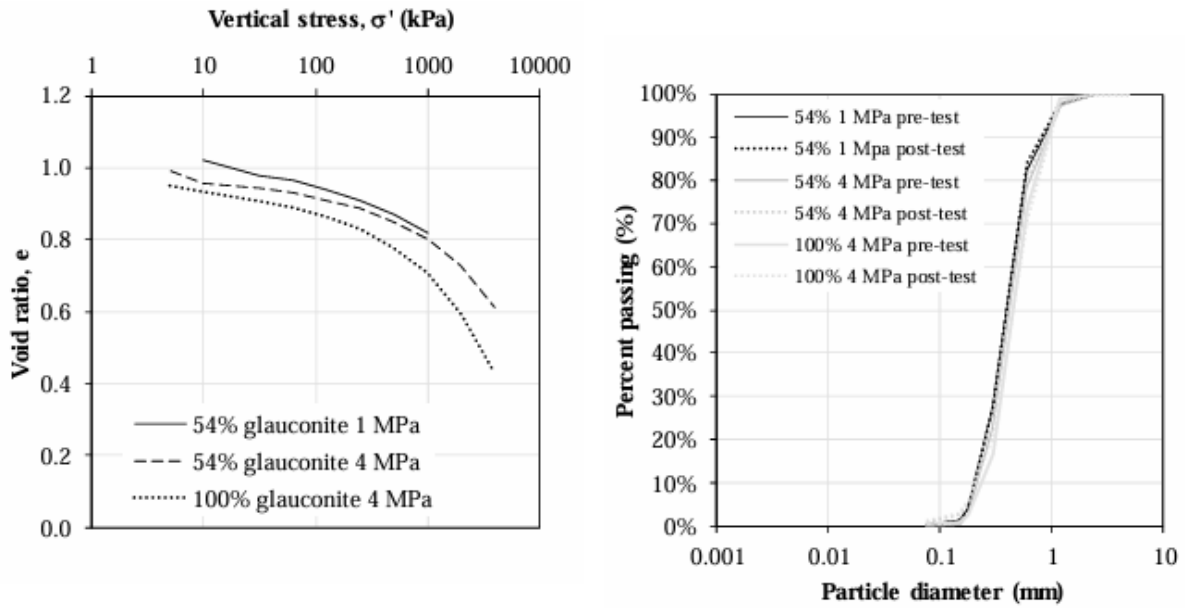


Figure 2.28: On the left, oedometer tests comparisons of Navesink glauconite sand prepared at concentrations of 54% and 100%. On the right, grain size distributions from before and after oedometer testing. Note that there is a negligible increase in fines content as the material is loaded to 4 MPa (Source: Westgate et al. 2022)

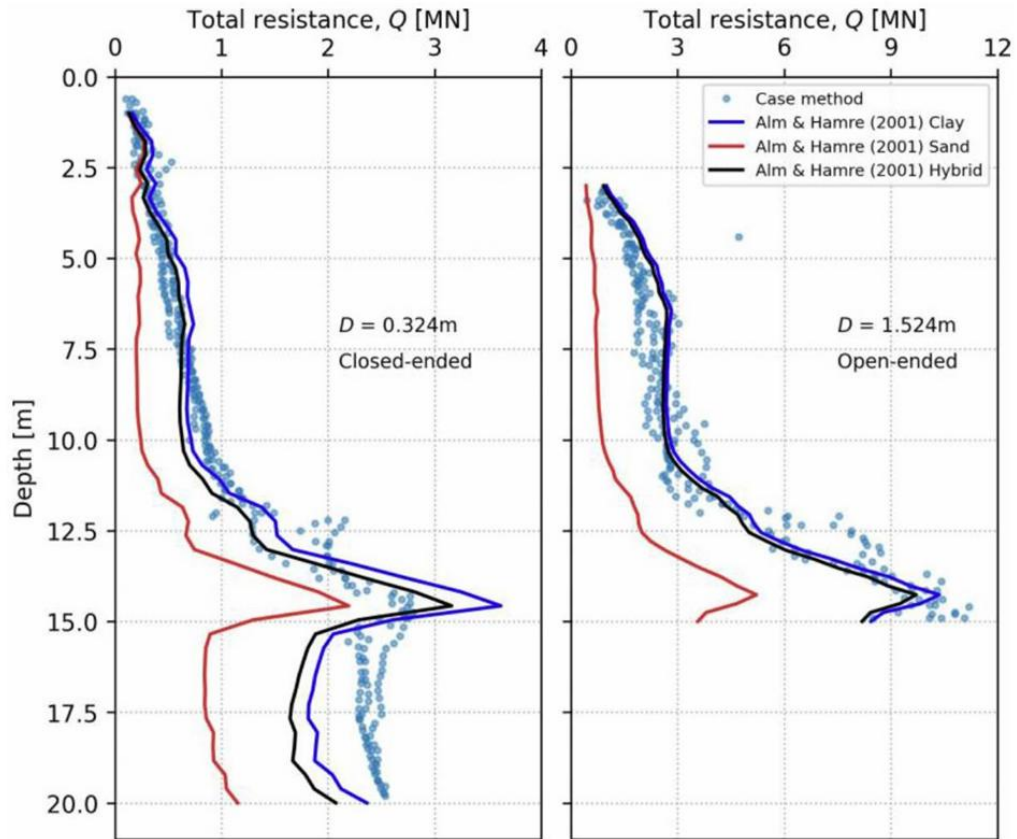


Figure 2.29: Two plots comparing predicted (Alm and Hamre 2001) to measured SRD from piles driven through New Jersey glauconite sand. Note that Alm & Hamre (2001) predicts SRD better if modeling glauconite as a clay or sand/clay hybrid as opposed to a sand (Source: Westgate et al. 2024).

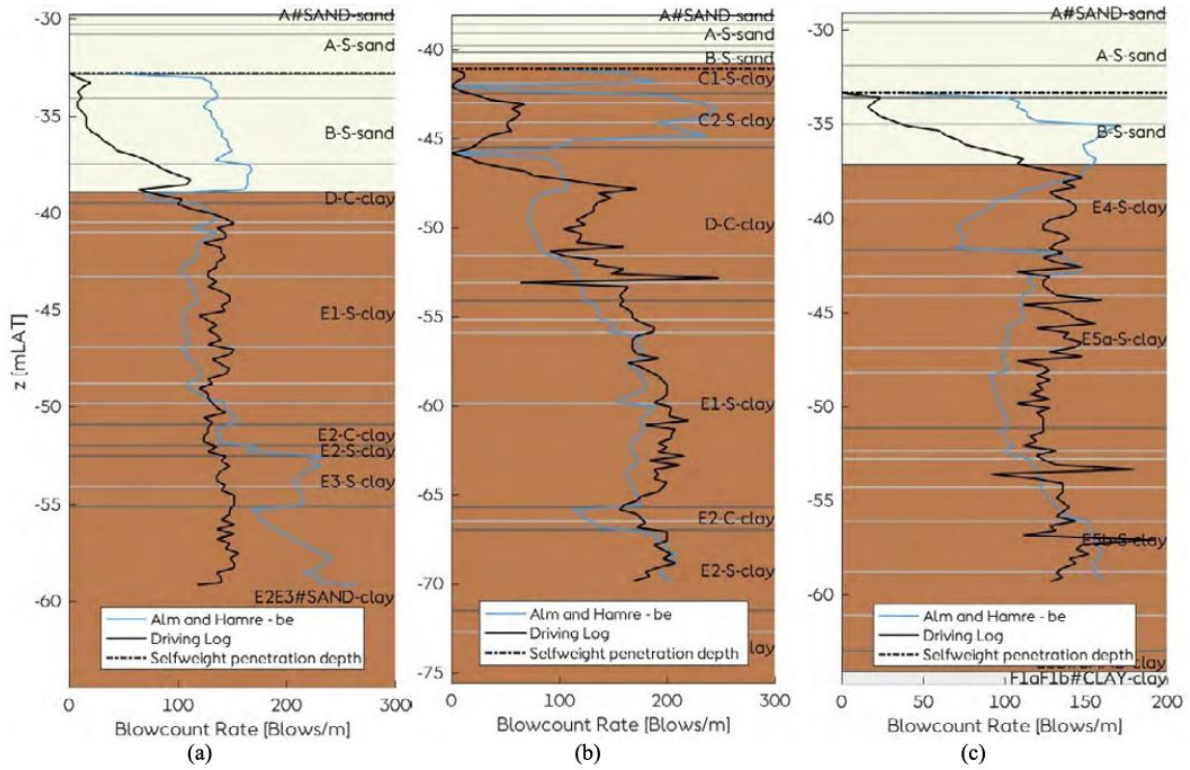


Figure 2.30: A plot of soil resistance to driving through North Sea glauconite sand as predicted by the Alm and Hamre (2001) clay model (blue) and as measured (black) (Source: Perikleous et al., 2023)

3 Materials and Specimen Preparation

This chapter describes the origins of the glauconite sand characterized in this study and the process that was employed for making samples. The glauconite sand that was characterized in this research originates from the Navesink Formation, a lightly-cemented Cretaceous glauconite marl that outcrops in Middletown, New Jersey. Subsequent sections of this chapter discuss the Navesink Formation and the geologic context of glauconite sand formation more generally. This is followed by a discussion of sample processing and specimen preparation. To investigate the effects of fines content on mechanical properties and behavior, samples were prepared with varying starting fines content using a resedimentation procedure developed by the TAG lab. In addition to the resedimentation procedure, the process for preparing sand samples at various relative densities is also discussed.

3.1 Geology of Glauconite Sands

This section explores the geologic formation of glauconite sands. These sands were deposited primarily during the Late Cretaceous (over 66 Ma) and Paleogene (66 to 23 Ma) periods. Authigenic glauconite mineralization during the Paleogene has been studied by Banerjee et al. (2020), who identified four major regions of glauconite formation:

1. The North American continental margin
2. The Paleo-Tethys, encompassing parts of northern Africa, southern Europe, the Middle East, and India
3. The Paleo-North Sea, extending from the United Kingdom in the west to northern Germany in the east
4. The high southern latitudes, including New Zealand and Argentina (Figure 3.1)

The authors constrained the age of glauconite deposits and correlated their formation with hyperthermal events (periods of elevated seawater temperature). These correlations were established by analyzing the concentrations of heavy oxygen and carbon isotopes (^{18}O and ^{13}C) in the shells of deep-sea benthic foraminifera.

In addition to elevated temperatures, glauconite formation requires prolonged chemical exchange between seawater and sediments, which is enabled by low sedimentation rates (Odin & Matter, 1981). As previously discussed, glauconite forms through the fixation of iron into a smectite or illite-like clay structure. Sub-oxic conditions, along with the availability of potassium and iron cations, are essential for this process (Figure 3.2). These conditions were facilitated by rising sea levels and the formation of warm, shallow seas during hyperthermal events of the Paleogene (Banerjee et al., 2020).

Geologists have proposed that glauconite sands primarily form from porous granular substrates such as fecal pellets and bioclasts (Banerjee et al., 2016; López-Quirós et al., 2020). Others suggest that glauconite may also form through the transformation of inorganic carbonate particles, mineral grains, or rock fragments (Odin & Matter, 1981). A widely accepted theory of glauconite evolution—developed by Odin & Matter (1981) and López-Quirós et al. (2020)—describes a process of precipitation, dissolution, and recrystallization.

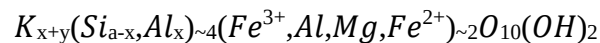
This process occurs in four stages, briefly discussed in Chapter 2: (1) nascent, (2) slightly evolved, (3) evolved, and (4) highly evolved glauconite sands particles (Figure 3.3).

Nascent particles form by the infilling of porous substrate (e.g., fecal pellets) with potassium- and iron-rich glauconitic smectite. These particles retain their original shape and are typically light in color. In the slightly evolved stage, organic matter decreases while potassium and iron increase. The pores become fully filled, and the particles darken and harden.

In the evolved stage, further recrystallization alters the shape and texture of the particles. Cracks form, and organic material is completely lost. The potassium content continues to increase, and the particles become darker green. In the highly evolved stage, previously formed cracks are filled with new crystals, increasing the grain size. A rim of less-evolved glauconite may form around the grain, giving it a more spherical appearance (Lamboy & Odin, 1975). These particles are shinier and harder due to cementation with iron, calcium, or phosphate minerals (Tedrow, 2002). Evolution typically halts after burial or prolonged exposure at the sediment–water interface (Hesselbo & Huggett, 2001; Rodríguez-Piedrabuena, 2024).

Figure 3.4 illustrates how potassium and iron diffuse through sediment pore fluids, originating from overlying seawater or adjacent mineral sources.

Despite its distinctive morphology, glauconite is a clay mineral structurally similar to illite. Illite is composed of tetrahedral and octahedral silica-alumino-silicate layers, typically bonded by sodium or potassium ions (Figure 3.5). Glauconite is essentially an iron-rich illite, in which Al^{3+} is extensively substituted by Fe^{2+} and the structure is enriched with interlayer potassium (K^+). The crystal structure of glauconite is shown in Figure 3.5. The molecular formula for glauconite is as follows:



Where x ranges from 0.2 to 0.6 and the sum of dioctahedral cations, y , ranges from 0.4 to 0.6 (Rodríguez-Piedrabuena, 2024).

3.2 The Poricy Navesink Glauconite

The Navesink Formation is a shelly marl rich in glauconite sand from the Upper Cretaceous (70 to 66 Ma). The sediments were deposited in this region, known as the Newark Basin, during a period of tectonic rifting, climatic warming, and sea level rise. The Navesink Formation is part of a belt of glauconite sands that spans about 100 miles, from the Navesink Highlands along the

southern shore of Raritan Bay to the Salem Province along the Delaware Bay. The glauconite belt ranges from 10 to 20 miles in width and dips southeast at approximately 25 to 60 feet per mile. Cook (1874) was the first to map the glauconite sand beds, dividing them into Lower, Middle, and Upper (Figure 3.6). The Navesink Formation is part of the Lower Glauconite bed. Clark (1892) drew cross sections through the glauconite beds. Figure 3.7 is a modified depiction of Clark's sections, with the Navesink Formation marked in bright green.

The samples for this research were taken from the Poricy Park fossil beds. The Poricy Brook cuts through the Navesink Formation, which outcrops along the riverbank. The Poricy Park fossil beds are located slightly northwest of Swimming River, which lies along section A to A' in Figure 3.7. The location of the Poricy Fossil Beds is marked by a green star in Figure 3.8.

Samples were collected from the Navesink Formation by walking along Poricy Brook until reaching a section of exposed bank. The bank is extremely stiff and difficult to sample even with a spade (Figure 3.9). The dark glauconite sand grains are visible within a gray clayey matrix. The material appears lightly cemented and over-consolidated. A 5-gallon bucket of the riverbank material was collected.

3.3 Sample Processing

Glauconite deposits in offshore wind lease areas on the North Atlantic Outer Continental Shelf (OCS) are primarily allogenic, meaning they have been reworked from their original depositional environment. These glauconite sands are typically buried beneath the seafloor. In one documented case, a glauconite-rich layer was encountered at depths between 15 and 35 meters below the seafloor, with glauconite contents reaching up to 85% in some samples. Figure 3.10 shows a sample collected during a site investigation for an offshore wind farm. The material appears exceptionally pure, with a high concentration of glauconite sand particles. Due to proprietary restrictions, this sample was not made available to the TAG Lab for further research.

Evidence indicates that many glauconite sand deposits on the North Atlantic OCS are relatively clean and occur in high concentrations. While some offshore deposits may include significant quantities of other minerals, this study focuses on characterizing specimens composed almost entirely of glauconite. The first objective was to isolate glauconite grains from the Poricy Navesink Formation, which are naturally embedded in a clayey matrix. Initial attempts to disaggregate the riverbank material by soaking and shaking proved too time-consuming for processing larger quantities, highlighting the need for a more efficient method.

A wet sieving method was developed to wash the silty and clayey fines off the glauconite particles. First, a 5-gallon bucket was filled with deionized water (DIW). An underwater electric pump was placed at the bottom of the bucket and turned on. The pump was connected to a length of rubber tubing used to circulate the water. Next, a stack of sieves was placed over the bucket of water on two steel rods. The sieves were arranged in decreasing size from top to bottom: No. 8, No. 10, No. 20, No. 50, No. 100, and No. 200. The key was to maximize the material retained on each sieve before any became overloaded and impermeable.

After the sieves were stacked, the unprocessed glauconite marl was placed on the topmost sieve. Water from the bucket was pumped through the tubing and washed over the material (Figure

3.11). The material was gently disaggregated while being washed to accelerate the process. Once a sieve became overloaded, the stack was carefully disassembled. Each sieve was washed in a secondary bucket to collect the disaggregated glauconite particles. Afterward, the sieve stack was reassembled, and a new chunk of glauconite marl was placed on top. When the water in the bucket became saturated with fine-grained particles, it was disposed of and replaced.

Material retained in the sieves often required multiple rounds of washing before the fines were completely removed. After collecting significant fines in the secondary bucket, the water was decanted, leaving wet glauconite sand. The material was transferred to a vacuum Erlenmeyer flask connected via plastic tubing through a rubber stopper to a second flask, which was in turn connected to the laboratory vacuum line. More DIW was added to the first flask, and the flask was gently shaken to suspend the fines. After 45 seconds of settling, the tubing was lowered into the suspension to remove the fine-grained particles. The water in the secondary flask was then disposed of. This process was repeated with additional DIW until the glauconite particles no longer generated fines upon shaking. Several kilograms of glauconite sand particles were collected in this way (Figure 3.12). Note that the sand fraction was later determined to be 86% glauconite by bulk mineralogy using XRPD (See Chapter 6).

3.4 Preparing Reconstituted Sand Specimen

Because the processed Navesink material is a highly evolved glauconite sand, it is relatively durable and can be left to air-dry without significantly increasing the fines content (Westgate et al. 2023). Since the processed Navesink glauconite sand is cohesionless, it was well suited for CRS and triaxial testing. The mechanical behavior of sands typically varies as a function of relative density. Relative density, expressed as a percentage, is the ratio between the minimum and maximum void ratios of the material, with 100% representing the maximum void ratio.

All sand specimens were prepared for CRS testing at the testing station by funnel-pouring approximately 14.3 grams of glauconite sand atop a porous stone covered in nylon filter within a rigid steel specimen cutting ring. The specimen was gently prodded vertically about 25 times with a thin steel rod to densify it. This method produced specimens with a relative density (D_r) ranging from 36 to 43%, which was deemed acceptable for the purposes of establishing repeatable compression behavior.

Preparing reconstituted sand samples for triaxial testing is more complex and was beyond the scope of this research.

3.5 Preparing Resedimented Specimens

One of the principal objectives of this research was to investigate the effects of fines content on the mechanical properties and behavior of glauconite sand. To isolate the influence of fines, specimens were prepared and tested at various starting fines contents using a resedimentation procedure. The following section outlines the method for generating glauconite fines and preparing the resedimented specimens.

Glaucinite fines were produced by crushing processed glauconite sand particles. Two methods were employed: malt-mixing glauconite particles in a Humboldt malt mixer, and shaking plastic bottles containing glauconite and ceramic balls in a wrist-action shaker. The malt-mixer, which applies shear loading to particles in slurry, was more effective over short timescales. The wrist-action shaker, by contrast, was more suited for longer durations, as it could be safely operated overnight. Photographs of these procedures are provided in Figure 3.13.

After crushing, the fines were separated from sand-sized particles by washing the crushed material over a No. 200 (75 μm) sieve. The collected fines were oven-dried at 100°C in ceramic bowls to determine the dry mass.

Once a sufficient mass of fine-grained glauconite had been collected, the resedimentation process was used to prepare the test specimens. This laboratory technique allows for the creation of reproducible soil samples with controlled void ratios, stress histories, and pore fluid salinities. While resedimentation is often used to “erase” disturbance from sampling intact specimens, in this study it was instead used to prepare specimens with precise starting fines contents, as all materials had already been processed.

The first step in resedimentation was to select the target fines content, water content, and pore fluid salinity. These parameters determined the ratio of glauconite sand, glauconite fines, sea salt, and distilled water needed. For this study, the salinity of the glauconite fines powder was assumed to be zero, a simplification justified below.

In typical practice at the Tufts Advanced Geomaterials Laboratory (TAG Lab), the salinity of materials such as Boston Blue Clay (BBC) or Gulf of Mexico Eugene Island Clay (RGoM-EI) is measured by centrifuging the sample, extracting the pore fluid, and measuring electrical conductivity using a probe calibrated with NaCl-based dilutions. However, salinity tests on glauconite sand produced conductivities near 370 mS/cm—over four times the 50 g/L reference conductivity of ~80 mS/cm. These readings were well outside the probe’s calibration range. One hypothesis for this spike is that glauconite crushing releases interlayer cations (e.g., K^+ , Mg^{2+} , Ca^{2+} , Fe^{2+}), increasing the ionic strength of the solution. Because the conductivity probe relies on ionic mobility and was calibrated only for NaCl, its readings for glauconite could not be accepted.

Despite this uncertainty, a salinity of 35 g/L was chosen for resedimentation—a representative value for marine clays from the North Atlantic. Natural sea salt was used to adjust the salinity.

Fines content was controlled by mixing a known mass of glauconite fines with a corresponding mass of glauconite sand. Water content was varied from 50% to 120%, depending on the fines content: lower water contents for 25% fines mixtures, and higher water contents for specimens composed entirely of glauconite fines. Note that the water content is computed as the mass of water divided by the mass of solids (the dry mass of the soil). The water content was varied to ensure that it was high enough for the slurry to flow during de-airing, but low enough to ensure proper consolidation.

To determine the exact mass of each component in the slurry, Dr. Germaine's custom spreadsheet, the Salt Mixing Guide, was used. This tool accounts for variables such as water temperature–dependent density.

The soil slurry was mixed for 20–25 minutes in a KitchenAid mixer, then covered with plastic wrap, labeled, and left to hydrate for 24 hours. After hydration, the slurry was remixed briefly and de-aired in a vacuum chamber. De-airing typically took less than 10 minutes but continued until no more bubbles were observed on the surface.

The de-aired slurry was then injected into a consolidometer. The inside of the consolidometer was lightly lubricated with silicone grease to reduce side friction and ease extrusion. The water bath of the consolidometer was filled with water of matching salinity (35 g/L). Porous stones and nylon filter were placed at the bottom of the cell to prevent slurry leakage (Figure 3.14). After injection, another porous stone and filter were placed on top of the slurry to apply the first load increment (McAneny, 2023).

Loading was applied incrementally, with the load doubled every 24 hours. Samples were typically consolidated to 98 kPa of effective stress and then unloaded to achieve an overconsolidation ratio (OCR) of 4 before constant rate of strain (CRS) or low-stress triaxial testing (Figure 3.15).

However, glauconite clay exhibited high stickiness, leading to increased sidewall friction and adhesion. These issues interfered with both extrusion and consolidation. Several samples consolidated to 98 kPa and unloaded for triaxial testing failed during setup. For future testing, the use of more silicone grease and a higher preconsolidation stress is recommended.

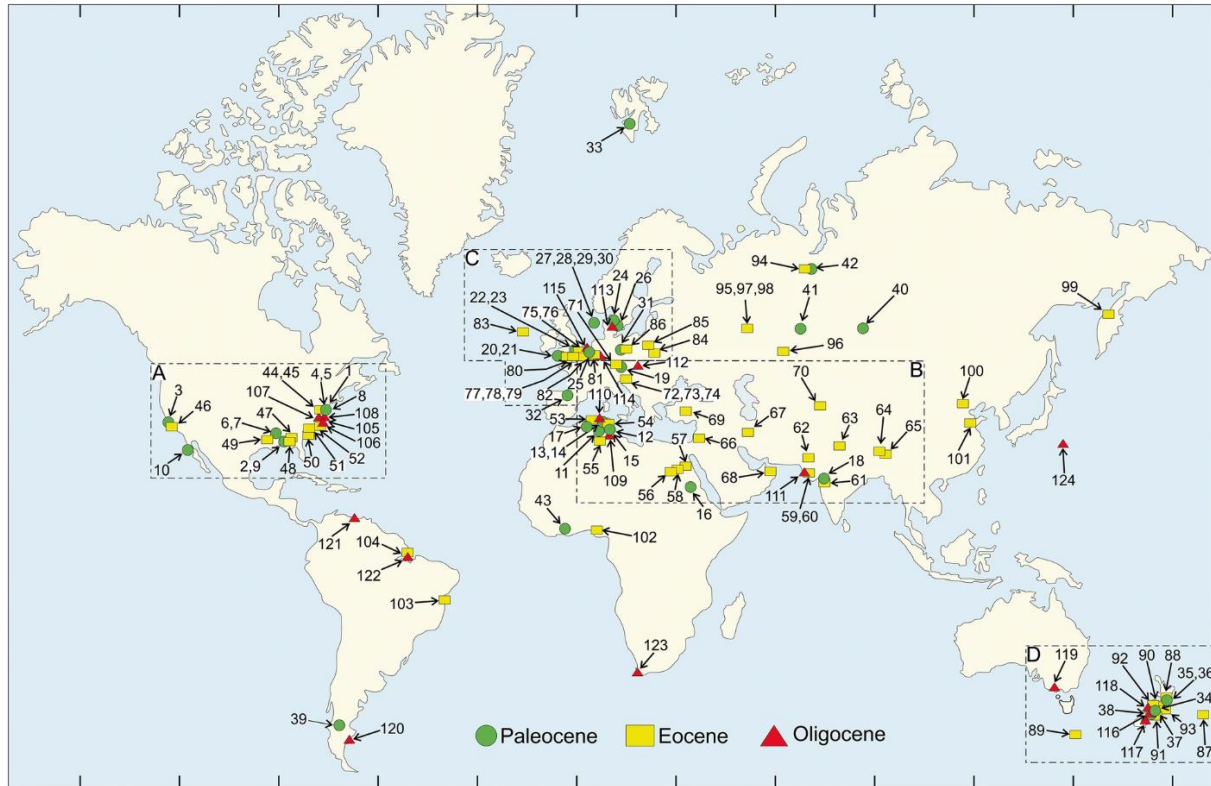


Figure 3.1: Map of Paleogene glauconite sand deposits throughout the world. Note that glauconite sands were deposited in four main regions. Each number corresponds to a distinct glauconite deposit as discussed in Banerjee et al., 2020.

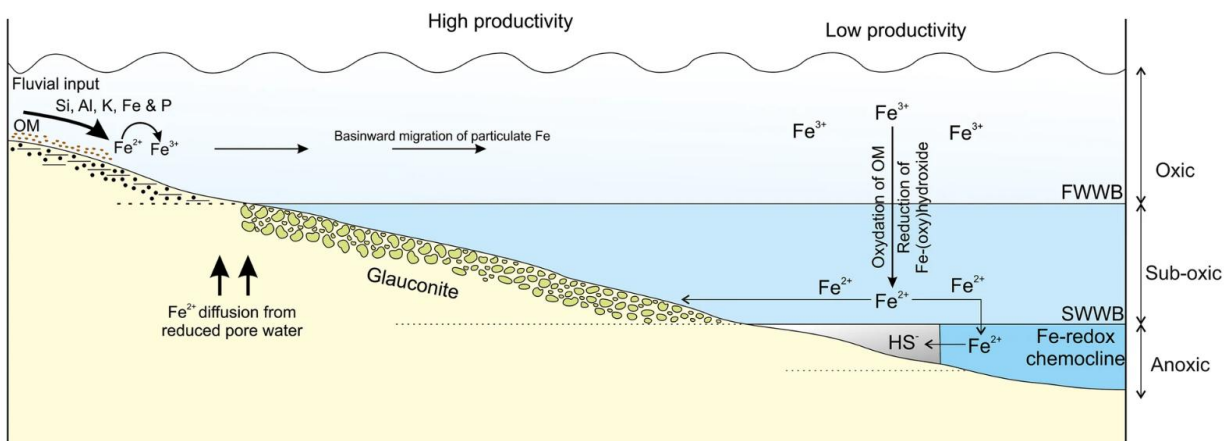


Figure 3.2: A depiction of how glauconite forms in shallow marine depositional environments under sub-oxic, reducing conditions (Source: Banerjee et al., 2020).

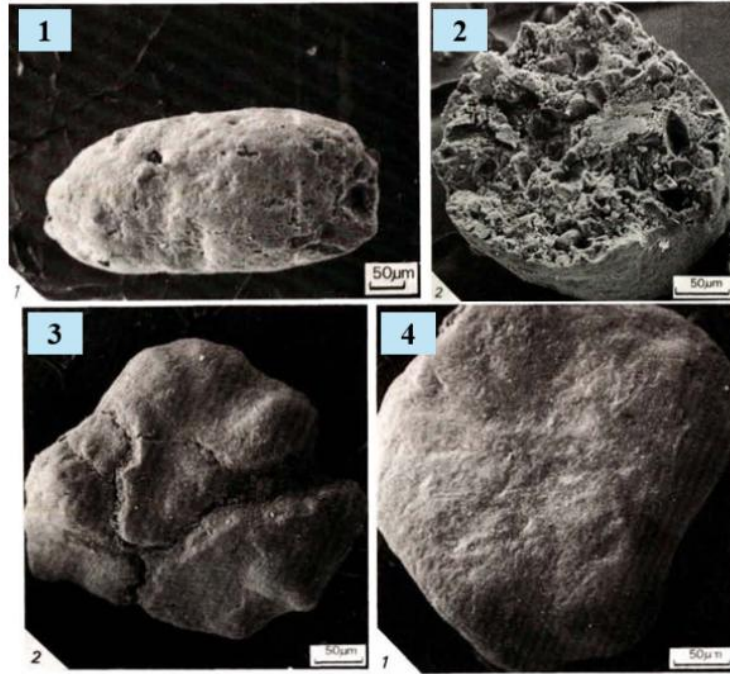


Figure 3.3: Microscope image of glauconite from the Gulf of Guinea during the (1) nascent (2) slightly evolved (3) evolved and (4) highly evolved stages (Source: Giresse et al., 1980).

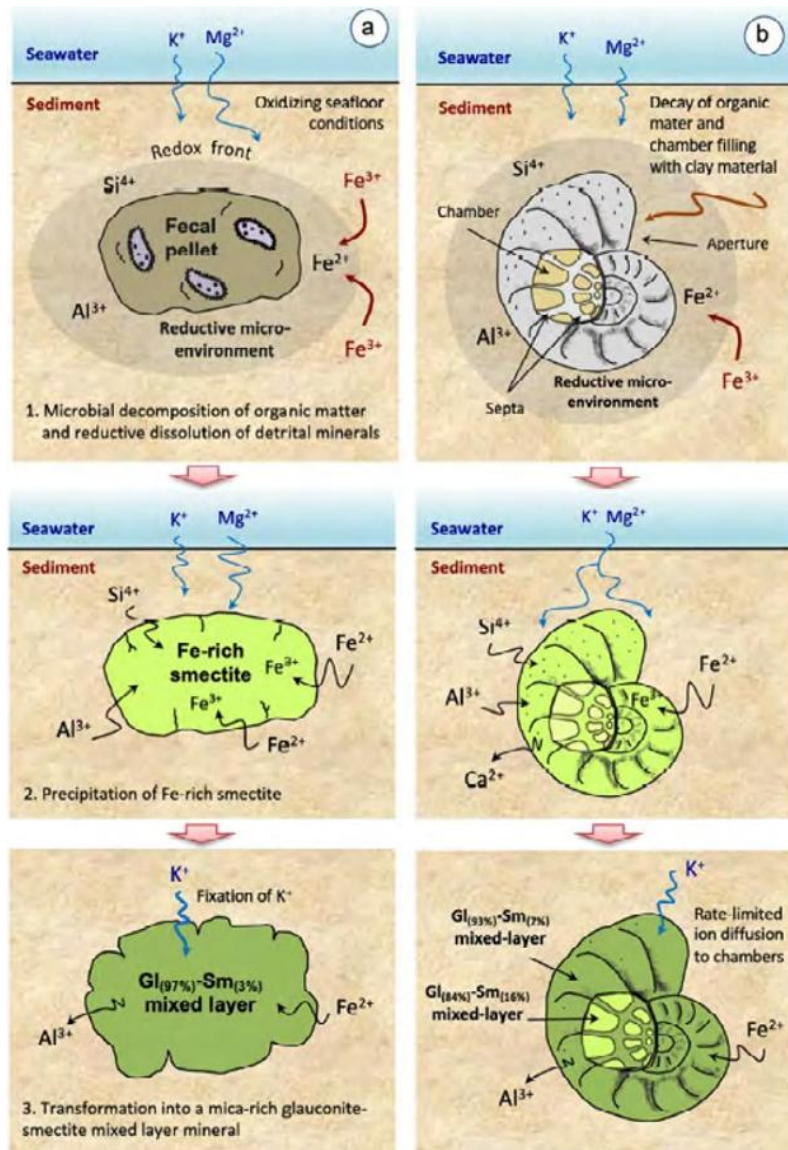


Figure 3.4: A schematic depiction of glauconitization process via (a) fecal pellet substrate and (b) foraminiferal substrate (Source: Fernández-Landero & Fernández -Calani, 2021).

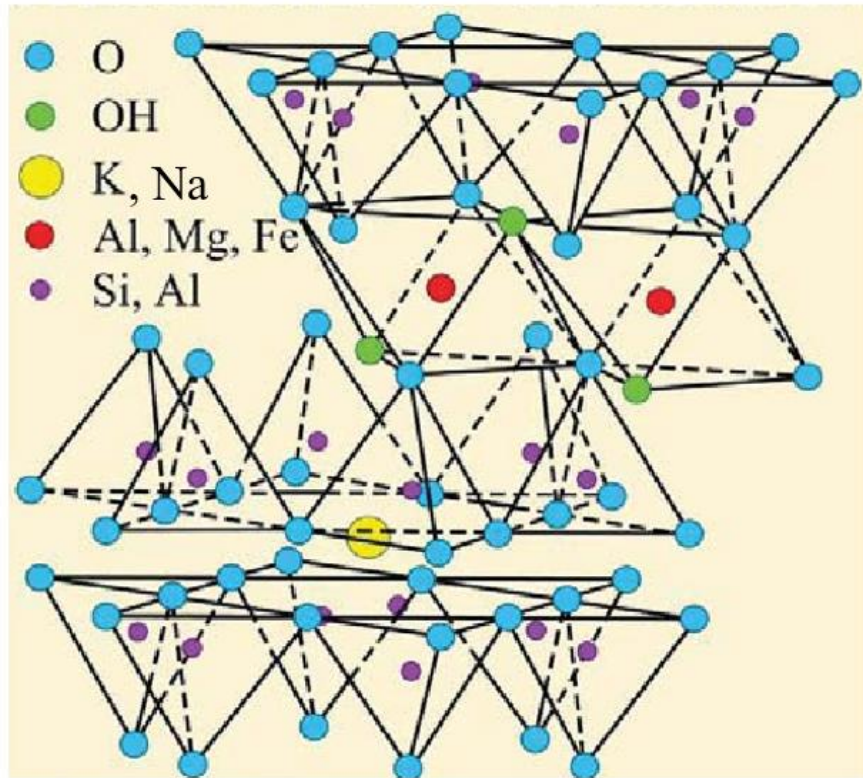


Figure 3.5: A diagram illustrating the crystal structure of glauconite clay, which consists of tetrahedral and octahedral silica-alumino-silicate units. These units are bonded by sodium or potassium ions. The structure bears resemblance to illitic clay due to the partial substitution of aluminum for iron within the lattice (Modified from USGS, 2001).

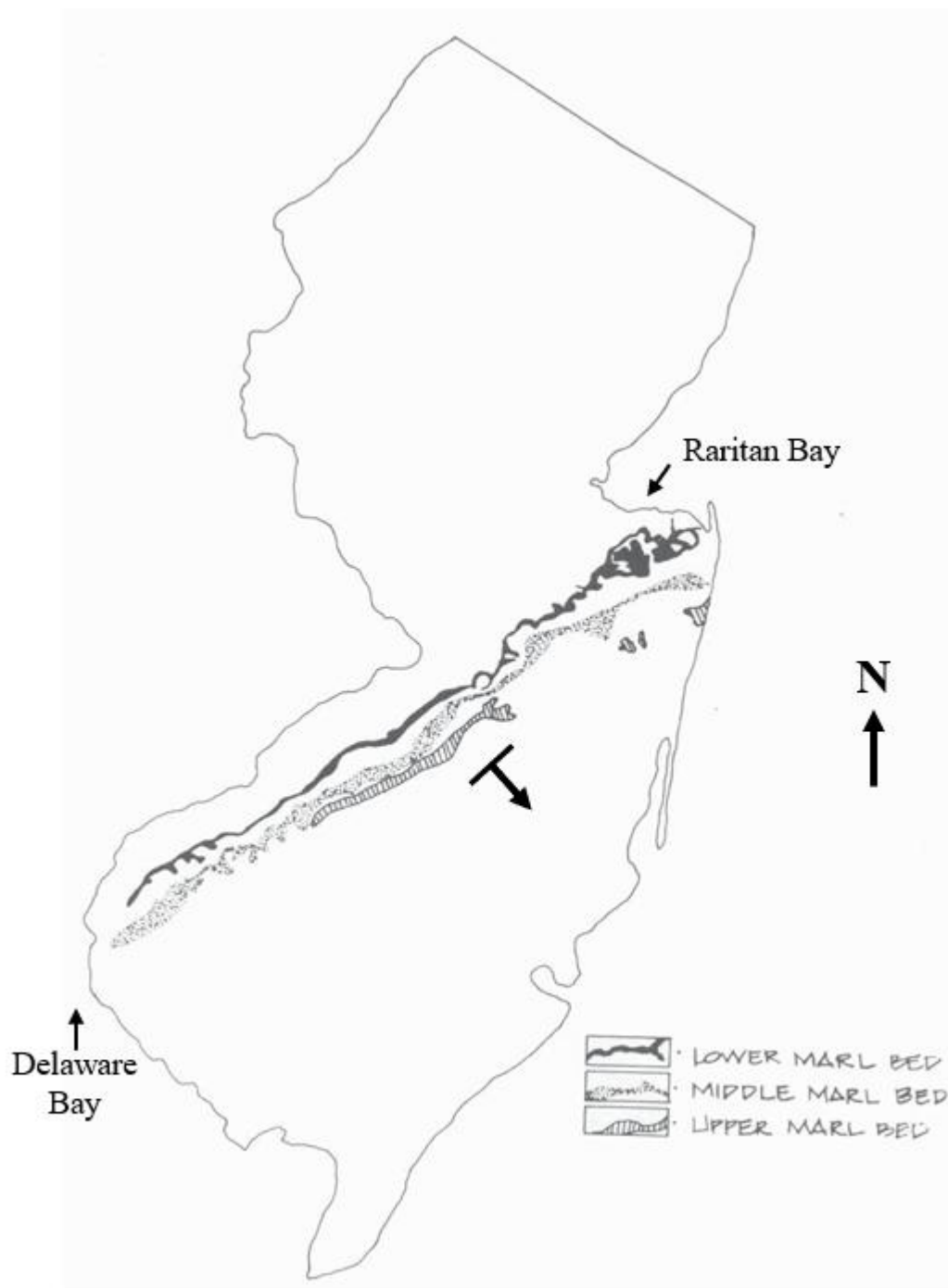


Figure 3.6: A map of the glauconite sand beds of New Jersey modified from Cook, 1874. The Lower marl bed approximates the Navesink and Marshalltown Formations, the Middle marl bed approximates the Hornerstown Formation, and the Upper marl bed approximates the Manasquan and Shark River Formations

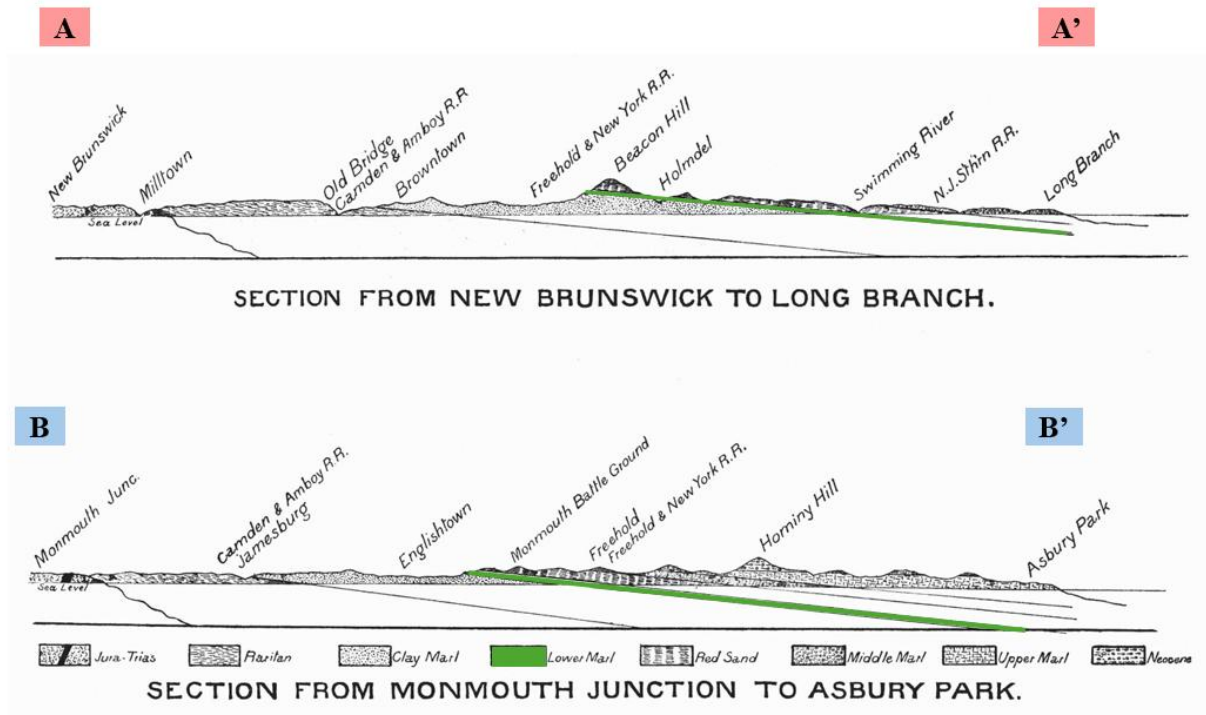


Figure 3.7: Two sections depicting the New Jersey glauconite sand belt from Clark (1892). The Lower Marl Bed, which contains the Navesink Formation is marked in green.



