STRENGTH CHARACTERISTICS OF A POLYMER BONDED SAND

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Thesis of Nicole F. Garcia:

Strength Characteristics of a Polymer Bonded Sand

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DEDICATION

This work is dedicated to the ancestors.
In nature, cementation occurs naturally through geological processes such as aging and chemical reactions that create bonds between soil particles. In Civil Engineering projects, cementation can be used to improve soil properties for the construction of pavement base layers, in the protection of slopes or earthen dams, and in the improvement of ballast layers for railroad tracks. A novel cementation technology is presented in this Thesis: quartz sand and polyethylene powder are mixed. The mix is heated to soften the polymer and allowed to cool to harden the polymer. The hardened polymer bonds the sands grains together, leading to a polymer bonded sand (PBS). Unconfined compression and triaxial compression tests were performed for this Thesis. The unconfined compression strength for PBS was found to increase with increasing polymer content, in a trend similar to that observed for mineral-cemented soils. However, the unconfined strength of PBS was found to be significantly higher than those of all other artificially cemented soils. The shear strength of PBS was found to vary with confinement, with cementation controlling the strength at low confinement and friction controlling the strength at high confinement. Mohr-Coulomb strength parameters of PBS were found to be significantly higher than those of all other artificially cemented soils. Acoustic Emission monitoring proved a useful tool for revealing the pre-failure strain associated with debonding, and therefore, for enabling potential healing of the material via heating.
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CHAPTER 1

INTRODUCTION

1.1 INTRODUCTION

In nature, cementation occurs naturally through geological processes such as aging and chemical reactions that create bonds between soil particles. The determination of the strength and stiffness provided to the soil due to cementation is important for geotechnical engineering applications. Often times soils at a construction site have poor bearing capacities, at the same time, the site may be far from a borrow site. In these cases, where project costs and environmental concerns may mean the end of a project, an alternative method of adding a cementing agent to the soil can be used to improve the soil. Portland cement has been the most often used and most successful technique employed to strengthen soil for improvement to base pavement layers, slope protection of earthen dams, and to bearing capacity of soils for foundation base layers.

Many studies have been conducted to determine the strength and stiffness of sands artificially cemented with mineral agents (e.g., Schnaid, Prietto, and Consoli 2001; Asghari, Toll, and Haeri 2004; Consoli et al. 2007). Indeed, studies conducted on cemented soil are generally done using artificially cemented specimens in a laboratory, due to difficulty with in situ sampling. Much of the research on cemented soil has been conducted on Portland cemented specimens using unconfined compression testing and isotropically consolidated triaxial tests. In the triaxial tests, specimens are placed between a top and bottom platen in a vacuum sealed chamber and then loaded from the top with a piston that is placed within a fitting on the top platen. Specimens are confined at a certain stress level and then loaded at a certain deformation rate. The result is a soil that is stronger at smaller confinements and strains than would be without the addition of the cementing agent. Even once the bonds are broken and deteriorated due to the load suffered the grains, such grains are still coated with...
the cementing agent. This fabric created by the breakdown of contact bonds is distinct and different than that of the same soil in an uncemented (i.e., restructured) state.

A new approach to artificial cementation is presented in this Thesis, where experimental studies are used to show the benefits of using polymer fines as the cementing agent, thus creating polymer bonded sands (PBS). This Thesis centers specifically on the experimental characterization of the strength behavior of PBS specimens via unconfined compression tests and drained triaxial tests on specimens subjected to compression under a range of confinements.

1.2 Scope

The scope is limited to specimens consisting of 20-30 Ottawa sand cemented moist with only one polymeric cement: polyethelene powder. Unconfined tests were conducted on specimens with 1, 2.4, and 5.6% cement by mass of sand, i.e., $CC = 1, 2.4, \text{ and } 5.6\%$ (to determine the role of $CC$ on unconfined compression strength). Triaxial tests were conducted on specimens with a single cement content $CC = 1\%$ and various confinements (to determine the role of confinement). Acoustic emissions (AE) monitoring was performed on selected triaxial tests as well as selected unconfined compression tests to gather insight into particle-scale mechanisms associated with fracture creation and failure.

1.3 Organization

This thesis is organized into five chapters. Chapter 2 presents a review of relevant literature. Chapter 3 details the materials and procedures involved in carrying out this study. Chapter 4 presents the results. Chapter 5 discusses and summarizes the conclusions of this research and offers recommendations for future work.
CHAPTER 2

BACKGROUND

2.1 CEMENTED SANDS: ANNOTATED BIBLIOGRAPHY

For the past forty years researchers have explored the engineering properties and behavior of cemented sands. A summary of key articles associated with the behavior of cemented sands is included next.

2.1.1 Static Properties of Lightly Cemented Sand

Author: Saxena and Lastrico (1978)

Purpose:
The purpose of this study was to examine the static properties of natural, lightly cemented silty sand from the Vincentown Formation in New Jersey.

Method details:
The study consisted of 92 isotropically consolidated undrained compression tests conducted on natural, calcite-cemented silty sands. Tests were conducted on lightly cemented specimens with various mineral fine contents under various confining pressures (292 to 388 kPa), in order to examine the strength behavior of lightly cemented silty sands in terms of the cohesion, dilatancy, and friction angle. A deformation rate of 0.25 mm/min was used.

Findings:
Cement content significantly affected the peak strength of specimens tested, with greater cement contents producing greater strengths. Peak strengths were also influenced by confining pressure, with greater strengths associated with higher confinements. The strength behavior of calcite cemented silty sands was found to be strain dependent. Stress-strain curves were observed to change from linear to nonlinear at approximately 1 % axial strain.
and pore pressure measurements reached a peak within that 1% boundary. Beyond this point, the pore pressure decreased even as strain increased, indicating a dense material, which tends to dilate. Below 1% strain, the cement content was the strength-controlling parameter (at a certain point, shear destroys the cement bonds to the extent that friction becomes the dominating component of strength).

2.1.2 Silicate-Stabilized Sands

**Author:**
Clough, Kuck, and Kasali (1979)

**Purpose:**
The purpose of this study was to investigate the stress-strain and strength behavior of silicate stabilized soils.

**Method details:**
Drained triaxial tests and unconfined tests were conducted on both naturally cemented and silicate stabilized soils. Triaxial tests were conducted at a constant strain rate of 0.2%/min. under confining pressures of 0, 69, and 138 kPa. The unconfined tests were conducted at different strain rates to determine the effect of those variations on the strength of the silicate stabilized sands. **Silicate grout** was mixed, as a percentage by volume (30, 50, and 70%) with two different commercial sands (Monterey 16 and 30), and a natural San Francisco bay sand. The grout mix consisted of sodium silicate, formamide, and water.

**Findings:**
The stiffness and strength of silicate stabilized sands were found to be significantly affected by (1) the grout content and (2) confining pressure. Increased silicate content resulted in increased stiffness and strength and decreased axial strain at peak. In addition, the failure was increasingly brittle with increasing silica content. Increased confinement translated to increased stiffness and strength of the cemented sands. Specimens tested under higher confining pressures had higher peak strengths, and peaked at larger strains than did specimens confined at lower pressures (similar results are presented in this Thesis, for polymer bonded sands). Volumetric expansion was observed post peak in all tested specimens. As the cemented specimens were subjected to large strains (> 10%) the stress-strain curves were observed to approach the uncemented sand’s stress-strain curves. The
initial density of cemented specimens was found to have a negligible effect on the strength and stiffness.

Results from the unconfined tests also demonstrated increasing stiffness and strength with increasing silica content. The effect of varying the strain rate showed that a decreased strain rate resulted in decreased strength by 50%, (for strain rates 0.2%/min vs. 0.0008%/min).

2.1.3 Dynamic Moduli and Damping Ratios for Cemented Sands at Low Strains

Author:

Purpose:
The purpose of this study was to ascertain the effects of cementation on the dynamic properties of sands at low strains. Results of resonant column (RC) tests were discussed. A new proposed relationship for maximum dynamic shear modulus was compared with reported relationships. Correlations between dynamic moduli and static strength from drained triaxial tests were developed for an effective confining pressure of 49 kPa.

Method details:
Monterey No. 0 sand and Portland type 1 cement was used to make the specimens tested. All specimens were made using the Ladd method of undercompaction. RC tests were conducted with a modified Drnevich-type Long-Tor resonant column apparatus.

Findings:
Cementation was observed to increase the dynamic moduli, with higher cement contents rendering greater dynamic moduli. Cementation also increased the damping ratio when cement contents were small (0 to 5 %), whereas higher cement contents lead to a decrease in the minimum damping ratio. The authors postulate that the increase in cement content did not decrease the damping ratio due to the cement creating “less clean” contacts between the grains, resulting in the shear wave expending more energy to travel through the cemented specimen. That is at higher cement content, the shear wave has to expend less energy to travel through the specimen, resulting in a lower damping ratio.
2.1.4 Effects of Cementation on Stress-Strain and Strength Characteristics of Sands

Author:
Reddy and Saxena (1993)

Purpose:
The purpose of this study was to determine the effect of cementation on the behavior of sand under various three-dimensional stress conditions.

Method Details:
This study consisted of a series of drained true triaxial device tests conducted on saturated cemented and uncemented sands, carried out to determine the effect of cementation on the behavior of sand under various stress paths. The stress paths consisted of conventional triaxial compression and hydrostatic isotropic compression, and tests along different octahedral planes. Tests were stress-controlled using a shear rate of 15 to 20 kPa for every 5 minutes, i.e., 3 to 4 kPa/min. Conventional triaxial tests were conducted under three different confining pressures (35, 69, 138 kPa). Specimens were prepared with Monterey No. 0 sand and 2% Portland type I cement based on the dry mass of the sand. Failure in this study was defined as a sudden increase in the major principal strain rate and using this definition, stress conditions at failure for all tests were determined and summarized.