Figure 3.9: Two photos showing Navesink glauconite (left) and the riverbank from which it was sampled (right).



Figure 3.10: A photo of glauconite sand sampled during an offshore wind site investigation. There appears to be an exceptionally high concentration of glauconite sand particles.

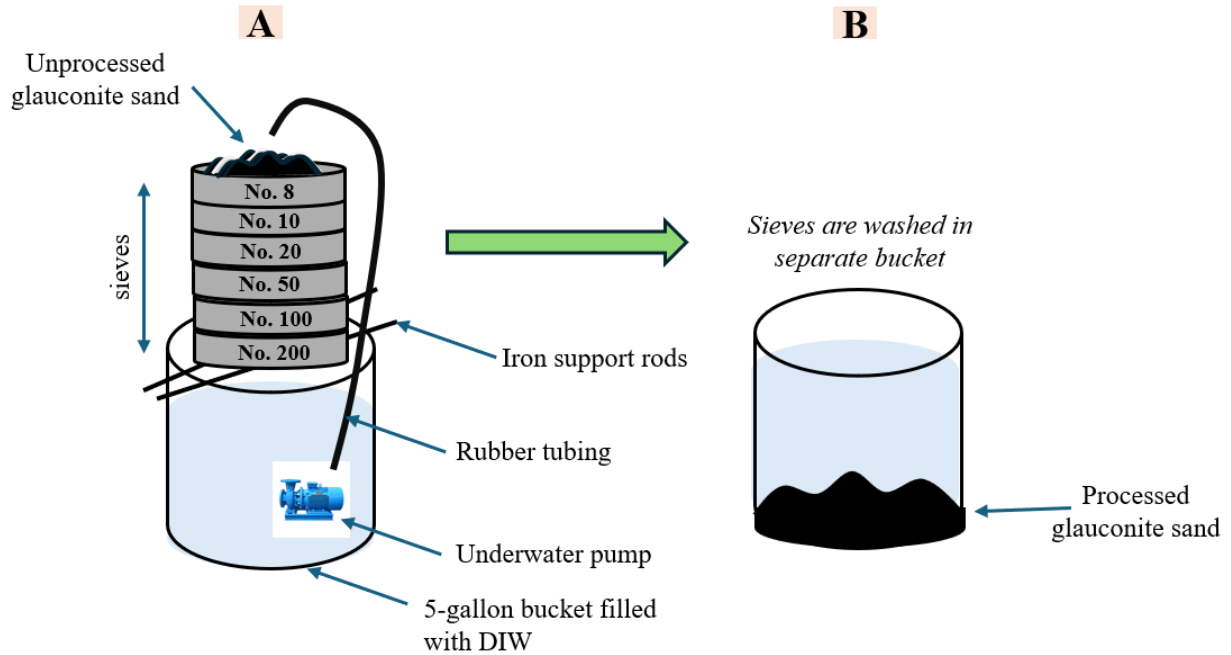


Figure 3.11: A schematic of the laboratory setup and workflow for separating glauconite sand from its clayey silty matrix.



Figure 3.12: Photos from the glauconite sand processing. The left photo shows glauconite marl before and after washing over the stacked sieves. The right is a close-up of the processed glauconite sand before the vacuum filtration method was applied.

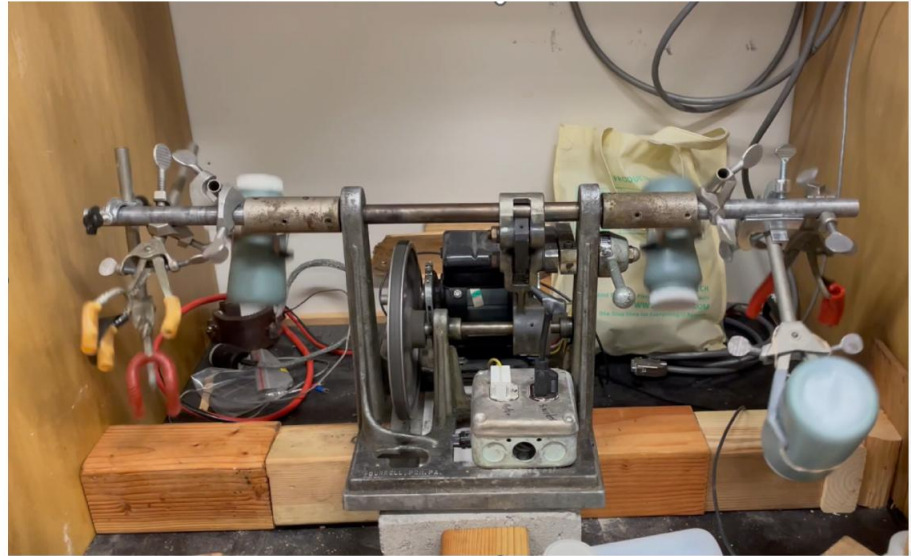


Figure 3.13: Two photos of different glauconite sand crushing methods. The left photo is glauconite sand that is being sheared in the Humbolt malt mixer. The right photo is a wrist-action shaker is being used to crush glauconite sand with ceramic balls in plastic bottles.



Figure 3.14: The resedimentation process, developed by the TAG Lab is summarized here in three photos. Left: First the glauconite sand and clay are mixed at the desired fines content, salinity, and water content. Middle: After mixing sufficiently, the soil slurry is de-aired using a vacuum. Right: the author used a syringe to deposit the de-aired soil slurry atop a porous stone and filter paper within a lubricated resedimentation tube.



Figure 3.15: A photo from the TAG lab showing a suite of resedimented specimens of glauconite sand and glauconite clay, prepared at different fines contents. Observe the weights that are applied to consolidate the specimens to the desired effective stress.

4 Laboratory Testing Equipment

4.1 Introduction to Laboratory Equipment

This section describes TAG Labs testing equipment that has been automated by computer control. The Constant Rate of Strain (CRS) testing methods and apparatus is based on technology that was developed at MIT (Wissa, Christian, Davis & Heiberg, 1971). The triaxial testing methods and apparatus discussed in this section are based on technology that was developed at MIT (Sheahan & Germaine, 1992).

Soils laboratory testing has been greatly improved by the addition of electronic instrumentation and automation of the testing process. Combining traditional testing equipment (load frames, triaxial cells, etc..) with electronic components (motors, transducers, analog-to-digital converters) has allowed TAG Labs to monitor and maintain stress states and strains with a high degree of accuracy and precision. The following sections provide a comprehensive overview of CRS test and triaxial test equipment.

4.2 Constant Rate of Strain Apparatus

The CRS tests performed in this study utilized a Trautwein brand C200000 CRS apparatus. This apparatus consists of a rigid cell composed of a steel specimen ring, a steel or plastic pressure chamber, a steel base with drainage ports, a steel top with a piston bearing, rubber O-rings seals, and bolts to hold everything together. During CRS testing, soil specimens are fitted into a rigid steel ring and contained on each end by a layer of nylon filter and a porous stone. The ring is fixed at the bottom of the device beneath a steel piston that imposes one-dimensional deformation on the specimen.

Before loading, the specimen is incrementally back pressured to 400 kPa to ensure saturation via a Pressure Volume Actuator (PVA). During the test, the sample is loaded slowly at a constant rate of strain, measured by a pair of external LVDTs. The CRS uses an external load cell to measure the load applied to the specimen. The load is applied using a PVA that is connected to a pressure jack that slowly raises the CRS apparatus. The steel piston pushes against the load cell and applies a reaction force to the soil specimen. The excess pore pressures generated during loading are measured at the bottom of the specimen by a diaphragm type pressure transducer. See Figures 4.1 and 4.2 for a diagram of a standard Trautwein CRS apparatus and a photograph of a typical TAG Labs CRS setup.

4.3 Medium Pressure Triaxial Apparatus

The triaxial test was carried out using a TAG Labs medium pressure triaxial testing apparatus (Figure 4.3). This apparatus is an automated device capable of testing soils up to 10 MPa cell pressure. The medium pressure apparatus used in this research was developed by Anderson (1991) to study the mechanical properties of frozen sands. The device was modified later by

Abdulhadi (2009). This device is operated using three hydraulic pistons (PVAs) that modulate pressure to a hydraulic load jack for the axial load, the cell pressure, and pore or back pressure. Just like the CRS, the motors controlling the PVAs are controlled by Maxon ESCON controllers which are powered with a 50V DC power supply. A custom-designed Strawberry Tree digital to analogue converter provides the command voltage from the computer to the motor controllers. The computer receives signals from several transducers (axial strain, axial load, volumetric strain, cell pressure, and back pressure) through a custom-made AD1170 A/D analog to digital converter (ADC). After receiving the signals, the computer compares the measured values to the target values and determines a corrective action, sending a signal to the control box that is converted to voltage and adjusts the PVAs. Having established a computer-controlled feedback loop, the state of stress and deformation of the soil can be controlled by QuickBASIC (QB) algorithms on the computer. Readings from each transducer are collected by a central data acquisition system (DAQ). The different components of the computer-controlled feedback loop and DAQ used to carry out CRS and triaxial testing are discussed in greater detail in subsequent paragraphs.

4.4 Transducers

There are three types of transducers used during CRS and triaxial testing: Linear Voltage Displacement Transducers (LVDTs), pressure transducers, and load cells. This section aims to provide an understanding of how each transducer works and how it is used in this research.

Two Transtek Inc. LVDTs are used within the triaxial testing apparatus to measure axial strain and volumetric strain of the specimen. Two LVDTs are used during CRS testing to measure axial strain. Axial strain is measured by monitoring the displacement of the piston and volumetric strain is measured using the displacement of the back pressure PVA piston. LVDTs are composed of a ferromagnetic core that moves within a tube composed of three solenoidal coils (Figure 4.4). As the core moved through the tube, the voltage changes based on the relative displacement from the zero position of the device.

LVDTs are capable of measuring displacements on the level of microns. They have low hysteresis, are nearly frictionless, and have a high linear range. LVDTs used in the TAG Lab have a 5 V DC range (-2.5 to +2.5 V). They are capable of measuring deformations up to one inch for axial strain and two inches for the volumetric strain LVDT. LVDTs are calibrated using a micrometer and the DAQ, and the devices are used with a 5 V DC input voltage.

The load cell used in the medium pressure triaxial cell is a Data Instruments JP-100, a 1000 lb (~454 kg) capacity load cell. Unfortunately, these models are no longer manufactured. Because the load cell is contained within the triaxial cell, the frictional effects caused by the gasket seal on the piston can be ignored, and the self-weight of the piston is greatly reduced by the upward pressure of the cell fluid (Figure 4.5).

CRS tests use a CAS Corporation (model SBA-2.5KLB-I) 2,500 lb (11.1 kN) capacity load cell. This s-shaped load cell is fitted with strain gages that are calibrated to measure load. The input and output voltages are monitored during loading, with the changes of voltage being proportional to the elastic deformation of the steel section. Because the load cell is external to the CRS apparatus, the self-weight of the piston applied to the soil during testing must be accounted for.

Omega™ PX-102 flush diaphragm pressure transducers are used to measure the cell pressure and the pore or back pressure during triaxial testing (Figure 4.6). These transducers have a 1000 PSI (6.89 MPa) capacity, and input voltage range of 5-6 V with a 100-mV output. They have been known to deviate less than 0.25% from the best-fit straight line (BFSL). Over the years, multiple students have recorded nearly constant calibrations factors, demonstrating the durability and accuracy of these measurement devices.

Data Instruments AB/HP diaphragm type pressure transducers with 200 PSI (1.38 MPa) capacities are used during CRS testing. These transducers are like load cells and consist of a steel diaphragm with two strain gauges. Changes in voltage are proportional to the elastic deformation of the diaphragm.

4.5 Pressure Volume Actuator

During triaxial tests, the axial load, cell pressure, and back or pore pressure are regulated by PVAs. The PVAs have a pressure tolerance of 14 MPa and a volumetric capacity of 47 cm³. A 0.5 tone Duff-Norton inverted ball-screw jack is driven by a Maxon Motors continuous output servomotor. The screw jack pushes a piston with a constant cross-sectional area of 2.85 cm² upwards into the hydraulic chamber of the PVA (Figure 4.7). Because of its constant area, volume change can be computed by measuring displacement.

The axial PVA generates hydraulic pressures within the PVA to push a jack which drives the base upon which the triaxial cell is situated against a stationary crossbar with a 10-ton capacity. The pressure is transferred to the specimen through the steel piston attached to the internal load cell. The axial PVA contains typical hydraulic oil.

Besides the axial load PVA, two other PVAs are used to control cell pressure and pore pressure. These two PVAs contain 20 centistokes silicone oil and salt-water. Silicone oil is used within the triaxial cell because it cannot permeate the membrane, due to its large molecular size and low polarity. It is also non-conductive, making it possible to operate electronics like the internal load cell within the device. The salinity of the pore fluid PVA is adjusted to match the salinity of the resedimented specimen, typically 35 g/L for offshore soils.

Given the nature of the CRS test, the pore pressure is not controlled by a PVA. The axial PVA contains hydraulic oil, but the cell PVA contains saltwater which is used to back pressure saturate the soil specimen.

4.6 DC Motor Controllers

The DC control boxes were developed and built within the TAG Lab by Dr. Germaine. Within the box, a control card sends commands to three Maxon Motor ESCON 50/5 servo controllers. These servo controllers produce a pulse width modulated signal (PWM) that controls the speed and torque of the motors. Connecting via USB to ESCON software allows the user to adjust the gain and output on the servo controllers. This allows the motor to move faster with the same control voltage coming from the computer. The box also contains an AC to DC power supply, a fan, and electronics that control the limit switches and manual switches on the outside (Figure 4.8).

4.7 Computer Control System

The triaxial control system was first automated at MIT (Sheahan, 1991). Over the years, improvements to the computer control program and feedback loop for the apparatus's driving system have enhanced quality control and precision. For example, during the K_0 consolidation phase of a CK₀UC triaxial test, the strain rate and cell pressure are computer controlled. In this phase, the feedback loop is governed by the axial and volumetric strain LVDTs. The computer processes voltage signals from the transducers via a 22-bit resolution ADC, capable of detecting changes as small as 1.2 μ V.

After reading the instrumentation, the computer compares axial strain to volumetric strain and determines a corrective action, adjusting cell pressure to minimize their difference. Using intermittent proportional control or continuous proportional-integral-derivative (PID) control, the QB program outputs a digital signal. This signal is sent through a Strawberry Tree Inc. D/A converter with 12-bit resolution and a ± 5 V range, which converts the digital signal into an analog voltage for the servo controllers. The servo controllers then drive the Maxon motors at a rate proportional to the command voltage, adjusting cell pressure PVA. This continuous closed-loop system ensures that the K_0 condition (zero lateral strain) is closely approximated throughout the consolidation phase of the test. See Figure 4.9 for a diagram of the computer-controlled triaxial testing system.

4.8 Data Acquisition System

The TAG Lab collects high precision data from pressure transducers, LVDTs, and load cell using a Hewlett Packard HP3497-based central data acquisition system (DAQ). The central DAQ is a highly accurate, low-noise data acquisition system that automatically adjusts to different signal levels. It uses dual slope integrating to eliminate noise and ensure high accuracy digital measurements. The DAQ has 200 channels, giving it the capability to collect data from 200 transducers throughout the lab simultaneously. The DAQ is connected to a Windows PC, containing custom-made software called Windap. Windap allows TAG Lab users to create data-recording tasks with desired channels, with a specified reading interval and total number of readings. The software also allows users to save data, take manual readings and retire tasks as

necessary. This ability to modulate reading intervals is particularly advantageous for capturing nonlinear soil behavior during triaxial testing.

4.9 Data Analysis

After collecting the .dat file from the central DAQ, voltages must be converted into engineering parameters of interest. The most straightforward method is to input the data into an Excel file that contains calibration factors for each transducer and formulas to convert normalized voltage to engineering parameters. For CRS testing, students use a standardized data reduction template which accepts each transducer readings and outputs the soil parameters of interest (e.g. coefficient of compressibility, permeability etc.). However, because of the additional parameters, it is less practical to reduce triaxial testing data with Excel. For this reason, all data reduction is performed using the QuickBASIC triaxial test data reduction code that was developed by Dr. Germaine and his students. This program accepts specimen dimensions, channels, normalized zero voltages, calibration factors, and transducer data and outputs all parameters of interest. Given the complexity of triaxial testing methodology, having a standardized code greatly reduces the probability of making calculation errors. The code was also altered by McAneny (2022) to account for the apparatus compressibility of the triaxial device. Overall, this QuickBASIC triaxial test reduction code greatly simplifies and increases the accuracy of the data reduction process.

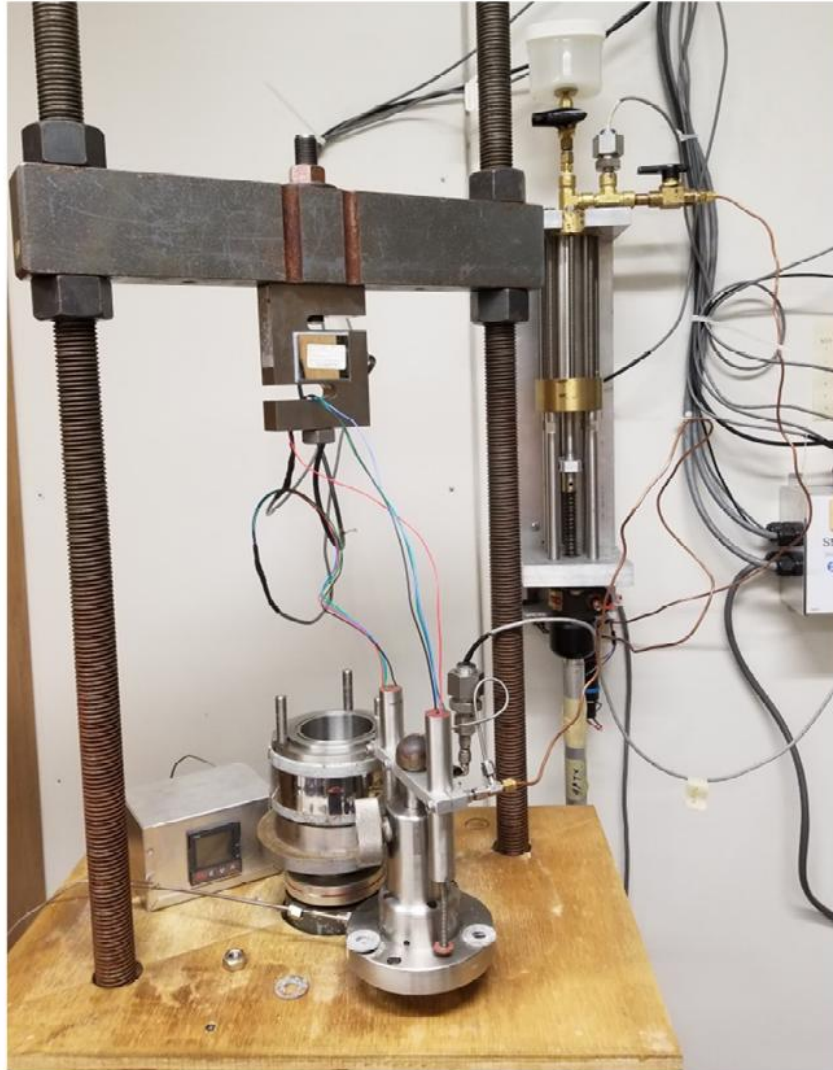


Figure 4.1: A photo of a typical Tufts Constant Rate of Strain (CRS) Compressibility Apparatus (adapted from Zablocki, 2018).

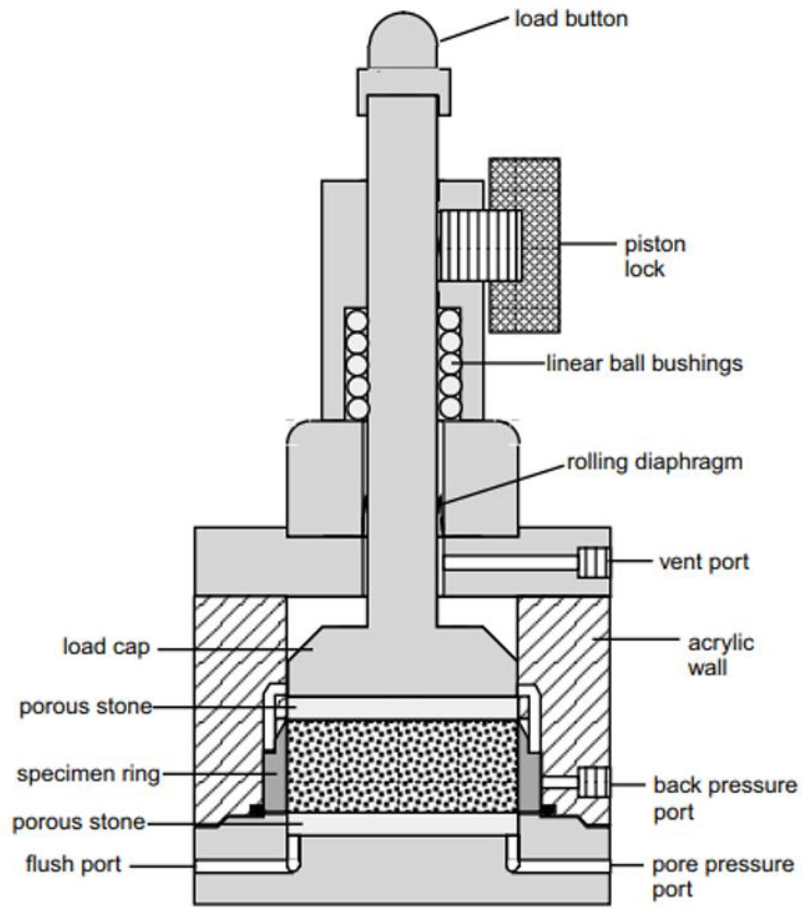


Figure 4.2: A cross-sectional depiction of a Trautwein C2000000 Consolidation Cell from manufacturer.



Figure 4.3: A photo of a medium pressure (0-10 MPa) triaxial testing apparatus (adapted from Doran, 2023).

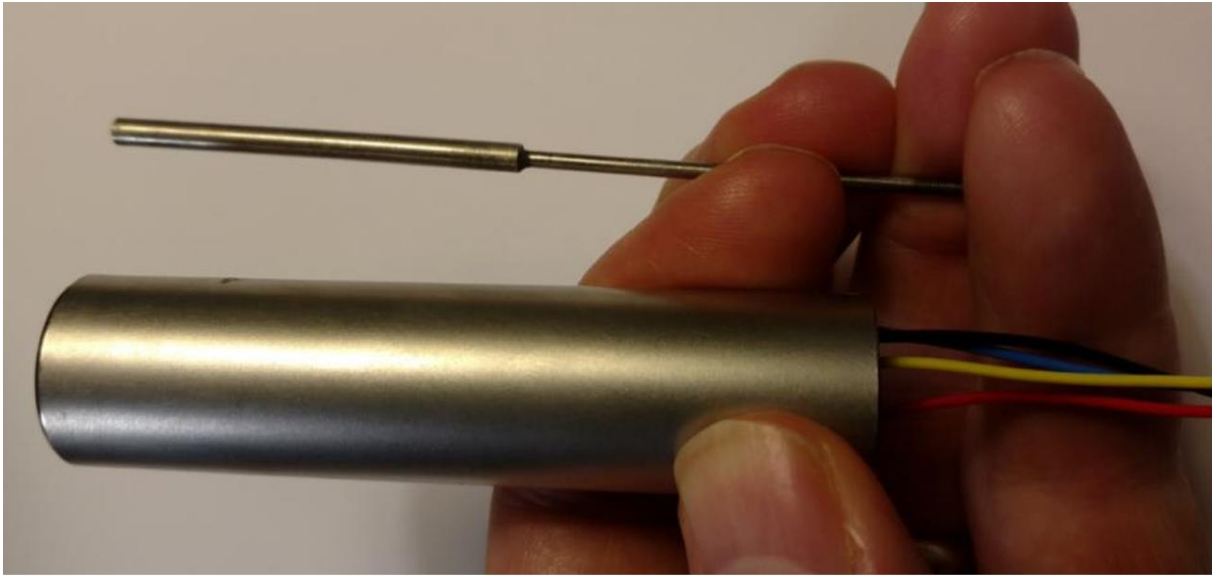


Figure 4.4: A photo of an LVDT. The ferromagnetic core above is slid into the tube below, producing a DC voltage change that is proportional (Dr. Germaine CEE 244 Course notes).

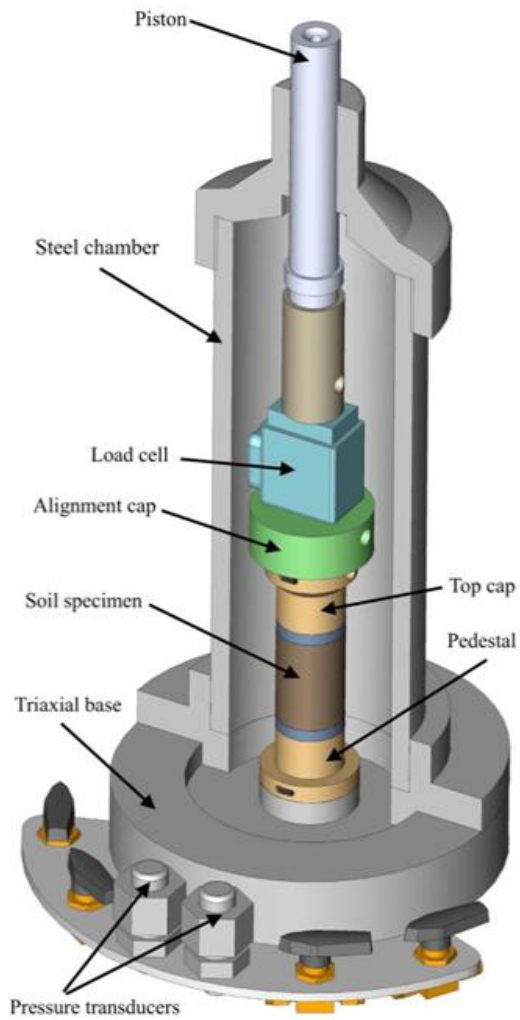


Figure 4.5: A cross-sectional depiction of a medium pressure triaxial cell (left). The load cell is attached to the piston and in nearly direct contact with the soil specimen (right).



Figure 4.6: A photo of a PX-102 1000 PSI pressure transducer from manufacturer.

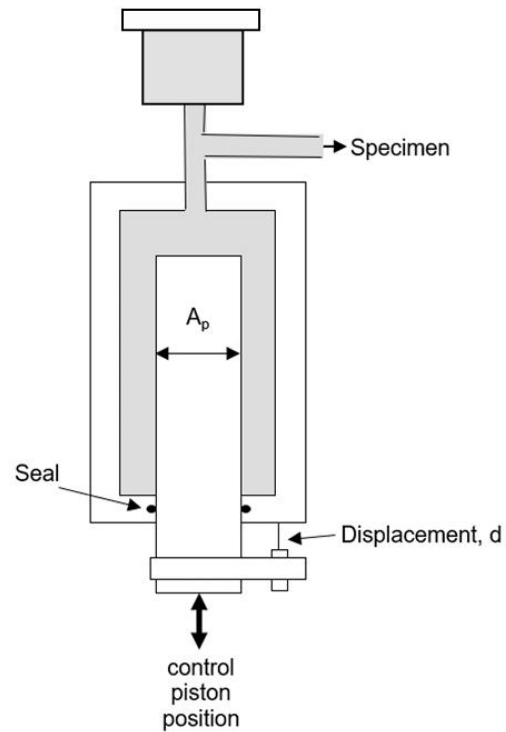
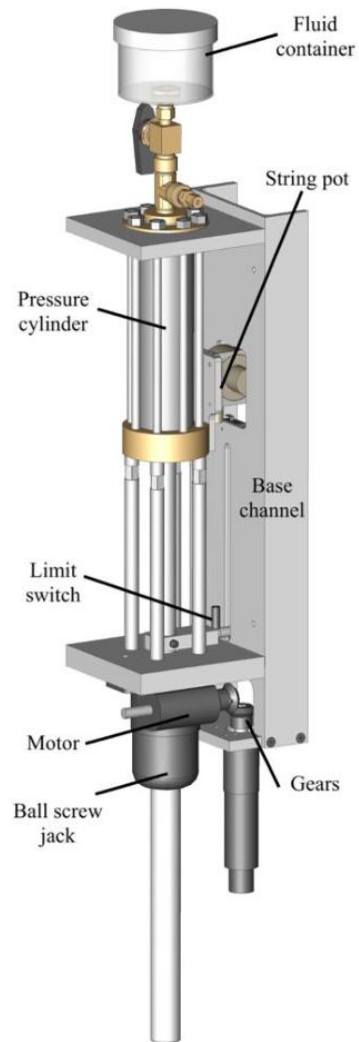


Figure 4.7: A diagram of a typical PVA setup used to regulate axial, cell, and back pressure during CRS and triaxial strength testing (left). On the right, a cross-sectional depiction of the pressure cylinder (adapted from Dr. Germaine's CEE 244 Course Notes).

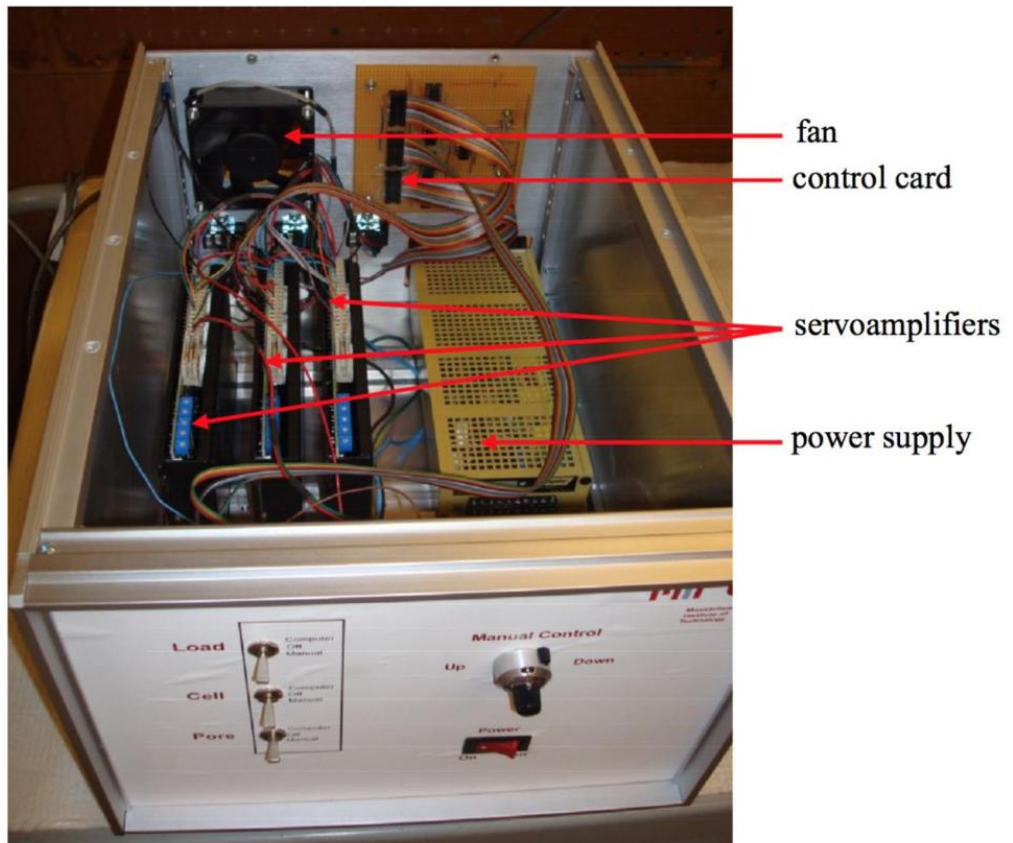


Figure 4.8: A photo of the DC motor control box. The control box accepts signals from the computer and adjusts the PVAs accordingly. Manual switches are on the front of the box.

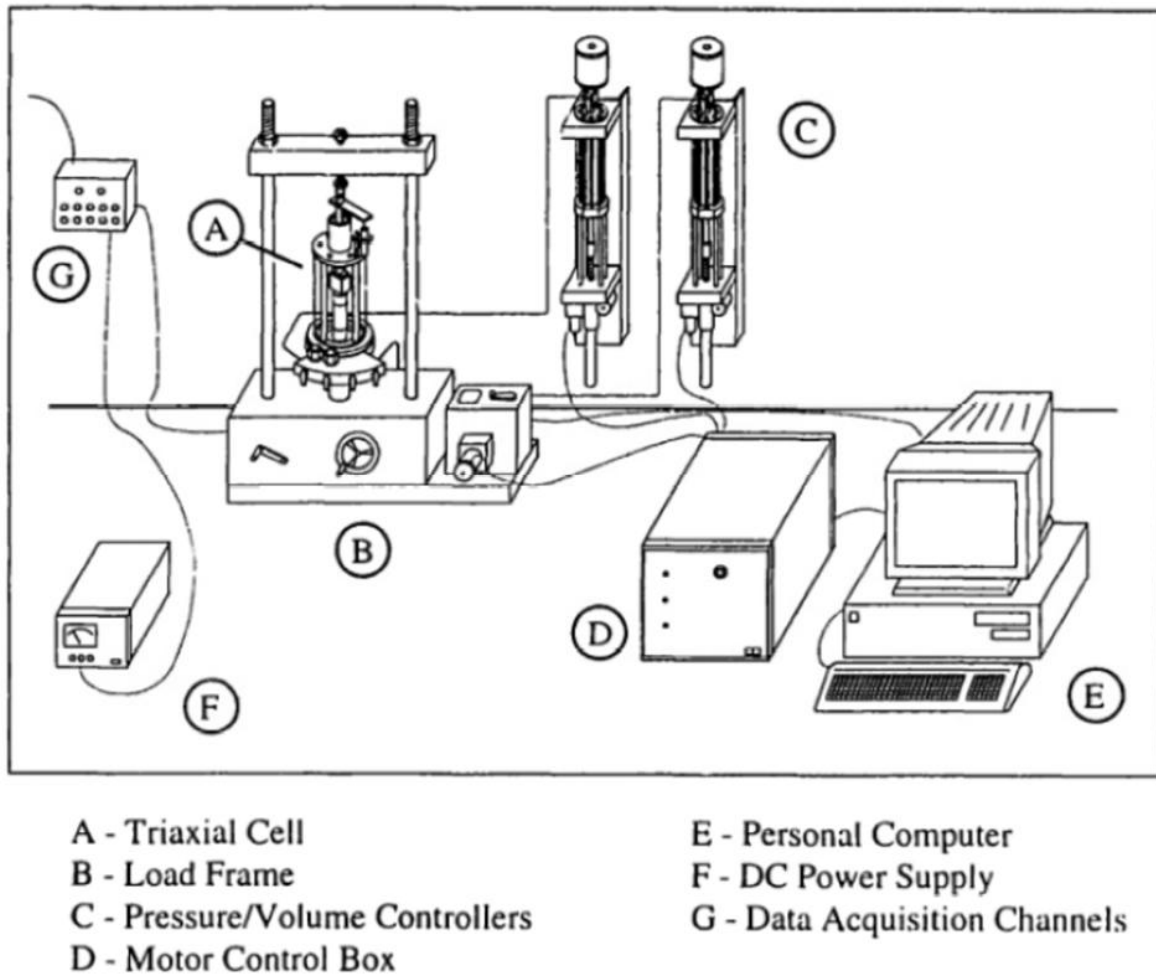


Figure 4.9: A depiction of the different components involved in a computer controlled triaxial testing apparatus (Adapted from Santagata, 1998).

5 Research Methodology

5.1 Introduction

The laboratory testing methods in this study follow ASTM standards and align with common geotechnical engineering practices. These tests fall into three categories: index property tests, consolidation tests, and triaxial shear strength tests.

Index property tests classify geomaterials and estimate properties such as compressibility, permeability, and shear strength using empirical correlations. Consolidation tests assess one-dimensional deformation and compression behavior. This study specifically employs constant rate of strain (CRS) compression tests, which also provide measurements of hydraulic conductivity.

Triaxial shear strength tests evaluate soil stiffness and internal friction mobilized during shearing. Unlike CRS tests, triaxial tests introduce an additional degree of freedom—confining stress—allowing for various shear strength assessments, including compression versus extension, lateral loading versus unloading, and drained versus undrained conditions. However, this research focuses exclusively on undrained compressive failure.