Findings:
Cemented sands were found to exhibit cohesion under all stress paths investigated. In triaxial compression, the internal angle of friction was found to be similar for both cemented and uncemented sand.

The stress-strain response of the cemented sands had a high initial stiffness, regardless of stress path. In contrast, the uncemented sands had similar stress responses regardless of stress path, until approximately 45% of the strength value was reached, whereupon different stress-strain responses were observed for different stress paths. Furthermore, the dilation at failure was always higher for the cemented sands; however the rate of volume change was similar for both the cemented and uncemented sands. The behavior of both the cemented and uncemented sands was found to depend on the stress state, and when subjected to the same stress state, different stress-path dependent behaviors were observed.
2.1.5 The Mechanics of Cemented Carbonate Sands

Author:
Coop and Atkinson (1993)

Purpose:
The purpose of this study was to investigate and compare the behavior of cemented and uncemented carbonate sand in triaxial tests.

Method details:
Triaxial tests were carried out on carbonate sand mixed with gypsum plaster (3.33 parts sand to 1 part gypsum plaster by weight) and cemented by adding distilled water. Tests with confining pressures up to 9 MPa were conducted.

Findings:
Under low confining pressures, cemented specimens displayed peaks; at higher confining pressures (i.e., \( \sigma_c > 300 \) kPa) the peaks disappeared. As confining stresses increased, cemented specimens transitioned from a shear band forming at failure to specimens failing in a more ductile manner, exemplified by barreling of specimens. A peak was not observed for uncemented specimens: failure was ductile.

The behavior of the cemented specimens at failure was brittle and specimens with higher cement contents displayed larger, sharper peaks, and increased strain-softening with increasing axial strain.

The cement matrix bears the confining stress initially. Once a certain strain is reached, cement bonds begin to break. This initial breakdown of cement bonds, termed the yielding point, was observable in test results. Under continued loading, degradation and breakdown of the cement matrix occurs and the load is transferred to individual particles. A transition from brittle to ductile failure was not easily defined due to the influence of the cement matrix post initial yield of cement bonds. In this thesis, such transition is well detected with microvibration measurements taken during loading, (see Chapters 3 and 4).

2.1.6 Yielding and Pre-Failure of Structured Sands

Author:
Cuccovillo and Coop (1997)
Purpose:

The purpose of this study was to evaluate the behavior of two different types of naturally structured sands from small strains to failure, paying special attention to the influence of structure on the shear stiffness and yielding.

Method details:

Two soils were studied in drained and undrained triaxial compression under a range of confining pressures in order to compare their behavior. The first, a calcarenite soil, is a weak grained soil, cemented naturally with calcium (a relatively strong cement). The second soil, a silica sandstone, is a strong grained soil cemented with iron oxide (a relatively weak cement).

In addition, cyclic undrained tests consisting of a series of loading-unloading shearing probes were performed on specimens to analyze the pre-yield stress-strain (i.e., pre-peak) response and the effect of bond degradation on the stiffness and yielding. Yield was defined as the initiation of bond degradation or destruction of cement bonds but prior to peak or failure of specimen. Specimens were loaded-unloaded, starting at the same mean effective stress $p'_{i} = 1760$ kPa and to successively higher stresses. An axial strain rate of approximately 0.003%/min was used to conduct the shearing probes.

Tests were conducted on both cemented and reconstituted specimens. Reconstituted specimens were made by rubbing the particles against one another to break the bonds (in the case of the calcarenite) and using a rubber pestle to break the bonds (in the case of the silica sandstone). In this thesis, reconstituted sand specimens were prepared by breaking up the bonded sand grains by rubbing them between fingers.

Findings:

Triaxial tests conducted at low confining pressures demonstrated a discernable yield point, whereas at high confining pressures ($\sigma_{c} > 60$ MPa) a yield point was not discernable.

Linear stress-strain behavior was observed in tests conducted at low values of $p'_{i}$ (below the yield stress), and increasing $p'_{i}$ did not change the linear behavior. Once shear began and the yield stress was reached the behavior was observed to become nonlinear.

Cement type had a significant effect on the shear stiffness. The cemented silica sandstone was stiffer when compared with the reconstituted silica sandstone and a yield point was observable for cemented specimens, while a yield point was not observable for the
reconstituted specimens. The calcarenite cemented soil was not stiffer than the same soil restructured.

**2.1.7 Low Strain Shear Moduli of Cemented Sands**

**Author:**
Baig, Picornell, and Nazarian (1997)

**Purpose:**
This study focused on measuring the shear modulus of cemented soils using piezo ceramic bender elements (BE) to circumvent RC-related coupling problems, and to compare the measured values to those reported in previous studies.

**Method details:**
Specimens were made by mixing Ottawa sand and **Portland type I-II cement** for relative densities 0 and 50%, and cement contents between 1 to 5% by mass. Specimens were mounted on a modified triaxial cell with end platens containing bender elements. Triaxial tests were conducted with confinements ranging from 70 to 700 kPa. Bender element measurements were conducted during loading and unloading. Square wave pulses were used to excite the transmitting bender element. The signal was detected by the receiver bender element and recorded by a digital analyzer. A sampling rate of 256 kHz was used to record the signal and an input voltage of 8 to 10 volts was used in all tests. Travel times were then used to determine the shear wave velocity $V_s$ and thus, the small-strain shear modulus:

$$G_{\text{max}} = \rho V_s^2$$

(2.1)

where $\rho =$ mass density. Piezo ceramic bender element measurements involved the use of a triaxial load frame and a push rod to apply a small seating load to keep the specimen in contact with the end caps. A range of confining pressures (70 to 700 kPa) were then applied. Tests with the bender element were conducted both during loading and unloading of the specimen. RC tests were also performed on certain specimens recovered from the triaxial chamber (after being loaded isotropically) and tested with the piezo ceramic bender elements. Results of the RC tests were compared to the results obtained with the piezo elements for the unloading portion of the triaxial tests.
Findings:

Test results obtained from bender element tests were close to results obtained using the RC device. The higher shear modulus values obtained with the piezo elements were attributed to the piezo elements inducing lower strain levels than the resonant column. Both type of test results indicate that confining pressure has a negligible effect on the $G_{\text{max}}$ of cemented sands (for the confining pressures imposed). Bender element tests demonstrated that the degree of cementation (i.e., the cement content) is the most important parameter in determining the $G_{\text{max}}$ of the cemented sand specimens.

2.1.8 Influence of Fiber and Cement Addition on Behavior of Sandy Soil

Author:

Purpose:
To evaluate the influence of fiber inclusion and cementation on the mechanical behavior of cemented soils.

Method details:
A weathered sandstone soil was cemented with Type IV Portland Pozzolanic (0 and 1%) and fiberglass fiber (0 and 3%) then subjected to drained compression triaxial tests at confining pressures of 20, 60, and 100 kPa. Specimens were tested were completely saturated.

Findings:
Peak strength, stiffness, and brittleness are increased by the addition of cement to the sandy soil. The effects of the fiber inclusion on the sandy soil is less pronounced, however there was a noticeable increase in peak and residual strength, as well as, ductility. When fibers were included in the cementation of the sandy soil the increase in residual strength was more effective. In addition, the inclusion of the fibers within the cemented specimens reduced the brittleness and the post-peak behavior was more ductile. An absolute measure of brittle versus ductile behavior is provided by the brittleness index ($I_B$) and is defined by the expression
\[ I_B = \frac{q_f}{q_u} - 1 \] (2.2)

2.1.9 Drained Probing Triaxial Tests on a Weakly Bonded Artificial Soil

Author:
Malandraki and Toll (2000)

Purpose:
This study was conducted to analyze the effect of changes in stress path direction during shear of an artificially bonded soil and restructured soil. Conventional and drained probing triaxial tests were conducted to analyze the effect of stress path direction on the soils behavior with respect to the strength, stiffness, and yield conditions. Note: in a probing test the load tracks along a specific, nonstandard stress path to hit the yield surface at different points.

Method details:
Drained probing triaxial tests were conducted on specimens cemented with fired Kaolin. Half of the tests followed a constant mean effective stress (p’) path, and the other half were conducted at a constant rate of change of deviator stress of 50 kPa. Conventional triaxial tests were conducted on bonded and restructured soil specimens. Bonded specimens were subjected to drained triaxial testing under confining pressures of 5 to 550 kPa and a shearing a rate of 1.5% per hour. In addition, conventional undrained and drained tests were also conducted on restructured sand-Kaolin specimens under confining pressures of 5 to 650 kPa. All specimens were prepared with void ratio = 0.6 by mixing sand and Kaolin (13%), which was then fired to 500\(^\circ\) (to produce a ceramic). The results of drained probing triaxial tests were then compared with the results of conventional triaxial tests.

Findings:
Bonded specimens reached higher peak strengths and demonstrated a higher stiffness then restructured specimens. Bonded specimens displayed a curved failure envelope at low mean effective stresses. The curve converged with the restructured failure envelope at high mean effective stresses. The convergence points to the friction between grains dominating strength at high mean effective stresses, whereas the bonding contributes to the higher strength and stiffness of the bonded specimens at low mean effective stresses.
2.1.10 Characterization of Cemented Sand in Triaxial Compression

Author:
Schnaid, Prietto, and Consoli (2001)

Purpose:
This study analyzed the mechanical response of an artificially cemented sandy soil and compared that response with the response of the same sandy soil uncemented, to determine if the influence of the cement could be expressed in reference to the measured behavior of the uncemented soil.

Method details:
Drained triaxial tests, unconfined compression tests, and photomicrograph analysis were performed with the aid of scanning electron microscopy (SEM). Results were then evaluated to determine the influence of cement content and the influence of the confining pressure, defined as the initial mean effective stress $p'_i$. Triaxial tests were conducted at confining stresses = 20, 60, and 100 kPa and strain rate 1.14%/min. Specimens were prepared by mixing a weathered sandstone soil with Portland cement, at contents of 1, 3, and 5 % (by dry mass of soil). SEM analysis was conducted on exhumed triaxial tested specimens.

Findings:
Triaxial test results showed a linear relationship between cement content and the cohesion intercept, with increased cement content resulting in an increased cohesion intercept. An increase in confining pressure produced a reduction in the sharpness of the peaks of the stress-strain curves of cemented specimens, (similar results are presented for the tests conducted in this thesis). Furthermore, the cemented soil was observed to fail in a brittle manner, with a well-defined shear band. Increased cement content also reduced the axial strain at failure (as the cement content was reduced, failure was increasingly ductile and the axial strain at failure increased). Cemented specimens initially contracted and then underwent dilation with the maximum volumetric strain rate occurring post peak. Once the dilation rate reached a maximum, the rate decreased until the dilation rate is nearly zero. In general, the friction angle of cemented sands was found to be higher than that of the uncemented sands.
Triaxial test results also showed different strength envelopes in a p’-q space, such that the uncemented and 1% cemented specimens fell along one envelope and the 3 and 5% specimens fell along another.

In addition, the unconfined compressive strength was observed to be a direct measure of the degree of cementation in triaxial compression. The authors used the cohesive-frictional nature of the cemented soil to express the shear strength of the soil as a function of the unconfined compressive strength and the uncemented friction angle where

$$q_f = \left( \frac{2\sin \phi}{1 - \sin \phi \ p_i'} \right) + q_u$$

(2.3)

A SEM analysis on images of exhumed cemented specimens from the triaxial determined that the differing cement contents produced both porous (uncemented and 1%), and dense fabrics (3 and 5%). The authors state that the differences in results of cemented specimens of varying cement contents indicate a need to analyze the effect of cement content on the critical state behavior of cemented soils.

2.1.11 Effect of Cementation on the Shear Strength of Tehran Gravelly Sand Using Triaxial Tests

Author:

Purpose:
This study was conducted to examine the effect of a lime cementing agent on the shear strength of Tehran gravelly sand soils.

Method Details:
Drained and undrained triaxial tests were conducted on cemented, uncemented, and restructured gravelly sand specimens under a range of confining pressures. Cemented specimens were prepared using hydrated lime as the cementing agent with cement contents of 1.5, 3, and 4.5%. The drained triaxial compression tests were carried out at a controlled deformation rate of 0.1 mm/hr and the undrained tests were conducted at a rate of 0.2 mm/hr.

Findings:
Cemented specimens were much stronger than uncemented specimens. Under low confining pressures, cemented specimens showed a brittle failure mode and were observed to have a shear zone. Under higher confining pressures, the failure was increasingly ductile. The
brittleness index proposed by Consoli et al. (1998), was used to describe the level of “brittleness”:

\[ I_B = \frac{(\sigma'_1 - \sigma'_3)_P}{(\sigma'_1 - \sigma'_3)_CS} - 1 \]  

(2.4)

where \((\sigma'_1 - \sigma'_3)_P\) is the deviatoric stress at peak and \((\sigma'_1 - \sigma'_3)_CS\) is deviatoric stress at critical (i.e., ultimate) state. A decreasing index is demonstrative of an increasingly ductile failure behavior (i.e., \(I_B = 0\) means full ductility, as no post peak softening is observed). Peak friction angles were slightly higher for the cemented specimens than for the uncemented specimens and the cohesion intercept was significantly higher for the former. The cement was found to have a greater effect at low confining pressures with that influence waning as confining pressure increased.

2.1.12 The Effect of Gypsum Cementation on the Mechanical Behavior of Gravely Sands

Author: Haeri, Hamidi, and Tabatabaee (2005)

Purpose: The purpose of this study was to evaluate the mechanical behavior of gravelly sand cemented with gypsum and compare such behavior to that of other cemented sand.

Method details: Gravelly sands were cemented with gypsum (1.5, 3, 4.5, and 6%) and then subjected to undrained and drained compression triaxial tests at confining pressures up to 500 kPa. Additionally, Brazilian tensile tests and unconfined compression tests were conducted on the cemented soil. Several parameters were used to describe the behavior of the cemented soil.

Findings: Cemented specimens showed brittle failure and were dilative after an initial contraction. Shear bands were observed in both drained and undrained test specimens. Furthermore, a decrease in cement content resulted in a decrease in the actual inclination of the shear band. Brittle behavior was exacerbated under low confining pressures as well as at higher cement contents. Barreling modes of failure were observed at confining pressures greater than 100 kPa. Results of cemented specimens showed a peak followed by a drop in
stress before leveling off to a final or ‘ultimate’ asymptotic. Lower cement contents and higher confining pressures were associated with a lower degree of strain softening. In addition, softening was greater in drained tests, indicating more brittle behavior (verifying that volumetric straining of the specimen results in earlier breakdown of the cemented specimens), than that of undrained tests. The brittleness index, an indicator of cemented soil brittleness, proposed by Consoli et al. (1998), was used, as defined in equation (2.2).

2.1.13 Stiffness and Deformation Characteristics of a Cemented Gravely Sand

Author:

Hamidi and Haeri (2008)

Purpose:

The purpose of this study was to investigate the stiffness and deformation characteristics of gypsum cemented gravely sand. The effect of the gypsum cement was compared with that of other cement types in the literature as a parameter affecting stiffness at bond yield.

Method details:

The gravely sand was cemented with gypsum contents of 1.5, 3, 4.5, and 6 %. Drained and undrained triaxial tests were conducted on dry and saturated specimens under confining pressures of 25, 100, 300, and 500 kPa. Triaxial tests were conducted at a deformation of 0.65 mm/min for drained and 0.2 mm/min for undrained.

Findings:

The behavior of cemented soil was found to be more brittle under drained conditions (versus undrained), with bond breakage occurring due to dilation of the specimen under shear. Clearly, for undrained tests, volume change is restricted. This inhibits bonds from breaking easily and results in a larger strain at peak. For drained triaxial tests, increased confining stress decreased the amount of dilation, (similar results are presented for the tests conducted in this thesis; Chapter 4).

The tangent stiffness of the cemented soil was found to be greater than that for the uncemented soil. This occurred for higher confining stresses but the difference was not as large as for low confining pressures. However, at approximately 300 kPa of confinement, such differences disappear. Indeed, under the higher confining pressures, the bonds break due
to confinement and shear, whereas at lower confining pressures, the bonds break predominantly due to shear.

Soil stiffness was found to be dependent on cement type. The rate of increase in tangent stiffness at bond yield, (defined as a sharp decrease in soil stiffness), was found to change with cement content for different cementing agents (gypsum vs. Portland cement vs. Lime). The paper’s authors advise that cement content and type should be considered when conducting settlement studies for structures built on cemented soil.

2.1.14 Cemented Soils: Small Strain Stiffness

Author: Rinaldi and Santamarina (2008)

Purpose: This paper concerns the role of cementing agents on the small-strain dynamic properties of soils.

Method details: The authors reviewed findings of tests conducted on cemented soils with a focus on small-strain parameters. They considered cases of cementation before loading and cementation after loading.

Findings: Cemented soils have different characteristics than do remolded soils, such that cement-dependent behavior is observed at low confinement and stress-dependent behavior is observed at high confinement. A decreased volume contraction during isotropic loading occurs for cemented soils and an increased tendency for dilation occurs under deviatoric loading. In addition, the high small-strain stiffness of cemented soils was observed to be independent of confinement; yet, once debonding occurred the stiffness was lost and irrecoverable.

2.1.15 An Experimental Investigation of the Behavior of Artificially Cemented Soil Cured under Stress

Author: Dalla Rosa, Consoli, and Baudet (2008)
Purpose:

Drained triaxial compression tests were conducted to study the stress-dilatancy, yield surface, and state boundary surfaces of a cemented soil cured under stress.

Method details:

Approximately 30 drained triaxial compression tests were conducted on cemented specimens cured under stress. Tests were also performed on uncemented specimens. To prepare specimens for testing, Botucatu residual sandstone sand was mixed with 0 to 3% rapid-hardening Portland cement by dry mass. Specimens were cured under confining pressures of 50, 250, and 500 kPa for 48 hours to simulate burial at depth. Triaxial tests were then conducted at confining pressures of 50, 250, or 500 kPa.

Findings:

Cemented specimens exhibited: (1) higher peak strengths and (2) a greater degree of dilatancy when cured under higher confining stresses. For cemented specimens sheared at high confining pressures, the difference in peak strength between the cemented and uncemented specimens was smaller than that for specimens sheared at lower confining pressures. This confirms (yet again) that cementation has a greater effect on strength behavior at low confining pressures. Stress-dilatancy effects were modeled using a stress-ratio dilatancy graph; dilatancy being based on total strains. The cement acted to hinder volumetric deformation up to peak and then, upon reaching peak, the specimens dilated with the dilation ratio remaining constant. This dilation indicates that deformation as a result of bond breakage has taken place.

The yield surfaces of the cemented sands investigated in this study were affected by curing stress. Specimens cured at 50 kPa (lowest confinement used) had the largest normalized yield and state boundary surfaces.

2.1.16 Characterization of Cemented Sand by Experimental and Numerical Investigations

Author:

Purpose:
The purpose of this study was to identify the micromechanics as to why cementation influences the strength parameters of cemented sands at peak and critical states, and governs stress dilatancy.

Method details:
Drained triaxial compression tests were conducted on saturated specimens under a confining pressure of 80 kPa. Specimens tested were prepared in a loose state Dr = 11% to ensure that the observed dilatancy (if any) corresponded to the influence of the cement. Ottawa 20-30 sand was mixed with Portland cement. The cement content was defined as the dry mass of cement divided by the mass of the dry sand. Cement contents of 1, 2, and 3% were used.