5.2 Index Property Tests

Index properties are measurable values used to classify soil. This classification process typically involves determining grain size distribution and soil behavior at various water contents. Coarse-grained soils are characterized using sieve analysis, while soils with significant fines content (>5% of particles smaller than 0.075 mm) require Atterberg Limits and hydrometer testing. Atterberg Limits testing identifies the water contents—the liquid limit and plastic limit—at which specific engineering behaviors occur. These values are essential for soil classification under the Unified Soil Classification System (USCS) and are standard practice in geotechnical engineering.

To support calculations in other tests, the specific gravity and maximum and minimum density of glauconite sand were determined. Additionally, an external laboratory conducted X-ray diffraction (XRD) testing on a glauconite sand specimen to assess its mineralogical composition. The interior microstructure of glauconite sand grains was also examined using a Scanning Electron Microscope (SEM) at the Tufts Micro and Nano Fabrication Facility (TMNF).

Throughout this study, glauconite sand refers to glauconite soils with less than 5% particles smaller than 0.075 mm. Although clays are strictly defined as particles with diameters smaller than 0.002 mm (Germaine & Germaine, 2009), the term glauconite clay in this study refers to glauconite soils with more than 50% particles smaller than 0.075 mm. This terminology aligns with the USCS definition. Resedimented specimens are labeled glauconite clay or glauconite sand with #% clay to indicate their composition.

5.2.1 Grain Size Analysis

The particle size distribution of glauconite sand and clay was determined through sieve and hydrometer testing. Each method is summarized below.

5.2.1.1 Sieve Analysis

A sieve test measures the distribution of coarse particles by placing an oven dried soil specimen at the top of a stack of sieves with progressively smaller openings. The stack is then shaken in a sieve shaker for 10 minutes. After shaking, for each sieve the total mass (mass of sieve and retained soil) is measured and recorded. The mass of the sieve is subtracted from the total mass to get the mass of the soil (M_T). The dry mass of the soil is calculated as Equation (5.1):

$$M_D = \frac{M_T}{(1 + \frac{\omega_c}{100})} \quad (5.1)$$

Where:

M_D = dry mass of specimen (gm)

M_T = total mass of specimen (gm)

ω_c = water content of companion specimen (%)

The percentage finer than each sieve size is determined by summing the masses retained on larger sieves, as shown in Equation (5.2):

$$N_m = \frac{M_D - \sum_{i=1}^m M_{d,i}}{M_D} \times 100 \quad (5.2)$$

Where:

N_m = percentage finer (%)

$M_{d,i}$ = dry mass of soil retained on sieve i (gm)

A grain size distribution plot is created by plotting the percent finer (ordinate) against the sieve size (abscissa).

5.2.1.2 Sedimentation Analysis

For soils with high fines content, sedimentation tests were conducted. These tests determine the distribution of particle sizes by measuring the change in specific gravity in a fluid column due to soil particle settling over time. The settlement of particles in fluid is governed by Stokes' Law, Equation (5.3), which relates the terminal velocity of a falling particle to the square of its diameter and the density difference between the particle and fluid:

$$V = \frac{d^2 g (\rho_p - \rho_s)}{18\eta} \quad (5.3)$$

Where:

V = velocity (m/s)

d = particle diameter (m)

g = acceleration due to gravity (m/s^2)

ρ_p = density of soil particles (kg/m^3)

ρ_s = density of the solution (kg/m^3)

η = dynamic viscosity of the solution ($\text{Pa}\cdot\text{s}$)

Since terminal velocity is proportional to the square of particle diameter, sedimentation results in a separation of particles by size within the fluid column. By measuring the suspension specific gravity via hydrometer at discrete time intervals, the particle size distribution can be back-calculated. The full methodology for the sedimentation test is detailed in Germaine & Germaine (2009) and ASTM-D7928 (*Particle Size Analysis of Soils*).

To ensure accurate results, 5 grams of Sodium Hexametaphosphate is added to the soil specimen to disperse flocculated clay particles. Additionally, temperature readings are taken with each hydrometer measurement to improve fluid specific gravity and viscosity calculations.

5.2.2 Atterberg Limits

Atterberg limits are used along with sieve analysis to classify soils according to the Unified Soil Classification System (USCS). The USCS specifies that Atterberg limits must be determined for soils with appreciable fines content ($>5\%$ smaller than 0.075 mm). These limits define water contents at which soil transitions between different states of material behavior.

Fine-grained soil behavior is highly dependent on soil-water interactions (Figure 5.1). As water content increases, interparticle spacing increases, causing the material to transition from a densely packed solid to a loosely packed liquid. As a result, soil strength decreases. While mechanical behavior is also influenced by mineral surface charge, particle size, and shape, Atterberg limits provide a standardized means of soil comparison and classification (Germaine & Germaine, 2009).

The Atterberg limits measured in this research include the plastic limit and the liquid limit, both defined in ASTM D4318:

- Plastic Limit (PL): The boundary between semi-solid and plastic behavior.
- Liquid Limit (LL): The boundary between plastic and fluid-like behavior.

5.2.2.1 Plastic Limit Determination

The plastic limit (PL) is determined by rolling a soil specimen against a glass plate until it forms a 3.2 mm diameter thread. If the soil crumbles at this diameter, it has reached the plastic limit.

The soil is then placed in a tare and covered to prevent evaporation. This process is repeated until at least 5 grams of material is collected. The specimen is oven-dried to determine its water content. At least three oven-dried specimens are required, and the average value is reported to the nearest integer as the plastic limit.

5.2.2.2 Liquid Limit Determination

The liquid limit is determined using the Casagrande cup and a grooving tool (Figure 5.2). This test simulates slope failure under dynamic loading. The number of blows (N) required to close a groove by 13 mm is recorded at different water contents.

Steps include:

1. Passing soil through a No. 40 sieve (0.425 mm).
2. Mixing with distilled water and allowing at least 16 hours for tempering.
3. Placing the soil in the Casagrande cup and leveling the surface.
4. Using the grooving tool to create a controlled slope.
5. Turning the crank at ~2 blows per second until the groove closes by 13 mm.
6. Extracting a soil specimen across the groove for water content determination.

At least three water content measurements (preferably four or five) are taken, ensuring data falls within the 15–35 blow range to maintain linearity. The water content (ω_c) is plotted against blow count (N) on a log scale. A best-fit line is drawn through the data points, and the liquid limit is taken as the water content corresponding to 25 blows.

The Plasticity Index (PI) is calculated as the difference between the liquid limit (LL) and the plastic limit (PL)

The PI is then plotted against the liquid limit on the Casagrande Plasticity Chart (Figure 5.3), which is required by USCS for fine-grained soil classification.

5.2.2.3 Applications of Atterberg Limits

Beyond classification, Atterberg limits are useful for assessing field variability in natural soil deposits. Comparing in situ water content to the plastic and liquid limits provides insight into soil behavior. Additionally, Atterberg limits help identify sensitive soils, soils with water contents exceeding their liquid limit. These soils are often found in regions affected by glacial rebound or marine sediment leaching.

5.2.3 Specific Gravity

Specific gravity (G_s) is the ratio of a soil's density to the density of distilled water at 20 °C, as defined by Equation (4). It is essential for computing phase relationships in geotechnical tests, such as void ratio variations with effective stress during compression testing. However, due to minimal variation between different soils, specific gravity is not typically used for soil classification. The method for determining specific gravity follows ASTM D854: Specific Gravity of Soil Solids by Water Pycnometer.

$$G_s = \rho_s / \rho_w \quad (5.4)$$

Where:

G_s = specific gravity at 20°C (dimensionless)

ρ_s = density of solids (g/cm³)

ρ_w = density of water at 20°C (g/cm³)

The water pycnometer method relies on Archimedes' Principle, which states that the volume of submerged soil particles equals the volume of displaced water. A crucial step in this method is accurately determining the volume of the iodine flask or pycnometer. This volume is experimentally obtained by measuring the mass of the container filled with water at various temperatures (18–30°C) and using Equation (5.5):

$$V_{B_{TC}} = \frac{M_{B+W_T} - M_B}{\rho_{w_T}} \quad (5.5)$$

Where:

$V_{B_{TC}}$ = volume of the volumetric at temperature T_C (cm³)

M_{B+W_T} = mass of the volumetric and water at temperature T (g)

M_B = mass of the volumetric (g)

ρ_{w_T} = density of water at temperature T (g/cm³)

Having established the volume of the iodine flask by calibration, three measurements are required to determine the specific gravity of the soil: the mass of the volumetric, water, and the soil, repeated three times, the temperature during each measurement, and the dry mass of the soil, determined after the test. The specific gravity at the test temperature and at 20°C is determined using Equations (5.6) and (5.7) (Germaine & Germaine 2009).

$$G_{S_T} = \frac{M_s}{(M_{B+W_T} + M_S) - M_{B+W+S_T}} \quad (5.6)$$

Where:

M_{B+W+S_T} = mass of the volumetric, water, and soil at temperature T, determined experimentally (g)

M_S = mass of dry soil (g)

M_{B+W+S_T} and M_{B+W_T} are taken at identical volumes and temperatures.

$$G_s = G_{s_T} \frac{\rho_{w_T}}{\rho_{w_{20}}} \quad (5.7)$$

Where:

G_{s_T} = specific gravity at the test temperature T

ρ_{w_T} = density of water at the test temperature T (g/cm³)

$\rho_{w_{20}}$ = density of water at 20°C (g/cm³)

5.2.4 Maximum Density and Minimum Density

The mechanical properties of granular materials are highly influenced by interparticle density. For example, dense sands tend to dilate, generating negative excess pore pressures during undrained shear, while loose sands contract, leading to positive excess pore pressures. This variation in density can alter the effective stress path and shear strength of the material (Figure 5.4).

Granular soil density is influenced by particle gradation, roundness, and deposition rate. Well-graded sands typically achieve higher dry densities than poorly graded sands during compaction (Figure 5.5). Additionally, rapid deposition increases hindrance, where particles are forced to stay in their initial position, while slow deposition allows particles to adjust into a denser configuration.

In this research, the maximum void ratio (e_{max}) was achieved using the funnel method (ASTM D4254 Method A), which maximizes hindrance by depositing soil with a small fall height. The minimum void ratio (e_{min}) was determined using the pepper shaker method, which minimizes hindrance by allowing particles to settle slowly (Germaine & Germaine 2009).

To compute the void ratios, the mass and volume of the container are first determined. The volume is measured by filling the container with water, subtracting the container mass, and dividing by the water density. The solid volume (V_s) is calculated using Equation (5.8)

$$V_s = \frac{M_s}{G_s \rho_w} \quad (5.8)$$

where V_s is the solid volume, M_s is the mass of the solids, G_s is the specific gravity of the soil, and ρ_w is the water density. The void volume is the difference between the container volume and the solid volume. The void ratio is calculated as the ratio of void volume to solid volume. The relative density is computed using Equation (5.9):

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100\% \quad (5.9)$$

Relative density allows comparisons of mechanical properties based on density and isolates effects such as gradation and particle shape.

5.2.5 X-Ray Powdered Diffraction (XRPD)

A specimen of malt-mixed, oven-dried glauconite sand was sent to the James Hutton Limited Institute (JHLI) in Cragiebuckler, Scotland, for mineralogical analysis using X-Ray Powder Diffraction (XRPD). XRPD is a technique used to identify crystalline materials based on their interaction with X-rays. The process directs a monochromatic X-ray beam at a powdered specimen and measures the diffraction pattern. This diffraction pattern serves as a unique "fingerprint" for each mineral, enabling phase identification and crystallographic analysis.

XRPD relies on Bragg's Law, Equation (5.10), which explains how X-rays interact with crystal planes:

$$n\lambda = 2d \sin \theta \quad (5.10)$$

Where:

n is the order of diffraction (typically one)

λ is the wavelength of the X-ray beam

d is the interplanar spacing of the crystal lattice

θ is the diffraction angle.

Once the powdered soil specimen is placed into the diffractometer, the X-ray beam is directed at the specimen. The diffractometer rotates about the specimen and the detector measures the intensity of diffracted X-rays at varying angles (2θ). When the condition $\lambda = 2d \sin \theta$ is met, X-rays diffract off adjacent crystal planes, resulting in constructive interference. This interference creates diffraction peaks, which are recorded by the detector (Figure 5.6). These diffraction peaks are plotted against 2θ values and referenced to a database to identify the crystal phases present in the specimen.

JHLI prepared the glauconite specimens differently based on the type of mineralogy being tested. For bulk mineralogy, the total specimen was used, and a dried representative portion was mounted on a glass slide for XRPD testing. For clay mineralogy, a sedimentation procedure was applied to separate the clay fraction from the silt and sand-sized particles. After the

sedimentation process, the clay suspension was dried and mounted on a glass slide for XRPD testing (Zablocki, 2018).

5.2.6 Scanning Electron Microscopy

Scanning Electron Microscopy (SEM) was employed to visualize the interior microstructure of glauconite sand particles. Glauconite sand particles were sprinkled into a mold containing unhardened epoxy and de-aired within a vacuum chamber for 5 minutes. After the epoxy hardened overnight, the solidified puck was sanded with progressively finer grit sandpaper to expose a cross-sectional view of the particles' interior.

The glauconite epoxy specimen was then taken to the Tufts Micro Nano Fabrication Facility (TMNFF), where it was sputter-coated with gold using a Denton Desk V Sputter Coater. Sputter coating minimizes charging effects, enhances secondary electron emissions, protects electron-sensitive specimens, and improves the signal-to-noise ratio.

Charging occurs when non-conductive specimens (e.g., clays) accumulate electrons after exposure to the SEM's electron beam. Since charging distorts images and damages specimens, a 15-20 nm thick gold layer was sputter-coated onto the specimen to dissipate excess charges.

Sputter-coating involves evacuating a chamber to 50 millitorr, introducing Argon gas, and applying a high voltage to a gold target to ionize the Argon gas into plasma. The positively charged Argon atoms accelerate toward the negatively charged gold target, knocking gold atoms off the target. These gold atoms deposit as a thin, uniform layer on the specimen's surface (TMNFF, 2017).

Once sputter-coated, the specimens were imaged using a Tescan VEGA3 SEM. The SEM generates a focused beam of electrons from a tungsten filament in the electron gun. The filament is heated, and a high voltage (5-30 kV) accelerates the electrons toward the specimen.

The electron beam is focused and scanned across the specimen's surface using electromagnetic lenses and deflector coils. The electron beam interacts with the specimen, generating secondary electrons, which provide high-resolution topographic imaging.

The TMNFF SEM can achieve resolutions below 10 nm. The captured secondary electron signals are converted into grayscale images, revealing detailed surface morphology and microstructural information. Care must be taken when operating the SEM stage, as contact between the specimen and optics can damage the tool (TMNFF, 2020).

5.3 Constant Rate of Strain (CRS) Testing

The Constant Rate of Strain (CRS) testing methodology was first developed by Smith and Wahls in 1969 (Smith & Wahls, 1969). This method applies a controlled rate of one-dimensional deformation to a soil specimen while measuring the change in effective stress. The ability to backpressure-saturate specimens prior to compression, developed by Wissa, Christian, Davis,

and Heiberg in 1971, allows for the calculation of hydraulic conductivity throughout the test. The complete CRS test procedure is outlined in ASTM D4186/D4186M.

To begin a CRS test, a thin film of vacuum grease is applied to the specimen cutting ring to minimize shear stress at the soil-steel interface. After recording the cutting ring's mass, the cutting ring is mounted on the CRS trimming stand and gently lowered onto the top of the specimen. It is then pushed down through the specimen, which should have a slightly larger diameter than the ring. This creates a tight seal between the specimen and the ring's sidewalls, and the excess material around the cutting ring is removed. Next, the top and bottom of the specimen are trimmed to a controlled height using a wire saw, and the mass of the cutting ring, now containing the specimen, is recorded again.

The specimen is then placed in the CRS testing apparatus, sandwiched between nylon filter and porous stones on each end. After assembling the CRS apparatus and attaching the drainage lines, the specimen is backpressure-saturated for a minimum of 24 hours. According to Henry's Law, increasing the partial pressure of gas increases its solubility in a liquid. Therefore, increasing the back pressure to ~ 490 kPa—well above atmospheric pressure—forces any trapped air to dissolve into the pore fluid (Doran, 2023). Resedimented specimens are typically consolidated to ~98 kPa and unloaded to ~98 kPa (OCR = 4). To minimize strain, backpressure saturation is conducted at an effective stress of ~98 kPa or less, with the back pressure, cell pressure, and axial stress increased in increments of ~196 kPa, each lasting 10 minutes.

Following back pressure saturation, the specimen is loaded at a constant rate of strain, typically 1% per hour. Specimens are trimmed quite short, typically to about 1.265 cm, which allows the device to accommodate very high strain levels. The bottom drainage line is closed during loading to ensure unidirectional drainage and measure the excess pore pressure. The strain rate is calculated from one of the LVDT's mounted to the piston to monitor the deformation within the specimen. The computer controller utilizes a PID feedback loop to maintain the constant rate of strain. A second LVDT, also mounted to the piston but not connected to the computer controller, records readings that are logged by the DAQ system. The average of the two LVDTs is used to calculate the strain when analyzing the data in Excel.

The bottom of the CRS apparatus is connected to a pressure transducer that measures excess pore pressures during loading. Excess pore pressure is computed as the difference between the cell pressure (held constant at ~392 kPa) and the pore pressure, measured at the base of the specimen. This pore pressure is used to calculate the effective stress within the specimen and its hydraulic conductivity at each stress level. The hydraulic conductivity of a soil specimen is calculated after the test using the following equations (Zablocki, 2018):

$$\sigma'_{a,n} = \sigma_{a,n} - \frac{2}{3}u_{m,n} \quad (5.11)$$

Where:

$\sigma'_{a,n}$ = axial effective stress at time n.

$\sigma_{a,n}$ = total axial stress at time n.

$u_{m,n}$ = base pressure at time n.

$$k_n = \frac{1}{2} \frac{\varepsilon_n^\circ H_n H_0 \gamma_w}{u_{m,n}} \quad (5.12)$$

Where:

k_n = hydraulic conductivity at time n

ε_n° = strain rate at time n

H_0 = initial drainage height of the specimen

H_n = drainage height at time n

γ_w = unit weight of water

The drainage height at time n is calculated as Equation (5.13):

$$H_n = H_0 - \Delta H_n \quad (5.13)$$

The change in height of the specimen is calculated by Equation (5.14) which includes corrections due to apparatus compressibility and cell uplift forces.

$$\Delta H_n = \delta_0 - \delta_{af,n} - \delta_{ap,n} \quad (5.14)$$

Where:

δ_n = axial deformation

$\delta_{ap,n}$ = apparatus compressibility correction due to loading

$\delta_{af,n}$ = apparatus compressibility correction due to cell uplift forces

The total axial stress at any point during the test is calculated as Equation (5.15):

$$\sigma_{a,n} = \frac{f_{a,n}}{A} \quad (5.15)$$

Where:

$f_{a,n}$ = the net force

A = the specimen area

An accompanying data reduction file converts the transducer readings, recorded by the DAQ, into measured strain, void ratio, cell pressure, pore pressure, hydraulic conductivity, and effective and total stresses using the appropriate equations.

5.4 Triaxial Testing

5.4.1 Introduction

Triaxial tests on glauconite clay were performed using a medium-pressure automated triaxial apparatus. Since this study compares the strength of granular and fine-grained materials (i.e., glauconite sand versus glauconite clay), it is important to note that triaxial test setup differs for cohesionless and cohesive materials. Having performed only one triaxial test on glauconite clay, this study focuses on the procedure for cohesive materials. Reviewing triaxial testing methodologies from previous TAG Lab theses helps avoid common mistakes in future research.

5.4.2 Triaxial Test Setup

Proper setup is crucial for accurate medium-pressure triaxial testing. Errors in specimen height, area, or sealing can significantly affect results. The setup consists of three key steps: specimen preparation, initial pressure-up, and backpressure saturation.

5.4.2.1 Specimen Preparation and Initial Pressure-Up

On the first day of testing, the specimen is extruded, trimmed, and placed in the triaxial device. Input parameters and normalized zero values are entered into the computer before beginning the pressure-up sequence. The following steps, sourced from McAneny (2022), apply specifically to intact or resedimented specimens:

1. Record the test number and relevant details (such as name, material, and project) in the laboratory logbook and complete the necessary sections of the triaxial testing datasheet.
2. Wrap the trimmed and measured specimen in wax paper and seal it in a plastic bag to prevent desiccation during setup. Place this bag inside another bag containing a damp paper towel.
3. Prepare a 350 mL sea salt solution in a 1L vacuum flask, ensuring the appropriate salt concentration. Stir until all salt is fully dissolved. A vacuum flask smaller than 1L may be unstable and prone to tipping or breaking. For this study, a salt concentration of 35 g/L was used.
4. Carefully transport the specimen, membrane stretcher, alignment jig, vacuum flask, scissors, two large adjustable wrenches, and a rubber mallet to the triaxial testing station. Using a utility cart simplifies transportation.
5. Collect two porous stones, two nylon filter, two 0.025-inch-thick triaxial membranes (1.4-inch diameter), six O-rings (1.4-inch diameter), and vacuum grease.

6. Apply a thin layer of vacuum grease to the sides of the brass top and bottom caps, then wipe hands clean with a paper towel.
7. Position the porous stones on the top and bottom caps and cover with the filters.
8. Place the specimen on the bottom cap and carefully position the top cap.
9. Using the membrane stretcher, encase the specimen with one 0.025-inch-thick 1.4-inch-diameter membrane. Secure the membrane using the O-ring stretcher to place two greased O-rings on both the top and bottom caps. Reapply grease to the ends.
10. Install a second membrane in the same manner. This time, place one additional O-ring on each end, positioned between the previously placed O-rings. This ensures a tighter seal, reducing the risk of leaks. Be precise when placing the O-rings to avoid missing the narrow gap.
11. Slide the top drain line over the specimen and hold the top cap in place using the alignment jig. Secure the drain line to both the top cap and base with a 3/8-inch box wrench. Apply grease to the O-rings on the drain line and remove debris from the threaded holes before tightening.
12. With the alignment jig still in place, use the vacuum flask and tubing with rubber stoppers to apply a vacuum to the pore pressure transducer port on the base. This removes trapped air from the membranes and ensures the top cap is in close contact with the specimen. Maintain the vacuum until step 24. As degassing occurs, the water will stop boiling. The tubing should then be lowered further into the saltwater solution, where it should bubble slightly as gas is extracted. If no leaks are present, bubbling will cease. Ensure that both the top and bottom drain valves remain open during this process.
13. Remove the alignment jig and verify the top cap is still centered.
14. Verify whether the piston was retracted following the previous test using the two large wrenches. Failure to retract the piston could lead to specimen damage when the cell is lowered onto the base.
15. Grease the large O-ring on the base, then use an electric winch to lift the triaxial cell.
16. Gradually lower the triaxial cell onto the base at reduced winch speed. When it is a few inches above the base, connect the internal load cell to the 9-pin connector. Monitor the load cell channel on the voltmeter as the cell is fully lowered to ensure the specimen is not being crushed.
17. Secure the triaxial cell to the base using a torque wrench, nuts, and washers. Tighten to 35 N-m, alternating between opposing pairs to maintain even pressure on the base O-ring.

18. Initiate the central data acquisition system to record readings from all transducers and power supply channels. The initial load cell zero reading should be taken when the piston is not in contact with the specimen. Additionally, record voltage readings from the TX station PC, as different A/D converters may produce slight variations.
19. Use a rubber mallet to lightly tap the piston downward until it contacts the specimen's top cap. Observe the load cell channel for a voltage change, indicating contact. At this moment, manually record the axial LVDT zero reading in the DAQ system. A depth micrometer measurement of the protruding piston can also verify top cap contact. The LVDT position is set using a steel dummy specimen to ensure it remains within its linear range throughout testing.
20. Bolt the load frame reaction bar into place, ensuring that it is sufficiently level. Then, using manual controls, raise the base of the triaxial cell until the piston contacts the load frame. The steel ball serving as the moment break should still rotate freely without excessive resistance.
21. Fill the triaxial cell with fluid by pressurizing the silicone oil chamber to 20 PSI and opening the Swagelok valve at the base of the cell. Monitor the overflow bubbler; once the cell is full, silicone oil will appear in the clear tube. At this point, first close the base Swagelok valve, followed by the top valve connected to the bubbler.
22. Open the Cell PVA valve and the corresponding valve on the TX base to allow communication with the triaxial cell. Perform a manual reading on the DAQ to record the zero reading of the cell pressure transducer.
23. Close the Cell PVA valve while keeping the base valve open to the cell. The Cell PVA can now be used to gradually increase the cell pressure.
24. Enter normalized zero voltage values for the cell pressure transducer, axial LVDT, and load cell into the TX station PC. Input specimen dimensions and relevant transducer calibration factors. In the pressure-up subroutine, set the desired pressure levels. A cell pressure that is ~98 kPa higher than the effective stress experienced before extrusion is applied. A deviatoric stress of approximately ~9.8 kPa is also applied to maintain piston contact with the top cap, preventing separation due to hydrostatic loading.
25. Switch the cell and axial motor controls to computer mode, allowing automated test operation. The pore PVA motor should remain off during this phase. To assist with pressurization, use the same cell fluid pressure chamber used for filling. This conserves the stroke length of the Cell PVA piston.
26. Once the target stress is reached, begin vacuum saturation. At this stage, the water in the vacuum flask should be bubble-free. Keep the top and bottom drains open to ensure the entire pore fluid system (including the specimen, tubing, and PVA) remains under vacuum.

Disconnecting the vacuum hose from the side of the flask will allow water to enter, saturating the system. After this, reattach the pore pressure transducer and hand-tighten it. Open the pore PVA valve to the atmosphere and manually record the zero value for pore pressure. This can be entered into the system using the keyboard shortcut Ctrl + Break, pressing F6, and inputting the normalized zero value for the “zpore” variable.

27. Close both the top and bottom drainage lines to allow pore pressure buildup. Keep the pore PVA motor off, and use the current volumetric strain LVDT voltage as the zero reference.
28. Clean up all tools and confirm that the computer control system is stable.
29. Maintain the setup for 24 hours. During this period, an axial strain of less than 0.4% (ϵ_a) and a pore pressure increase of approximately ~98 kPa should be observed. If, after this duration, the pore pressure equals the cell pressure, an internal leak may be present, allowing cell fluid to enter the specimen. In such cases, dismantle the setup, first reducing axial stress, then cell pressure. Repeat the setup process with a new set of specimen measurements and mass recordings.

5.4.2.2 Backpressure Saturation

Backpressure saturation is achieved by incrementally increasing cell and pore pressure in small steps while maintaining constant effective stress. The pressure is increased by ~98 kPa every 5–10 minutes until reaching ~392 kPa. The specimen is held at ~392 kPa pore pressure for 24 hours or until volumetric strain stabilizes.

Saturation is assessed using the Skempton B parameter:

$$B = \frac{\Delta u}{\Delta \sigma_c} = \frac{1}{1 + n \frac{C_w}{C_{sk}}} \quad (5.16)$$

Where:

Δu = the change in pore pressure

$\Delta \sigma_c$ = the change in cell pressure

n = porosity

C_w = the compressibility of water

C_{sk} = the compressibility of the soil skeleton

A B-value greater than 0.96 indicates full saturation. If the B-value is too low (below 0.90), additional backpressure increments are applied, and saturation continues for at least 24 hours until the desired B-value is reached. Note that B-values less than 0.90 have been observed in the TAG Lab for fully saturated specimens that are highly stiff and over consolidated. If the B-value exceeds 1.0, an internal leak is suspected, and the setup must be re-evaluated.

5.4.3 K_0 Consolidation

K_0 consolidation maintains a constant cross-sectional area during consolidation, requiring a leak-free system and precise PID control. Before initiating K_0 consolidation, the pore pressure transducer is monitored with drainage lines closed to check for leaks.

The K_0 consolidation subroutine dynamically adjusts cell pressure to maintain equal volumetric and axial strain changes, assuming the specimen remains cylindrical. The process is conducted at a strain rate of 0.06%/hr to prevent excess pore pressure buildup. Due to significant axial strain, K_0 consolidation can take up to three weeks to complete.

5.4.5 Undrained Shear

Following consolidation, undrained shear tests are performed under strain-controlled loading conditions. The specimen is sheared in compression while maintaining constant cell pressure. The strain rate for undrained shear is typically set at 0.5% per hour.

During shearing, axial strain increases, compressing the specimen while excess pore pressures are measured. It is important to correct for membrane effects during consolidation and shear. The principal stress difference is corrected for membrane effects using Equation (5.17):

$$\Delta(\sigma_1 - \sigma_3)_m = (4E_m t_m \varepsilon) / D_c \quad (5.17)$$

Where:

E_m = the membrane modulus

t_m = the thickness of membrane

ε = axial strain

D_c = the specimen diameter at the end of consolidation.

The corrected principal stress difference is calculated as Equation (5.18):

$$(\sigma_1 - \sigma_3)_c = \frac{P}{A} - (\sigma_1 - \sigma_3)_m \quad (5.18)$$

Where:

P = the applied force

A = the corrected specimen area.

$\Delta(\sigma_1 - \sigma_3)_m$ = the membrane correction

Note that the membrane correction is also applied during the consolidation stage of the test, but the equations are not provided here.

To accurately calculate the load, one must divide the applied force by the correct specimen area. The specimen area after consolidation is calculated using a cylindrical correction, while the specimen area during shear is calculated using a parabolic correction (Zablocki, 2018; Germaine & Ladd, 1988).

The specimen height at the end of consolidation is determined using Equation (5.19):

$$H_c = H_0 - \Delta H_0 \quad (5.19)$$

Where:

H_c = Height of the specimen at the end of consolidation

H_0 = Initial specimen height

ΔH_0 = Change in the height of the specimen during consolidation

The cross-sectional area at the end of consolidation, based on the cylindrical area correction, is given by Equation (5.20):

$$A_c = (V_0 - \Delta V_{sat} - \Delta V_c)/H_c \quad (20)$$

Where:

V_0 = Initial specimen volume

ΔV_{sat} = Change in specimen volume during saturation, calculated by the computer from the measured axial strain during saturation

ΔV_c = Change in specimen volume during consolidation, calculated by the computer from the measured axial strain during consolidation

The cross-sectional area during and after shear in compression, is calculated using a parabolic area correction as Equation (5.21):

$$A_{parabolic} = \left(-0.25 + \frac{\sqrt{25 - 20\varepsilon - 5\varepsilon^2}}{4(1 - \varepsilon)} \right)^2 \quad (5.21)$$

Where:

A_c = Specimen cross sectional area at the end of consolidation

ε = Axial strain

Undrained tests provide key insights into the effective stress behavior of the soil, allowing the determination of shear strength parameters under controlled loading conditions.

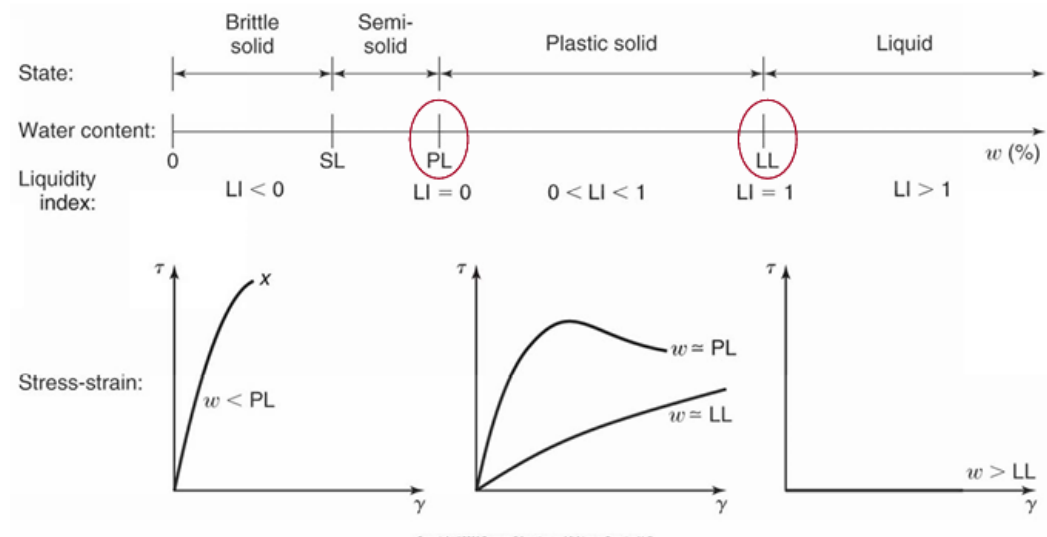


Figure 5.1: A generalized depiction of what stress-strain behavior is expected at the plastic and liquid limits (Source: Dr. Jen's Course Notes, CEE042).



Figure 5.2: A picture of the Casagrande cup, plastic grooving tool, mixing spatula, water content tares, and resilience tester (Source: Germaine & Germaine 2009).

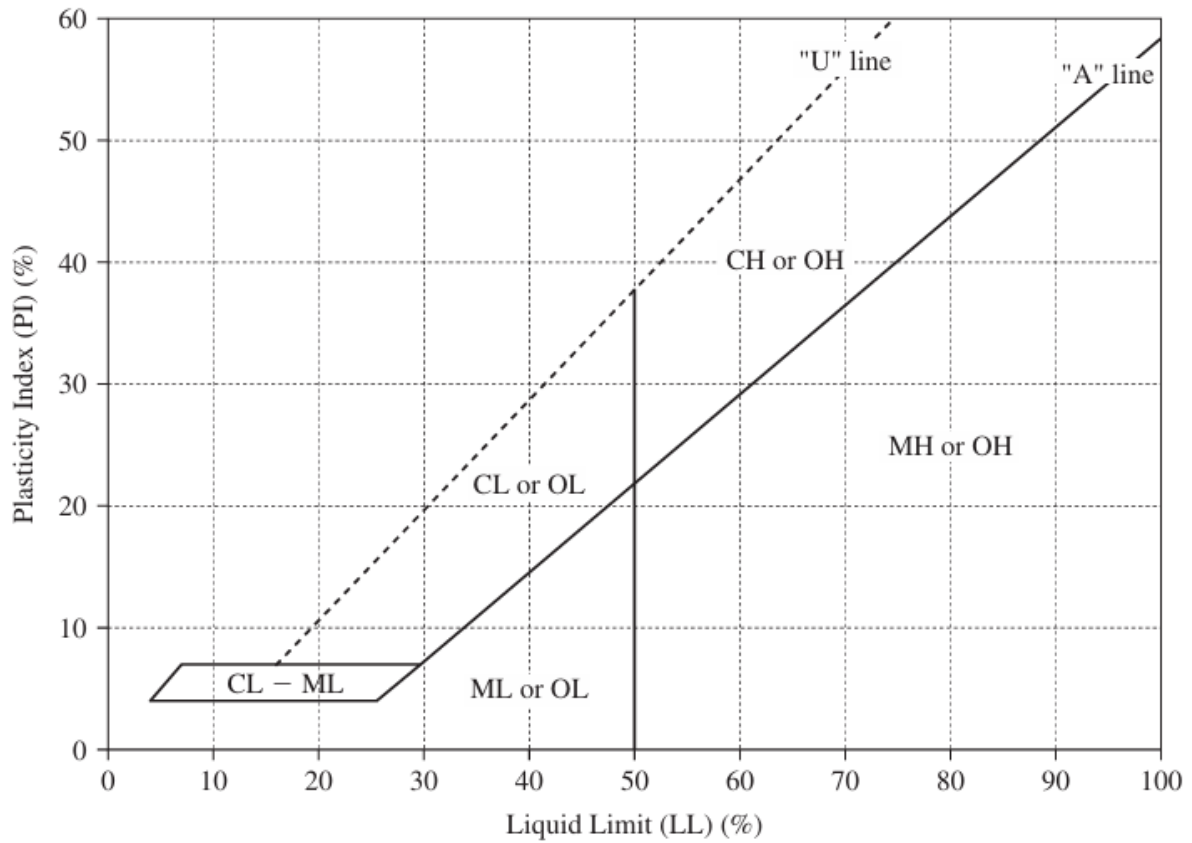


Figure 5.3: The Casagrande Plasticity Chart used to classify fine-grained soils according to the USCS

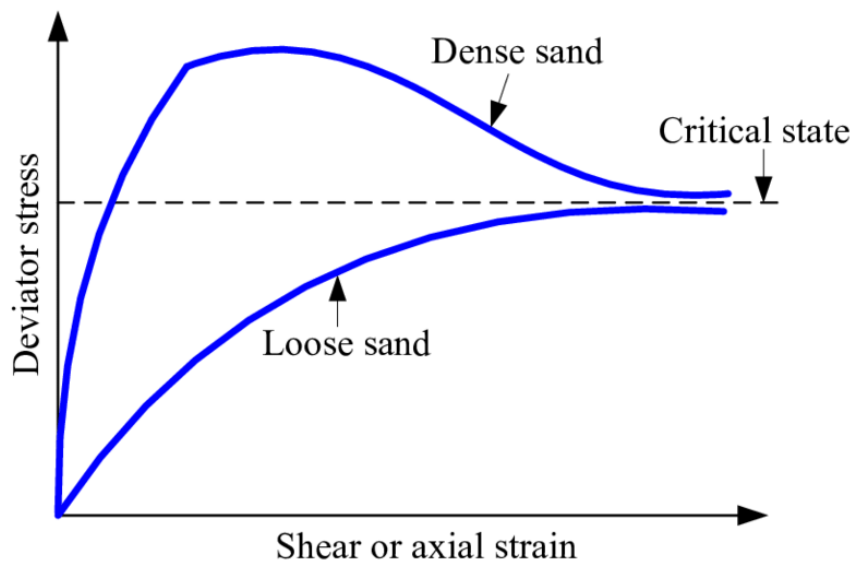


Figure 5.4: A conceptual graph depicting the effect of gain density on the shear strength of sand (Rahimi, 2014).