Findings:
Cementation was found to increase the friction angle and cohesion at the critical state ($\phi'_c$). Increased cement content resulted in an increased $\phi'_c$. The authors postulate that such increase is due to (1) the cement particles still being connected to the sand particles, which increases the particle surface roughness, and (2) various sized sand clusters still being present at this ultimate (critical) state provide support (through a stronger force-chain network), and thus a higher strength. Peak state strength was found to be governed by competing and related processes: the bond breakages lead to a decrease in overall strength but also create subsequent volumetric dilation, which increases the strength (the dilation is caused by bonded clusters moving relative to one another). The post peak softening emerges because the strength loss due to the further breaking of bonds is greater than the strength increase due to dilation.

Peak strength and the maximum dilatancy rate did not occur at the same axial strain: a “delay” of maximum dilatancy was observed in all cemented specimens, and explained as follows. Initially, debonding is spatially random and dilation is hindered by the still widely-bonded specimen. Prior to peak, cumulative debonding translates to decreased strength; yet, the same debonding enables dilation and thus increased resistance. The latter prevails. At peak, the strength loss caused by debonding balances the resistance owing to dilation, hence the peak. Post-peak, the resistance owing to dilatancy cannot compensate the strength loss caused by the extent of debonding that has taken place within the shear band, hence the strain
softening. Thus, the peak strength is controlled by the structure, which is influenced by interparticle cementation rather than density.

DEM results indicated that bond breakage is initially mild and random. Once peak strength is surpassed, many bond breakage events take place, most of which occur along a shear band within the specimen. At this point, the loss in strength due to the many debonding events surpasses the gain in strength due to the contribution of dilation; this causes a lowering of the strength from the peak value, even at the maximum dilatancy rate (hence the “delay”).

### 2.1.17 Fundamental Parameters for the Stiffness and Strength Control of Artificially Cemented Sand

**Author:**

Consoli et al. (2009)

**Purpose:**

The purpose of this study was to quantify the influence of the amount of cement and porosity on the effective strength parameters and initial shear modulus.

**Method Details:**

A number of unconfined compression test and triaxial tests were carried out. Triaxial compression tests with measurements of initial stiffness were also completed on specimens under distinct effective confining pressures (i.e., 20, 200, and 400). A uniform nonplastic fine sand (SP) was mixed with Portland cement (Type III). The cement content was based on the dry mass of soil and target moisture content.

**Findings:**

The voids/cement ratio is introduced (and defined as the volume of voids divided by the volume of cement), and the unconfined compression strength can be represented as a function of the voids/cement ratio ($V_v/V_{ce}$). Triaxial test stress-strain results were also evaluated in terms of the $V_v/V_{ce}$ ratio. Increased $V_v/V_{ce}$ ratio resulted in a reduction in the brittleness of cemented soil. A lower $V_v/V_{ce}$ ratio changes from dramatically brittle to nearly ductile at higher $V_v/V_{ce}$. The cohesion intercept ($c'$) and the angle of shearing resistance ($\phi'$) were reduced with increased $V_v/V_{ce}$ ratio.
2.1.18 Shear Strength of Artificially Cemented Sands

Author:
Lee, Choi, and Lee (2009)

Purpose:
The purpose of this study was to investigate the effect of cementation on the shear behavior of artificially cemented sands with aims toward development of an analytical model by which to predict the shear strength of the cemented sands using the unconfined compressive strength.

Method Details:
Cemented and uncemented specimens were tested in drained triaxial conditions under a range of confining pressures, from 50 to 400 kPa. Deviatoric loading was imposed with a strain rate of 0.1%/min. Specimens were prepared using two types of sand (K7 and Busan), and gypsum cement with 5% and 10% cement content.

Findings:
Increases in cement content resulted in the cemented sand response becoming increasingly brittle. Cement content influenced the dilation (by way of delaying), of specimens under shear due to the strength of the cement bonds. This “delaying” of dilation occurs due to the competing contributors to strength: dilation and bonding resistance, as explained by Wang and Leung (2008). The test results showed specimens with a cement content of 5% had relatively larger changes in volume when compared to the volume change of specimens with 10% cement content. However, specimens with 10% cement content were observed to undergo a sudden expansion near peak, whereas the 5% cemented specimens did not show a sudden expansion near peak: more cement results in more dilation. Increases in confining pressure reduced the softening tendency, generally resulting in a more ductile response.

2.1.19 Characterization of Weakly Cemented Sands Using Nonlinear Failure Envelopes

Author:
Sharma et al. (2011)

Purpose:
The purpose of this study was to characterize the shear strength of weakly cemented sands using nonlinear failure envelopes.

**Method details:**

Triaxial compression, unconfined compression, and indirect tension tests were conducted on weakly cemented specimens. Specimens were prepared by mixing silty sand and ordinary **Portland cement** in contents of 0, 1, and 2.5%. Specimens were prepared to densities 1.8, 2.1, and 2.25 gm/cc.

Triaxial compression test were conducted on saturated specimens under a range of confining pressures up to 400 kPa, at an axial strain rate of 0.005 %/min. Brazilian tension tests were carried out at a strain rate of 0.2 %/min and unconfined compression (UC) tests were carried out at a strain rate of 0.5 %/min. All tests were conducted under drained conditions.

**Findings:**

A nonlinear strength function was introduced to characterize the shear strength of a weakly cemented sand, defined as:

\[
\tau = P_a A \left( \frac{\sigma'}{P_a} + T \right)^n
\]  

In equation (2.5), \( \tau \) is the shear strength for a given effective stress \( \sigma' \), \( P_a = \) atmospheric pressure, and the remaining parameters (A, n, and T) are nondimesional, nonlinear strength parameters. The cohesion intercept obtained from the nonlinear function was termed the “real cohesion” \( c_r \). Comparisons between the nonlinear strength function and the linear Mohr-Coulomb function resulted in different cohesion intercepts.

The nonlinear failure envelope was used to fit the experimental data and was also used to fit data from two deep wells, obtained from the oil industry. The nonlinear failure envelope fit the data reasonably well and “real cohesion” intercept values were observed to increase with increasing cement content and density. The cohesion intercept obtained using the linear Mohr-Coulomb theory was overestimated due to the nonlinear behavior of the cemented sands at lower confining pressures. The nonlinear function rendered cohesion values that were more accurate than the cohesion values obtained with the Mohr-Coulomb function.
2.1.20 Influence of Cement-Voids Ratio on Stress-Dilatancy Behavior of Artificially Cemented Sand

Author:
Consoli et al. (2012)

Purpose:
This study aimed to quantify the influence and interaction of both the amount of cement and the porosity on the stress-dilatancy behavior of artificially cemented sand by way of a cement-voids ratio parameter defined as the ratio between the volume of cement and the volume of voids of a mixture \((V_{ce}/V_v)\). The inverse of this parameter is used to quantify cement content in this thesis; Chapter 3.

Method Details:
A uniform fine sand and was mixed with Portland Type III cement and tap water to make the samples. Eighteen isotropically consolidated, drained triaxial compression tests were conducted on cemented sand specimens under a range of confining stresses up to 400 kPa, void ratios (0.69 to 0.82), and cement contents (3.0 to 10.3 % by mass of dry soil). Specimens were subjected to a slow deformation rate of 0.0173 mm/min to ensure drained conditions. Pore pressures and confining stresses were monitored with pressure transducers and the deviatoric load was measured with a load cell with capacity 10 kN and a resolution of 0.005 kN. Axial strains were measured with two independent systems: an internal system using two local deformation transducer (LDT) sensors, (with a resolution smaller than 1 micrometer), and an externally mounted linear variable differential transducer (LVDT), with a resolution smaller than 10 micrometer. The latter was used to measure the axial deformation.

Unconfined compression (UC) tests on “saturated” (submerged in water for 24 hours; then dried quickly just prior to testing with an absorbent cloth), specimens were conducted using an automatic loading machine with a maximum capacity of 50 kN, and proving rings with capacities of 10 kN and 50 kN, with resolutions of 0.005 kN and 0.023 kN, respectively. A deformation rate of 0.14 mm/min was used for UC tests.

Findings:
Test results showed that the volume of cement as well as the volume of voids significantly affected the compressive strength of cemented sands, such that (1) small
increases in cement content resulted in a significant strength gain and (2) decreasing the volume of voids increased the compressive strength of the cemented sand. A dominant shear band was observed post peak strength in all triaxial test specimens.

Tests conducted on cemented specimens with different volume of cement and porosity values but the same $V_{ce}/V_v$ showed similar peak strengths, demonstrating that $V_{ce}/V_v$ controls the stress-strain behavior up to peak: (1) a larger $V_{ce}/V_v$ resulted in a larger peak deviator stress and more pronounced post peak softening; (2) sands with large $V_{ce}/V_v$ were more dilative than sands with low $V_{ce}/V_v$; (3) specimens confined to the same stress but with differing $V_{ce}/V_v$ resulted in (a) an increased stress ratio $q/p'$ (at peak), and (b) a greater maximum dilation rate when $V_v/V_{ce}$ was large (Note: this paper defines $q = \sigma'_1 - 2\sigma'_3$ and $p' = (\sigma'_1 + 2\sigma'_3)/3$). In general, smaller confining pressures resulted in larger stress-ratios (as also shown by the results obtained in this thesis) and thus, a greater maximum rate of dilation. For one test, as the stress-ratio decreased and a shear band developed post peak, the rate of dilation decreased faster than the stress-ratio. This is indicative of the frictional nature of the critical state strength of the soil.

Post peak, the soil strength tended towards a critical state value with friction as the controlling resistance mechanism. The understanding that emerged from the tests was that cemented sand behavior is brittle at lower confining pressure, transitioning to ductile at higher confining pressures.

**2.1.21 Drained Behavior of Cemented Sand in High Pressure Triaxial Compression Tests**

**Author:**
Marri, Wanatowski, and Yu (2012)

**Purpose:**
The purpose of this study was to characterize the mechanical behavior of cemented sand at high confining pressures.

**Method Details:**
A series of drained isotropically consolidated triaxial tests were carried out on saturated specimens of uncemented and cemented sand to examine strength behavior in relation to cement content and confining pressure. Cemented specimens were prepared using a Portaway quartz sand mixed with 5%, 10%, and 15% (by mass of dry sand) ordinary cement.
Portland cement as the cementing agent. Specimens were confined to 1, 4, 8, and 12 MPa and sheared under drained conditions, keeping confinement constant, at a deformation rate of 0.02 mm/min. In addition, isotropic tests were conducted on both cemented and uncemented sand specimens with different void ratios and the effects of high confining pressure on particle crushing and cement bond breakage were examined using scanning electron microscopy (SEM).

Findings:

Isotropic compression tests revealed that the effect of initial void ratio became less significant as confining pressure increased, evidenced by the isotropic compression curves with different initial void ratios converging at high pressures in an e vs log (p’) plot. However, cement content did play a role in the compressibility of the cemented sands. At high confining pressures, the compressibility of the cemented specimens decreased with increasing cement content.

Increasing the confining pressure increased the peak strength as well as the amount of volumetric compression during shear. SEM observations showed that the confining pressure stage had little effect on the breakage of cement bonds in triaxial tests, as most crushing of sand particles and cement bonds were broken during shear. In addition, SEM analyses of both cemented and uncemented sands revealed that during shear, the uncemented sand underwent a larger degree of particle crushing than did the cemented sand (due to the protection offered by the cement).

The drained isotropically consolidated triaxial test results confirmed the profound effects of cement content and confining pressure on the stress-strain and volume change behavior of cemented sands. For confining stress ($\sigma_c < 12$ kPa), increasing the cement content increased the peak strength, but reduced the initial volumetric compression of the specimen. For high confining stresses ($\sigma_c = 12$ MPa), all specimens were contractive and the role of cement content on the volumetric change behavior was not significant. The stress-strain behavior of the cemented sands was observed to become increasingly ductile with increasing confinement and was evidenced by the transition of a pointed peak to a smooth peak in the deviatoric stress vs. strain curve, with the effects of the cementation being less pronounced at high confining pressures. Shear banding was observed in all the drained isotropically consolidated triaxial cemented specimens. However, complex modes of failure
were observed (“brittle”, “transition”, and “ductile”) depending on cement content and confining pressure.

2.2 ACOUSTIC EMISSIONS: ANNOTATED BIBLIOGRAPHY

A summary of relevant articles associated with Acoustic Emissions (AE) monitoring for soil processes subjected to loading is included below.

2.2.1 Acoustic Emissions in Stressed Soil Samples

Author: Koerner and Lord (1974)

Purpose: This study centered on the AE response of soil specimens under various stress conditions.

Method details: An accelerometer was used to detect acoustic emissions from soil specimens undergoing hydrostatic compression and triaxial compression. The piezoelectric accelerometer captured waves from the stressed soil specimens via a rod sensor; the signal was amplified and recorded. Triaxial compression tests were conducted using confinements of 34.5, 68.9, and 103.4 kPa. The hydrostatic compression tests were conducted by applying air pressure to the water inside the triaxial cell chamber, in increments.

Findings: Higher AE counts coincided with increased pressures during hydrostatic tests. Higher confining pressures in triaxial compression tests also resulted in greater AE counts (during shear).

2.2.2 Scanning Electron Microscope and Acoustic Emission Studies of Crack Development in Rocks

Author: Fonseka, Murell, and Barnes (1985)

Purpose:
This study used both scanning electron microscope (SEM) observations and (AE) monitoring to analyze the development or growth of cracks in rock specimens under compression.

**Method details:**

Triaxial compression tests were carried out under a confining pressure of 157 MPa and a strain rate of $10^{-5}$ sec$^{-1}$. SEM observations were conducted on specimen’s post triaxial loading and post uniaxial compression. AE monitoring was conducted during uniaxial compression on samples of dolerite, microgranodiorite, and marble.

**Findings:**

AE were categorized according to behavior during loading. At the beginning of loading, the AE rate sharply increased and then decreased until leveling off at approximately 75% of the load, whereupon the AE rate increased exponentially until failure occurred. Prior to failure, the AE rate from all rock specimens under load, dramatically increased. In addition, volume changes of the specimen agreed well with AE events in that rapid increases in volume change corresponded to sharply increased AE rates. Marble, which is a more ductile rock then the other two rock types tested, produced less pronounced AE’s than the other rocks tested. Each AE event described in the study was not due to an individual crack forming, but resulted from cumulative AE events within a certain time period (such experimental feature is also featured by the AE measurements made in this thesis).

SEM studies revealed that rock specimens displayed few observable cracks along grain-boundaries or within grains for stresses up to about half the failure stress. Beyond half the failure stress, cracks along grain boundaries began to form. At this point, transgranular (serrated) cracks also began to appear until the load was high enough that the cracks joined and formed large cracks. This continued until the specimen began to break.

**2.2.3 Yielding of Soil as Determined by Acoustic Emission**

**Author:**

Tanimoto and Tanaka (1986)

**Purpose:**

The purpose of this study was to introduce an alternative method that examines the AE energy released from soil specimens during shear in order to determine the on-set of
yield of the soil. The yield stress $\sigma_{\text{yield}}$, was defined as the stress below which the specimen responds elastically to shear. The soil tested was a SW, with $G_s = 2.66$, $C_u = 29$, and average particle size 0.41mm.

**Method details:**

Drained triaxial tests conducted with a constant mean principle stress were performed to determine the yield point using AE measurements. Results from these tests were then compared to results from constant cell pressure triaxial tests. A range of confining pressures were applied (100, 200, 300, and 400 kPa), to specimens with various overconsolidation ratios (OCR of 6, 3, 2, 1.5, and 1.2). Before shear, specimens first underwent isotropic compression and were pre-loaded up to 600 kPa to eventually bring the specimens to a specific OCR. AE’s were monitored with a piezoelectric transducer mounted in the pedestal of the triaxial cell and acoustic emissions that exceeded a predetermined threshold were counted and recorded for a period of time during loading (in this thesis, a similar AE methodology is adopted).

**Findings:**

Constant confinement triaxial test results showed that increasing the confining pressure caused the specimen to undergo a slight change in volume. As each specimen was sheared, it underwent large changes in volume along with sharply increased AE counts. The stress-strain curve correlated well with the AE count, which increased as the strain increased. The volume change experienced by specimens in the early stages of loading (when specimens are initially contractive) appeared to be associated with the elastic region of the soil.

### 2.2.4 The Role of Acoustic Emission in the Study of Rock Fracture

**Author:**

Lockner (1993)

**Purpose:**

The purpose of this study was to examine rock fracture processes using AE.

**Method details:**

A review of existing literature on AE was conducted to evaluate pros and cons associated with using AE to study the process of rock fracture and to predict rock failure.
Findings:

Studies conducted on AE have shown that a rise in AE rates correlates strongly to inelastic strain. The use of AE, a natural by-product of brittle fracture and micro-growth rate, to provide information as well as further the understanding of the process of rock fracture, is an adequate non-destructive alternative to existing methods of study.

2.2.5 Characterizing Bond Breakages in Cemented Sands Using a MEMS Accelerometer

Author:
Wang, Ma, and Yan (2009)

Purpose:
The purpose of this study was to record AE emitted by cemented sand grains as bonds were broken during triaxial loading.

Method details:
Drained triaxial compression tests were conducted on cemented and uncemented sand specimens under a confining pressure of 30 kPa, with a strain rate of 0.1%/minute. Bond breakage events and associated AE were captured during shear. Acoustic Emissions were recorded using a MEMS accelerometer ("MEMSA") as well as a commercially available piezoelectric transducer, ("PZT") to enable comparisons. One MEMSA and one PZT were attached (close together) to cemented specimens to record the AE. AE signals were captured when amplitudes exceeded a certain threshold. Acoustic emissions were recorded for 90 seconds and the AE rate was determined by the total number of hits within this time.

Findings:
For the uncemented sands both the PZT and MEMSA detected increasing AE rates with increasing axial strain, until approximately 10%, at which point the AE leveled off. At $\varepsilon_a > 10\%$, the AE’s were deemed a result of particles sliding.

For cemented sands, both the PZT and the MEMSA showed small AE in the beginning and then increasing AE with increasing strain. The AE rate was high up to the peak and then the AE rate decreased as a shear band developed (where many of the bonds were already broken) and the region outside the shear band underwent elastic unloading. The AE resulted mainly from bond breakages. Similarly to the case for uncemented sand, the AE
response of cemented sands was in general, aligned with the stress-strain response of the specimen.

Cemented specimens that were stronger were better able to resist bond breakages at small strains. This was evidenced by a “delay” in the rise in AE for the stronger cemented specimens. Stronger cement bonds also exhibited a smaller range over which a high AE rate was observed, possibly because the strength loss of cement bonds was more severe (for strong specimens), and allowed local weaknesses to develop readily.

The PZT and MEMSA had notable differences. The PZT was more sensitive to recording AE, whereas the MEMSA detected the rise in AE rate earlier than the PZT. The MEMSA detected a distinct rise in the AE rate with stronger cemented specimens, indicating that breakages begin with weaker bonds (Note: the MEMSA has a lower attenuation which allows it to capture low frequency waves). As the loading continues, bond breakages continue, and the larger bonds begin to break. The PZT recorded higher frequency signals better than the MEMSA. Yet, the MEMSA was deemed to be a reliable AE detection sensor; especially good for capturing low frequency AE.

### 2.2.6 Monitoring the Oedometric Compression of Sands with Acoustic Emissions

**Author:**
Fernandes, Syahrial, and Valdes (2010)

**Purpose:**
The purpose of this study was to introduce a technique whereby mechanical grain interactions (mainly crushing), could be captured during oedometric compression tests.