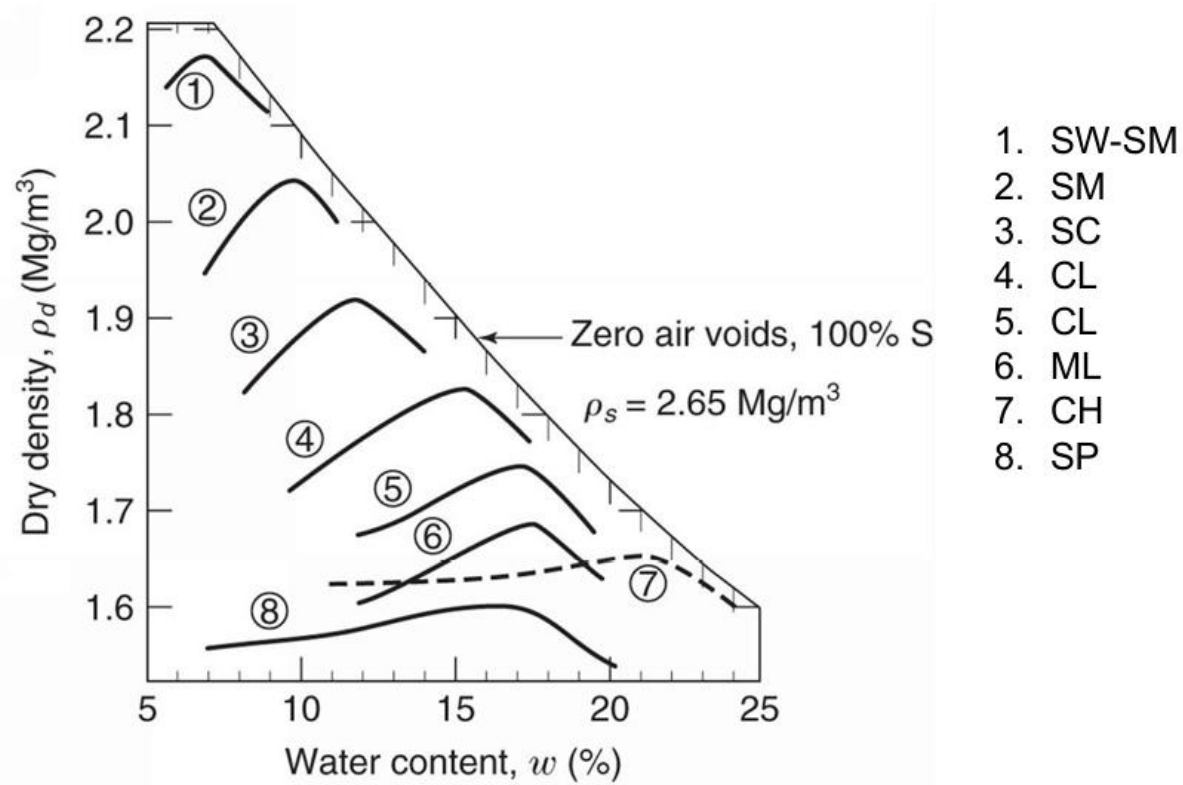


Figure 5.5: A plot of how compaction behavior varies by soil type during Proctor testing. Observe how poorly graded sands (SP) achieve significantly lower dry densities than well-graded sands (SW) (Source: Johnson & Sallberg, 1960).

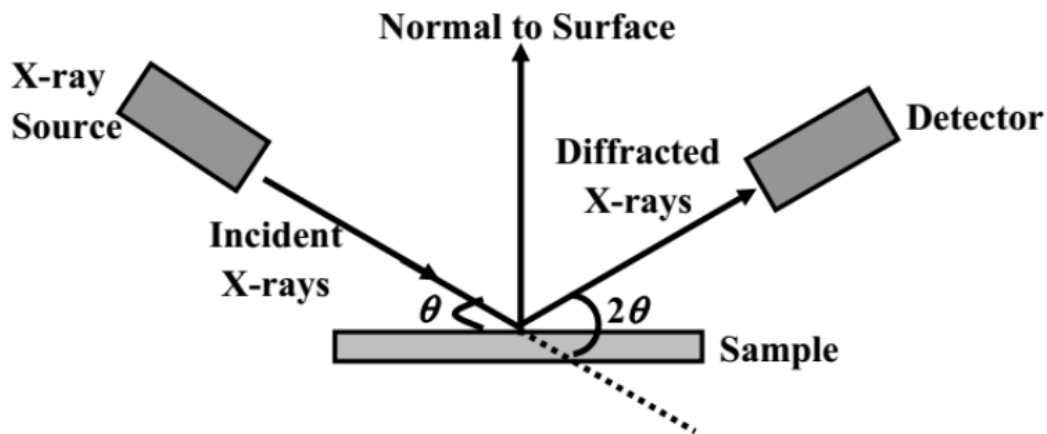


Figure 5.6: A schematic representation of the principles of X-Ray Powdered Diffraction (XRPD) technique. (Source: Andualem, 2020).

6 Experimental Results and Discussion

The following chapter is a full account of the experimental results of this study followed by a discussion of their significance to the problem of pile refusal in glauconite sands. A complete list of tests conducted during this study is provided in Table 6.1 for the reader to reference. The chapter begins with a discussion of effort to characterize the processed glauconite sand material via grain size analysis and determination of maximum and minimum void ratios and specific gravity. Next, a discussion of efforts to crush processed glauconite sand grains and study the fine-grained particles via sedimentation, Atterberg limits, and XRPD testing and SEM imaging. Then, a discussion of results of CRS testing of processed glauconite sand and resedimented specimens and the determination of parameters such as compression index (C_c), recompression index (C_r), secondary compression index (C_{ae}), permeability (K), and coefficient of compressibility (c_v). This is followed by a discussion of triaxial testing results from one CK₀UC triaxial test on glauconite sand with 75% fines content. Difficulties encountered during resedimentation and triaxial testing procedures are also discussed. The properties of fine-grained glauconite are compared with the properties of in-house geomaterials: Resedimented Boston Blue Clay (RBBC) and Resedimented Gulf of Mexico-Eugene Island (RGoM-EI) Clay. Finally, the chapter concludes with a discussion of the implications of this research for pile drivability in glauconite sands.

6.1 Overview of Experiments

6.1.1 Characterization of Processed Glauconite Sand

After an extensive washing process, described in Chapter 3, to remove the fine-grained riverbank material from the glauconite particles, the author produced about 4 to 5 kilograms of glauconite sand. A simple sieve analysis was conducted on the processed glauconite sand (Figure 6.1). The material was 98.9% sand and 1.1% fines. A coefficient of curvature (C_c) of 1.14 and a coefficient of uniformity of 2.57 was determined. The material was classified as uniform or poorly graded sand (SP) according to the USCS.

Digital microscope images of the processed glauconite sand are provided in Figure 6.2. The grains are dark green to black in color and well-rounded. Fissures or cracks can be observed across the grains' surfaces. To view the interior microstructure, the grains were frozen in epoxy, shaved down with sandpaper, then imaged with the TMNFF SEM (Figure 6.3). The SEM image in Figure 6.3 indicates that the cracks propagate through most of the particles, but there is considerable variability in how deep the cracks penetrate. These cracks are a likely place for failure to occur during particle crushing.

The minimum and maximum void ratios were determined as 0.860 and 0.558 respectively. This corresponded to a minimum and maximum dry density of 1.25 and 1.733 g/mL respectively.

The specific gravity of the processed glauconite sand material was 2.70, compared to 2.78 for the unprocessed riverbank material. Both specific gravity values are significantly less than the value of 3.04 reported in Westgate et al. (2023) (See Table 2.2).

6.1.2 Analysis of Particle Crushing

The processed glauconite sand was ground with a mortar and pestle and imaged using a digital microscope as shown in Figure 6.4. Fracturing the particles in this manner generated significant silt-sized ($< 75 \mu\text{m}$ and $> 2 \mu\text{m}$) and some clay-sized ($< 2 \mu\text{m}$) particles. Note that clay-sized particles could not be photographed with this device.

Figure 6.5 is an SEM image taken at a scale of $20 \mu\text{m}$, demonstrating how silt and clay-sized particles were generated after shearing with sandpaper. These particles can be seen flaking off the exposed surface. Note that this was observed macroscopically by the author while shaving down the glauconite sand epoxy puck, as green dust became suspended in the air.

The author conducted several sedimentation tests to understand how the nature of mechanical work applied to the bulk sand material (compressive versus shear loading) influences the degree of fines generation. HY 225 is a sedimentation test that was carried out on material that had been sheared in the malt-mixer for 3 hours then further crushed using ceramic balls and a wrist-action shaker device for 24 hours. HY 226 is a sedimentation test that was carried out on material that had been axially compressed to 300 MPa using a hydraulic press (Figure 6.6). HY225 observed an 80% increase in fines content, with 20% due to an increase in silt and 60% due to an increase in clay. HY 226 observed a ~45% increase in fines content with ~30% being silt-sized particles and ~15% being clay-sized particles. These tests are not directly comparable but reveal that continued shearing leads to progressive deterioration of particles (i.e., sand to silt to clay in the case of HY 225). It is also notable that extremely high stresses, 300 MPa, were only enough to induce 45% fines generation.

HY 227 is a sedimentation test that was conducted on crushed glauconite sand collected during an offshore surveying campaign in the New York Bight. The material was adhered to the side of a CPTu cone and has a fines content of about 50% (Figure 6.7). Notably, the grain size distribution closely matches that of HY 226, with about 35% silt-sized particles and 15% clay-sized particles.

Cone tip resistances are typically much less than 50 MPa in offshore glauconite deposits. Nevertheless, the degree of crushing was similar to what was observed in the lab under stresses as high as 300 MPa. These data suggest that the processed glauconite sand particles used in this study are significantly stronger (i.e. less crushable) than the particles that were collected offshore.

To approximate the degree of particle crushing at a given effective stress, the author conducted a series of CRS compression tests up to various stress levels (6.9, 11.8, 14.7, 24.5, 34.3, 44.1 MPa)

on the processed Navesink glauconite sand. The fines generation was measured after each test. A plot of fines generation as a function of axial effective stress (MPa) is depicted in Figure 6.8.

The result of these experiments was a power law function with an R^2 value of 0.96. The Navesink glauconite sand reached a fines content between 18-20% at an effective stress of ~45 MPa (the capacity of the CRS device).

As previously discussed indicates that the Navesink glauconite sand is not capable of achieving the 50% fines content generated during pile driving or piezocone penetration testing, where stresses are typically much less than 50 MPa. This result was consistent with Westgate et al. (2024) which observed modest fines generation at the pile wall when pile driving through the Navesink glauconite sand in New Jersey.

Alm and Hamre (2001) predict that stresses experienced at the pile toe during pile driving in sands range from 0.35 to $0.55 \cdot q_T$, where q_T is the net cone tip resistance as determined from CPTu data. The lower and upper bounds corresponded to loose and dense sands respectively. Figure 6.9 is a plot that relates net cone tip resistance (MPa) to fines generation (%) at the pile toe. Note that the amount of fines generation ranges from about 4 to 15% over a range of cone tip resistances from 5 to 50 MPa.

6.1.3 Characterization of fine-grained glauconite

As indicated during crushing and sedimentation experiments, compressive loading generates fine-grained glauconite at a predictable rate. However, the gradation of glauconite fines varies depending on the method of preparation. Specifically, compressive loading generates silt-sized fines and modest clay, while shear loading (via malt-mixing) generates a more uniform clay-sized material.

Malt-mixing, being the more efficient method to generate fine-grained glauconite, was used to prepare fine-grained material for Atterberg limits, CRS, and triaxial testing. Because of this, it should be noted that the fine-grained glauconite used in this study is likely more uniform and clay-sized than the material generated during pile driving and CPTu testing.

The fine-grained glauconite had a liquid limit of 98 and a plasticity index of 71. These values are significantly higher than the values reported by Westgate et al. (2023) (See Table 2.3).

Data from an XRPD test on fine-grained glauconite is reported in Tables 6.2 and 6.3. Tables 6.2 and 6.3 report the bulk mineralogy and clay mineralogy respectively. Observe that the bulk material is 86.3% glauconite or glauconite/smectite by bulk mineralogy. Other mineral constituents like calcite and quartz represent non-glauconite particles in the coarse fraction. The mineralogy of the clay fraction is 44% glauconite/smectite and 56% glauconite.

6.1.4 CRS Tests on glauconite sand

Three CRS tests were conducted on glauconite sand, CRS 1770, 1771, & 1772. For each test, the sand was prepared at a relative density ranging between 36 to 43%, then loaded to an effective

stress of about 44 MPa at 2%/hour strain rate. After loading to 44 MPa, the material tested in CRS 1771 was unloaded to 2.5 MPa and reloaded to 44 MPa three times, to understand the effects of cyclic loading. CRS 1772 included an 11 hour hold stress phase at 2 MPa and three 4 hour hold stress phases at 7.4, 20, and 44 MPa to measure the rate of secondary compression. Figures 6.10, 6.11, and 6.12 show the compression curves from CRS 1770, 1771, & 1772 respectively.

During each test, a buildup of excess pore pressure was observed, beginning around 25 to 30 MPa (Figure 6.13). This was attributed to particle crushing which decreases the permeability of the material. According to Figure 6.8, an effective stress of 25 to 30 MPa corresponds with about 12.5% to 15% fines generation. This fines generation and buildup of excess pore pressure coincided with a significant drop in permeability. For CRS 1770, the permeability dropped from 10^{-13} to 10^{-18} m² as the porosity decreased from 0.14 at 16 MPa to 0.06 at 44 MPa (Figure 6.14).

Although excess pore pressures during virgin compression were modest, reaching a maximum value of 0.032 MPa during CRS 1770, they increased significantly during cyclic loading in CRS 1771. In this test, the first reload cycle generated 0.05 MPa, the second 0.10 MPa, and the third 0.17 MPa of excess pore pressure. During unloading, negative excess pore pressures as low as -0.06 MPa were observed (Figure 6.15), which is typical behavior for over consolidated (OC) clays.

Several factors explain the increase in excess pore pressure with each reload. One is related to CRS test controls. As shown in Figure 6.16, the axial strain increment decreased with each reload. Because loading was applied at a constant strain rate of 2%/hour, a lower strain increment effectively increased the rate of loading in each cycle. Additional contributing factors include a reduction in porosity and a possible increase in fines content with each reload. Together, these effects likely led to reduced permeability and the observed rise in excess pore pressure.

Hold stress intervals during CRS 1772 were used to derive the secondary compression index (C_{ae}) at 2, 7.4, 20, and 44 MPa, reported in Table 6.4. The secondary compression index describes the slope of the change in void ratio, Δe , divided by the change in log time, $\Delta \log_{10} \text{time}$. Plots of secondary compression of glauconite sand are provided in Figure 6.17. Notably the secondary compression index drops sharply as stress increases going from 0.0155 at 7.4 MPa to 0.0055 at 44 MPa. This trend suggests a higher rate of particle crushing and rearrangement at lower stress levels but significant stiffening as the soil compresses further and fines content increases.

6.1.5 CRS Tests on Resedimented Glauconite Sand with Glauconite Fines

CRS tests 1809, 1816, and 1808 were conducted on glauconite sand with fines contents of 25%, 50%, and 75%, respectively, while CRS 1784 was performed on 100% fine-grained glauconite (Figure 6.18). The increasing fines content in these tests directly influenced the swell index (C_s),

recompression index (C_r), compression index (C_c), and secondary compression index (C_{ae}) as summarized in Table 6.5.

The swell and recompression indices were determined by performing an unload–reload cycle at 44 MPa and calculating the change in void ratio (Δe) over the change in log-effective vertical stress ($\Delta \log_{10} \sigma'_v$). Note that the values for C_s and C_r for glauconite sand with 75% fines content specimen were obtained from CRS 1794 rather than CRS 1808.

It is important to note that the swell and recompression indices were not all measured at the same over consolidation ratio (OCR), which can affect the values of C_s and C_r . CRS 1809 and 1816 were unloaded to an OCR of 44, CRS 1794 to an OCR of approximately 15, and CRS 1784 to an OCR of 88. For greater consistency, future researchers are advised to measure elastic behavior at the same OCR across tests.

The swell index and recompression index both increased dramatically with increasing fines content, consistent with the observation that more fines lead to more capacity for structural readjustment and rebound. The compression index was measured over the virgin compression portion of the curve from 1 to 44 MPa. It increases dramatically from 0.173 to 0.531 as the fines increased from 25 to 100%. This indicates a loss of grain-support and emergence of more clay-supported behavior.

C_{ae} was measured at 44 MPa for all specimens. It was relatively consistent, ranging from 0.00272 to 0.00319. Notably the secondary compression index for glauconite sand at 44 MPa was 0.00554, greater than all the resedimented specimens. This is unusual, as fine-grained materials typically exhibit greater rates of secondary compression than coarse-grained materials

Plots of secondary compression for CRS 1809, 1816, 1808, and 1784 are provided in Figure 6.19. Each hold stress phase was approximately 4 hours long. It should be noted that the automated CRS device is computer controlled to maintain total stress and therefore not ideally suited for measuring secondary compression. As shown in Figure 6.20, each resedimented specimen developed excess pore pressures of about 1.7 MPa when loaded to 44 MPa effective stress at 1% strain rate. Given these large excess pore pressures, the hold stress phase does not capture true secondary compression as the system is undrained and total stress is held constant. Nevertheless, the secondary compression indices calculated for resedimented samples are useful for relative comparisons of time-dependent compression of glauconite sand with varying fines contents.

The permeability, k , and coefficient of compressibility, c_v , both decreased for resedimented specimens with increasing fines content (Figures 6.21 and 6.22). As previously discussed, the permeability of glauconite sand decreases by four orders of magnitude (from 10^{-14} to 10^{-18} m²). Resedimented specimens were another order of magnitude less permeable (10^{-19} m²) at high stresses. Increasing the fines content from 25% to 50% to 75% to 100% did not significantly decrease permeability. The coefficient of consolidation, which describing the rate of

consolidation in (m^2/s), differs significantly at low stress then converges upon a value between 4×10^{-8} to $2 \times 10^{-7} \text{ m}^2/\text{s}$ for all specimens at 10 MPa.

6.1.6 Resedimentation and Determination of Maximum Past Pressure:

The author noted that many of the resedimented specimens were not fully consolidated to 98 kPa, the standard past pressure that TAG lab specimens are loaded to before unloading to an OCR of 4. Figure 6.23 provides several plots from CRS 1809, 1816, 1808, and 1784 that relate total work to axial effective stress to determine the maximum past pressure. Notably the maximum past pressure for CRS 1809 (25% fines) was 59 kPa and for CRS 1816 and 1808 (50 and 75% fines) was 78 kPa. One interpretation of this is that insufficient silicone grease was applied to the consolidometer wall, leading to frictional effects that prevented full consolidation throughout the specimen. However, the re-occurrence of this problem suggests that glauconite fines are especially adhesive, an observation that is supported by high plasticity and roasted marshmallow-like consistency.

This failure to adequately consolidate fine-grained glauconite specimens caused TX 1591 to fail during setup. This specimen, 100% fine-grained and consolidated to 98 kPa, was not stiff enough to withstand any shear stress and failed immediately when rolling up the membrane. After this failure, the author consolidated triaxial specimens to 294 kPa with better results.

6.1.7 Triaxial Test on Glauconite Sand with 75% fine-grained glauconite

Eight triaxial tests were attempted, but seven failed due to issues such as unconsolidated specimens, internal leaks at the top cap, and motor control malfunctions (see Table 6.1). To date, the author has not successfully completed a standard triaxial test on glauconite sand, primarily due to the challenges of reconstituting specimens within the triaxial cell. However, one CKoUC triaxial test, TX 1601, was successfully conducted on glauconite sand containing 75% fines. This specimen was pre-consolidated to approximately 294 kPa in a consolidometer before being unloaded to an OCR of 4, and it demonstrated sufficient strength to support the top cap during setup.

Despite this, analysis using the strain energy method estimated a maximum past pressure between ~147 kPa and 196 kPa during K_0 consolidation, lower than expected. This discrepancy is attributed to frictional losses in the consolidometer. Because of these frictional losses, the K_0 consolidation subroutine was run until 20% strain. However, the specimen was only consolidated to about 1.6 MPa instead of the anticipated 6 MPa.

Typically, K_0 consolidation is carried out to achieve about 10% axial strain before the specimen is sheared at the desired stress level. To address these issues, the author recommends that future researchers resediment fine-grained glauconite specimens to maximum past pressures 33–50% greater than the intended target stress. This adjustment would help compensate for frictional losses in the consolidometer, produce a stiffer specimen, and preserve sufficient stroke of the axial PVA for undrained shearing.

The TX1601 specimen was K_0 -consolidated to 0.6 MPa, unloaded to 0.1 kPa, an OCR of 6, then reloaded to about 1.6 MPa (Figure 6.24). A K_0 coefficient of 0.4 was measured during the initial loading portion of the test (Figure 6.25). K_0 is typically higher for high plasticity, fine-grained materials, but a value of 0.4 is not unusual for these materials at low stresses.

After K_0 consolidation, the specimen was sheared at a strain rate of 0.05% per hour until about 3.6% strain. A motor control issue disrupted further shearing beyond 3.6%. During undrained shearing, the specimen exhibited a contractive or strain-hardening response, typical of normally consolidated (NC) clays (Figure 6.26). A buildup of excess pore pressure to about 325 kPa was observed (Figure 6.27). The internal friction angle (ϕ') converged upon a value close to 30° . This internal friction angle is relatively high for silty-clayey soils, though not unheard of in low-stress tests. Based on Jaky's formula ($K_0 = 1 - \sin\phi'$), however, K_0 should be closer to 0.5.

6.2 Comparison to In-House Materials

This section compares the plasticity, compression behavior, permeability of fine-grained glauconite with resedimented Boston Blue Clay (RBBC) and resedimented Gulf of Mexico-Eugen Island Clay (RGoM-EI). RBBC is an illitic clay and RGoM-EI is a smectitic clay. Both materials that have been researched extensively by the TAG Lab. As discussed in Chapters 2 and 3, glauconite mineral is considered part of the illitic family of clays. However, XRPD data from the crushed Navesink glauconite sand suggests that it is closer in composition to smectite. The clay mineralogy was classified as 56% glauconite, 44% glauconite/smectite and 0% illite or illite/smectite. This data is supported by its mechanical properties and behavior.

Figure 6.28 compares the liquid limit and plasticity index of fine-grained glauconite with those of RBBC and RGoM-EI Clay. The material is even more plastic than RGoM-EI on the Casagrande Plasticity chart.

The compression curve of fine-grained glauconite is compared with those of RGoM-EI and RBBC in Figure 6.29. Fine-grained glauconite is more compressible, beginning with a higher void ratio than both other materials. Its compression curve crosses over that of RBBC at 2 MPa and that of RGoM-EI at about 8 MPa. Fine-grained glauconite and RGoM-EI are concave up while RBBC is essentially linear in void ratio (e) log-effective stress ($\log_{10}\sigma'_v$) space.

The permeability of the three materials is compared in Figure 6.30. Fine-grained glauconite is less permeable than RBBC and similar in permeability to RGoM-EI at low stresses; at high stresses it is significantly more permeable than RGoM-EI. Interestingly, the slope of K is much shallower suggesting that permeability of fine-grained glauconite does not decrease as fast as the two clays. Perhaps fine-grained glauconite has more silt-sized particles than RBBC and RGoM-EI, allowing for better drainage and a slower reduction in permeability with decreasing porosity.

Glauconite sand is less permeable than the RBBC and RGoM-EI but becomes similar in permeability to RBBC at 40+ MPa. As previously discussed, this is likely due to particle crushing at high stresses.

These comparisons of plasticity, compression behavior, and permeability suggest that fine-grained glauconite is more like smectitic RGoM-EI clay than illitic RBBC. This is consistent with the XRPD test results which indicated that the material was 44% glauconite/smectite.

6.3 Discussion of Results

As discussed in Chapter 2, previous research has shown that glauconite transitions from granular to cohesive behavior due to particle crushing. This has presented several challenges for geotechnical engineers. First, the material cannot be reliably classified as either sand or clay using CPTu-based soil behavior type (SBT) classification charts (e.g. Robertson, 2016). Second, soil static resistance to driving (SRD) is often underestimated from CPTu parameters leading to excessive blow counts or pile refusal during installation. Third, the properties of glauconite can be significantly altered during pile installation, creating uncertainty regarding foundation performance. This study has characterized the crushing behavior of glauconite sand and the impact of increasing fines content on compression behavior and permeability.

As previously discussed, the compression behavior of fine-grained cohesive materials like RBBC and RGoM-EI clay is concave up in void ratio (e) log-effective stress ($\log_{10}\sigma'_v$) space. In contrast, coarse-grained cohesionless materials like silica sand exhibit concave down behavior in void ratio (e) log-effective stress ($\log_{10}\sigma'_v$) space. Because glauconite is a granular material at low stress and a cohesive material at high stress, its compression behavior transitions from concave down to concave up. Figure 6.31 compares the compression behavior of glauconite sand (CRS 1771) with that of MIN-U-SIL 40, a manufactured silica silt with an average particle size of 20-25 μm . The manufactured silica silt compression curve was measured by Mark Zablocki in CRS 1571.

To capture this transition from concave down to concave up behavior the author measured the compression index (C_c), the slope of the compression curve in void ratio (e) log-effective stress ($\log_{10}\sigma'_v$) space, at discrete stress levels (Figure 6.32). Then, using the power law function that relates effective stress to fines generation, C_c was related to the fines content. Figure 6.33 shows the nonlinear relationship between compressibility and stress level and compressibility and fines content. The compression index increases up to 7 MPa or about 7-8% fines, then decreases from 7 to 33 MPa or ~15.5% fines. See Table 6.6 for the values of stress (MPa), fines content (%), and compression index (C_c) used in this analysis.

Notably, the compressibility of glauconite sand peaks very close to 7.4 MPa, the stress at which the maximum secondary compression index (C_{ae}) was determined. This suggests that the highest rate of particle breakage for this material is observed around 7 MPa effective stress.

As discussed in Chapter 2, the crushability of glauconite sands has been known to vary significantly with particle age and maturity. This is reflected by HY 227, the grain size analysis of the crushed glauconite sand from the New York Bight which achieved a fines content as high as 50% from stresses applied during piezocone penetration testing. Although achieving this fines content was not practical for the Navesink glauconite sand that was characterized in this study, the

resedimented specimens provide insights into how compression behavior might vary when conducting CPTu or pile driving through a more crushable glauconite sand.

Figure 6.34 overlays the compression curve for glauconite sand (CRS 1771) with the compression curves of the resedimented specimens containing both glauconite sand and fines. As previously discussed, compressibility increases with increasing fines content. Interestingly, however, the resedimented sand specimen with 25% fines (CRS 1808) is less compressible than the glauconite sand specimen up to ~14 MPa. This suggests that the fine-grained glauconite has filled the pore space, maximizing stiffness and density. As the stress level increases beyond 14 MPa, the glauconite sand specimen becomes the densest. Perhaps the crushed glauconite particles immediately fill in the voids without having time to hydrate and expand as resedimented specimens would.

Although transition from grain-supported to fines supported behavior is observed, determining the critical fines content is challenging. One potential method involves measuring the associated fines increase after each specimen was compressed to 44 MPa. The glauconite sand and resedimented CRS test specimens were dissolved in sodium hexametaphosphate, a deflocculating agent, then washed over the number 200 (75 μm) sieve. The increase in fines content was plotted as a function of the starting fines content in Figure 6.35. Notably only Glauconite sand and glauconite sand with 25% fines experienced an increase in fines content, 17% and 4% respectively. Specimens with 50% starting fines content or more did not experience an increase in fines suggesting that glauconite sand particles were not in contact and therefore incapable of experiencing further crushing. This analysis suggests that the critical fines content, or the point at which the soil matrix transitions from grain supported to fines supported, occurs between 25% to 50% fines.

Figure 6.36 plots the compression index as a function of fines content for both glauconite sand (CRS 1771) and the resedimented specimens. Notably the compression index reaches a global minimum value at 29% fines content, the updated fines content for the resedimented specimen with a starting fines content of 25%. This indicates that specimens that are closest to the critical fines content may have reduced compressibility under high confinement.

Table 6.1: The table reports the test number, the material that was tested, and comments or observations about each test.

	<u>Test</u>	<u>Material/Comments on Test</u>
	Simple sieve analysis	
1	Test 1	Processed glauconite sand, not logged in logbook
	Sedimentation Analysis	
2	HY225	Processed glauconite sand after 3 hours malt-mixing (MM) and 24 hours wrist-action shaking (WAS)
3	HY226	Processed glauconite sand, axially compressed (AC) to ~300 MPa
4	HY227	Crushed glauconite sand adhered to Bluepoint Wind CPT equipment
5	HY228	Native Poricy Riverbank material (MM 1 minute)
6	HY229	Native Poricy Riverbank material (MM 5 minutes)
7	HY230	Native Poricy Riverbank material (MM 10 minutes)
8	Max and Min Void Ratio Tests	Processed glauconite sand, not logged in logbook
	Specific Gravity	
9	SG209 ($G_s=2.70$)	Processed glauconite sand
10	SG210($G_s=2.78$)	Native Poricy Riverbank material
	XRPD	Conducted by James Hutton Limited; Not logged
	SEM	Processed glauconite sand, dry powder from AC test, not logged in logbook
	Atterberg Limits	
11	Test 1	Material that was used for HY225, malt-mixed for 3 hours, wrist-action shaken for 24 hours, not logged in logbook
12	Test 2	Processed glauconite sand that was malt-mixed, and wet sieved (100% fines content)
	Pore Fluid Salinity	
	6.24.24 Test	100% fines content powder, conducted by Matthew, readings were above calibration range of probe, not logged
13	8.7.24 Test	100% fines content powder, readings were above calibration range of probe, not logged
	CRS	

14	CRS1770	Processed glauconite sand, starting $D_r=42\%$, loaded to 39.2 MPa at 2% strain rate
15	CRS1771	Processed glauconite sand, starting $D_r=35\%$, loaded to 39.2 MPa, with 2% strain rate, and 3 unload reload cycles (from 39.2 to 2.5 MPa), buildup of pore pressure observed during unload reload cycles likely due to this being a rate-controlled test on stiffened material
16	CRS1772	Processed glauconite sand, starting $D_r=36\%$, loaded at 2% strain rate with three unload reload cycles at (from 2, 7.8, & 19.6 MPa to 0.5MPa); secondary compression allowed at 2, 7.8, 19.6, and 44.1MPa.
17	CRS1778	Processed glauconite sand, malt-mixed & wet-sieved until 100% fines content, $e_0=1.96$, loaded to 7.4 MPa and 353 kPa pore pressure at 1% strain rate, significant ($> 50\%$ strain) was observed, σ'_p appears much lower than 98 kPa suggesting consolidometer was poorly lubricated, very high adhesion between glauconite clay and consolidometer tube
18	CRS 1784	100% fines content loaded to 41.2 MPa and 1.3 MPa pore pressure at 1% strain rate, σ'_p appears to be closer to 98 kPa, specimen may be from different section in consolidometer with less friction than CRS 1778 specimen, $e_0=1.74$, minor control issue in the middle of the test, 2 hours of hold stress was measured at 41.2 MPa.
19	CRS 1790	100% Glauconite Clay, test failed
20	CRS 1794	Processed glauconite sand with 75% fines content, loaded to 41.2 MPa and 1.8 MPa pore pressure at 1% strain rate, four hours of hold stress at 44.1 MPa, test setup appears to have been performed incorrectly, however NC region, is well-defined
21	CRS 1804	Processed glauconite sand with 75% fines content, test failed
22	CRS 1808	Processed glauconite sand with 75% fines content loaded to 44.1 MPa, four hours of hold stress at 44.1 MPa, test was paused for eight hours at ~ 14.7 MPa due to axial piston running out of stroke, however compression curve geometry is well defined, σ'_p appears to be close to 98 kPa
23	CRS 1809	Processed glauconite sand with 25% fines content, loaded to 44.1 MPa and excess pore pressures as high as 1.8 MPa, four hours of hold stress at 44.1 MPa, Ch. 84 LVDT appears to have given incorrect readings (measuring increased displacement during unloading, Ch. 84 LVDT gave accurate resulting in well-defined curve geometry, σ'_p appears to be closer to 74 kPa.
24	CRS1810	Processed glauconite sand with 50% fines content, test failed

25	CRS 1816	Processed glauconite sand with 50% fines content loaded at 1% strain rate to 44.1 MPa and 1.7 MPa pore pressure, four hours of hold stress observed at 44.1 MPa σ'_p appears to be closer to 74 kPa
26	CRS 1819	Ticino sand, data not reduced
27	CRS 1820	MIN-U-SIL 40 (Silica Sand), data not reduced
28	CRS 1821	MIN-U-SIL 40 (Silica Sand) with 10% Glauconite Sand, data not reduced
29	CRS 1822	Processed glauconite sand, loaded to 14.7 MPa at 1% strain rate to measure crushing, data not reduced
30	CRS 1823	Processed glauconite sand, loaded to 24.5 MPa at 1% strain rate to measure crushing, data not reduced
31	CRS 1824	Processed glauconite sand, loaded to 31.4 MPa at 1% strain rate to measure crushing, data not reduced
32	CRS 1827	Processed glauconite sand, loaded to 6.9 MPa at 1% strain rate to measure crushing, data not reduced
33	CRS 1828	Processed glauconite sand, loaded to 11.8 MPa at 1% strain rate to measure crushing, data not reduced
	Resedimentation	
34	RS 797	100% fine-grained glauconite, prepared with a water content of about 220% (~2.2 times the liquid limit) for CRS testing, successfully tested
35	RS 798	100% fine-grained glauconite, prepared with a water content of about 220% (~2.2 times the liquid limit) for CRS testing, successfully tested
36	RS 814	100% fine grained glauconite, prepared for triaxial testing. Soil slurry was prepared at 120% (~1.2 times the liquid limit). Specimen was consolidated to 294 kPa before unloading to ensure stiffness, <i>not yet tested</i>
37	RS 815	Processed glauconite sand, with 75 % fines content, slurry was prepared at 120% prepared for triaxial testing, specimen was consolidated to 294 kPa before unloading, successfully tested
38	RS 816	Processed glauconite sand, with 25 % fines content, soil slurry was prepared at water content of 65% for triaxial testing, loaded to 294 kPa before unloading, <i>not yet tested</i>
39	RS 817	Processed glauconite sand, with 25 % fines content, soil slurry was prepared at water content of 65% for CRS testing, successfully tested
40	RS 818	Processed glauconite sand with 50 % fines content, soil slurry was prepared at water content of 100% for CRS testing, successfully tested
41	RS 822	100% fine-grained glauconite, soil slurry was prepared at a water content of 150%, sample was consolidated to 3.1 MPa for medium pressure triaxial testing, <i>not yet tested</i> .

42	RS 823	Processed glauconite sand with 75% fines content, soil slurry was prepared at a water content of 150%, sample was consolidated to 2.9 MPa for medium pressure triaxial testing, <i>not yet tested</i> .
	Triaxial Testing	
43	TX 1590	CIUC medium pressure triaxial test on processed glauconite sand, failed
44	TX 1591	CK ₀ UC low pressure triaxial test on 100% fine-grained glauconite, failed during setup, cause of failure: specimen not fully consolidated
45	TX 1592	CIUC medium pressure triaxial test on Ticino sand, failed
46	TX 1594	CIUC medium pressure triaxial test on Ticino sand, failed
47	TX 1595	CIUC medium pressure triaxial test on Ticino sand, failed, B value of 0.39 observed, cell pressure motor ran out of stroke during consolidation suggesting internal leak.
48	TX1596	Leak check for MP triaxial setup, cause of leak was identified as faulty top cap drainage port
49	TX 1599	CK ₀ UC MP triaxial test on Intact BBC, internal leak was observed likely due to faulty top drainage line, and test was reset with new drainage line. Test failed during consolidation due to pore motor fluctuations, pore motor control settings adjusted by George after test
50	TX 1601	First successful CK ₀ UC triaxial test on processed glauconite sand with 75% fines content. The specimen (RS815) was consolidated to 294 kPa, but still highly compressible. The specimen was K ₀ – consolidated at a strain rate of 0.05% per hour. The axial motor fluctuated midway through shearing, possibly due to electronic noise. This disrupted the remainder of the test, but the peak shear stress and internal friction angle could still be determined.
51	TX 1602	CIUC MP triaxial test on glauconite sand, test failed. The specimen was reconstituted within the triaxial cell using a metal brace with vacuum suction to keep a rubber membrane open. Particles were configured within membrane using pepper shaker method. After placing top cap and O-rings, specimen was crooked and had trouble maintaining the vacuum seal.

Table 6.2: A table showing the bulk mineralogy of a specimen of processed glauconite sand:

Quartz	Plagioclase	K-feldspar	Calcite	Dolomite	Siderite	Pyrite
3.0%	0.6%	1.5%	4.5%	0.2%	1.0%	1.0%
Halite	Amphibole	Kaolinite	Chlorite(Tri)	Muscovite	I+I/S-ML	Glauconite +G/S
0%	0%	0%	1.9%	0%	0%	86.3%

Where Chlorite(Tri) is Trioctahedral Chlorite, I+I/S-ML is Illite + Illite/Smectite – Mixed Layered, and Glauconite + G/S is Glauconite + Glauconite/Smectite.

Table 6.3: A table showing the relative percentage of clay minerals in the clay fraction ($< 2 \mu\text{m}$) sampled from crushed glauconite sand.

Chlorite(Tri)	Kaolinite	Illite	I/S-ML	Glauconite	Glauconite/Smectite
0%	0%	0%	0%	56%	44%

Table 6.4: A table showing how the secondary compression index (C_{ae}) of glauconite sand changes as a function of stress level.

Stress (MPa)	2	7.4	20	44
Secondary Compression Index (C_{ae})	0.01161	0.01554	0.00748	0.00554

Table 6.5: A table showing the swell index (C_s), recompression index (C_r), compression index (C_c), and secondary compression index (C_{ae}) of resedimented glauconite sand specimens with varying fines contents. Note that C_{ae} was measured at 44 MPa during four-hour hold stress phases. All specimens were unloaded and reloaded from ~ 44 MPa to compute C_s and C_r . However, CRS 1809 and 1816 were unloaded to an OCR of 44, while CRS 1794 and 1784 were unloaded to OCR's of ~ 15 and 88 respectively.

Fines Content (%)	25	50	75	100
Test Number(s)	CRS 1809	CRS 1816	CRS 1808; CRS 1794	CRS 1784
Swell Index (C_s)	0.034	0.073	0.117	0.190
Recompression Index (C_r)	0.057	0.090	0.152	-
Compression Index (C_c)	0.173	0.234	0.353	0.531
Secondary Compression Index (C_{ae}) at 44 MPa	0.00277	0.00272*	0.00319*	0.00316

*Measured after unload-reload cycle

Table 6.6: A table showing how the compression index (C_c) of glauconite sand changes as a function of stress (MPa) and fines content (%). Note that each value of C_c was calculated over a range of ± 1 MPa for a given stress.

Stress (MPa)	Fines Content (%)	Compression Index (C_c)
4.1	7.1	0.38
5.3	7.9	0.40
6.9	8.7	0.40
9.0	9.6	0.37
11.6	10.5	0.34
15.1	11.6	0.30
19.6	12.8	0.27
25.4	14.1	0.24
33.0	15.6	0.23

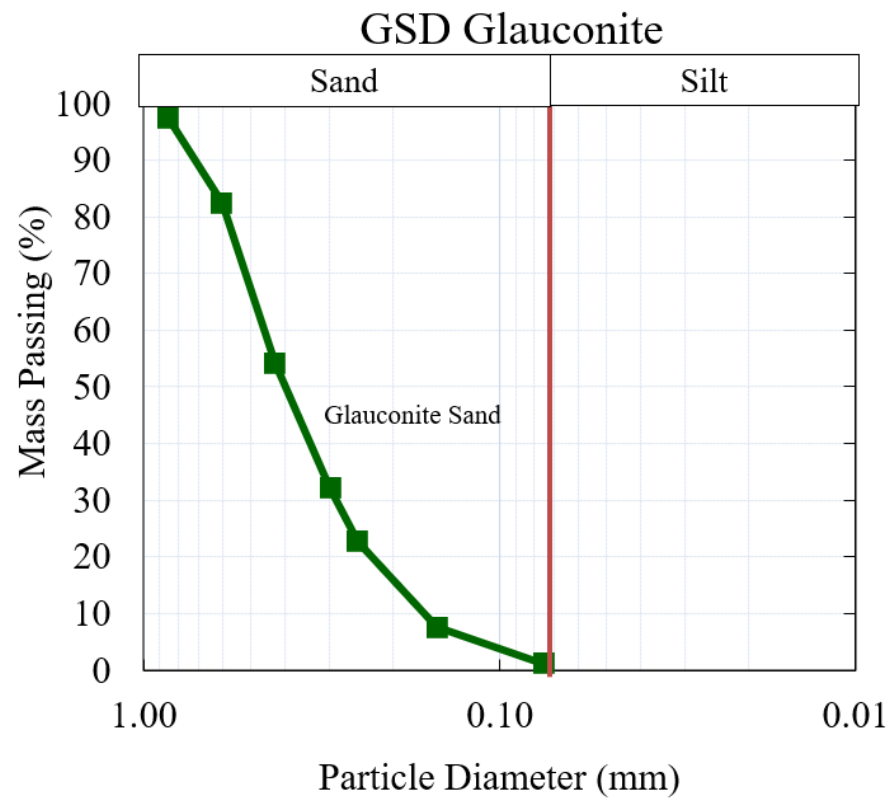


Figure 6.1: A grain-size distribution plot of processed glauconite sand.