**Method details:**
Three types of coarse sand with different grain strengths and shapes were tested. The first was a homogenous Ottawa sand of high strength, the second was a heterogeneous natural beach sand with a medium average strength, and the third was a homogenous low strength sand with internally-porous grains. All specimens were prepared dense then loaded with a constant deformation rate $v = 0.05 \text{ mm/sec}$ and then unloaded with $v = 0.1 \text{ mm/sec}$. A piezo transducer placed under the oedometer’s bottom platen was used to detect AE from mechanical grain interactions during oedometric compression. The load-deformation response was then compared to the AE data.
Findings:

AE were counted for points that exceeded an amplitude threshold, defined as the background noise in the laboratory (noise recorded prior to loading). Note this approach is used to analyze the AE recorded during tests associated with this Thesis. Results plotted in the form of stress-time curves showed a pre-yield ($\sigma<\sigma_{yield}$) regime, a clastic ($\sigma_{yield}<\sigma<\sigma_k$) regime, and a “stiffness regain” ($\sigma>\sigma_k$) regime. Severe breakage occurred initially during the clastic regime and the soil lost stiffness. At a higher stress ($\sigma = \sigma_k$), the breakage that had occurred lead to a rise in coordination, resulting in increased stiffness as the soil continued to be loaded. The authors found that a certain degree of breakage must be reached for each soil before such stiffness regain occurs. AE results showed that the AE signal and the count-time data curve can be used to (1) estimate the pre-yield regime (where the AE count rate rises with the increasing stress); (2) detect the onset of the clastic regime (where there is a decrease in the AE count rate with increased stress); and (3) detect the onset of the stiffness regain regime (where the AE count rate is relatively constant).
CHAPTER 3

MATERIALS AND PROCEDURES

3.1 MATERIALS

Ottawa 20-30 quartz sand (0.6 mm < d < 0.85 mm) with specific gravity $G_s = 2.65$, classified as poorly graded sand (SP) via the Unified Soil Classification System (Figure A.1), and polyethylene powder with $G_s = 1.2$, 20 μm < d < 60 μm, from Tiger Drylac Co. (Figure A.2) were used as the soil and cementing agent, respectively. Deionized water was used in the preparation of all specimens.

3.2 SPECIMEN PREPARATION: CEMENTATION

Each PBS specimen was prepared in the laboratory, as follows. The polymer fines and the uncemented sand grains (Figure A.3) are mixed in a beaker with a volumetric cement content $f_v = V_c/V_s = 2.2\%$, where $V_c =$ volume of cement and $V_s =$ volume of solids (i.e. volume of sand plus volume of cement). Water is added to the mixture (water content $w = m_w/m_s \approx 6\%$) so as to enable capillary forces to hinder segregation. The mixture is then agitated by hand using a metal spatula and then partitioned. Each portion is then placed into a split-aluminum mold (diameter = 50 mm and height = 100 mm) lined with Safeway brand wax paper so that the specimen can be removed easily from the mold following the heat treatment stage. The soil is then tamped inside the mold using a cylindrical aluminum rod (mass = 415.55 grams) in three separate layers. Special care was taken to scarify the tops of the bottom two layers, to promote strong adhesion amongst adjacent layers. This was done to decrease the likelihood that localization failures could occur at layer interfaces. The preparation of each specimen took approximately 30 minutes. The mold is then placed in an oven with $T = 40^\circ$ C for one hour to hasten the water evaporation, i.e., the drying, process. Indeed, drying is an important component of the proposed cementation treatment because the polymer fines are carried through capillary action to the sand-sand contacts as water recedes.
to form menisci. The specimen is then allowed to air-dry at room temperature (outside the oven) until the water content is negligible \((w = m_w/m_s = 0.03 \%)\). The top of each specimen was left exposed during the \(T = 40^\circ\) oven heating stage and the air-drying stage. Once the water content was deemed negligible, the specimen is topped with a steel piston (mass = 186.75 grams) and the mold is placed in the oven with \(T = 150^\circ\) C for one hour. This heating stage was implemented to soften (i.e., melt) the polymer fines. The specimen is then removed from the oven, and allowed to cool at room temperature, to allow the softened polymer to harden and thus bond the sand grains. Each specimen is then carefully extracted from the mold by removing the screws that connect the split mold and then splitting the mold. When the specimen is removed, some of the lining wax paper is adhered to the specimen walls, and thus carefully removed by hand. Finally, the specimen is weighed and its dimensions are measured to determine the density. A picture of a PBS specimen is shown in Figure A.4 and a picture of a cluster of bonded sand grains is shown in Figure A.5. Experimental details for prepared specimens are presented in Table A.1.

Note: The gravimetric cement content \(CC\) is commonly used in the literature, and is defined as \(CC = m_c / m_{sa}\), where \(m_c\) = mass of cement and \(m_{sa}\) = mass of sand only, i.e., \(m_{sa} = m_s - m_c\). \(CC\) can be written as a function of \(f_v\) as follows. First, expanding the definition of \(f_v\) leads to:

\[
f_v = \frac{V_c}{V_s} = \frac{V_c}{V_c + V_{sa}} = \frac{m_c}{\rho_c} \frac{m_s + m_{sa}}{m_c + m_{sa}}
\]

(3.1)

Rearranging (3.1)

\[
f_v \frac{m_c}{\rho_c} + f_v \frac{m_{sa}}{\rho_{sa}} = \frac{m_c}{\rho_c}
\]

(3.2)

Solving (3.2) for \(m_c\) yields

\[
m_c = f_v m_{sa} \rho_c \frac{1 - f_v}{m_s \rho_{sa}}
\]

(3.3)

Rearranging,

\[
\frac{m_c}{m_{sa}} = CC = \frac{f_v \rho_c}{(1 - f_v) \rho_{sa}}
\]

(3.4)

The volume of voids to volume of cement ratio \(V_v/V_c\) is used in the literature because it captures both the cement content and the void ratio of the soil (Consoli et al. 2009). The ratio
Vv/Vc is used in this Thesis (see Chapter 2) to describe the degree of cementation. A formulation for Vv/Vc as a function of CC and the void ratio is derived as follows. First, rewriting the definition of CC:

\[ CC = \frac{m_c}{m_{sa}} = \frac{V_c \rho_c}{V_{sa} \rho_{sa}} \]  

(3.5)

Solving for Vsa yields:

\[ V_{sa} = \frac{V_c \rho_c}{CC \rho_{sa}} \]  

(3.6)

The definition of the void ratio e is now expanded:

\[ e = \frac{V_v}{V_s} = \frac{V_v}{V_{sa} + V_c} = \frac{V_v}{\frac{V_c \rho_c}{CC \rho_{sa}} + V_c} = \frac{V_v}{\frac{V_c \rho_c + V_c CC \rho_{sa}}{CC \rho_{sa}} + V_c \rho_c + CC \rho_{sa}} \]  

(3.7)

Rearranging (3.7) yields:

\[ \frac{V_v}{V_c} = \frac{e(\rho_c + CC \rho_{sa})}{CC \rho_{sa}} \]  

(3.8)

And simplifying (3.8) yields:

\[ \frac{V_v}{V_c} = e \left( \frac{\rho_c}{CC \rho_{sa}} + 1 \right) \]  

(3.9)

3.3 Specimen Preparation: Testing

Each PBS specimen was subjected to a drained triaxial test (dry) or to an unconfined compression test. Drained triaxial tests were carried out on eleven dry PBS specimens. Each specimen was fitted with a tubular latex membrane as follows. First, the membrane is fitted onto a steel tube, by inserting the membrane into the tube and folding the membrane’s ends onto the outside of the tube. This tube is then placed vertically over the bottom platen of the triaxial. Then, the specimen is placed inside the tube (thus inside the membrane) and on the bottom platen of the triaxial. The membrane is then carefully pulled off the bottom of the tube and onto both (1) the sides of the bottom platen and (2) the sides of the bottom half of the specimen. The top plate is then placed on top of the specimen, and the membrane is pulled off the tube and onto both (1) the sides of the top plate and (2) the sides of the top portion of the specimen. Care was taken to make sure no dislodged sand grains remained at the interfaces between the specimen and the top and bottom plates, thus ensuring flat
interfaces. The triaxial chamber was then assembled, filled with deaired water, and pressurized to a desired confinement (with panel valves left open to ensure drained conditions).

Each “restructured” specimen was prepared as follows. A triaxial-tested specimen is selected and desstructured by rubbing bonded clusters together by hand into a glass beaker. The sand grains are then weighed to determine the mass. Next, the polymer-coated sand grains (Figure A.6) are poured into a cylindrical split-mold lined with a rubber membrane. The cylindrical mold is seated on the loading frame around the bottom platen and a small circular mesh is placed onto the bottom platen to keep any sand grains from falling into the drainage perforations of the triaxial’s bottom platen. A vacuum is applied to a port located on the side of the mold causing the membrane to hug the mold so that the sand grains can freely fill the mold. A circular mesh is placed on top of the specimen followed by the top platen. The vacuum and mold are then removed, and the height and diameter of the restructured specimen are then recorded using calipers.

3.4 Testing Procedures

PBS and restructured specimens were confined to a range of stresses, between 55 kPa and 586 kPa (see Table A.1), and then sheared in drained conditions with an axial displacement rate of 0.014 mm/sec at constant confinement. In each test, the axial deformation was recorded with a linear differential transducer (LDVT; resolution = 10 μm) mounted on the bottom platen of the loading frame (Durham-Geo MODEL S-600). The deviatoric load, applied axially to impose shear, was recorded with a load cell (MODEL 1000) positioned externally, between the piston of the triaxial and the top, fixed brace of the load frame. The volumetric strain was determined by monitoring the water level inside the pressure panel (Durham-Geo MODEL S-500) burette associated with the triaxial chamber. Water fluctuations inside the burette were deduced to correspond with changes in the volume of the specimen. Lubricated ends were not used.