Figure 6.2: Digital microscope images of processed glauconite sand. No scale was included with this device but note that all particles are sand-sized ($> 75 \mu\text{m}$).

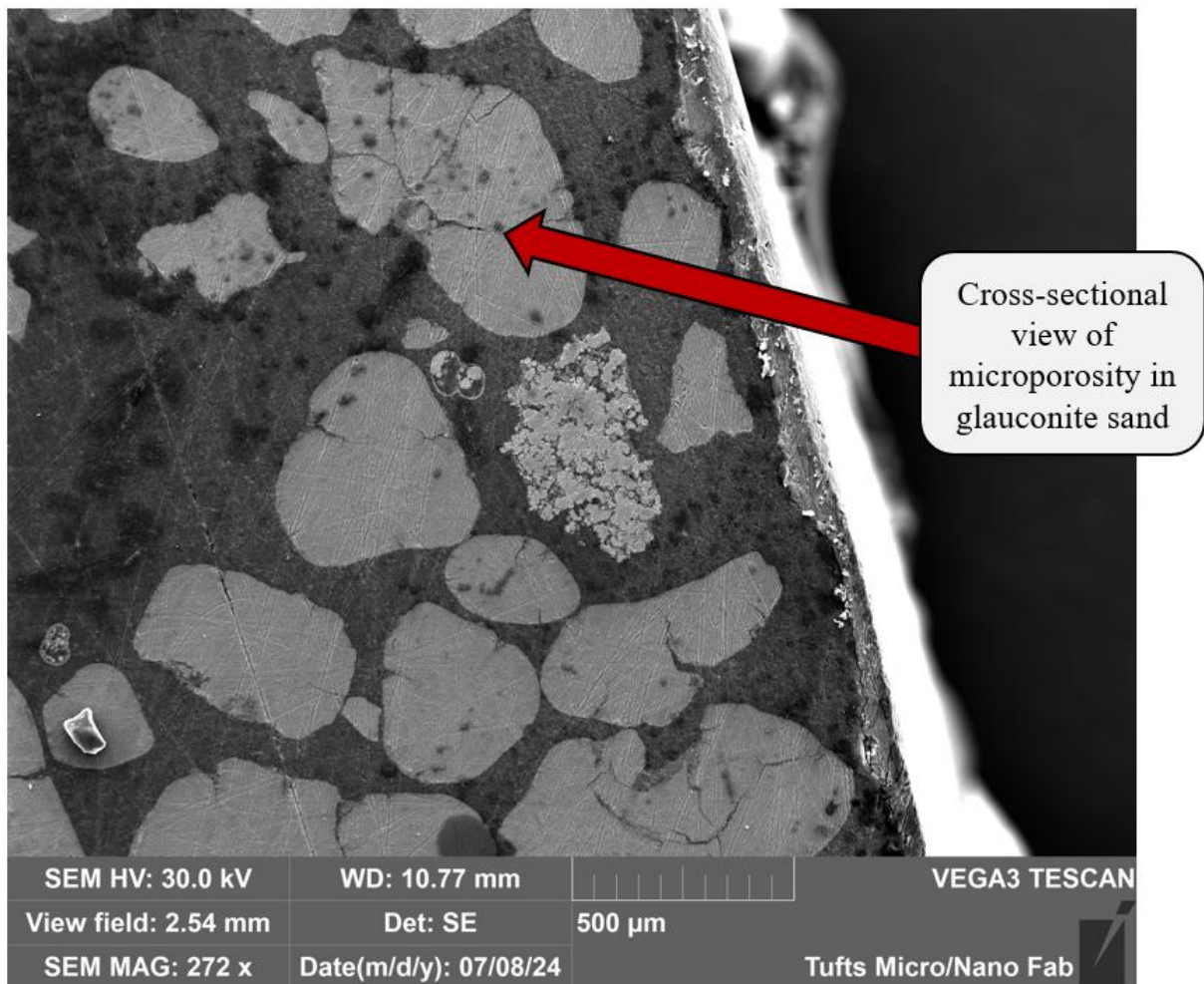


Figure 6.3: An SEM image of processed glauconite sand, frozen in epoxy, and shaved down with sandpaper to reveal its interior microstructure.

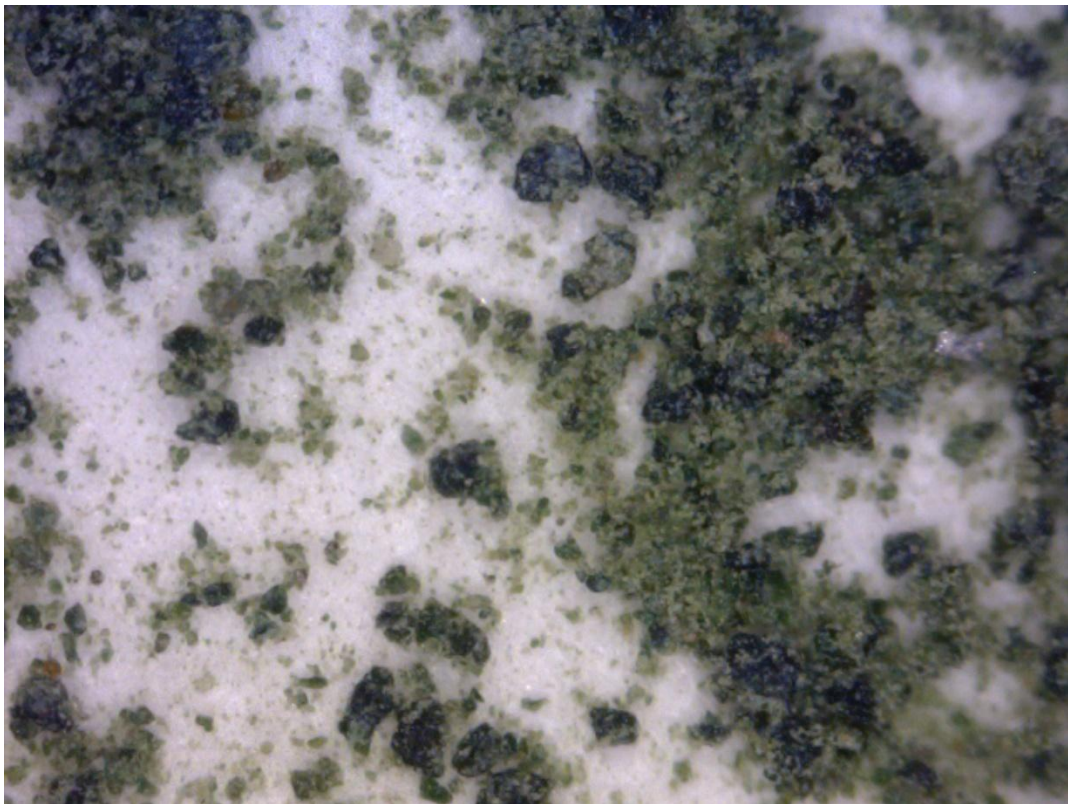
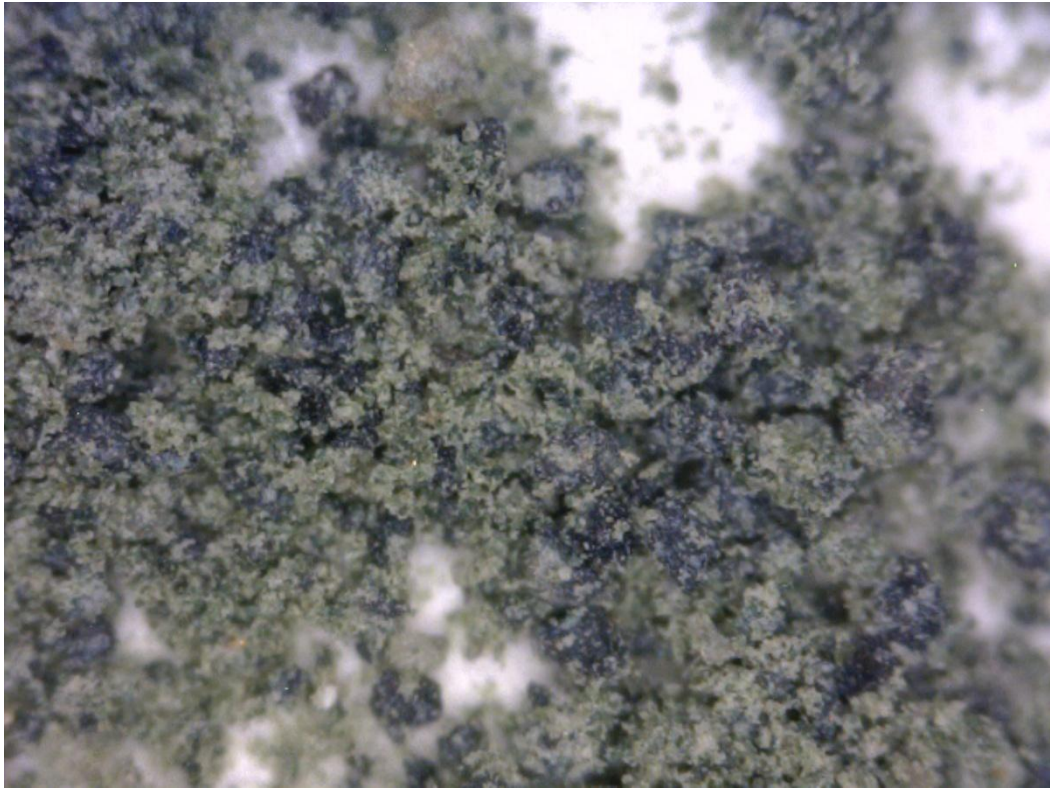


Figure 6.4: Digital microscope images of the particles from Figure 6.2 after being ground with a mortar and pestle.

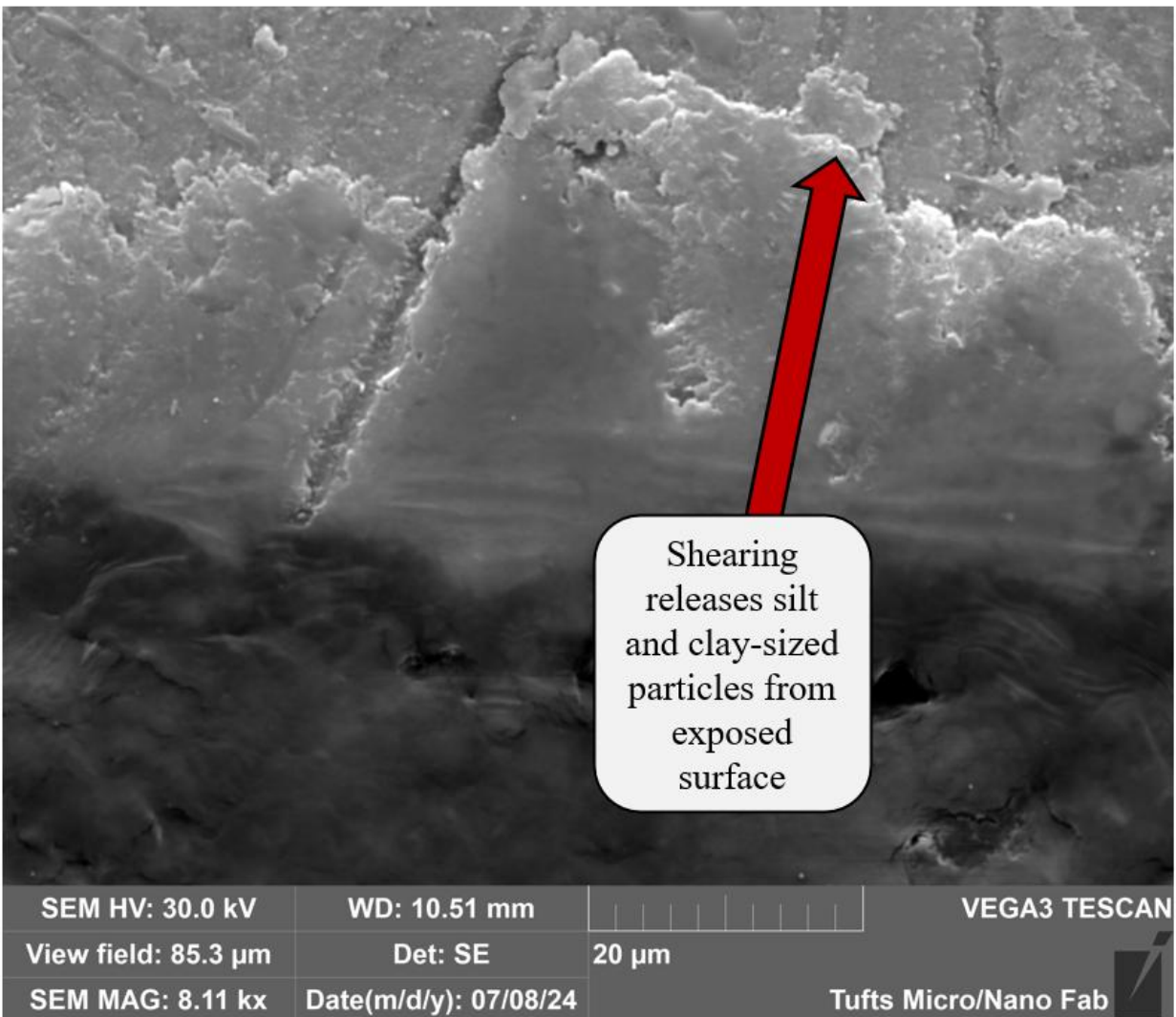


Figure 6.5: A SEM image of processed glauconite sand, frozen in epoxy, and shaved down with sandpaper. Shearing the particles with sandpaper releases silt ($<75 \mu\text{m}$)- and clay ($<2 \mu\text{m}$)-sized particles (see scale of $20 \mu\text{m}$).

Summary of Grain Size Analysis

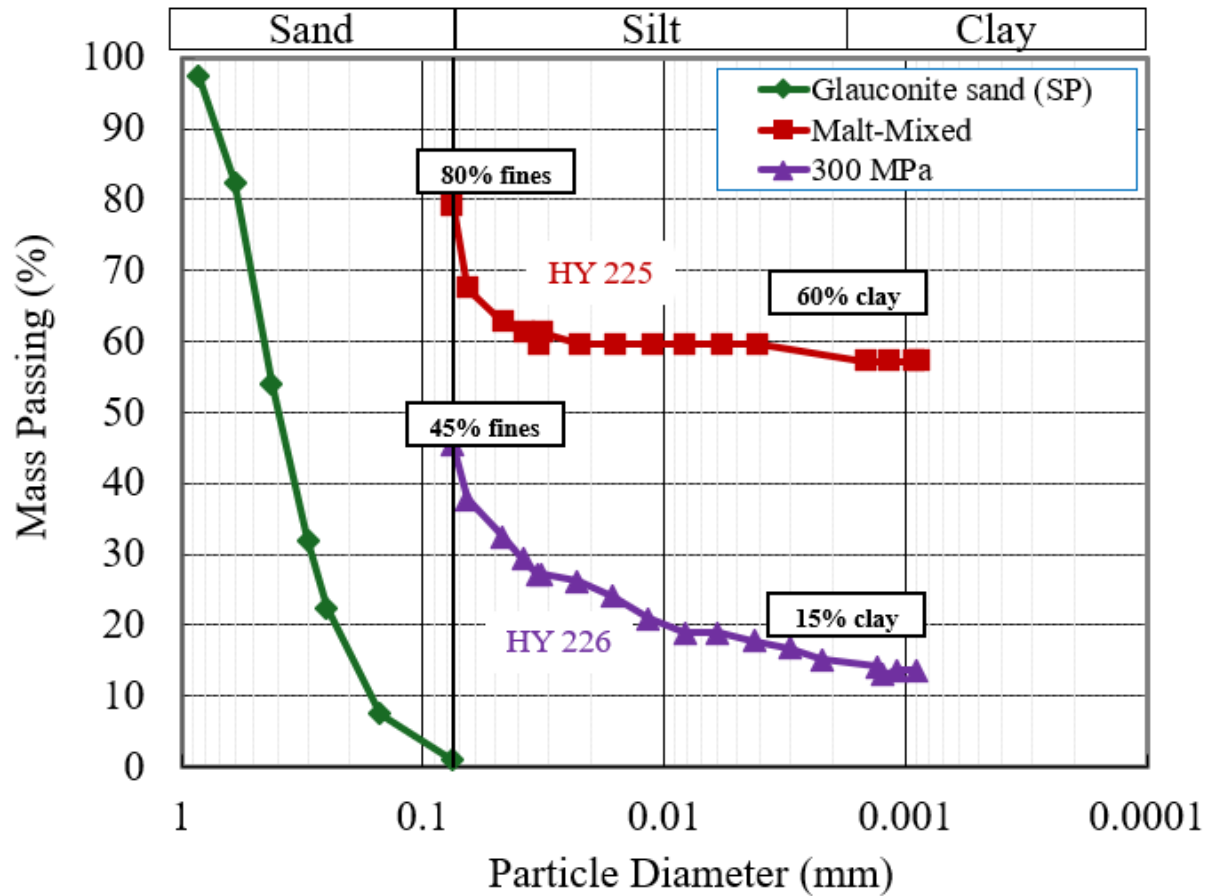


Figure 6.6: Sedimentation tests exploring the effects of different crushing methodologies on fines generation. The green curve is the processed glauconite sand, the red curve is processed glauconite sand that was malt-mixed for 3 hours and shaken with ceramic balls for 24 hours, and the purple curve is processed glauconite sand that was compressed up to 300 MPa.

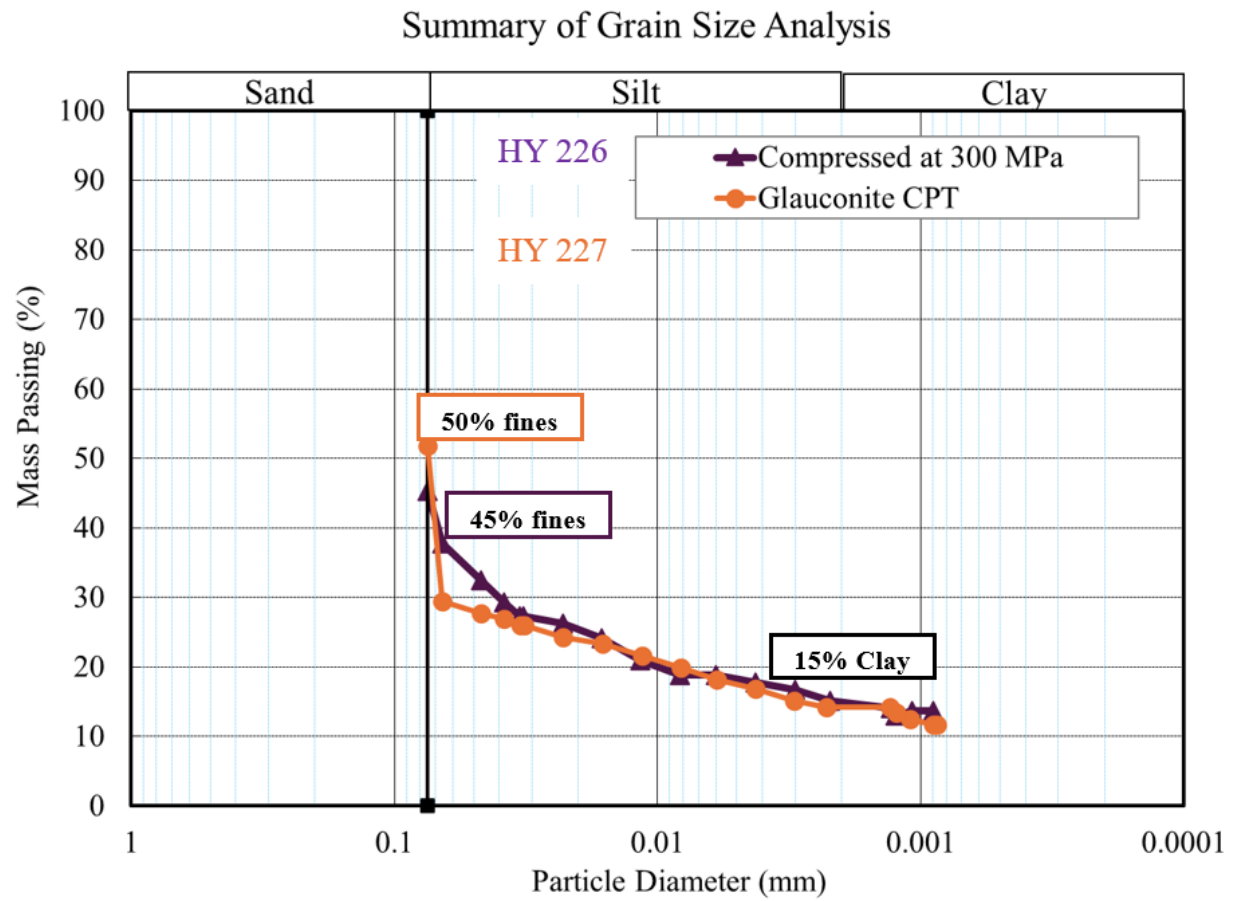


Figure 6.7: A comparison of the grain size distributions of the processed glauconite sand crushed at 300 MPa (purple) and crushed glauconite sand collected during an offshore wind site investigation campaign (orange).

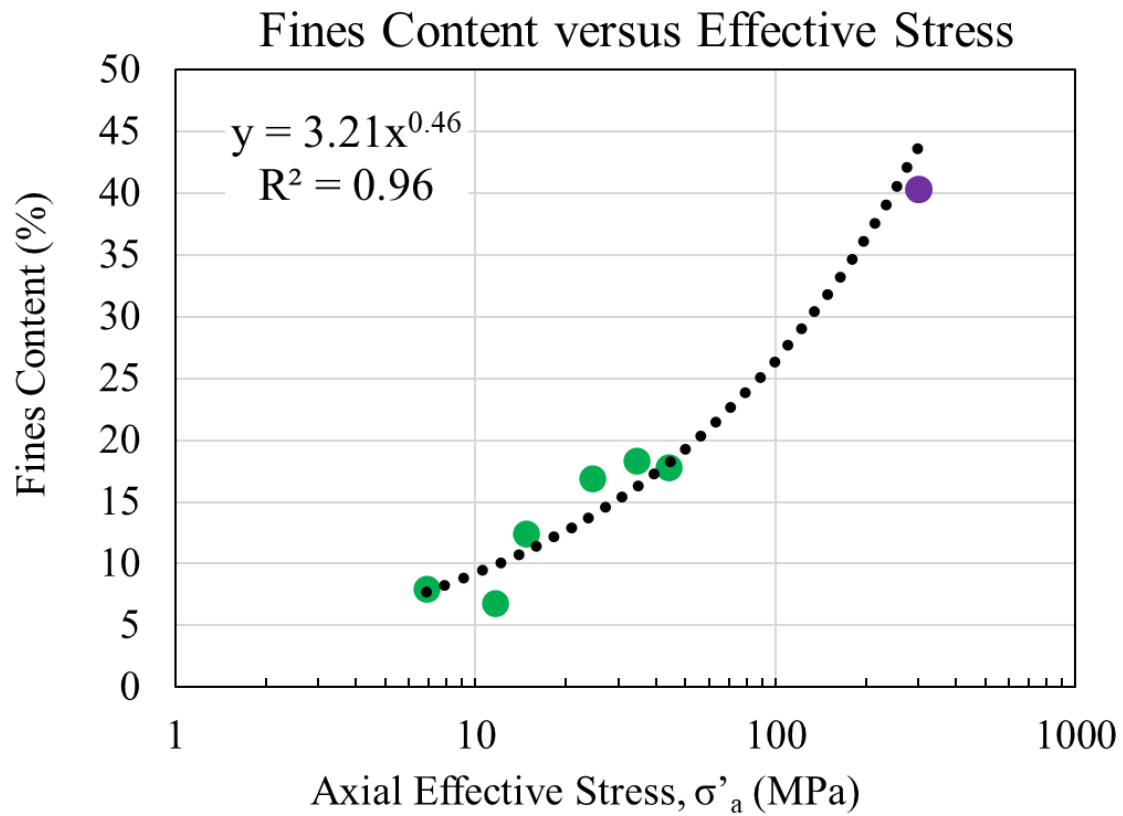


Figure 6.8: A plot showing the power law relationship between axial effective stress and the associated fines generated due to crushing of glauconite sand. Green points are from CRS tests while the purple point is from a hydraulic press test conducted on dry material.

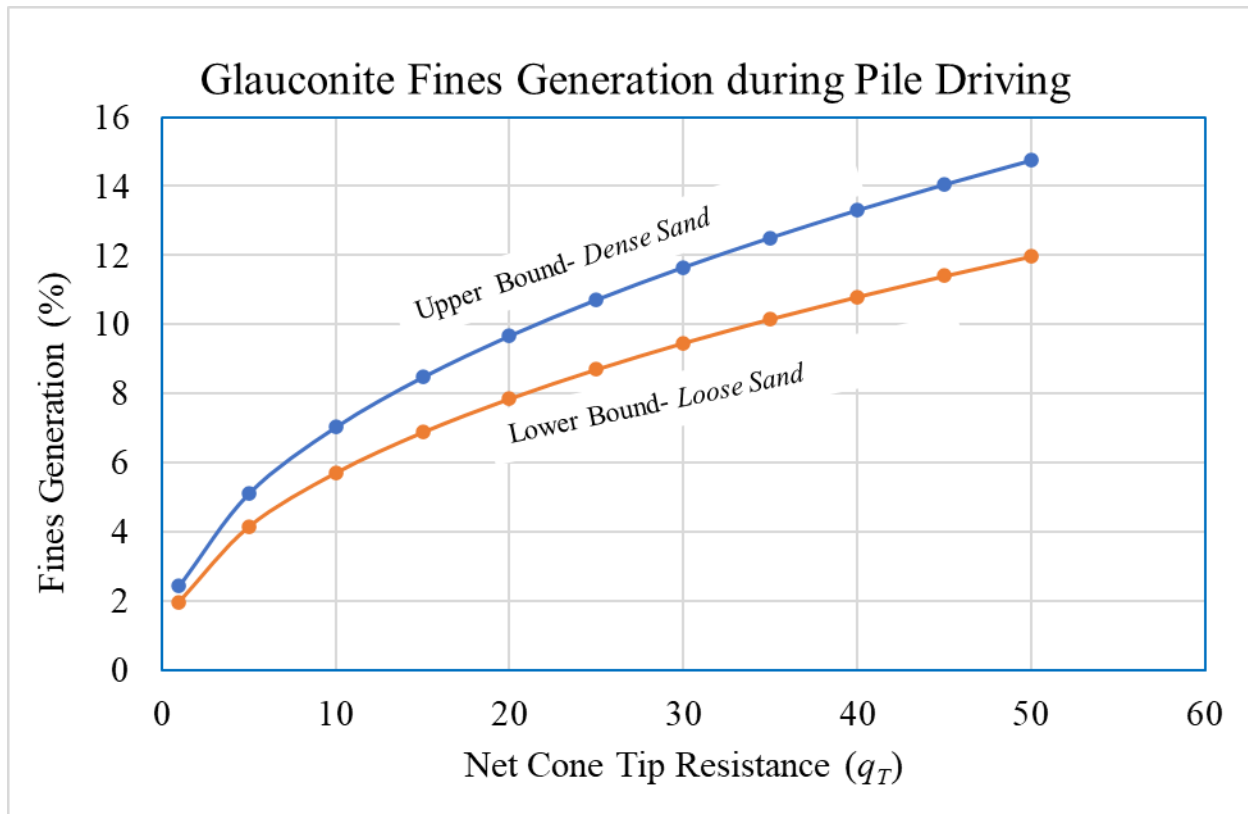


Figure 6.9: A plot relating net cone tip resistance during CPTu testing to glauconite sand crushing during pile driving where stress at the pile toe is approximated as $0.35 \cdot q_T$ for piling in loose sand and $0.55 \cdot q_T$ for piling in dense sand.

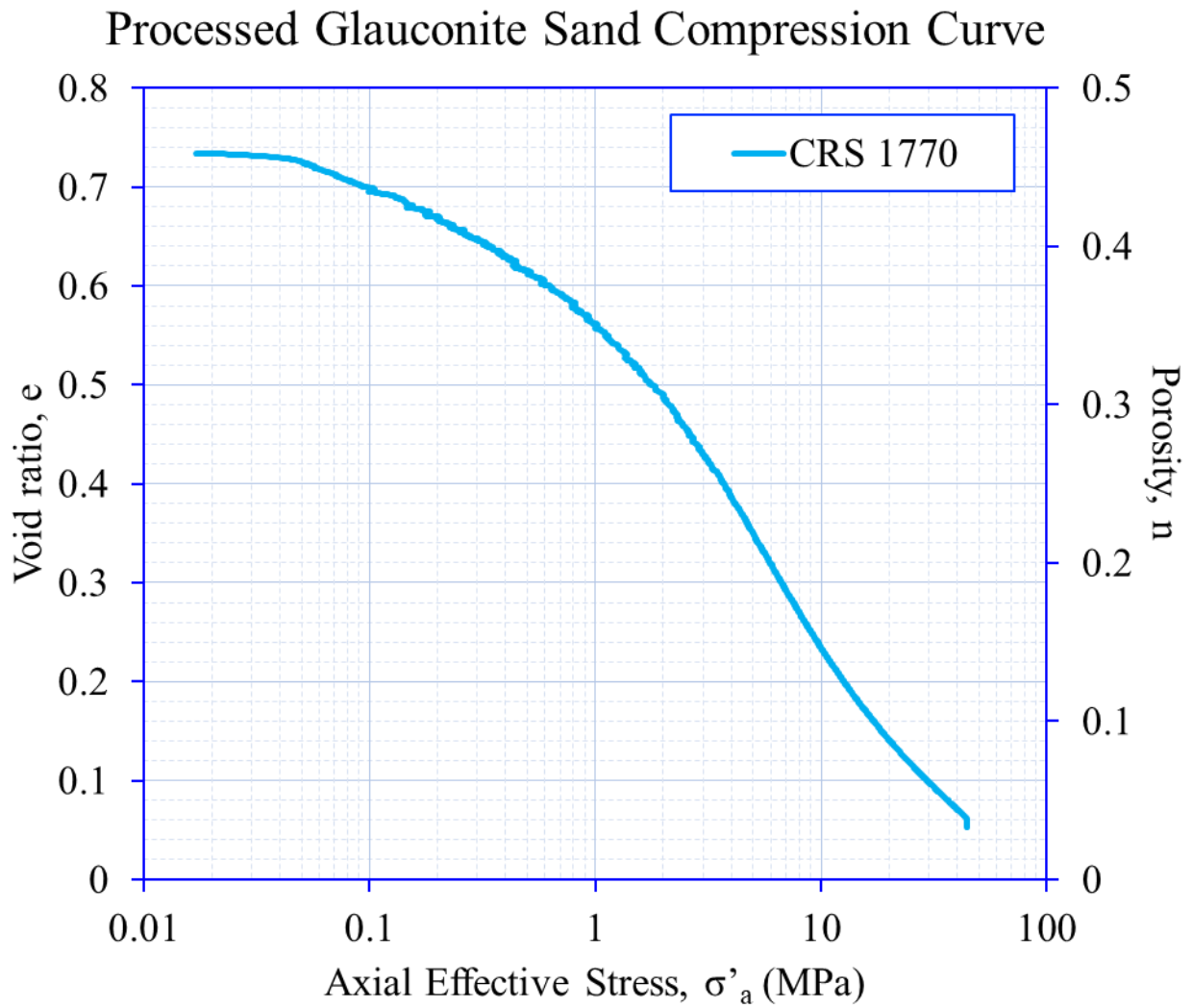


Figure 6.10: A compression curve of glauconite sand in void ratio log-effective stress space (CRS 1770). Note that the material is extremely compressible, decreasing from 0.73 to 0.05 void ratio as it is loaded to 44 MPa.

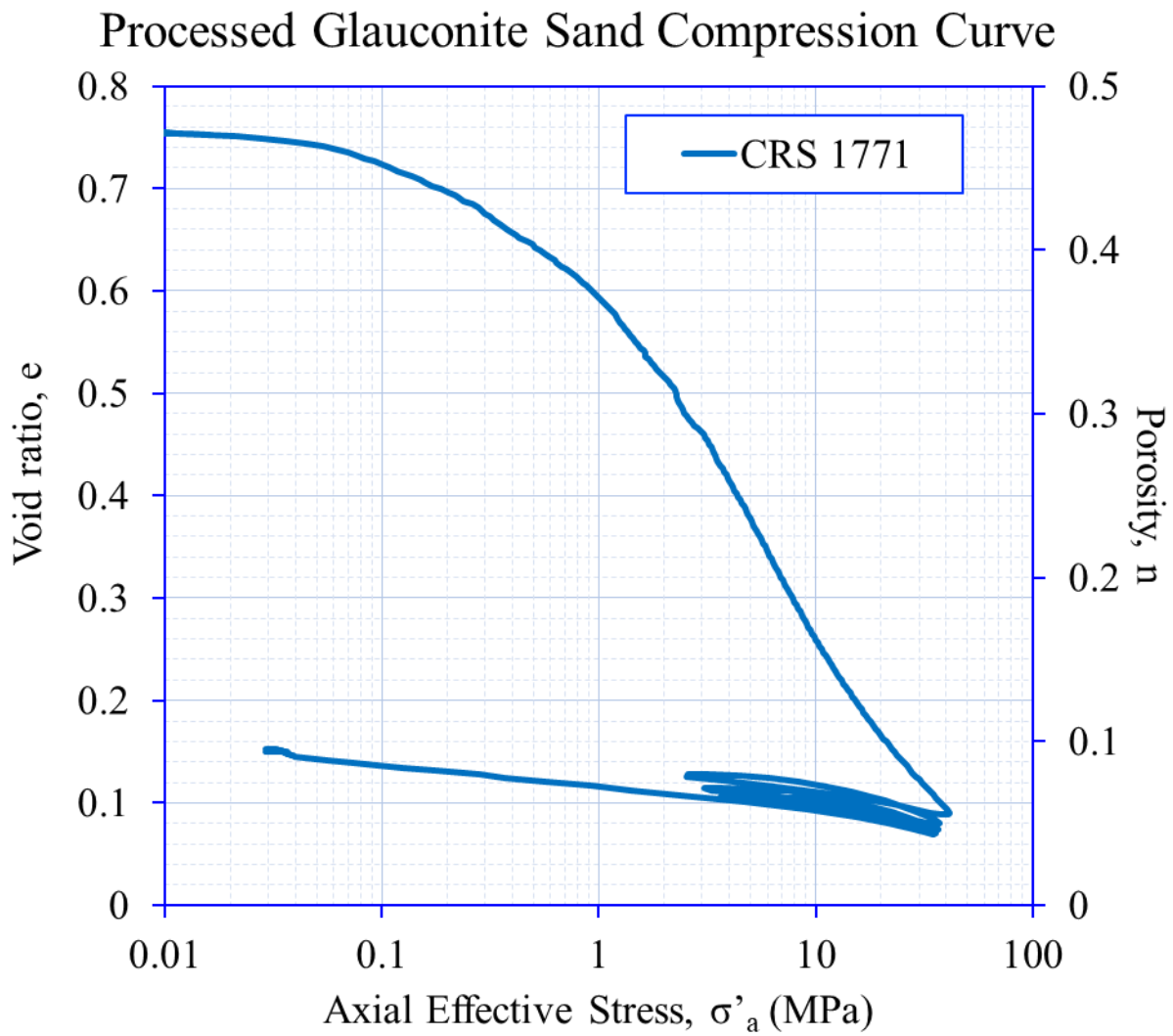


Figure 6.11: A compression curve of glauconite sand in void ratio log-effective stress space (CRS 1771). Three unload-reload cycles from 44 to ~2.5 MPa were conducted after loading to 44 MPa.

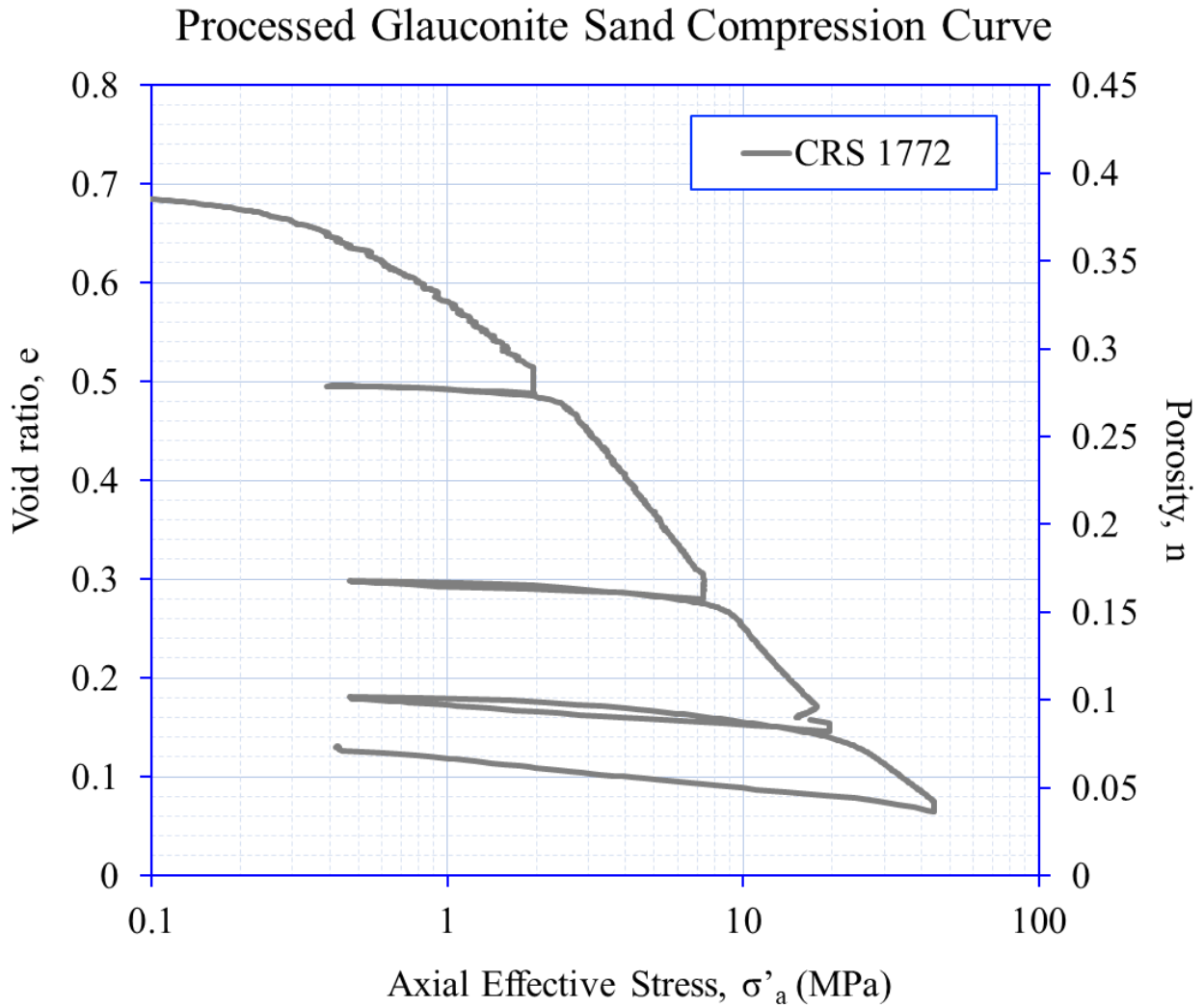


Figure 6.12: A compression curve of glauconite sand in void ratio log-effective stress space (CRS 1772). Secondary compression was observed at 4 stress levels: 2, 7.8, 20, and 44 MPa.

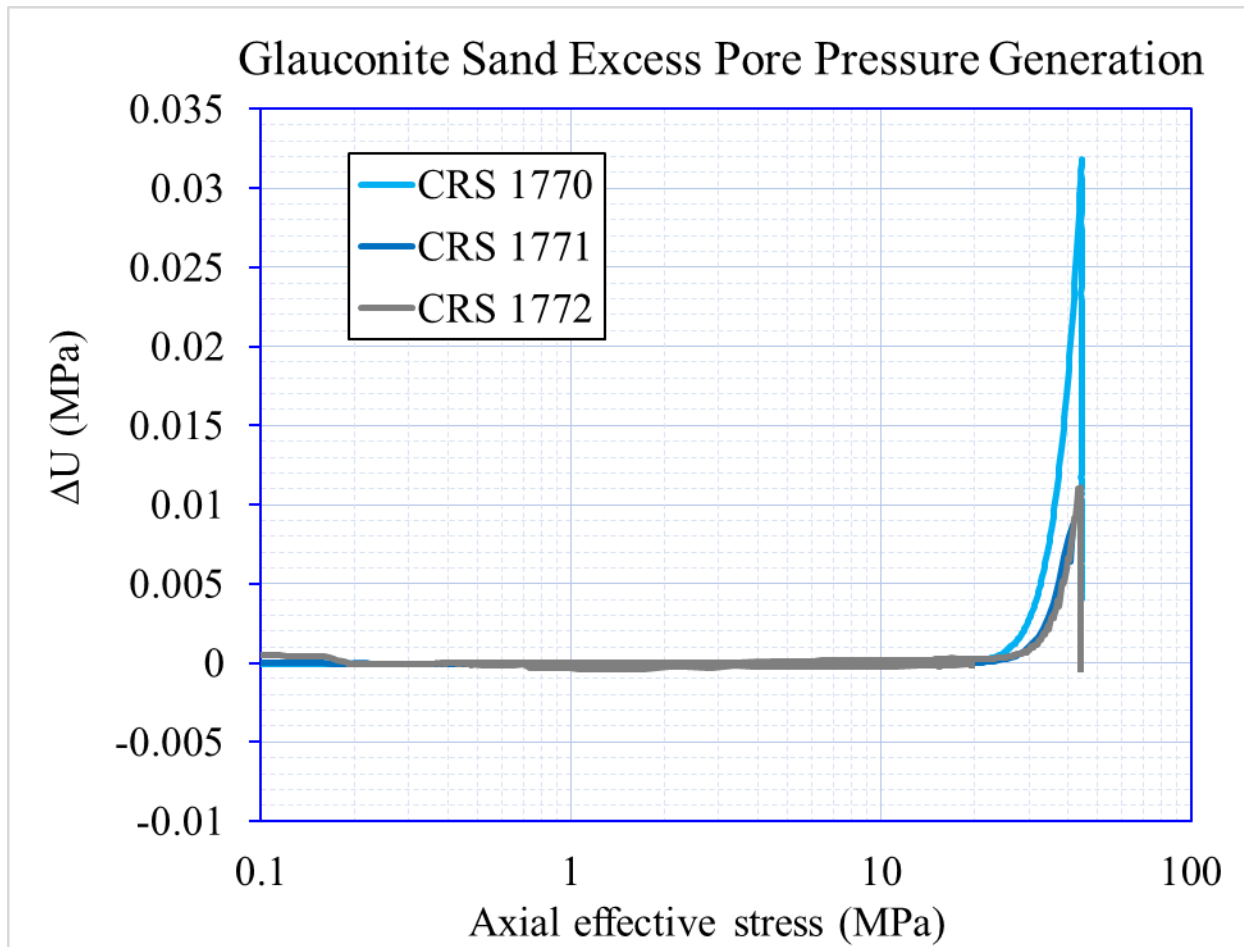


Figure 6.13: A plot showing buildup of excess pore pressure during CRS testing of glauconite sand at 2% strain rate.

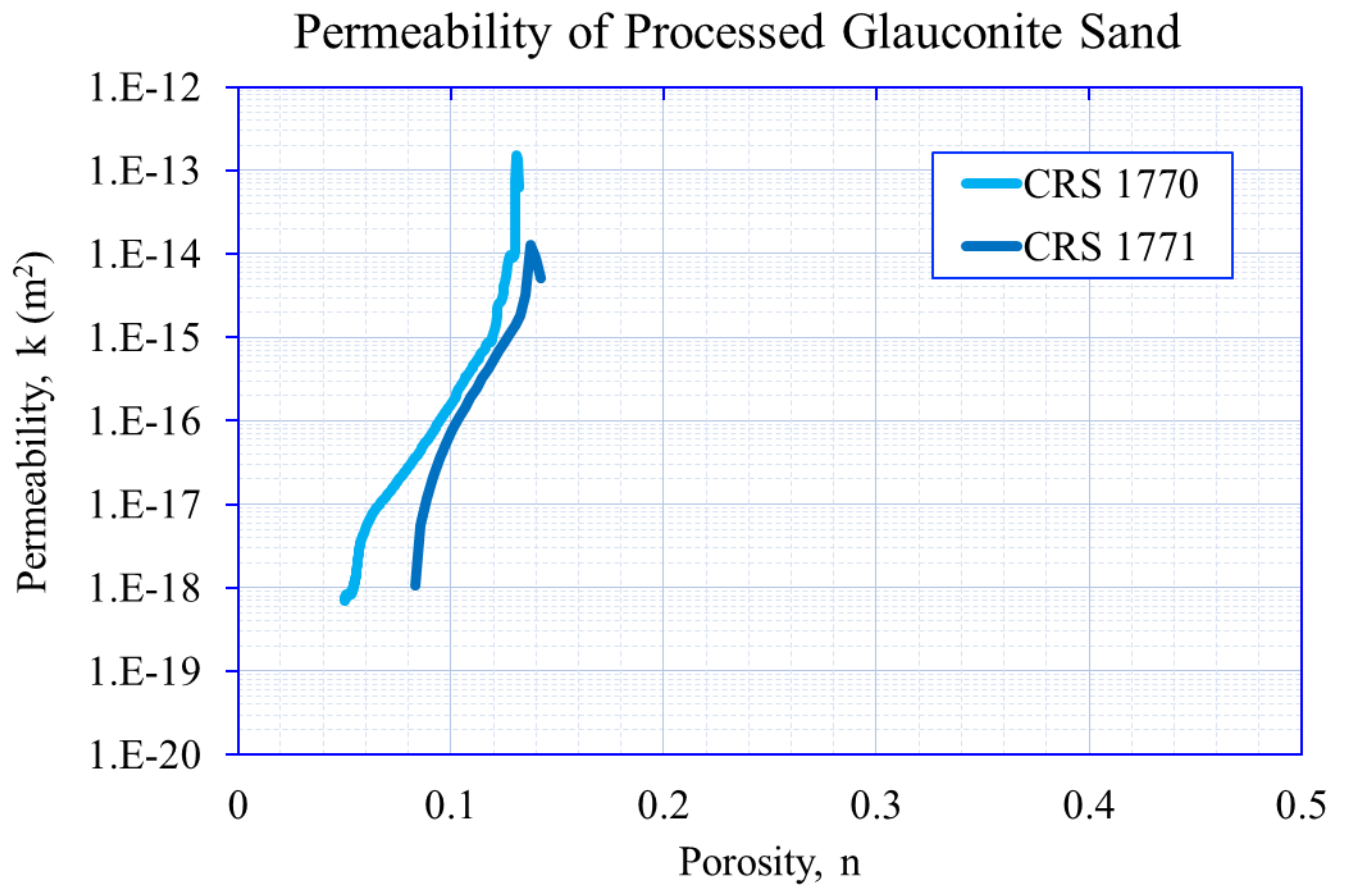


Figure 6.14: A plot showing the significant decrease in permeability observed during CRS testing of glauconite sand.

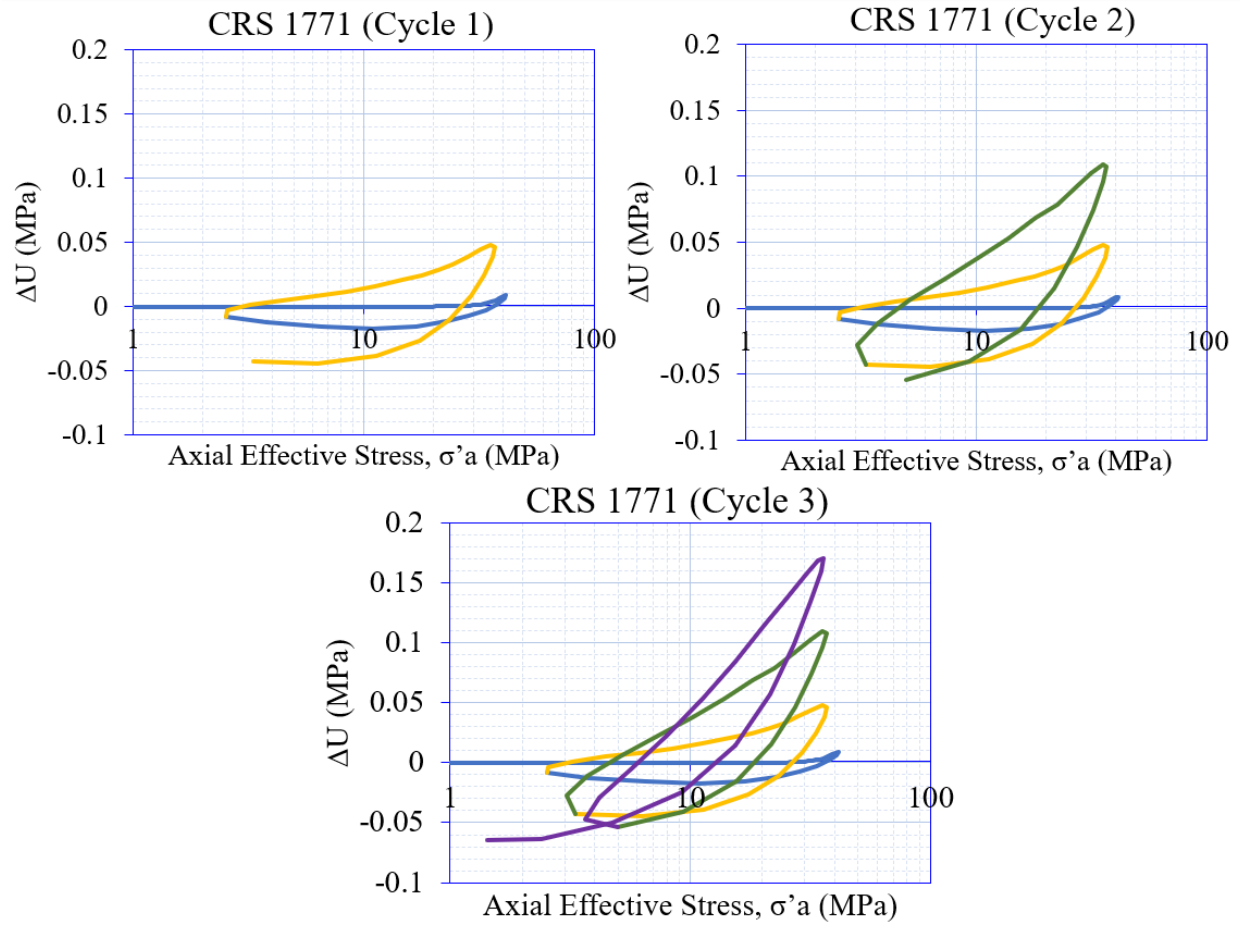


Figure 6.15: A plot of excess pore pressure during cyclic loading of glauconite sand during CRS 1771. Note that the rate of loading is accelerating despite the constant 2% rate of strain.

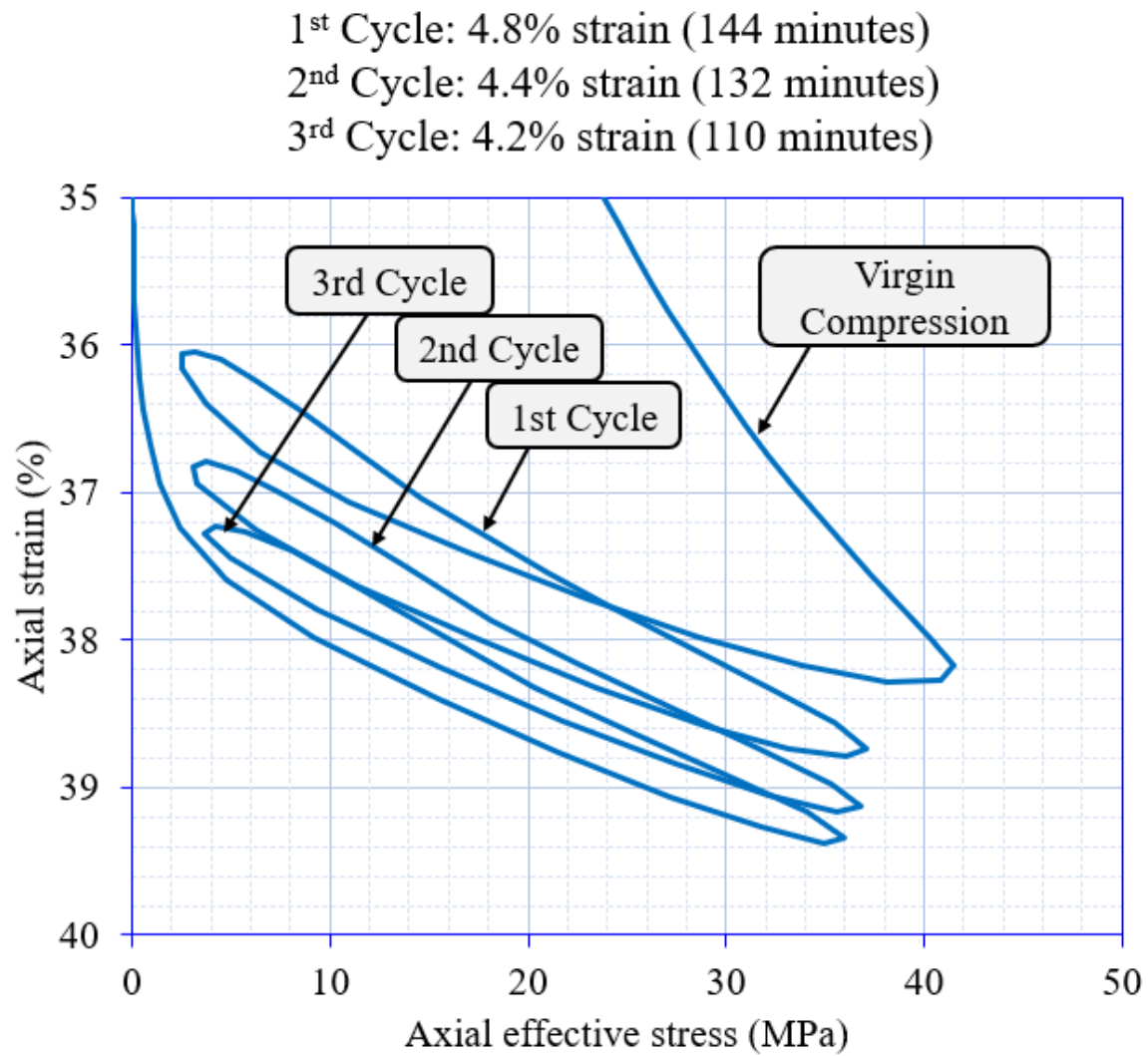
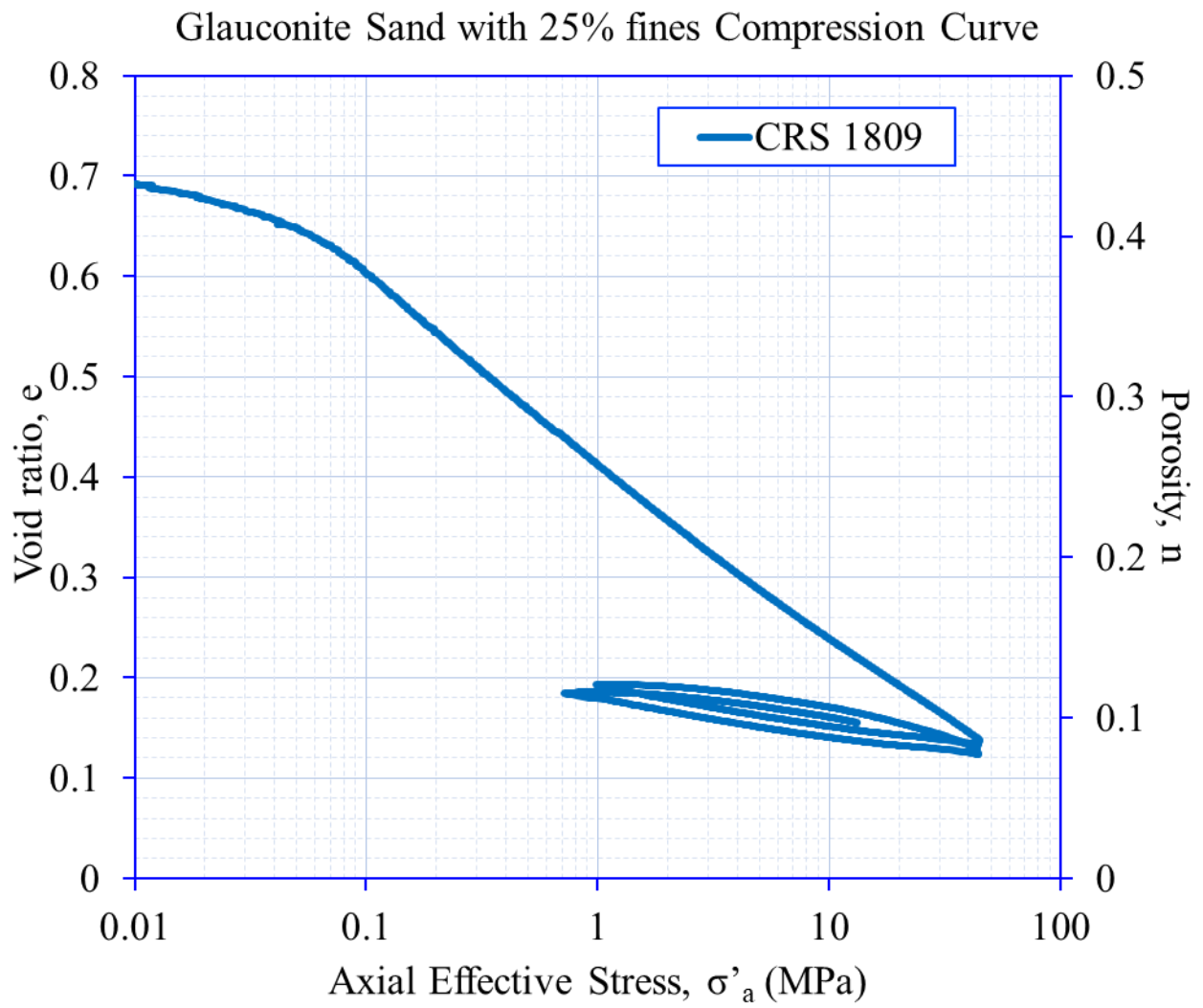
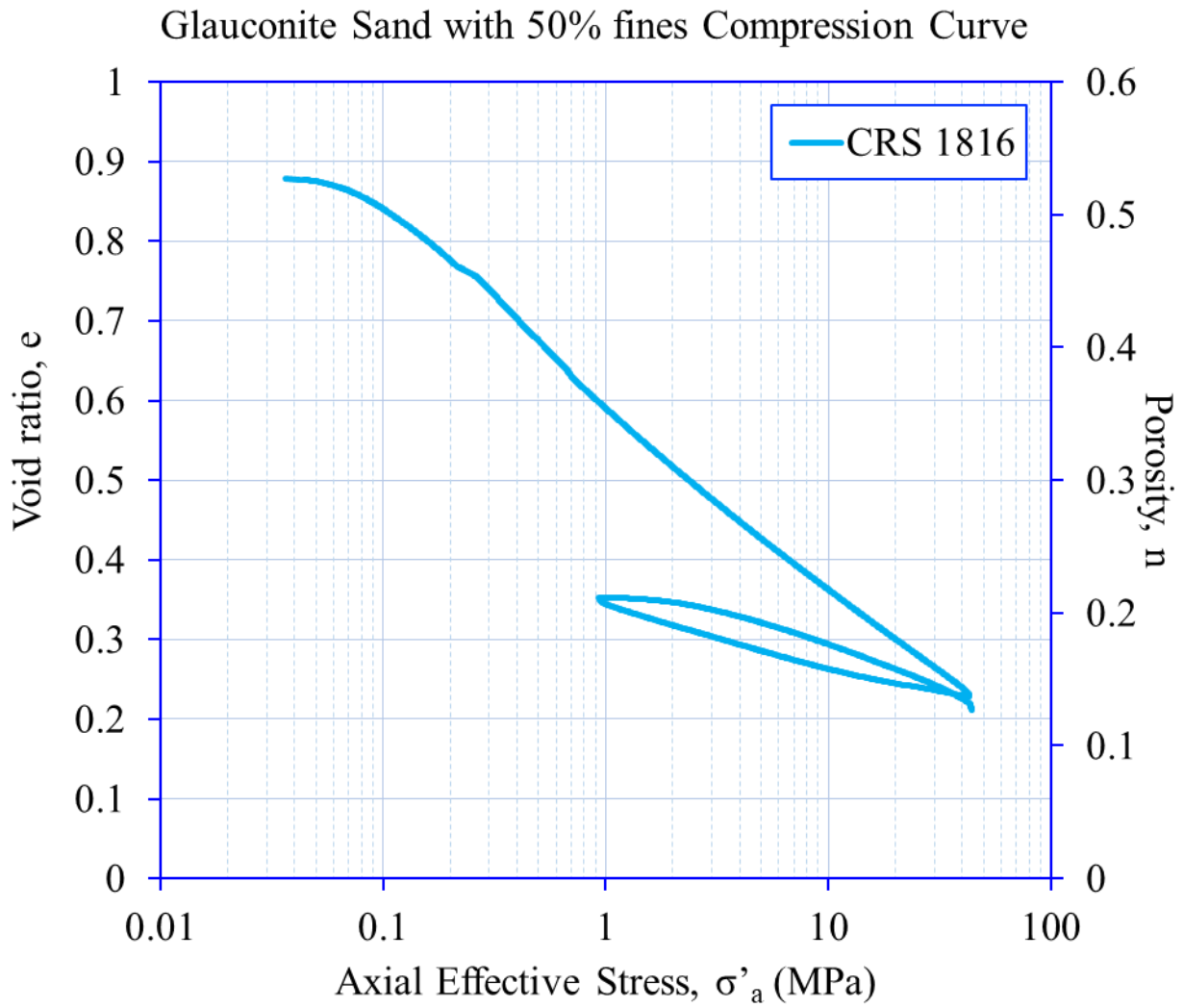
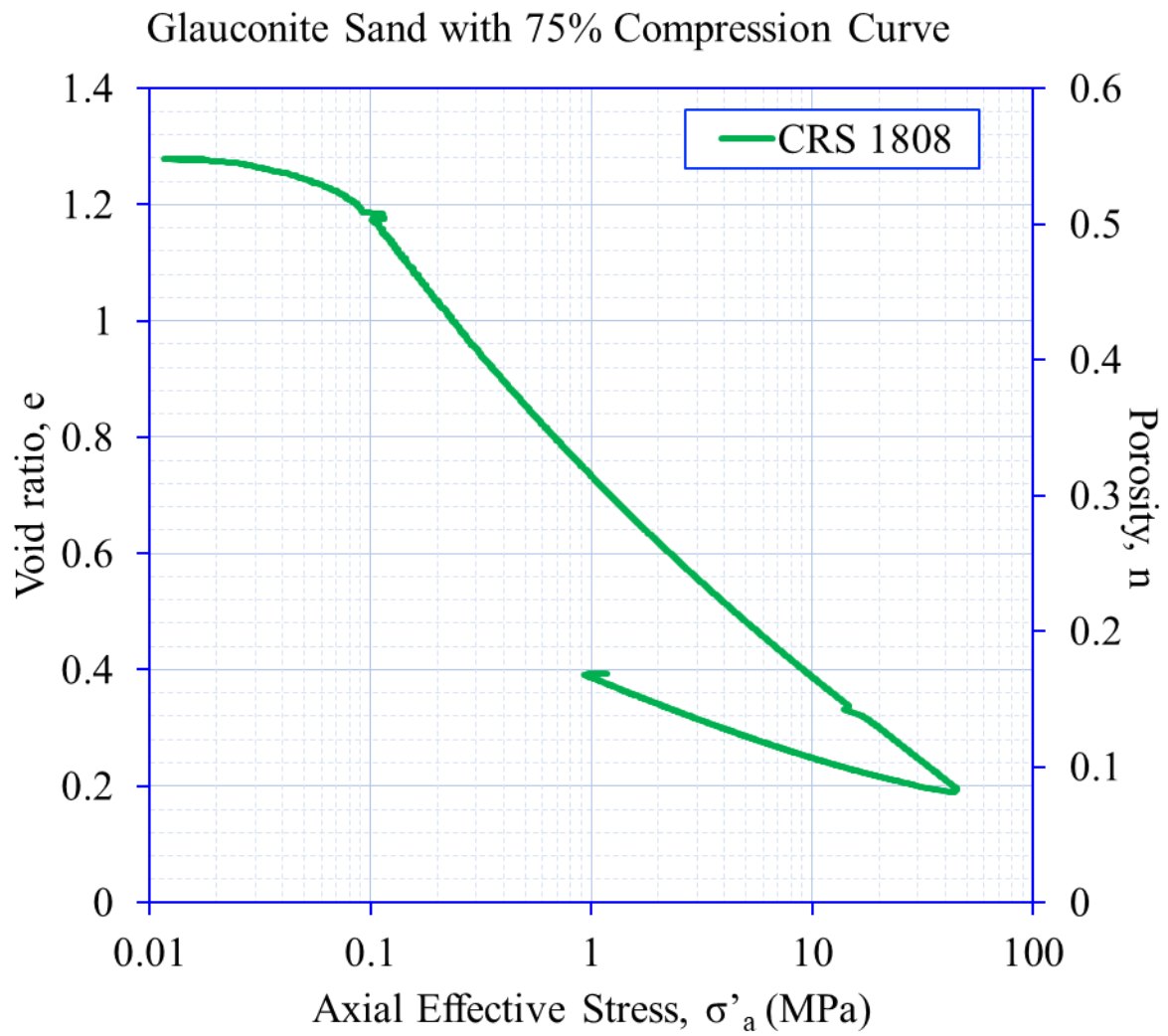


Figure 6.16: A plot of axial strain as a function of axial effective stress during cyclic loading of processed glauconite sand. Because CRS 1771 was conducted at a strain rate of 2% per hour, the author has calculated the length of time for each strain increment during reloading (see above).







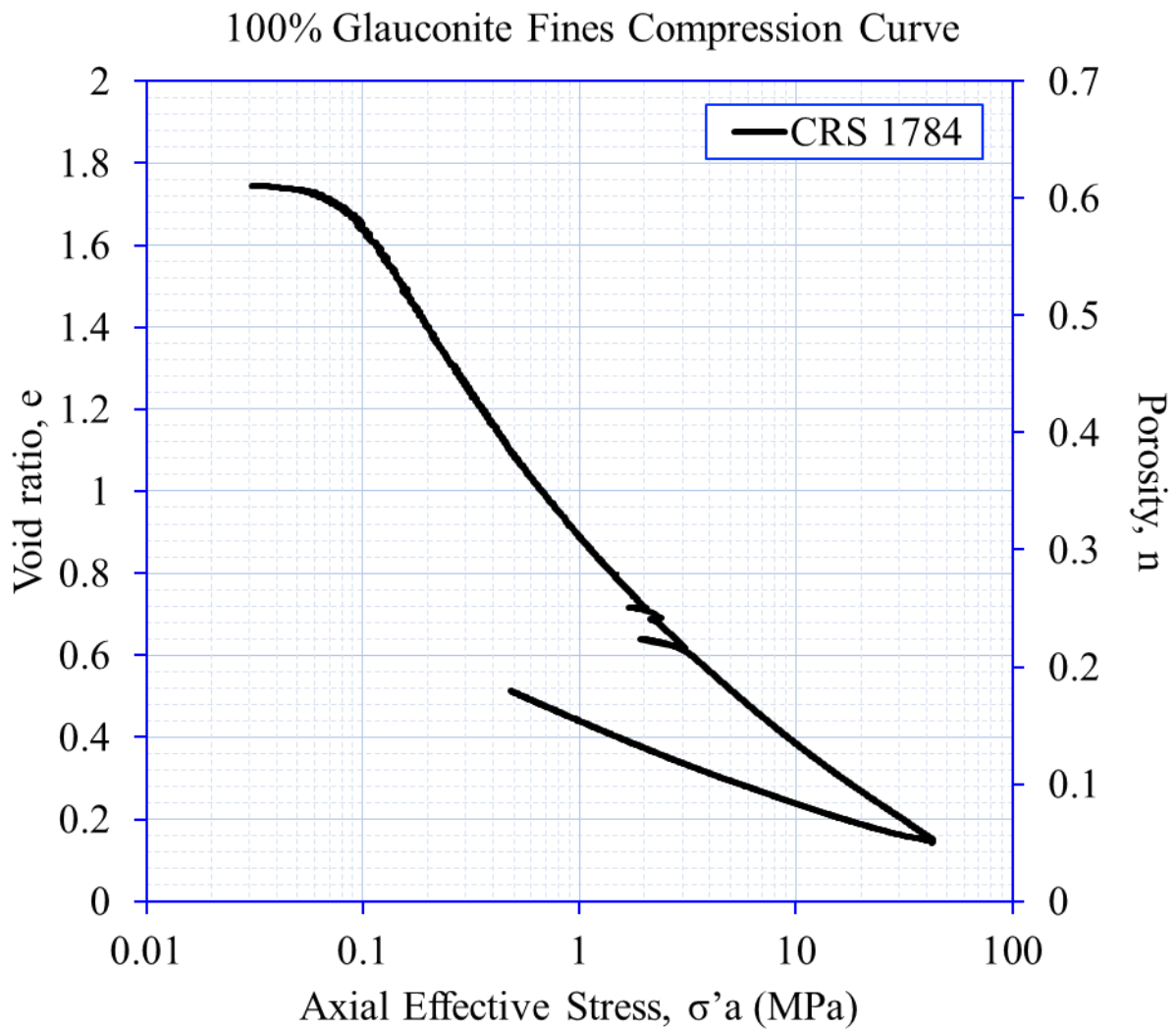
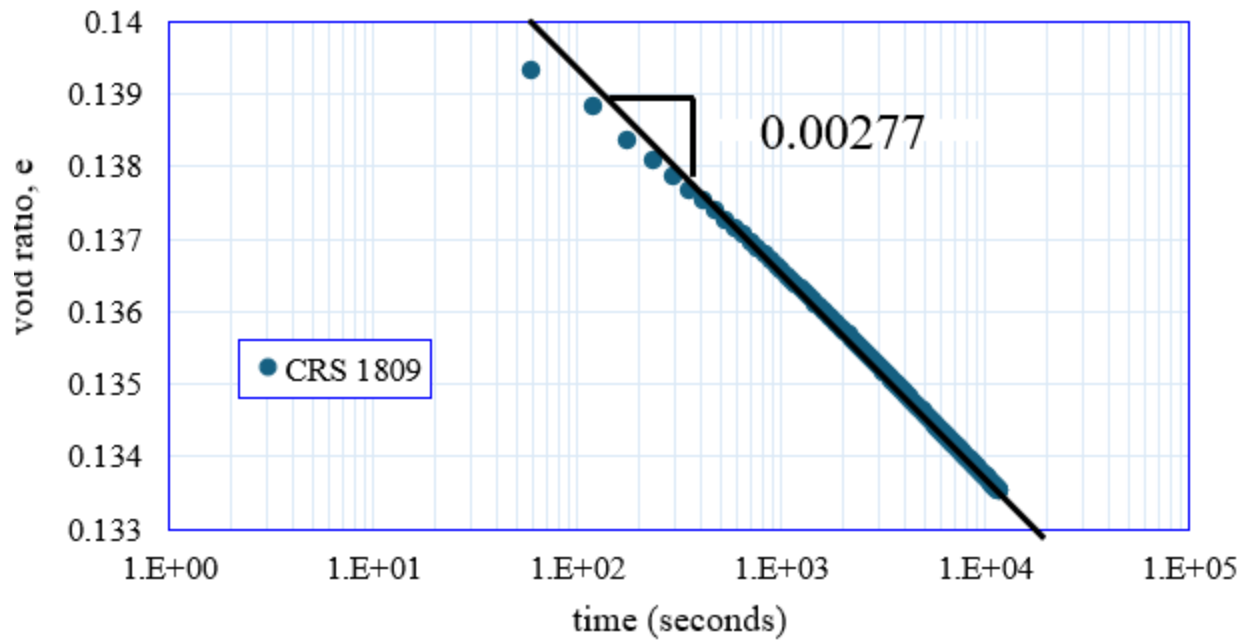
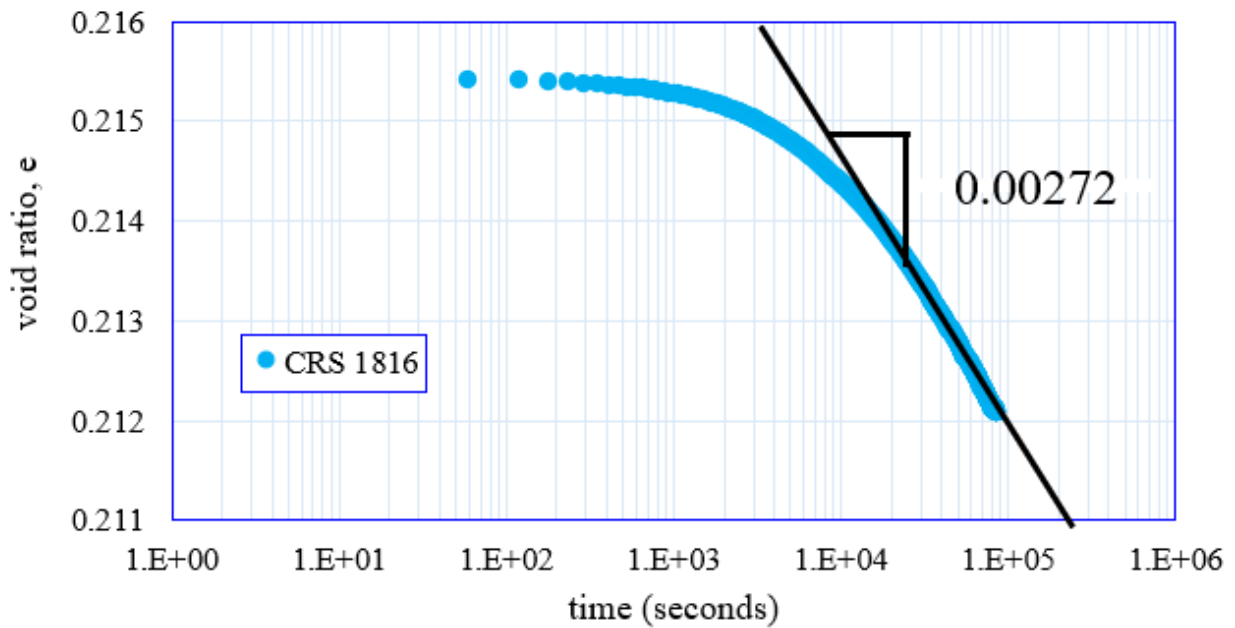


Figure 6.18: Four compression curves from CRS 1809, 1816, 1808, and 1784. The fines content was increased from 25%, to 50%, to 75%, to 100% for each test.