Unconfined compression specimens were tested under a controlled deformation rate of 0.07 mm/sec. Deformation was recorded with an LDVT internal to the load frame (Instron model 3382). The deviatoric load was recorded with a load cell internal to the load frame.
Microvibrations, hereafter denoted acoustic emissions (AE), were measured during both the confinement and shearing stages of the triaxial tests and during unconfined tests using a ceramic shear accelerometer (PCB model 352A24, \( f_{\text{res}} \geq 30 \text{ kHz} \)), attached with paraffin wax (1) onto the bottom platen of the triaxial cell directly beneath the specimen (Figure A.7) or (2) attached onto the bottom platen of the load frame for unconfined compression tests. The accelerometer was connected to a line-powered, eight-channel digitally controlled amplifier (ICB transducer systems, MODEL PCB 482). This amplifier was connected to the microphone input of the computer (Dell laptop model PP04X), as a line input. The signal recorded during testing was acquired with the software program CEP with a sampling rate of 192 kHz. The AE analysis was performed by counting the number of times the signal amplitude exceeded a threshold \( T \), which was set as the ambient noise level (see Figure A.8), following the methods described by Fernandes, Syahrial, and Valdes (2010). Note: Such noise was recorded for a few seconds prior to the shearing stage of the tests. A MATLAB code proprietary of the Geo-Innovations Research Lab was then used to determine the count \( C \); that is, cumulative sum of the number of times the signal amplitude \( A \) exceeded \( T \):

\[
C(t) = \sum_0^t (A > T)
\]  

(3.10)

3.5 Mohr-Coulomb Strength Parameters

In geotechnical engineering Mohr-Coulomb strength parameters are used to describe the shear strength of soils. In terms of effective stresses, the shear strength function is called the Mohr-Coulomb failure envelope and is written as:

\[
S = \sigma' \tan(\phi') + c'
\]  

(3.11)

In eq. 3.11, \( \sigma' = (\sigma - u) \), is the effective stress. \( \sigma \) is the total stress applied normal to the shear plane, and \( u \) is the pore water pressure acting at the point in question. The angle of internal friction is referred to as \( \phi' \), and describes the relationship between the effective stress and the strength of the soil (the tangent of \( \phi' \) is the slope of the failure envelope, i.e., the slope of the line described by eq. 3.11). The angle of internal friction is dependent on soil gradation, particle shapes, and soil fabric. Two friction angles can be defined, depending on whether or not there is strain softening upon shear strain: the peak friction angle, \( \phi'_p \), and the
critical state friction angle, $\phi'_{cv}$. The critical state friction angle corresponds to a post-peak state, where the soil offers a constant resistance upon shearing. The cohesion intercept ($c'$) is a fitting parameter: the intercept of the straight line – described by equation 3.11– on the shear stress axis (y-axis). The cohesion intercept is the shear strength of the soil when the effective stress is zero; thus, it captures the strength of the bonds between grains, if any exists. In this Thesis, the strength of the polymer bonds shared by sand grains is captured by a nonzero cohesion intercept.
CHAPTER 4

RESULTS

It is important to note that in the tests conducted both unconfined and triaxial compression, the stiffness is low at low strains is likely due to the top of the specimen not being perfectly horizontal.

4.1 UNCONFINED COMPRESSION TESTING

The experimental details associated with unconfined compression tests are listed in Tables A.1 and B.1 in Appendix A and B, respectively. Stress-strain curves for PBS specimens are shown in Figure B.1. Peak strength for all tests occurred at axial strains between 0.7% and 1.7%, with the strains at peak increasing with increasing CC (B.4). Figure B.2 shows the unconfined compression strength ($q_{uc}$) as a function of the cement content (CC). The results show that the cement content has a significant effect on the strength of the cemented soil. In addition, the $q_{uc}$ increases linearly with CC (in the CC range tested). The trend is also shown in Figure B.3 along with results reported by Consoli et al. (2007) and Park (2011) for sands cemented with Portland cement. Note that the $q_{uc}$ of PBS is significantly larger than that of the other Portland-cemented sands.

The $q_{uc}$ of a cemented specimen is expected to be dependent on the porosity of the specimen. Indeed, $q_{uc}$ is sensitive to the specimen’s volume of voids $V_v$, as shown in Figure B.5. There is a better defined correlation of $q_{uc}$ and the inverse of $V_c$, as shown in Figure B.6. This reflects the role of CC on $q_{uc}$. A combination of parameters yields the unconfined compression strength of PBS specimens as function of the voids-cement ratio defined by equation 3.9 in Chapter 3. The data are shown in Figure B.7, with the trend obtained by Consoli et al. (2007) for Portland – cemented sands with 1% < CC < 7%. Note that the $q_{uc}$ vs. $V_v/V_c$ relationships follow a similar, nonlinear trend: the sharp rise in $q_{uc}$ with decreasing $V_v/V_c$ reflects the increasing interconnectivity of the cement in the void space with increasing CC (i.e., as $V_v/V_c$ decreases; Garcia, Valdes, and Cortes 2015).
4.1.1 Acoustic Emissions

Figures B.8, B.9, and B.10 show the stress-time curves for three selected tests (with different CC) aligned with the corresponding AE signals and counts. Each AE signal corresponds fairly well with the stress-time curve. In general, AE activity begins as soon as loading commences and increases during loading. This suggests that the mode of failure is the creation of multiple fractures that then combine to render the post peak resistance degradation. Indeed, note that (1) the highest AE count rate (i.e., largest slope of the count curve) occurs prior to peak and (2) the AE count rate immediately decreases post peak. In addition, a comparison of Figures B.8, B.9, and B.10 reveals that the count values pre-peak increase with increasing CC. This suggests that the number of contact points required to debond for failure to occur increases as CC increases.

4.2 Triaxial Testing

The experimental details associated with unconfined compression tests are listed in Tables B.1 and C.1 in Appendix B and C, respectfully. Shear localization planes formed on PBS specimens, as observed in Figure C.1, and bulging (not shown) was observed in restructured specimens. Strain softening was not observed for the restructured specimens, indicating that failure for such was ductile (rather than brittle).

Figure C.2 shows the axial strain - strength (\(\varepsilon_a vs q = \frac{(\sigma_1' - \sigma_3')}{2}\)) curves and the axial strain-volumetric strain (\(\varepsilon_a vs. \varepsilon_{vol}\)) curves for PBS specimens confined to a variety of stresses (\(\sigma_3' = 55\ kPa\) to 586 kPa). Restructured specimen test results are also included for reference. Each curve corresponds to one test at the denoted confinement (in kPa). Initially, the stress-strain behavior of PBS specimens rises linearly up to a peak \(q\), i.e., \(q_{peak}\), beyond which the soil suffers strain softening: the stress \(q\) decreases until a critical state is reached. Both \(q_{peak}\) and the axial strain at peak increase with increasing confinement.

Figure C.3 (bottom) shows that all PBS specimens display contractive behavior followed by dilative behavior. For PBS specimens, the maximum dilation rate takes place just prior to the peak and dilation continues until the volumetric strain stabilizes (rate \(\approx 0\)). Increasing the confining pressure results in (1) an increase in the contractive strain prior to the inflection point, (2) a decrease in the dilatancy rate after the inflection point (when dilation commences), and (3) a decrease in the critical state dilation strain. For restructured
specimens, the maximum dilation rate takes place prior to the maximum \( q \) reached and dilation continues, never appearing to reach a constant value (for the axial strains imposed).

Effective stress ratio vs. axial strain curves are shown in Figure C.3 (top). The peak ratio decreases with increasing confining pressure. A pronounced transition that occurs for \( 55 \text{kPa} < \sigma'_{3} < 200 \text{kPa} \) can be observed in Figure C.3 (bottom), which shows peak stress ratios vs. confinement. This transition signals the emergence of friction as a contributor to peak strength.

### 4.2.1 Acoustic Emissions Alignment

The results of the triaxial tests were analyzed to examine the relationship between AE and the stress-strain-volume change characteristics of the PBS. Figures C.4, C.5, C.6, C.7, and C.8 show deviatoric stress-time curves aligned with AE signals, AE count curves, and volume time strain curves.

The AE alignment data clearly show the effects of confinement. Figure C.4 (\( \sigma'_{3} \approx 1 \text{kPa} \)) shows that AE activity begins as soon as loading commences and rapidly increases until peak, at which point the AE activity drastically diminishes (i.e., the count rate is \( \approx 0 \)). Figure C.5 (\( \sigma'_{3} =55 \text{kPa} \)) shows that AE activity begins as the stress increases. The AE count, however, continues to increase through peak until a point post peak (\( t = 110 \text{ sec} \)), where the AE count rate decreases. An increasing count post peak signals the establishment of a shear band (Figure C.1); which is immediate for negligible confinement (e.g., \( \sigma'_{3} = 1\text{kPa} \)), and which requires some axial strain for confined specimens (e.g., \( \sigma'_{3} \geq 55 \text{kPa} \)). For \( \sigma'_{3} = 55 \text{kPa} \), the AE count continues to decrease as the critical state is reached. Figures C.6, C.7, and C.8 (for \( \sigma'_{3} = 207, 345, \) and \( 586 \text{kPa} \)) show trends similar to those of the \( \sigma'_{3} = 55 \text{kPa} \) specimen, in that AE activity continues beyond the peak stress; yet, the AE count curve at and beyond the peak becomes increasingly gradual with increasing \( \sigma'_{3} \). This means that friction plays an increasingly important role in controlling the shear resistance as \( \sigma'_{3} \) increases. The AE data corresponds well to the volumetric strain vs. time data in that AE are produced as the specimen contracts, but not at a high rate, as opposed to the high AE count rate observed while dilation takes place.
4.2.2 Strength Parameters

Figure C.9 shows the failure envelopes for the PBS specimens tested in this study. The failure envelopes are linear and the peak friction angle is greater than the CS friction angle. In addition, the CS friction angles for PBS and for restructured specimens are essentially the same. Failure envelopes from previous research data conducted by various authors are compared with the results of this study in Figure C.10. It can be observed from the previous studies that using another type of cement, like Portland, results in strengths lower than those achieved for PBS, even when using CC = 1% for PBS and CC > 3% for Portland. This is clearly shown in Figure C.11, which indicates that the strength of the PBS specimens is significantly larger than those of sands cemented with other agents, even when those specimens have low $V_v/V_c$ values (as low as $\approx 3$) as compared to those of the specimens tested ($\approx 29$).