Secondary Compression (25% fines content) at 44 MPa



Secondary Compression (50% fines content) at 44 MPa



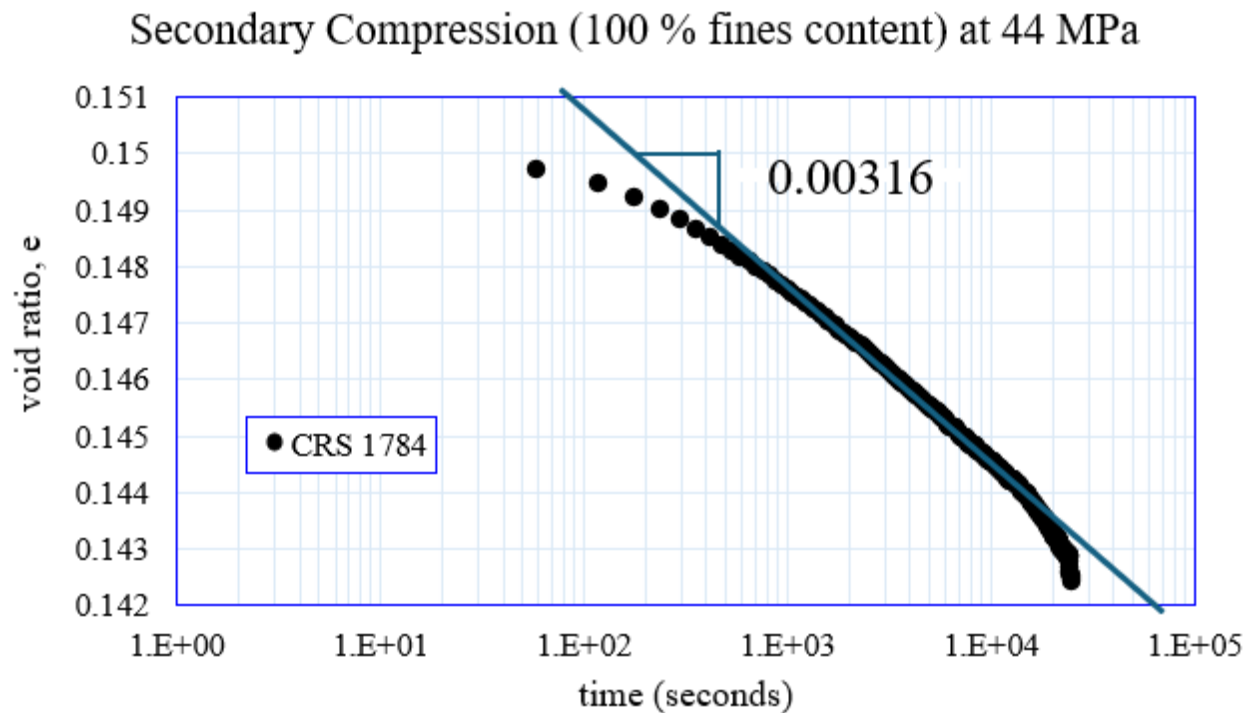
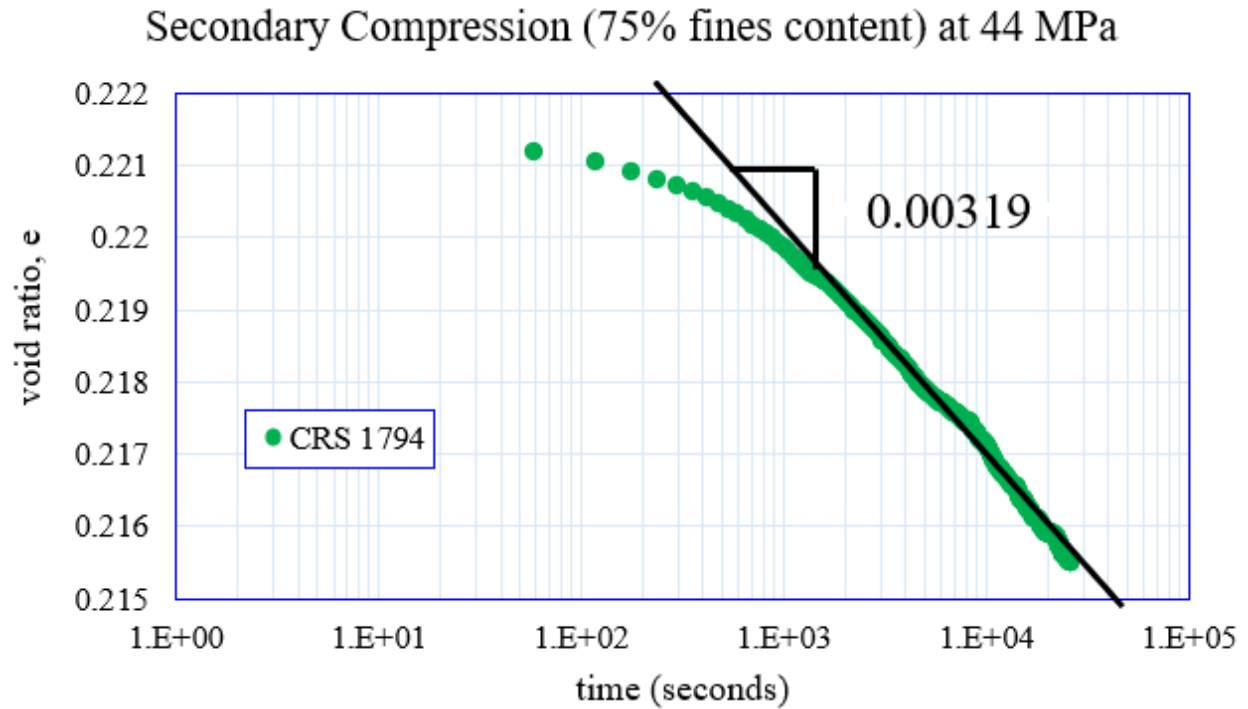


Figure 6.19: A plot of secondary compression index for resedimented specimens with 25, 50, 75, and 100% fine-grained glauconite. Note that CRS 1816 and CRS 1794 use data from after unload reload cycles.

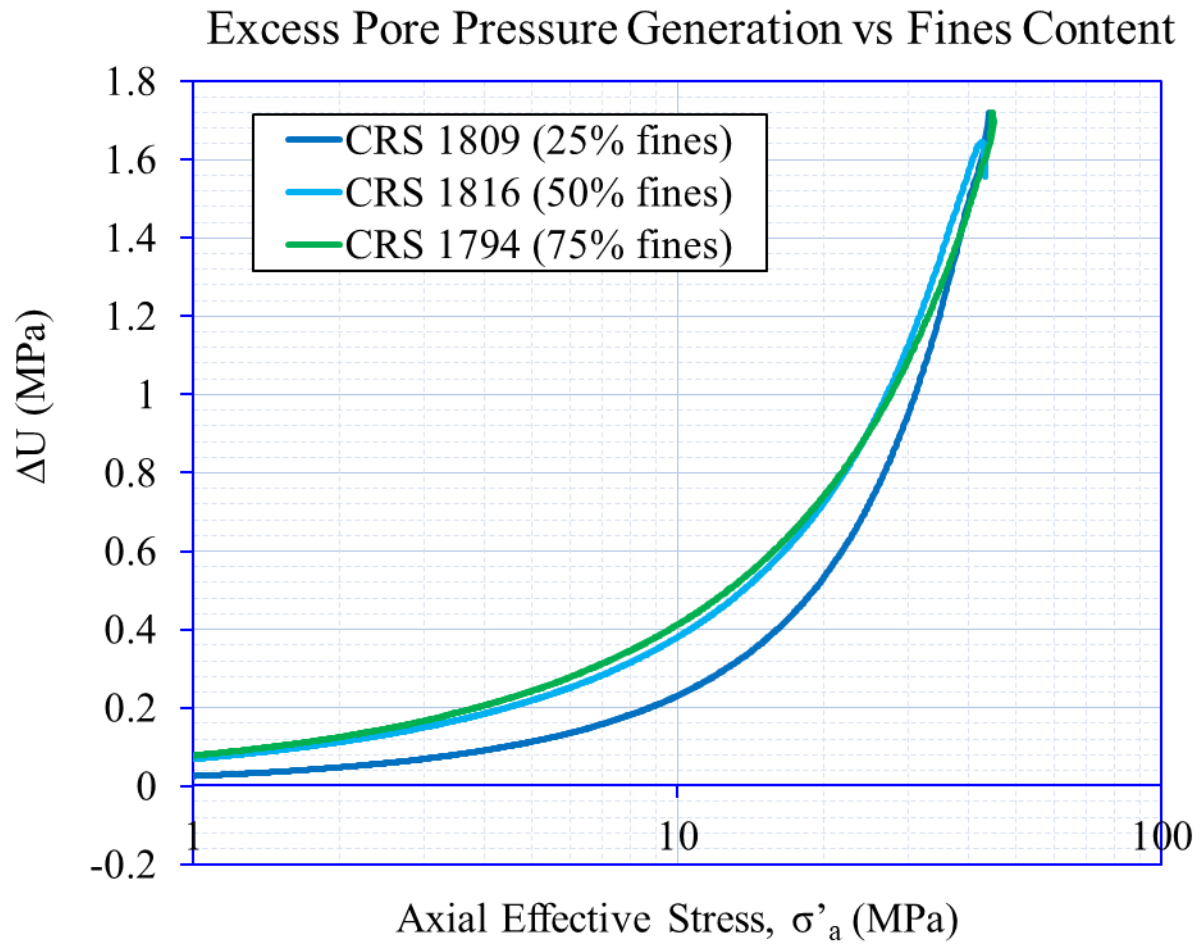


Figure 6.20: A plot showing the excess pore pressure response when loading resedimented specimens of glauconite sand with varying glauconite fines to 44 MPa effective stress at 1%

strain rate.

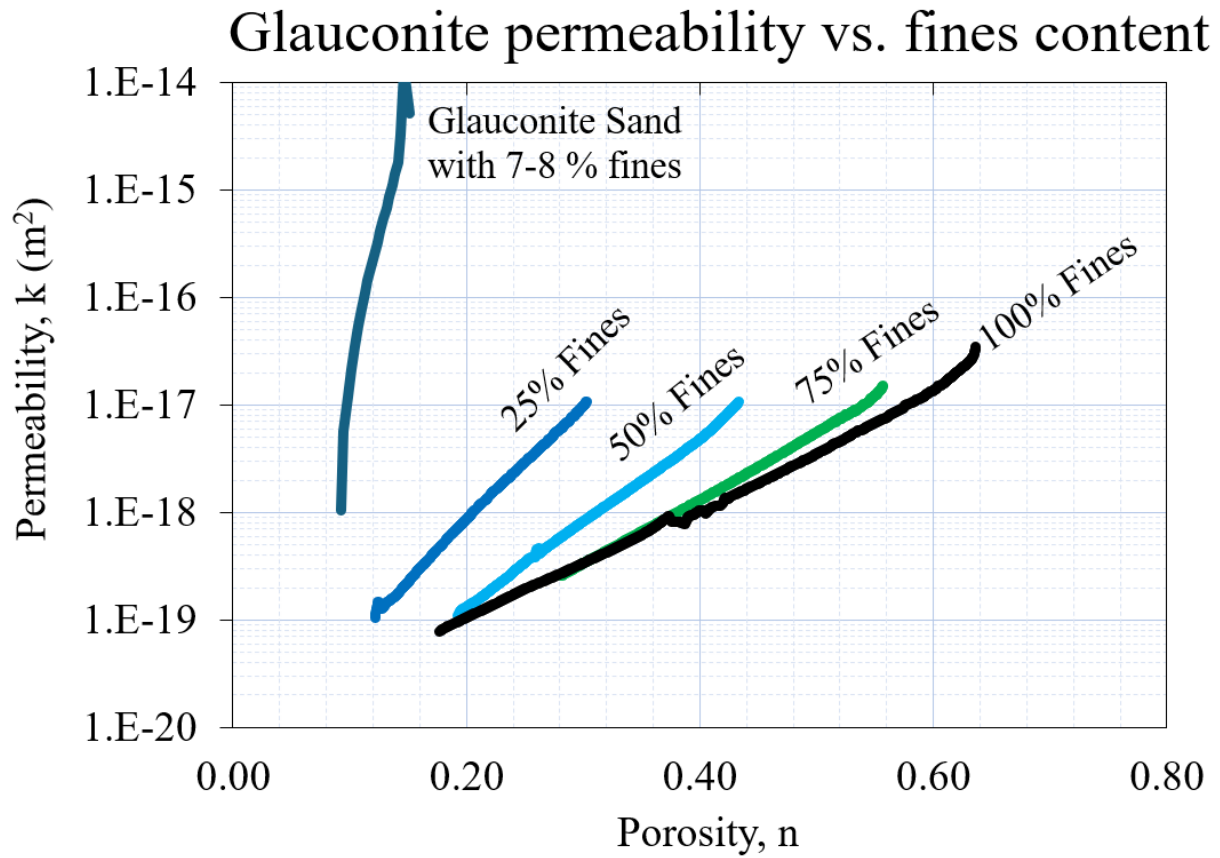


Figure 6.21: A plot showing permeability as a function of porosity from CRS tests on glauconite sand, fine-grained glauconite, or glauconite sand with varying fines content.

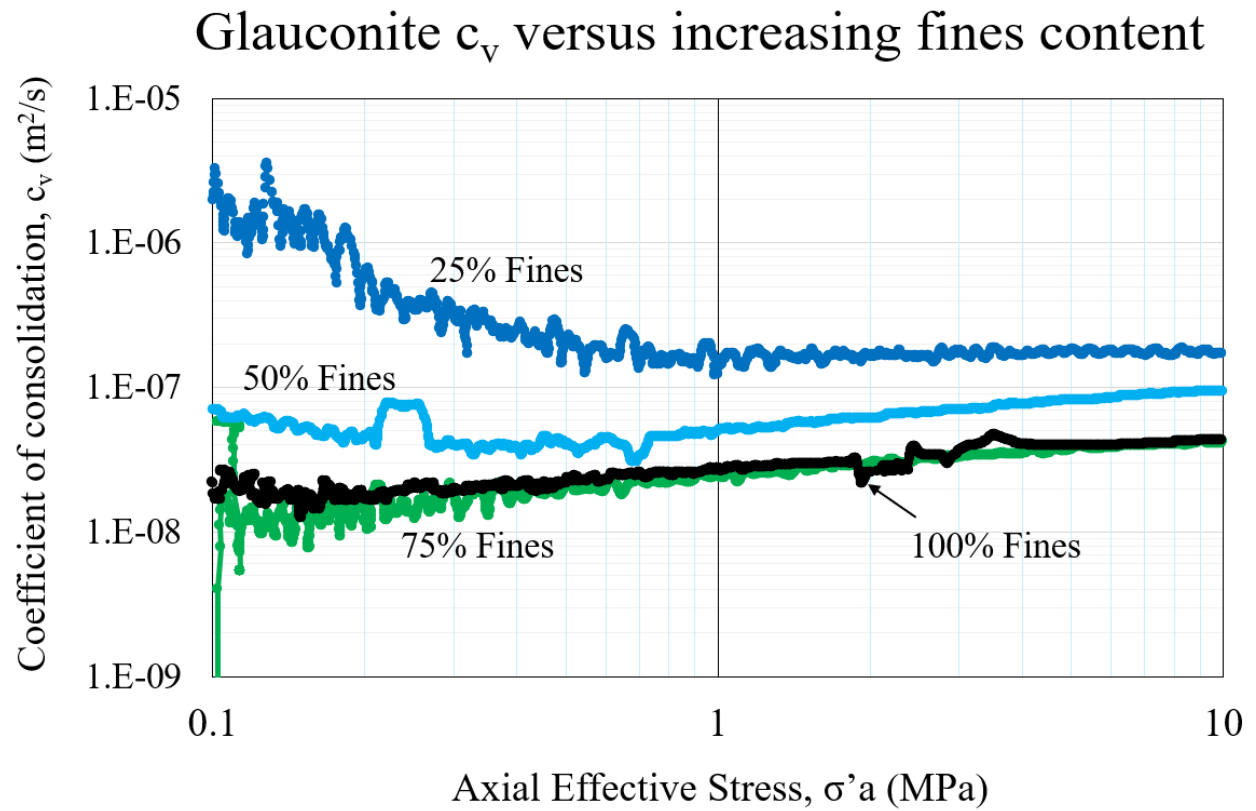
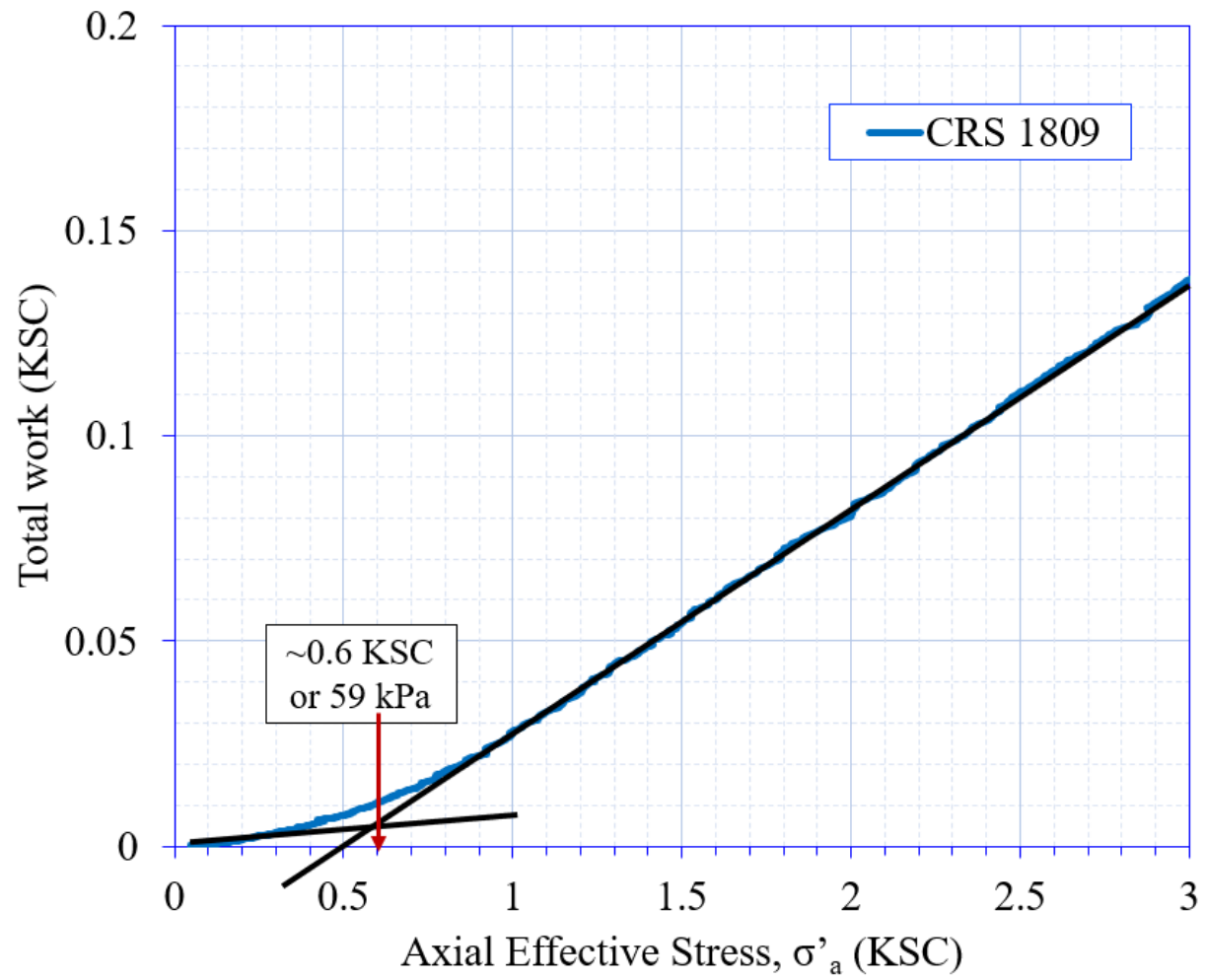
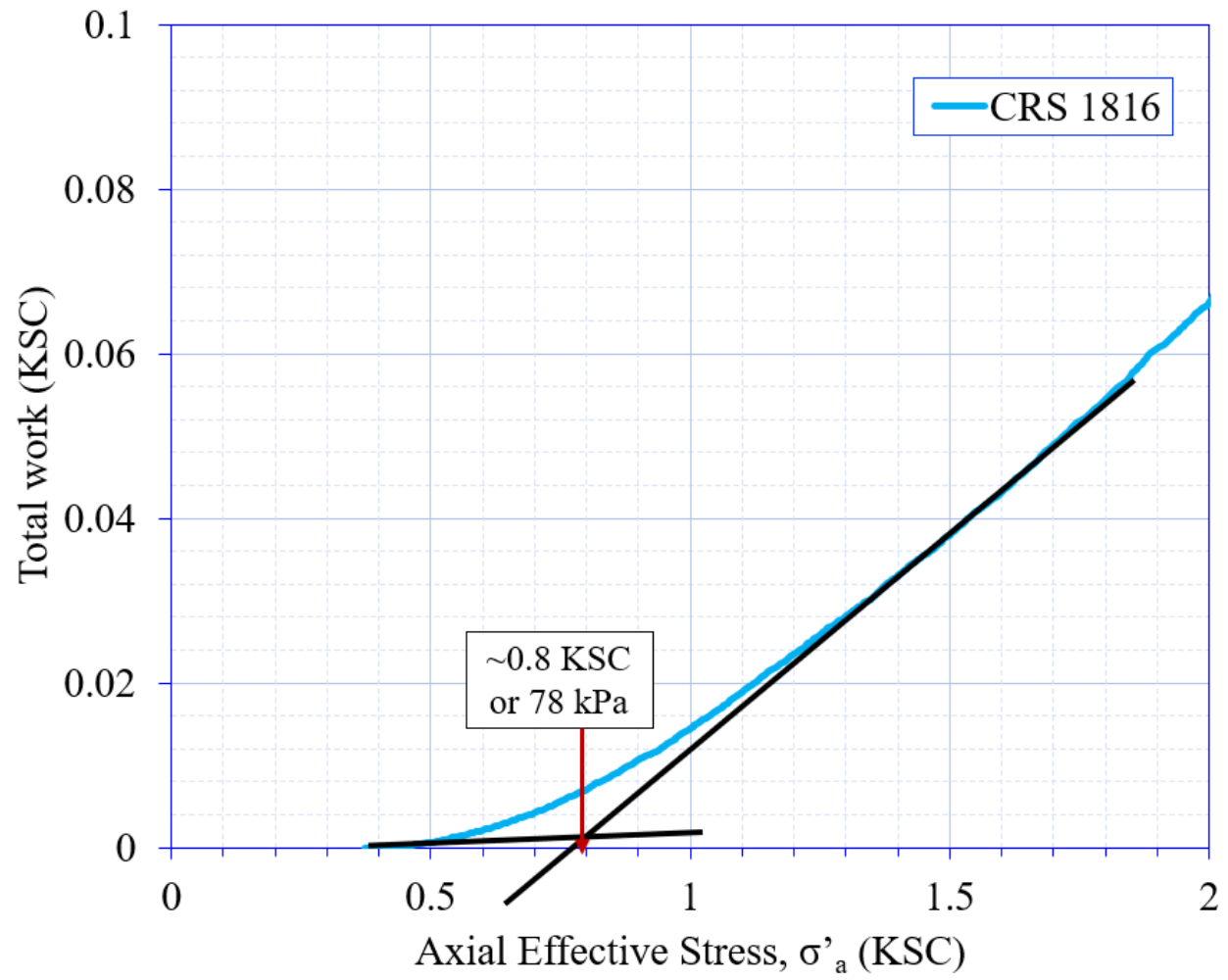


Figure 6.22: A plot showing coefficient of consolidation as a function of axial effective stress from CRS tests on fine-grained glauconite and glauconite sand with varying fines contents.

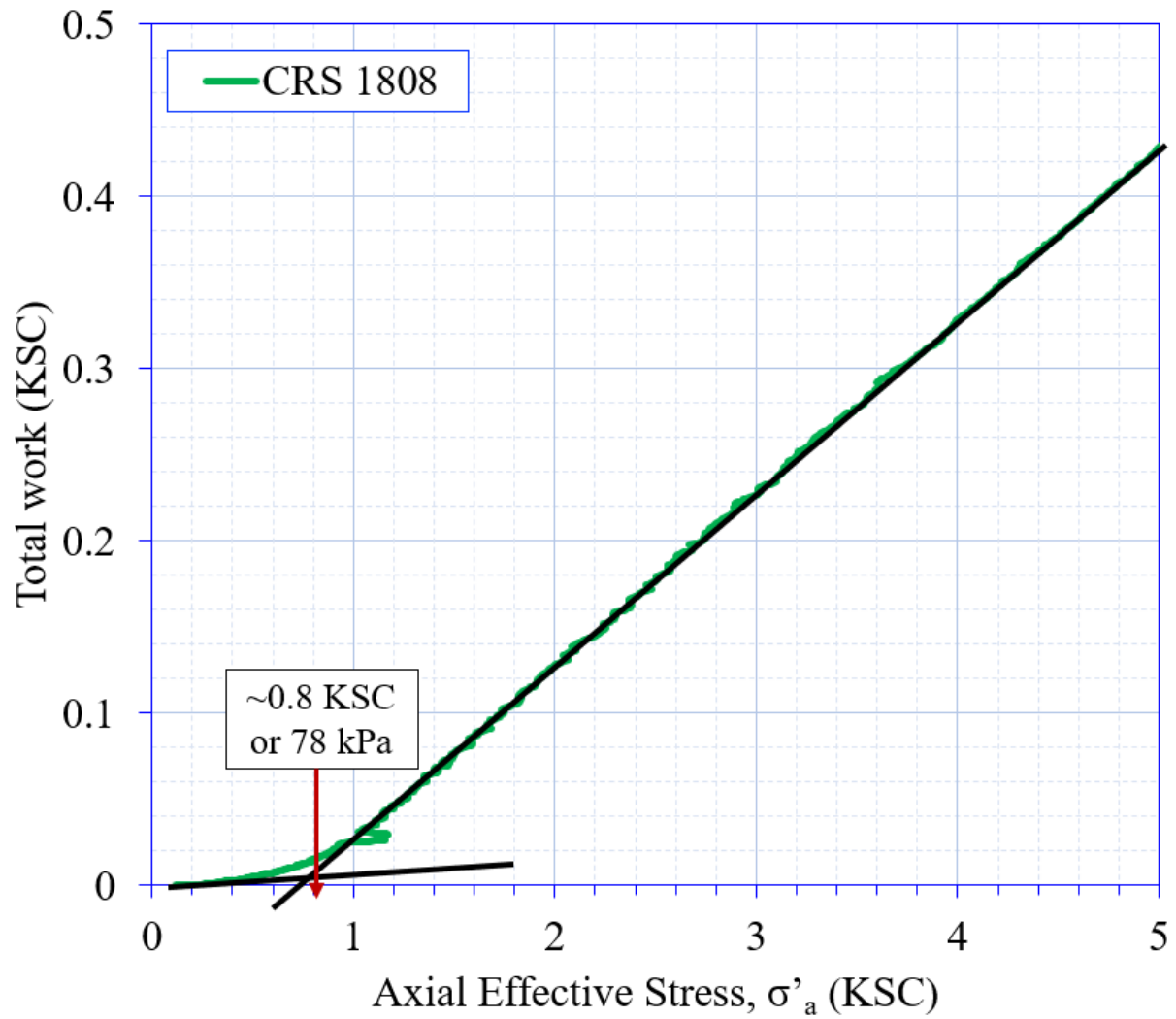
Strain Energy Determination of Maximum Past Pressure



Strain Energy Determination of Maximum Past Pressure



Strain Energy Determination of Maximum Past Pressure



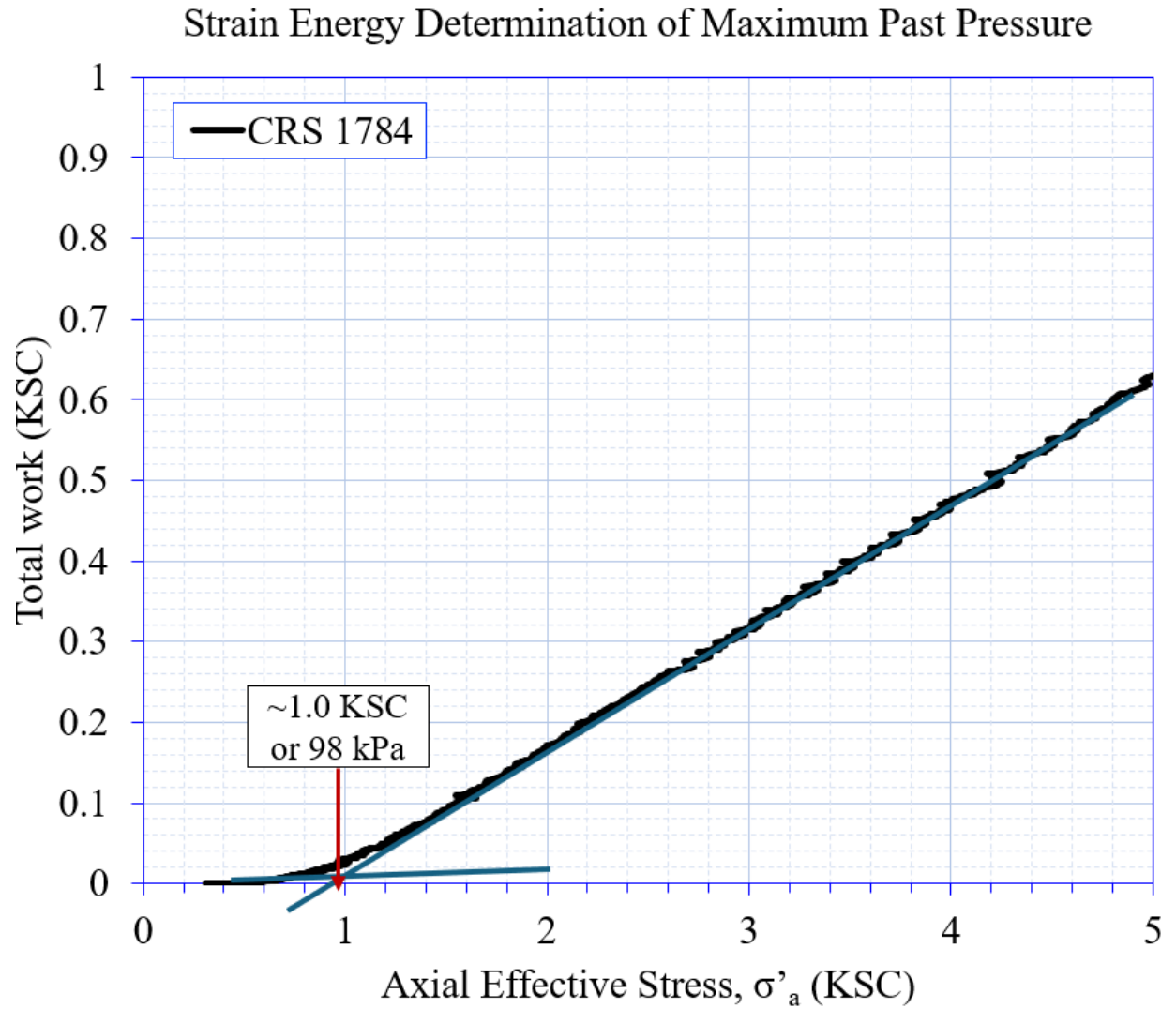


Figure 6.23: Four plots which relate total work (KSC) to axial effective stress (KSC) and show how the strain energy method can be used to determine the maximum past pressure ($\sigma_{p'}$) from a CRS test. Note that $\sigma_{p'}$ is shown in both KSC and kPa.

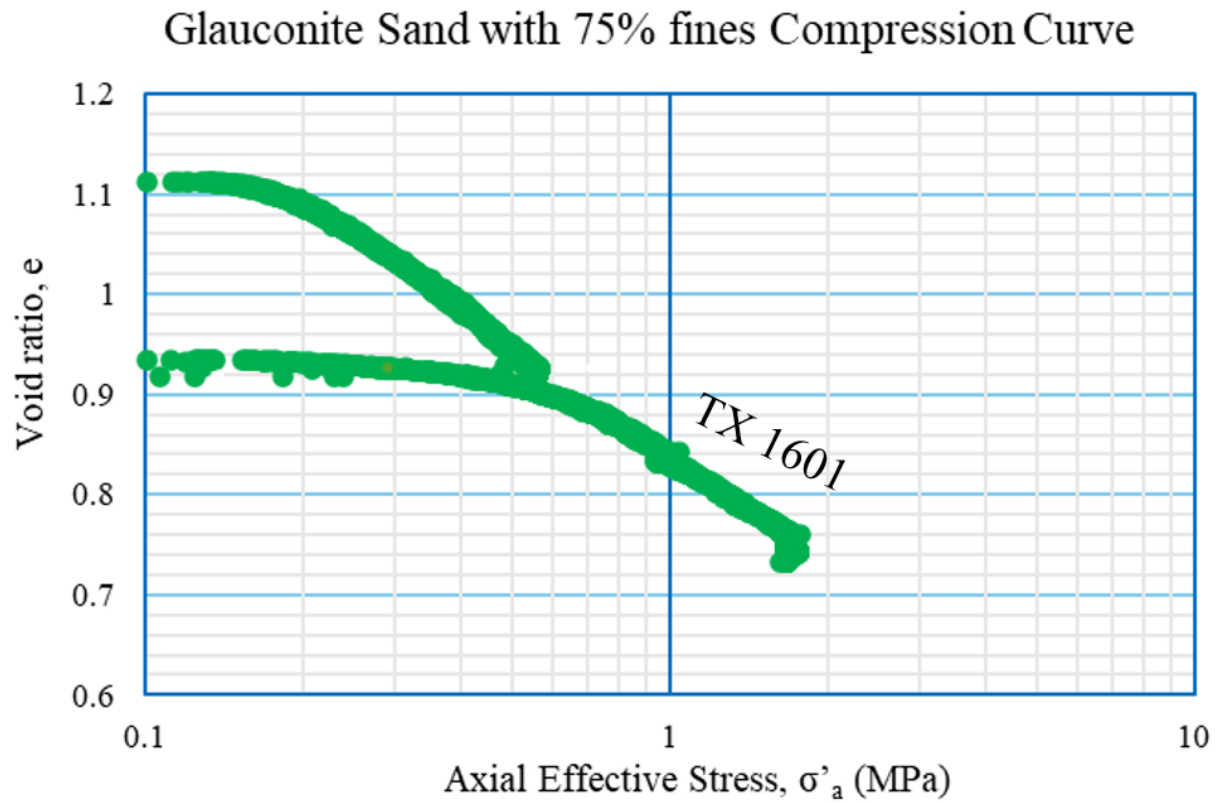


Figure 6.24: A compression curve from TX1601, a CK_0 UC triaxial test on glauconite sand with 75% fine-grained glauconite.

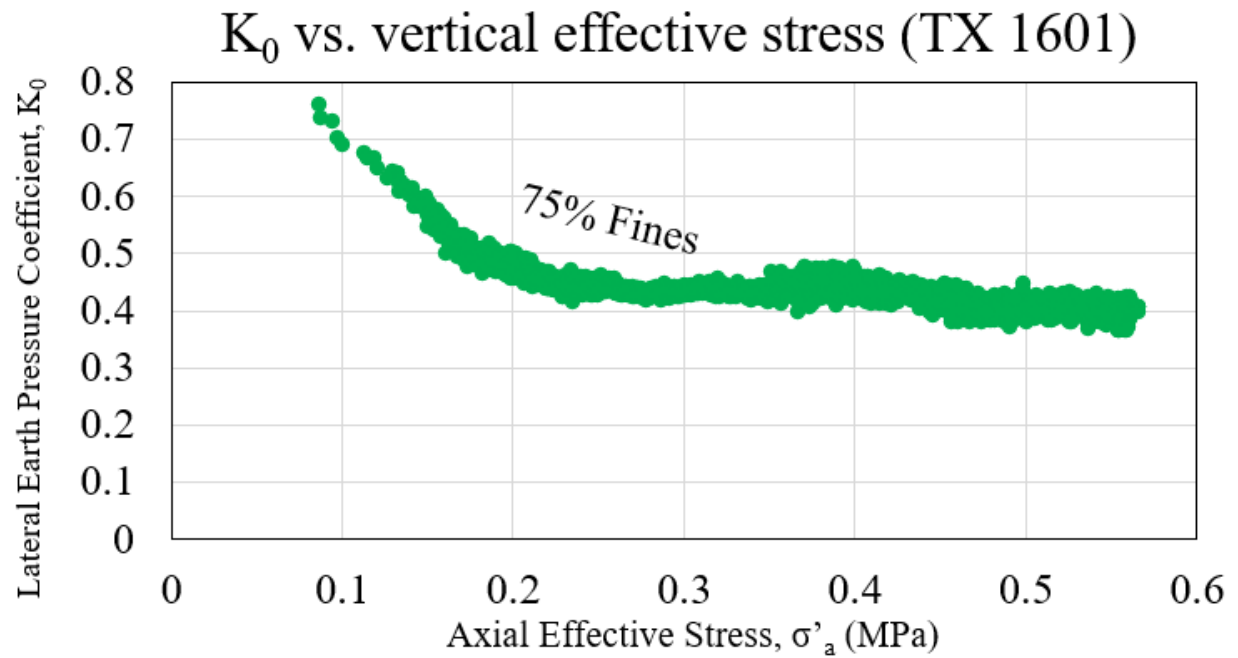


Figure 6.25: A plot of K_0 values determined from the K_0 consolidation phase of TX 1601.

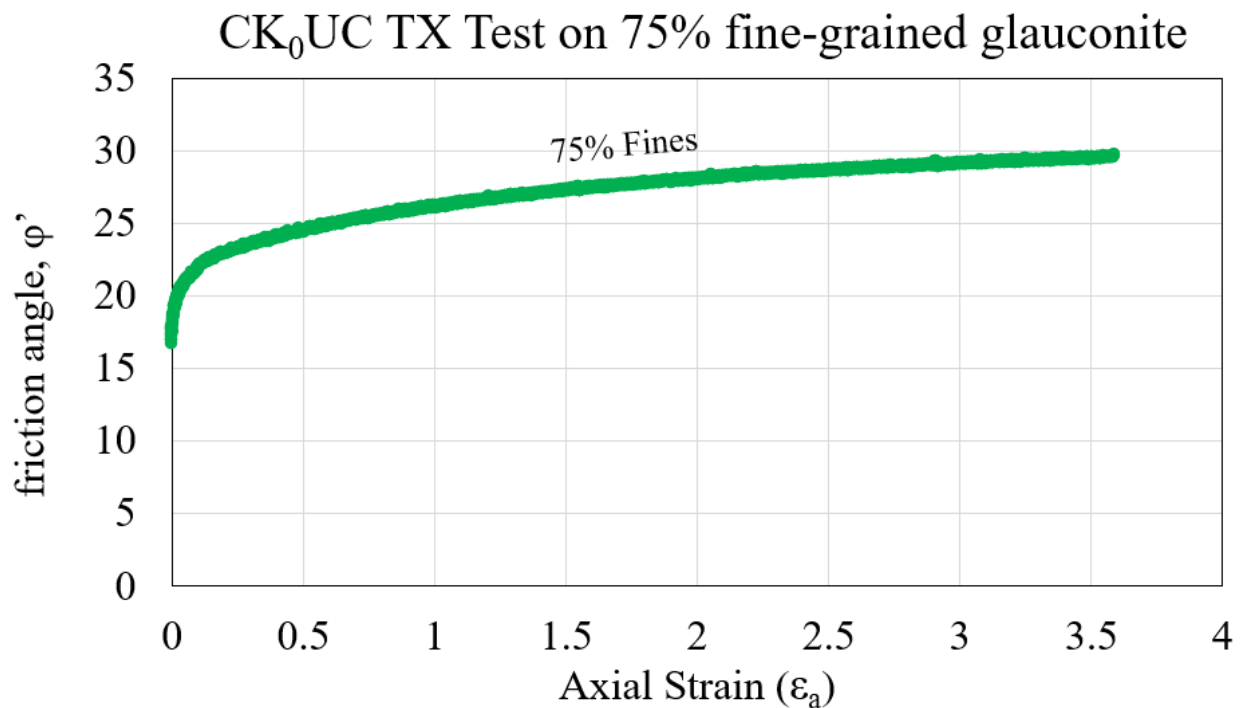


Figure 6.26: A plot of the internal friction angle as a function of axial strain from TX 1601.

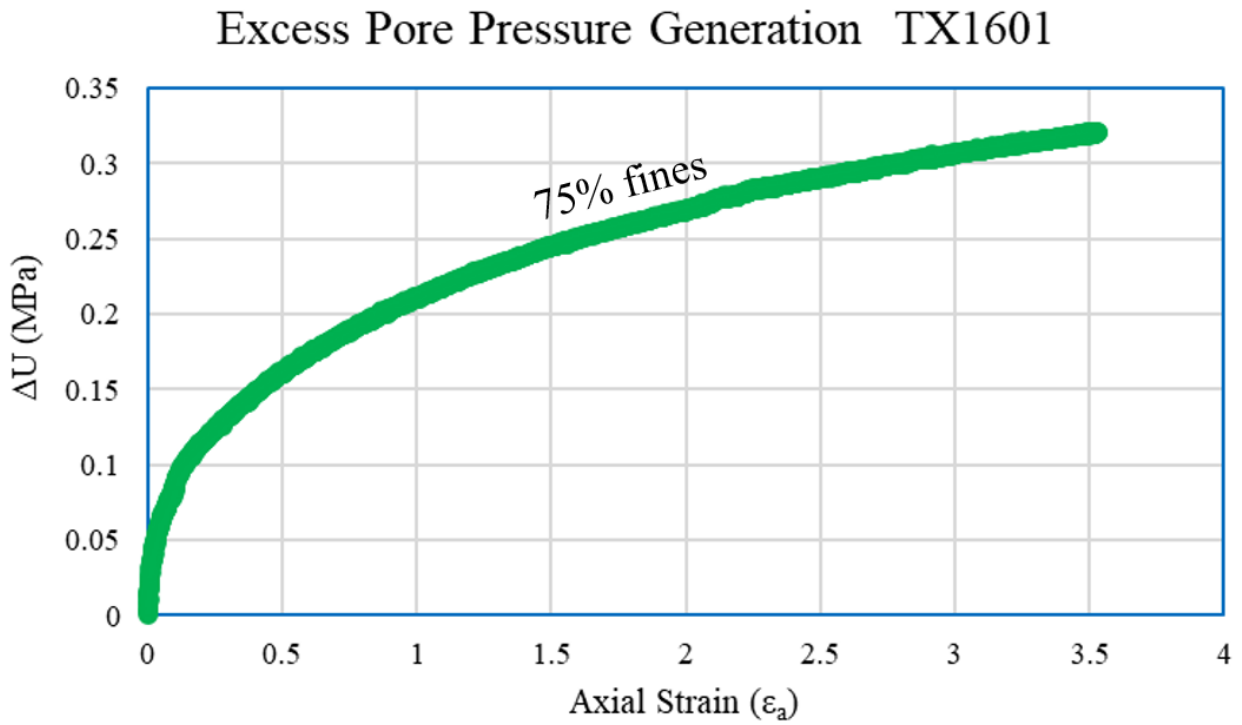


Figure 6.27: A plot showing the buildup of excess pore pressure during undrained shear from TX1601

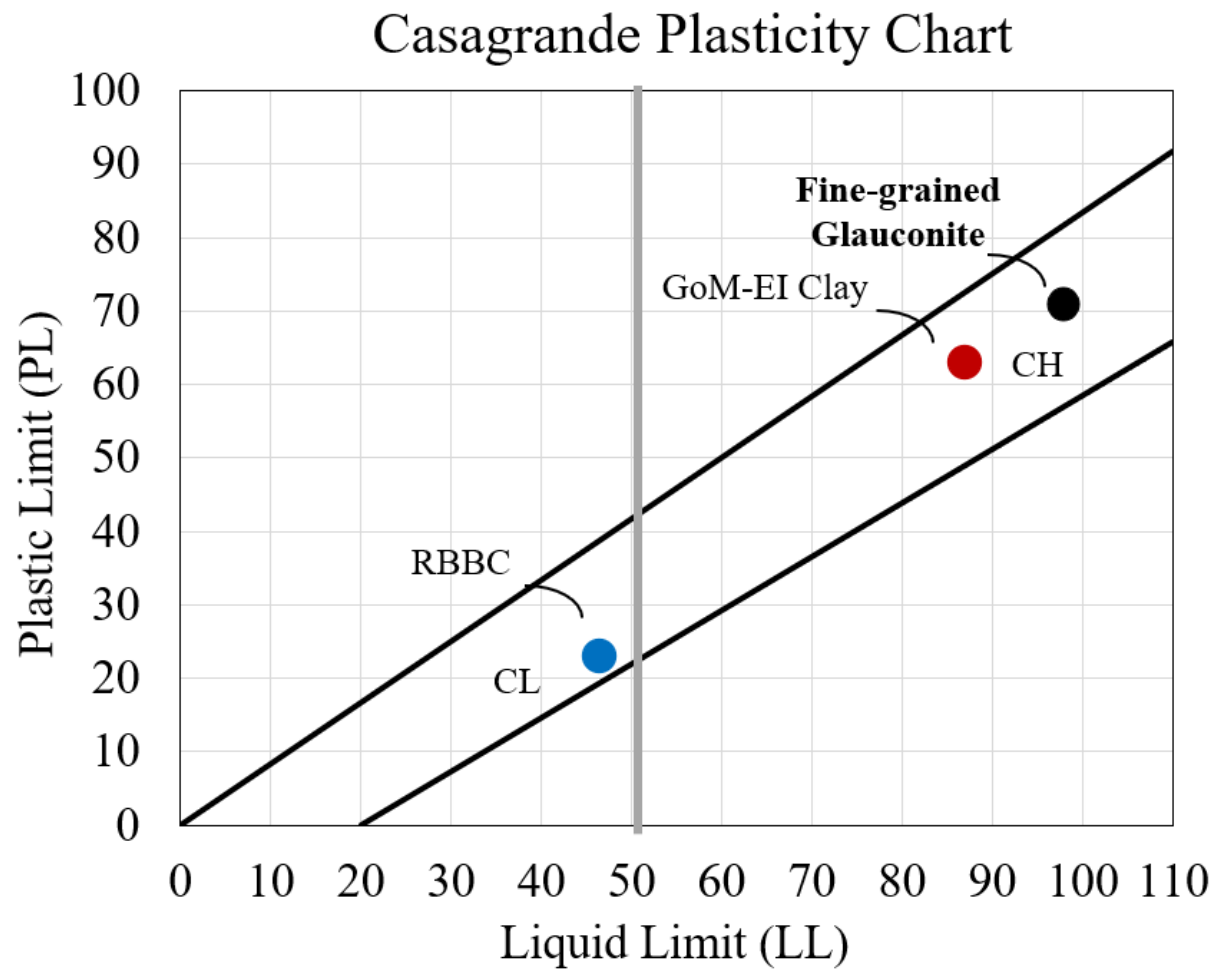


Figure 6.28: A Casagrande Plasticity chart comparing the plasticity of glauconite clay (i.e. fines) with RBBC and RGoM-EI Clay.

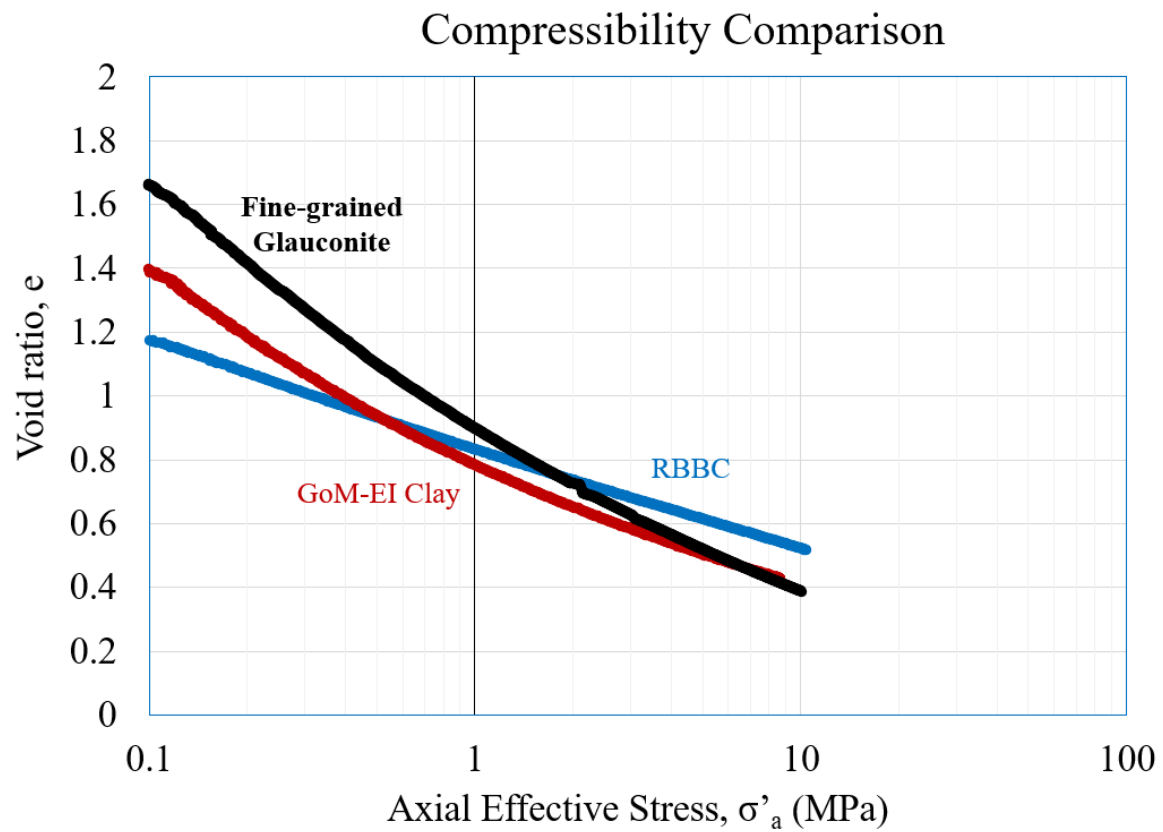


Figure 6.29: A plot comparing the compression curves of fine-grained glauconite (CRS 1784) with RBBC and RGoM-EI.

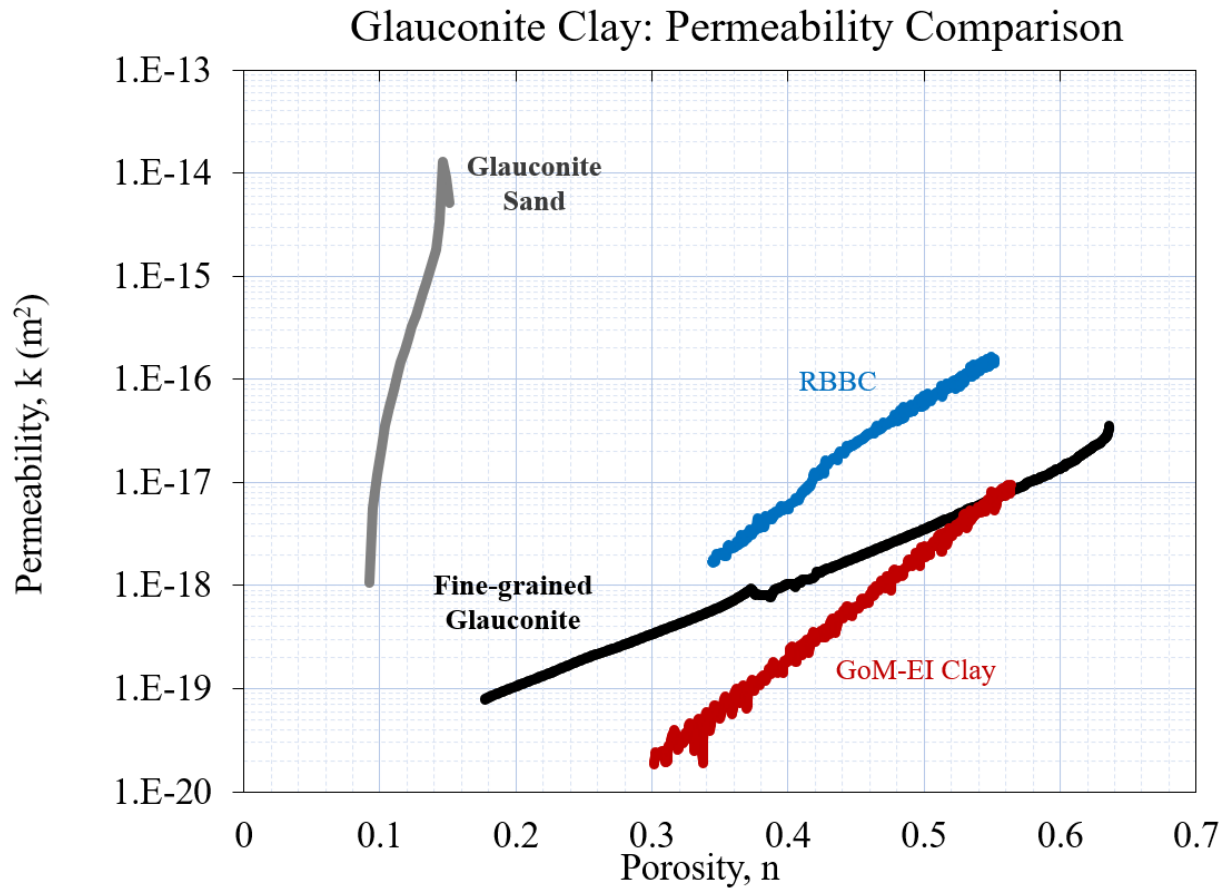


Figure 6.30: A plot comparing the permeability, K , of glauconite sand (CRS 1771), fine-grained glauconite (CRS 1784), RBBC, and RGoM-EI at various porosities.

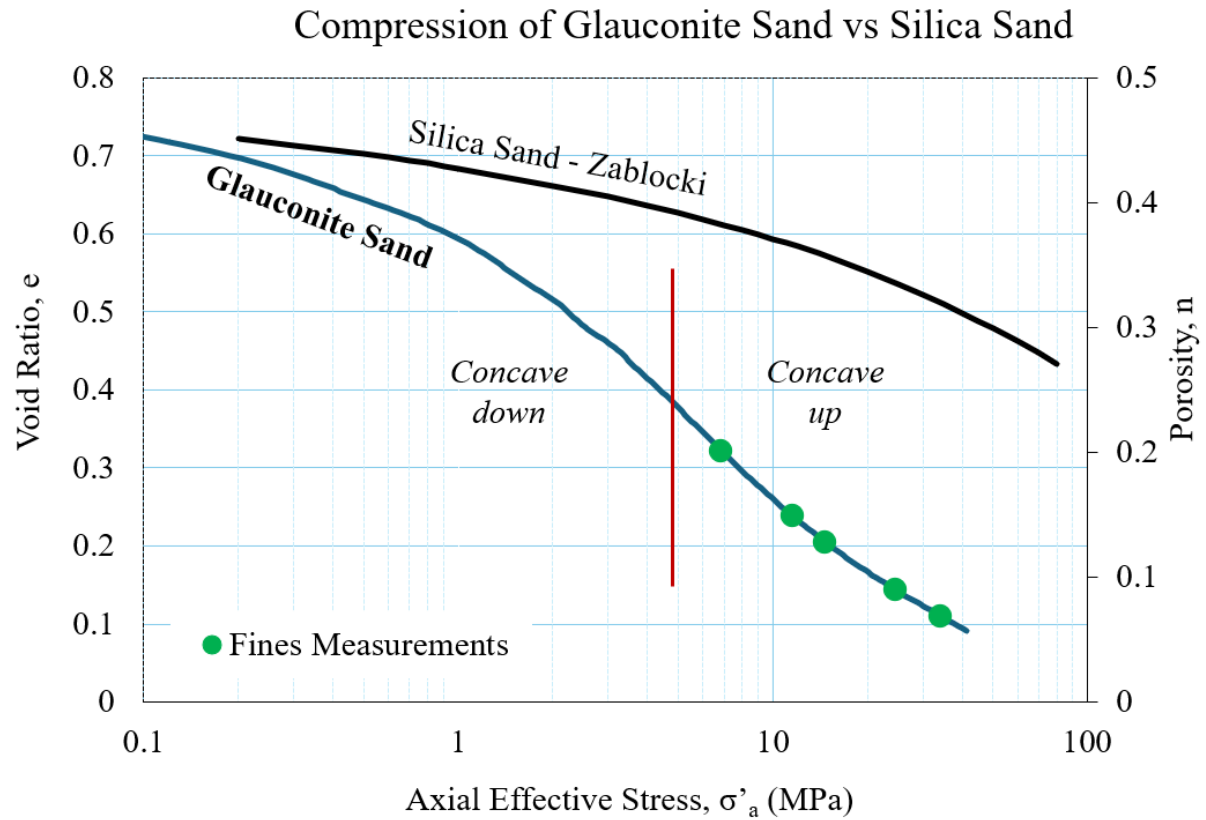


Figure 6.31: A plot comparing the compression behavior of glauconite sand (CRS 1771) with that of MIN-U-SIL 40, a manufactured silica silt. Note the transition from concave down to concave up behavior due to fines generation. The green dots are stress levels where fines content was measured (See Figure 6.8).

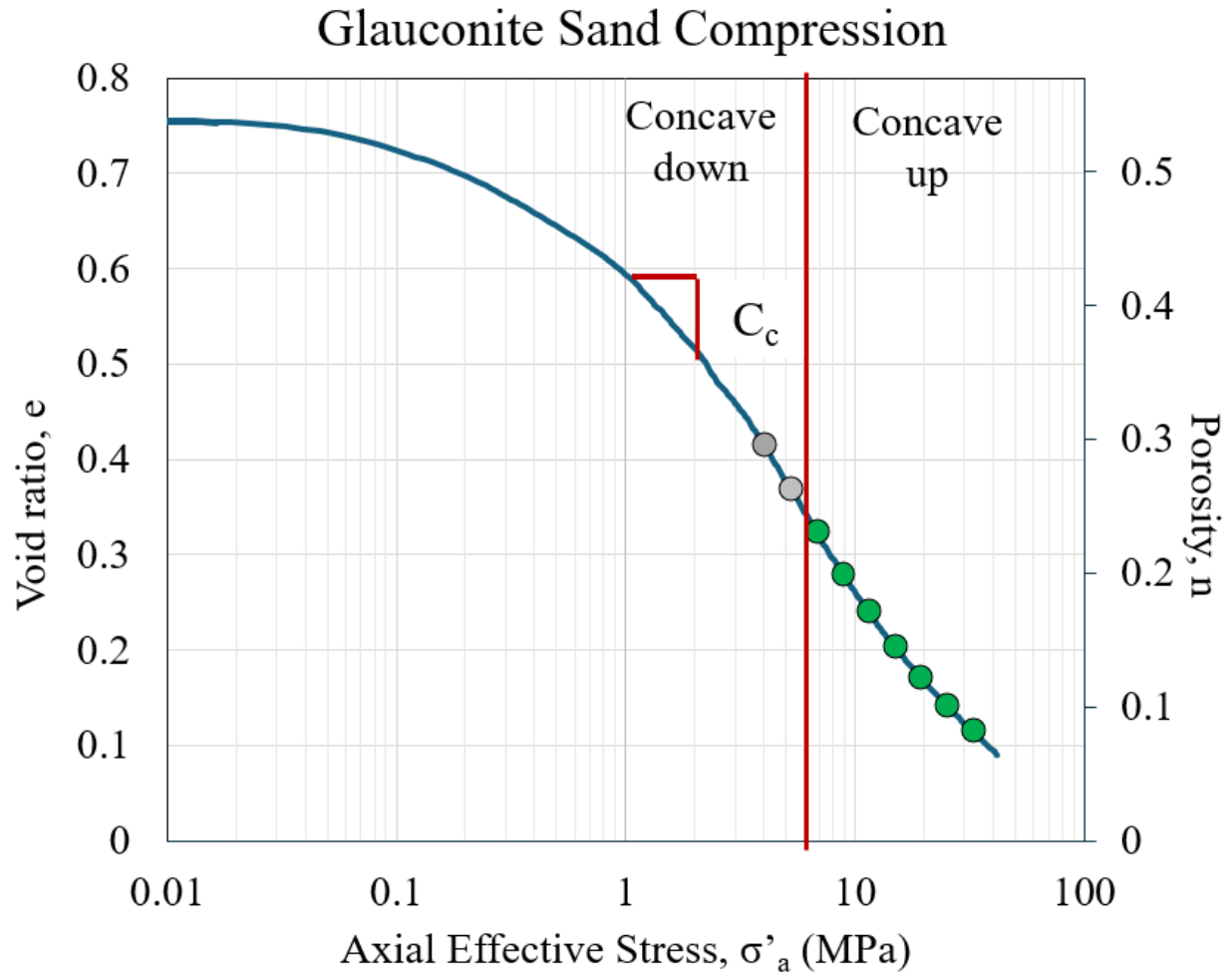


Figure 6.32: A plot showing how the compression index, the slope of the compression curve in void ratio (e) log-effective stress ($\log_{10}\sigma'_v$) space, changes with increasing stress. The gray points indicate where C_c increases while the green points indicate where C_c decreases.

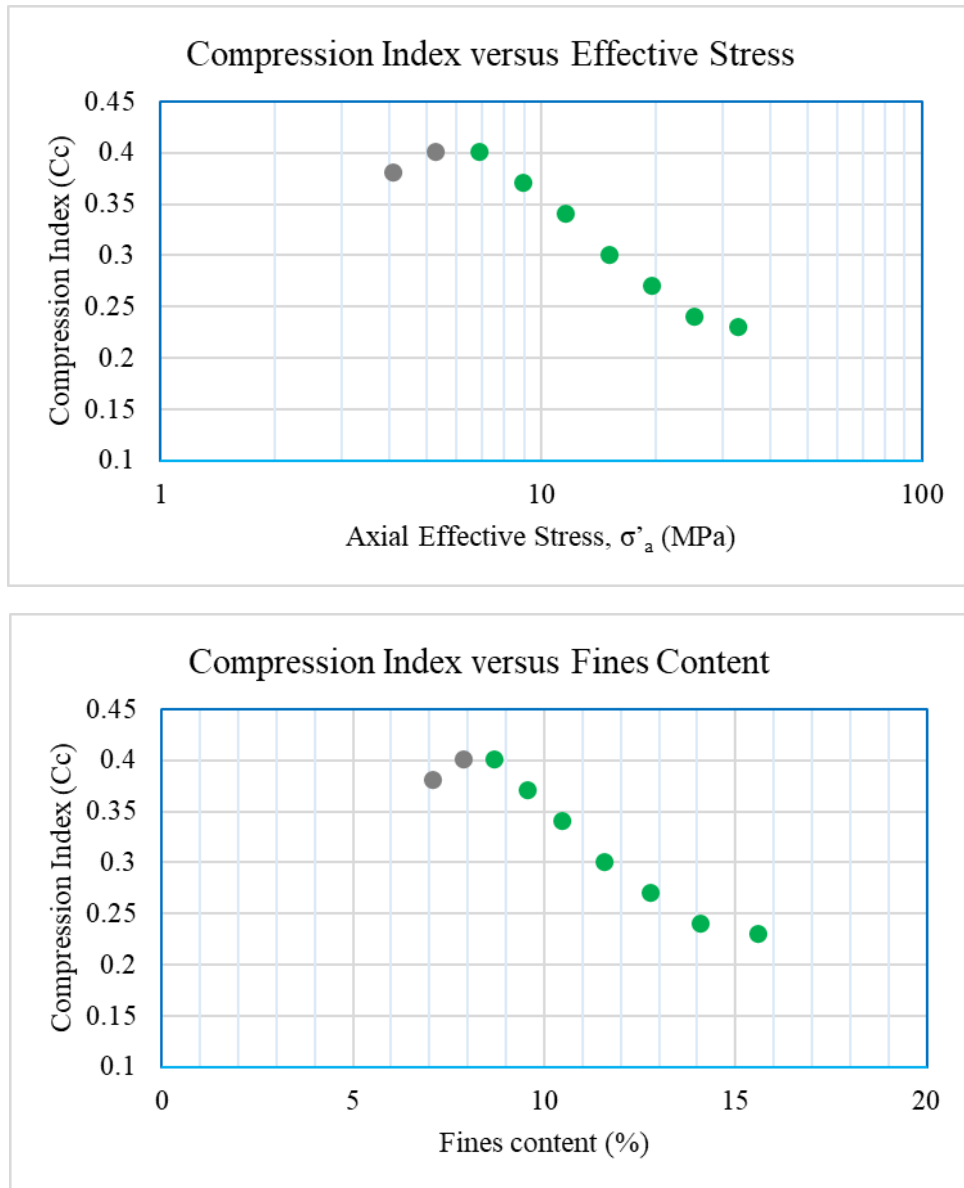


Figure 6.33: Two plots showing how the compression index (C_c) changes as a function of stress level (above) and fines content (below).

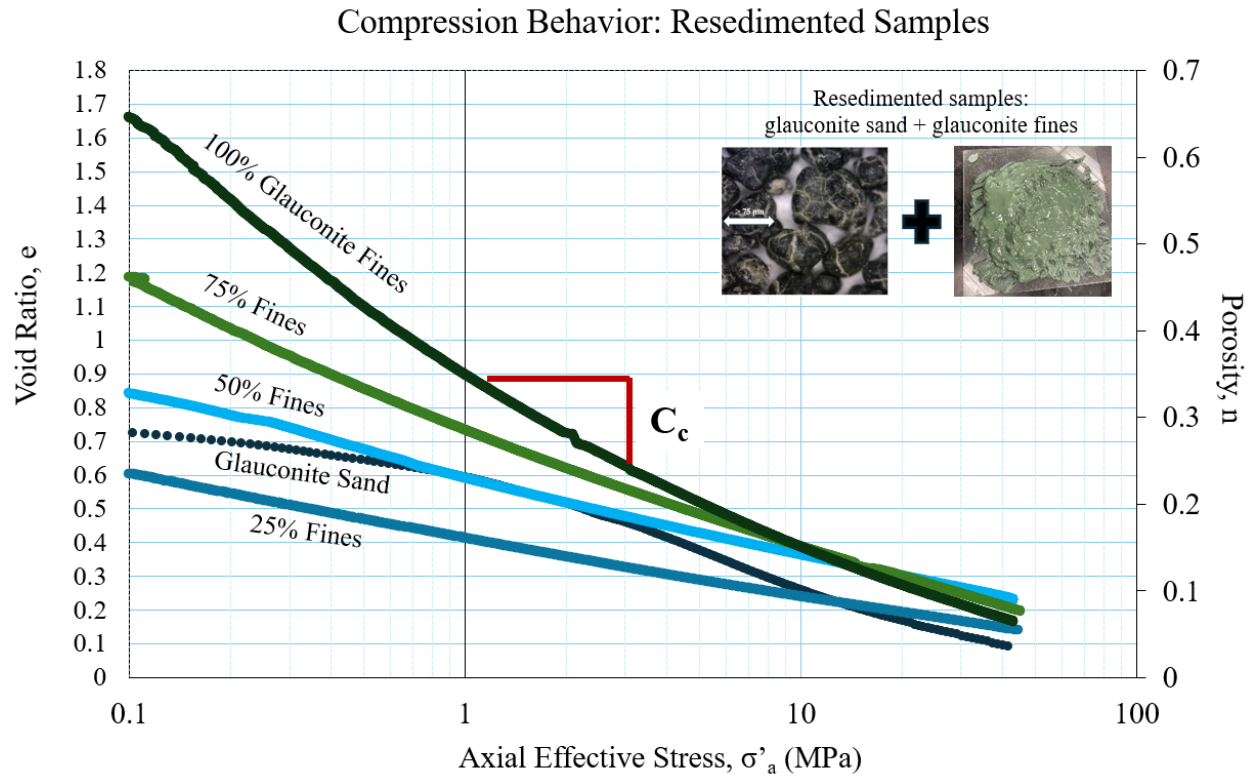


Figure 6.34: A summary plot comparing the compression curves of glauconite sand (CRS 1771) with glauconite sand with 25% fines (CRS 1808), glauconite sand with 25% fines (CRS 1816), glauconite sand with 25% fines (CRS 1809), and 100 % fine-grained glauconite (CRS 1784).

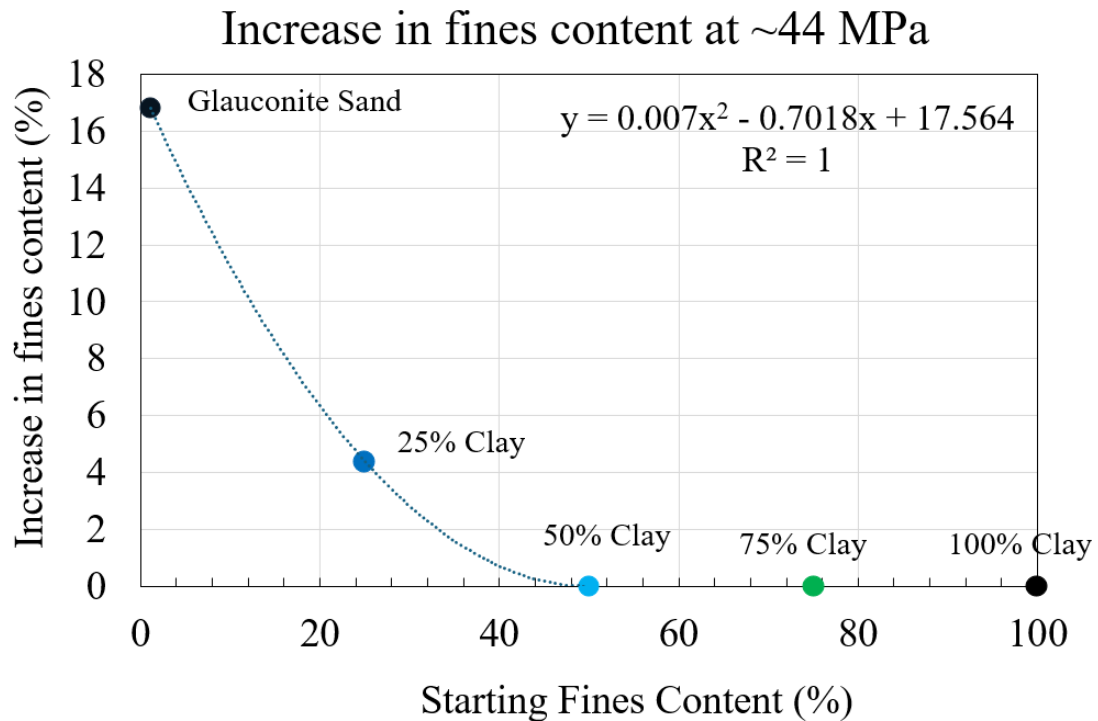


Figure 6.35: A plot indicating whether the fines content at the end of the test, after compressing to 44 MPa, was greater than the starting fines content.

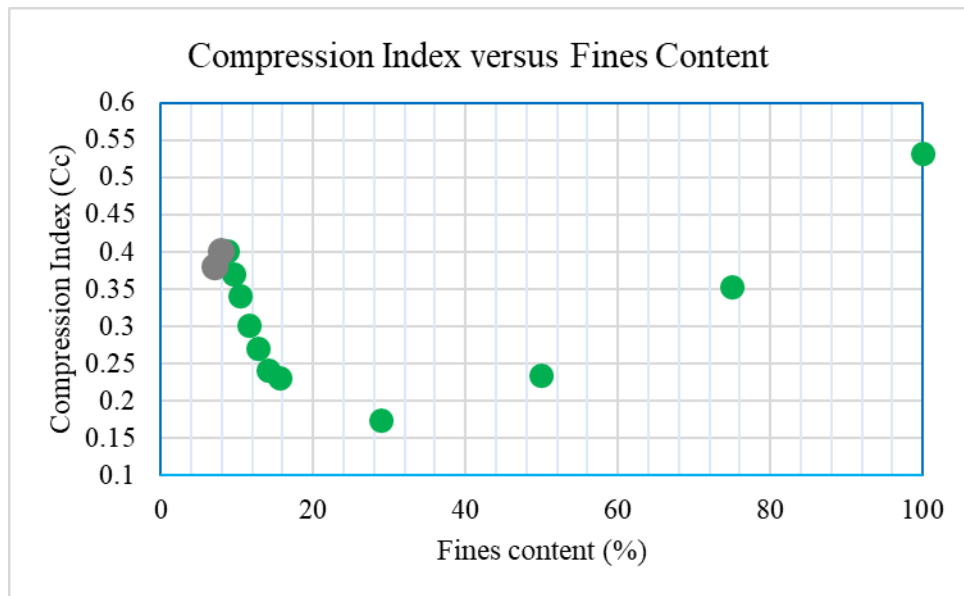


Figure 6.36: A plot showing the compression index as function of fines content. Note that the cluster of points below 20% fines content are from CRS 1771, a test on glauconite sand; the points to the right are the compression indices from CRS 1808, CRS 1816, CRS 1809, & CRS 1784 respectively.

7 Conclusions and Recommendations

7.1 Motivation for Conducted Work

This thesis presents a laboratory investigation into the mechanical behavior of Navesink glauconite sand, a highly evolved, glauconite sand sampled from Poricy Park in Middletown, New Jersey. Glauconite sand exhibits unique mechanical behavior due to its tendency to crush under stress, transitioning from granular to cohesive behavior. This transformation challenges conventional geotechnical classification and design practices in several key ways:

1. **Classification Uncertainty:** Glauconite cannot be reliably classified using CPTu-based Soil Behavior Type (SBT) charts (e.g., Robertson, 2016), as it behaves differently from typical sands or clays.
2. **Underprediction of SRD:** CPTu-based models for predicting Soil static Resistance to Driving (SRD), such as Alm & Hamre (2001), often underestimate SRD in glauconite sands, leading to excessive blow counts or even pile refusal.
3. **Uncertain Foundation Performance:** The mechanical properties of glauconite sand evolve during pile installation, increasing uncertainty in foundation performance and long-term behavior.

As discussed in Chapter 2, CPTu data are routinely used during offshore wind site investigations to classify soils and estimate SRD for monopile design. Reliable prediction of SRD is crucial to avoid pile refusal, a costly and time-consuming outcome in offshore construction. Given glauconite's abundance across the Northeast Atlantic, where concentrations as high as 85% are reported, a detailed understanding of its crushing behavior under stress and its mechanical and hydraulic behavior is essential for improving offshore monopile design.

7.2 Summary of Results

This study has systematically characterized the stress-dependent behavior of glauconite sand, with a particular focus on particle crushing, compression behavior, and permeability under increasing fines content. These findings are novel contributions to geotechnical literature.

Material was sampled from a riverbank outcrop of the Navesink Formation, wet-sieved to remove non-glauconitic fines, and found to be ~86% glauconite or glauconite/smectite by bulk mineralogy. SEM imaging revealed a friable microstructure prone to breakage. Crushing tests demonstrated that the mode and duration of loading significantly affects the type and amount of fines produced. Shearing (malt mixing) induced greater crushing, yielding predominantly clay-sized particles, whereas axial compression produced more silt-sized particles and a broader gradation.

CRS testing of glauconite sand revealed significant reductions in permeability and increases in pore pressure beginning at ~25 to 30 MPa, with secondary compression peaking around 7.4 MPa, reflecting ongoing particle crushing at constant effective stress. At higher stress levels (20 to 44

MPa), secondary compression diminished, indicating reduced particle rearrangement or densification of the crushed matrix.

Resedimented specimens with increasing fine-grained glauconite (25–100%) exhibited progressively more cohesive behavior, with increased compression indices (C_c , C_s , C_r), decreased permeability, and excess pore pressures as high as 1.72 MPa. The secondary compression index (C_{se}) was greater for glauconite sand than fine-grained resedimented specimens, contrary to the typical behavior of granular materials, which usually exhibit less secondary compression.

A CK₀UC triaxial test was conducted on glauconite sand with 75% fines. The material exhibited a contractive response during shear. An internal friction angle of 30°, relatively high for silty-clayey material, and a K_0 value of 0.4, lower than expected, was determined.

Fine-grained glauconite was compared with illitic RBBC and smectitic RGoM-EI clays. It showed higher plasticity and compressibility than both, with permeability falling between the two, consistent with its mixed glauconite/smectite composition.

Compression curve geometries in void ratio (e) log-effective stress ($\log_{10}\sigma'_v$) space illustrate the material's transitional behavior. While smectic clays exhibit concave-up and granular quartz silts concave-down compression curves, glauconite sand transitions from concave-down to concave-up around 7 MPa, corresponding to ~7-8% fines content. The compression index peaked at ~7 MPa, coinciding with the highest observed rates of secondary compression, then declined to a minimum near 33 MPa (~15.5% fines). These trends indicate that compression behavior is strongly influenced by particle crushing. At fines contents above 25%, the material increasingly resembled a fine-grained matrix, suggesting a shift from grain-supported to fines supported fabric.

7.3 Implications for Pile Design in Glauconite Sand

This study has identified several mechanisms relevant to pile drivability in glauconitic sands. Laboratory testing demonstrates that glauconite particles are susceptible to crushing under compressive loading, as evidenced by increased fines content. This particle breakage is accompanied by reduced permeability and altered compressibility, suggesting that pile driving may induce undrained conditions in the field.

A key uncertainty remains in translating laboratory results from highly evolved Navesink glauconite sand to field behavior in other glauconite deposits. While Navesink specimens show relatively limited crushing under high effective stresses, less evolved glauconite sands appear to be more crushable and may behave differently.

Current industry practice often models glauconite sand as sand at the pile toe and as clay along the shaft, a simplification supported by field observations (Westgate et al. 2024; Perikleous et al. 2023). However, inferring crushing susceptibility from in-situ data (e.g., CPTu Q_c values) remains challenging without further laboratory studies on less evolved glauconite sands.

Based on the grain size distributions presented in this study, the material appears to transition from a granular sand to a fine-grained granular soil, composed of dense, angular silt-sized particles and some clay-sized material. Improved understanding of this transition from grain-supported to fines-supported behavior, through continued laboratory testing and advanced numerical modeling, will help engineers select more appropriate design parameters and drivability models for glauconite-bearing soils.

7.4 Areas for Future Research

Further research is needed to characterize the full range of mechanical and hydraulic behavior in glauconitic sands, particularly under conditions more representative of pile installation. Possible research areas include:

1. Undrained Strength and Effective Friction Angle

Additional triaxial testing could focus on determining undrained shear strength and effective internal friction angles across a range of fines contents.

2. Shear and Compression Wave Velocities

Triaxial tests using velocity actuators and geophones could be used to quantify shear modulus degradation and damping characteristics as a function of fines content.

3. Crushing and Permeability Tests

Constant-head permeability tests at varying stress levels within the CRS cell could help quantify how permeability decreases with particle crushing.

4. Scanning Electron Microscopy (SEM)

SEM analysis of epoxied CRS specimens could clarify how crushed fines occupy pore space and offer insights into the failure mechanisms of glauconite particles.

5. Laboratory Testing of Less Evolved Glauconite

Less mineralogically mature glauconite sands appear more susceptible to crushing and warrant a separate suite of laboratory tests. Future work could explore whether younger New Jersey glauconites more closely resemble offshore glauconite deposits.

6. Micromechanical Modeling of Crushing Behavior

Discrete Element Method (DEM) simulations could be used to investigate particle-scale crushing and its influence on bulk mechanical and hydraulic behavior. Calibrating these models would require lab testing of individual particle strength under compression, tension, and shear. Accurately modeling particle roundness, rolling resistance, friction, and angularity remains a key challenge.

7. Numerical Simulation of Pile Driving

Material Point Method (MPM) models are suitable for simulating large-strain behavior such as lateral soil displacement during cone penetration or pile driving. Incorporating triaxial test data into MPM simulations could enable quantification of lateral stresses and prediction of static soil resistance (SRD) in glauconitic sands. MPM predictions could be compared to CPTu-based SRD predictions (e.g., Alm & Hamre 2001; Perikleous et al. 2023).

8. Correlation of Laboratory and CPTu Data

Offshore engineers commonly correlate CPTu parameters (e.g., Q_c) with relative density (D_r). Strength testing at various relative densities could help develop glauconite-specific correlations. Note that Navesink glauconite may have different maximum and minimum void ratios ($e_{\max}$ and $e_{\min}$) than offshore glauconite deposits.

9. Correlation of Laboratory and Field Permeability Data

Offshore engineers measure pore pressure dissipation times during pile driving. Analyzing dissipation times in conjunction with hammer blow frequency (typically ~25 blows/minute) could improve correlations between laboratory-derived hydraulic properties of “clean” Navesink glauconite and offshore glauconitic sands.

Together, these research objectives would contribute to a more comprehensive framework for evaluating pile drivability in glauconitic sands by linking particle-scale processes to field-scale behavior.

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