Figure C.12, shows the brittleness index (see equation 2.3 in Chapter 2) as a function of $\sigma'_3$. Note that the brittleness index decreases as $\sigma'_3$ increases, confirming that the material is brittle and that the overall behavior is dependent on the confining pressure. The brittleness index drops pronouncedly for $55 < \sigma'_3 < 200$ kPa (note the log scale). This confinement range, points to the emergence of friction as a contributor to peak strength. Recall that this can also be interpreted from the stress ratio trend in Figure C.3 (bottom). Friction becomes the controlling contributor to peak strength at higher confinements. Indeed, the transition in Figure C.13 suggests that friction becomes largely responsible for peak strength when $\sigma'_3 > \sim 350$ kPa for the tested specimens with CC = 1%. Higher transition confinements are expected for specimens with higher CC.
CHAPTER 5

CONCLUSIONS

5.1 CONCLUSIONS

The following conclusions are derived from this Thesis research:

- The proposed technology, heat-induced cementation of sand with added polymer fines, is effective, in that high strengths are achieved for polymer bonded sand (PBS) specimens prepared in the laboratory.

- The unconfined compression strength \( (q_{uc}) \) of PBS increases approximately linearly with cement content (CC), and decreases with the voids volume-to-polymer volume \( V_v/V_c \) ratio, following a power law, as does the \( q_{uc} \) of Portland cemented sands.

- The \( q_{uc} \) of PBS specimens with CC = 1% was higher than those reported in the literature for other cement types, even when such specimens have CC > 1%.

- The peak shear strength of PBS sands with CC = 1% depends on confining pressure.

- Post-peak strain softening occurs for PBS specimens with CC = 1% confined to \( \sigma_3' < 580 \) kPa. This means that (1) cementation controls the stress-strain resistance up to peak, and (2) after the peak, the response tends towards a critical state, where the resistance is controlled by friction.

- PBS with CC = 1% exhibits a gradual transition between a cement-controlled strength regime at low confinements and a friction-controlled strength at high confinements. This transition (55 kPa < \( \sigma_3' < 200 \) kPa) is observed via stress ratio data and also evidenced by features of acoustic emissions (AE) collected during testing.

- The cohesion intercept and the effective friction angle of PBS with CC = 1% confined with \( \sigma_3' < 580 \) kPa are significantly higher than those reported in the literature for cemented sands with other binding agents, even when such specimens have CC > 1%.

5.2 RECOMMENDATIONS FOR FUTURE WORK

The following recommendations are made to extend this Thesis research:

- The sand used to prepare the PBS specimens tested herein was uniform, with a single mineralogy (quartz). Future work to quantify the role of mineralogy and
gradation features on the response of PBS is recommended. Perhaps there is an optimal grain size distribution to achieve the highest strength for a given CC and mineralogy.

- For a given soil, there is likely a characteristic CC for which additional cement does not produce additional strength. Beyond such CC, the strength of the PBS is namely the strength of the polymer. Studies to determine this CC and the soil properties that control it (gradation and mineralogy) are recommended.

- The PBS specimens tested were cemented with one cement content, i.e., CC = 1%. Future work on the role of CC on shear strength is recommended, placing particular emphasis on the pore network. An increase in CC corresponds to a strength increase but also to a decrease in pore sizes and therefore a decrease in hydraulic conductivity. Low k translates to poor drainage and excess pore pressures. This means that the loading response for a PBS specimen with high CC can temporarily alternate between drained and undrained during triaxial testing.
REFERENCES


## APPENDIX A

### CHAPTER 3

Table A.1. Summary of polymer bonded specimen details.

<table>
<thead>
<tr>
<th>Test$^{[1]}$</th>
<th>Specimen</th>
<th>$\sigma'$ (kPa)</th>
<th>$\rho$ (g/cm$^3$)</th>
<th>$f_v$ (%)</th>
<th>CC (%)</th>
<th>$e$ ( )</th>
<th>$V_v/V_c$ ( )</th>
</tr>
</thead>
<tbody>
<tr>
<td>T_14</td>
<td>PBS</td>
<td>1$^{[2]}$</td>
<td>1.60</td>
<td>2.2</td>
<td>1</td>
<td>0.637</td>
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<td>0.640</td>
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</table>
Figure A.1. Ottawa 20-30 quartz sand used in preparation of specimens.

Figure A.2. Polyethylene Powder used as the cementing agent in specimen.
Figure A.3. Uncoated Ottawa 20-30 sand grains.
Figure A.4. Cemented specimen with cement content of 1%.
Figure A.5. Pictured above is a) one of many clusters of polymer bonded cement grains; and b) a SEM view of the polymer bond joining two sand grains.
Figure A.6. Cement coated sand grains.
Figure A.7. Schematic diagram of the experimental setup of triaxial cell and attached accelerometer.
Figure A.8. AE signal recorded during shearing stage of triaxial test conducted with $\sigma'_3 = 55$ kPa.
### APPENDIX B

### CHAPTER 4 UNCONFINED TESTING

Table B.1. Experimental results obtained from unconfined compression tests.

<table>
<thead>
<tr>
<th>Test</th>
<th>Class</th>
<th>$D_{50}$ (mm)</th>
<th>$D_{max}$ (mm)</th>
<th>$D_{min}$ (mm)</th>
<th>Type</th>
<th>CC (%)</th>
<th>$e$</th>
<th>$\sigma_1'$ (kPa)</th>
<th>$\varepsilon_a$ (peak)</th>
</tr>
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<td>0.6</td>
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<td>1</td>
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</table>
Figure B.1. Stress-strain curves of PBS specimens cemented with various cement contents, i.e., CC = 1, 2.4, 5.6%.
Figure B.2. Variation of unconfined compressive strength with cement content for PBS specimens.
Figure B.3. Unconfined compression strength vs. CC for PBS and other cemented sands. The triangles denote cemented specimens that contain fibers.
Figure B.4. The relationship between axial strain at peak and cement content for PBS specimens tested in unconfined compression.
Figure B.5. Relationship between specimen’s voids volume and unconfined compression strength (for PBS specimens).
Figure B.6. Variation of unconfined compressive strength with the inverse of the volume of cement for PBS specimens.

\[ y = 1440.6x^{-1.013} \]

\[ R^2 = 0.9651 \]
Figure B.7. Variation of unconfined compressive strength with voids/cement ratio.
Figure B.8. Stress curve aligned with AE signal and AE count for PBS specimen with CC = 1%.
Figure B.9. Stress curve aligned with AE signal and AE count for PBS specimen with CC = 2.4%. 

---

Figure B.9. Stress curve aligned with AE signal and AE count for PBS specimen with CC = 2.4%.
Figure B.10. Stress curve aligned with AE signal and AE count data for PBS specimen with CC = 5.6%. 
# APPENDIX C

## CHAPTER 4 TRIAXIAL TESTING

Table C.1. Summary of experimental results obtained from triaxial compression tests conducted for this study and those by selected investigators.

<table>
<thead>
<tr>
<th>Ref.</th>
<th>Class</th>
<th>Soil</th>
<th>Cement</th>
<th>Triaxial Results</th>
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[1] Schnaid, Prietto, and Consoli (2001) report $C_u = 32$, $D_{60} = 0.0896$ mm, and $D_{10} = 0.0028$ mm.
[3] $\phi'$ and $c'$ for Marri, Wanatowski, and Yu (2012) calculated assuming a linear failure envelope, as shown in Figure C.10.
[4] Feng & Montoya (2015) grouped specimens with cc (%) = 0.9 to 1.4, 2.2 to 3, and 4.3 to 5.3.

\[ \sigma'_3 = 1 \text{kPa} \quad \sigma'_3 = 207 \text{kPa} \quad \sigma'_3 = 345 \]

Figure C.1. Pictured here are PBS cemented specimens post triaxial testing at three confining pressures. Shearbands evident.
Figure C.2. Strength-axial Strain-Volumetric Response for PBS, with CC = 1%, at various confinements. The letter C denotes confinement to the stress (kPa) denoted following the C. The letter R stands for restructured specimens. The volumetric strain, $\varepsilon_{\text{vol}}$, for the restructured specimen confined at 55 kPa was not recorded.
Figure C.3. Top: stress ratio vs axial strain for triaxial tests on PBS and restructured specimens conducted at different confining pressures, as denoted (in kPa). Bottom: peak stress ratio vs confining pressure.
Figure C.4. Stress curve aligned with AE signal, AE count, and volume change data for test conducted with $\sigma_3 \approx 1$ kPa.
Figure C.5. Stress curve aligned with AE signal, count, and volume strain data for test conducted with $\sigma'_3 = 55$ kPa.
Figure C.6. Stress curve aligned with AE signal, count, and volume strain data for test conducted with $\sigma'_3 = 207$ kPa.
Figure C.7. Stress curve aligned with AE signal, AE count, and volume change data for test conducted with $\sigma'_3 = 345$ kPa.
Figure C.8. Stress curve aligned with AE signal, AE count, and volume change data for test conducted with $\sigma'_3 = 586$ kPa.
Figure C.9. Peak, critical state, and restructured failure envelopes for PBS, with CC = 1%.
Figure C.10. Failure envelopes of cemented sands extracted from selected studies in the literature as well as the envelopes for PBS that pertain to this study.
Figure C.11. $V_{v}/V_{c}$ ratio vs. peak $q$ values for this study ($V_{v}/V_{c} \approx 29$), as well as selected data from the literature.
Figure C.12. Brittleness index vs. confinement for selected data from literature as well as from this study.
Figure C.13. Variation of peak q with confinement for PBS specimens w/ CC = 1%.