SERRI
Southeast Region Research Initiative

SERRI Report 70015-006

DEVELOPMENT OF AN EMERGENCY CONSTRUCTION MATERIAL FOR DISASTER RECOVERY

OAK RIDGE
National Laboratory

Mississippi State University
CMRC
Construction Materials Research Center
"An Industry, Agency & University Partnership"

SERRI Project: *Increasing Community Disaster Resilience Through Targeted Strengthening of Critical Infrastructure*

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This material is based upon work supported by the U.S. Department of Homeland Security under U.S. Department of Energy Interagency Agreement 43WT10301. The views and conclusions contained in this document are those of the authors and should not be interpreted as necessarily representing the official policies, either expressed or implied, of the U.S. Department of Homeland Security.

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SERRI Project: Increasing Community Disaster Resilience
Through Targeted Strengthening of Critical Infrastructure

DEVELOPMENT OF AN EMERGENCY CONSTRUCTION MATERIAL FOR DISASTER RECOVERY

Performing Organization Report No. CMRC-10-03

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Date Published:

June 2012

Prepared for
U.S. Department of Homeland Security
Under U.S. Department of Energy Interagency Agreement 43WT10301

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UT-BATTELLE, LLC
for the
U.S DEPARTMENT OF ENERGY
under contract DE-AC05-00OR22725

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ACKNOWLEDGEMENTS

Thanks are due to many for the successful completion of this project. The authors are especially grateful for the financial support provided by the *SERRI* program. In addition, due gratitude is extended to everyone employed at *DHS* and *ORNL* who worked diligently with the authors to make this project a success. A great deal of the success of this research can be attributed to the efforts of *DHS* and *ORNL* personnel.

Thanks are owed *Grace Construction Products*, Jimmy Hill of *Fiber Reinforced Soils*, Vince Perez of *CTS Cement Manufacturing Corporation*, and *Holcim Cement* for material donation. In addition to material donation, James Boyd and Donnie Walker of *Holcim Cement* (Artesia MS plant) and Alissa Collins (Theodore, AL plant) assisted in development of specialty cement chemistries, collecting test data, and with multiple technical matters and are owed special thanks for their invaluable efforts.

Joe Ivy of *Mississippi State University (MSU)* assisted with many activities including fabricating specimen molds and construction of the environmental chamber. Sandra L. Ortega-Achury of *MSU* assisted with obtaining water properties used in the report. Dr. Michael Cox with *MSU's* Plant and Soil Sciences Department provided guidance in pH measurement. Their efforts were very timely and are owed thanks.

Dennis Boehm of *Hayward Baker* provided industry data and reviewed portions of the work associated with *Hayward Baker* activities. Mr. Boehm's efforts are appreciated. Dr. Kent Newman of the *Engineer Research and Development Center* reviewed select portions of the work and is owed due gratitude.

Numerous *MSU* students assisted with laboratory testing, data reduction, data analysis, and similar tasks and their efforts were essential to the completion of the work. Among them were Ben Cox, Heather Craft, Brittany Gunkel, Trey Jordan, Clay Sanders, Justin Shaw, Katie Strickland, Josh Stubbs, and Tim Woolman.

SYMBOLS

A	activity of clay
A_i	converted cross sectional area of specimen
A_F	attachment adjustment factor
A_o	original cross sectional area of specimen
Amp	Amperage
C-A-S-H	calcium aluminate silicate hydrate
CaO	calcium oxide
CaOH	calcium hydroxide
$CaSO_4$	gypsum
CBR	California Bearing Ratio
CEC	cation exchange capacity
CF	clay fraction or percent clay
CH	high plasticity clay
CKD	cement kiln dust
CL	low plasticity clay
CLSM	controlled low-strength material
CS	calcium sulfoaluminate cement
C_T	total stabilization content as a percentage of unstabilized slurry weight
C_3A	tricalcium aluminate
CSA	calcium sulfoaluminate cement clinker
C-S-H	calcium silicate hydrate
D	cementitious dosage rate referencing pre-treatment volume (kg/m^3)
D_w	cementitious dosage rate referencing pre-treatment and grout volume (kg/m^3)
Dial	Pocket Geotester
DSM	dry soil mixing
E	elastic modulus of stabilized soil slurry (MPa)
F70	70 mm long fiber
F20	20 mm long fiber
F - NF_{ratio}	behavior ratio between fiber reinforced and non-fiber reinforced specimens
FP	fibrillated polypropylene
GGBFS	ground granulated blast furnace slag
GPa	giga Pascal
Group 1	strongest *Soil 1* group
Group 2	intermediate *Soil 1* group
Group 3	weakest *Soil 1* group
G_s	specific gravity of solids
kCl	potassium chloride
kPa	kilo Pascal
LL_{OD}	liquid limit obtained after oven drying same specimen tested prior to drying
LL	liquid limit
LOI	loss on ignition
L_o	original length in axial direction
M	maturity

MH	high plasticity silt
ML	silt
MPa	mega Pascal
M(t)	temperature-time factor accumulated up to time *t* (C-hr)
NC	normal consistency
NP	normal placement protocol
OH	organic clay
OL	organic silt
OMC	optimum moisture content from Proctor compaction test
P	applied force
PFM	pneumatic flow mixing
PI	plasticity index
PL	plastic limit
P_{max}	maximum force during *UC* testing
PoP	Plaster of Paris
ppt	parts per thousand
P_{ult}	force at σ_{ult}
PSD	particle size distribution
PVA	poly(vinyl)
R	hand held gage reading
R^2	coefficient of determination
Ring	Pocket Penetrometer
RP	rapid placement protocol
RPM	revolutions per minute
SAC	semi-adiabatic calorimetry
SB-HB	Standard Blend-Hayward Baker
SC	specialty cement
SEM	scanning electron microscope
SGM	super geo-material
Shear	Pocket Vane Shear Set
SiO_2	silicon dioxide
SP	poorly graded sand
SP-SM	sand with silt
SO_3	sulfur trioxide
SSA	specific surface area
T_a	air or water temperature adjacent to specimen during curing
T_c	curing period
T_i	maturity temperature (C)
T_I	temperature at center of curing slab specimen (C)
T_r	temperature of reference specimen during soil slurry *SAC* test (C)
T_{ref}	temperature of reference specimen during cement paste *SAC* test (C)
T_s	temperature of stabilized soil slurry specimen during *SAC* test (C)
T_{samp}	temperature of cement paste specimen during *SAC* test (C)
T_o	reference temperature (C)
$TS_{\%}$	total solids (solid mass divided by total mass) expressed as a percentage
TSF	ton per square foot

TTF	symbol to denote temperature-time factor (C-hr)
UC	unconfined compression test
V	volt
VBC	Vicksburg buckshot clay
W_{wts}	weight of water in final slurry
W_{sts}	weight of soil solids in final soil slurry
W_t	weight of soil slurry to be mixed
W_{ws}	weight of soil in barrel including residual moisture
W_{wb}	weight of water to be batched or added into the soil slurry
WSM	wet soil mixing
XRD	X-ray diffraction
XRF	X-ray fluorescence
Zone 1	specialty cement considered stronger than control
Zone 2	specialty cement could be stronger than control
Zone 3	specialty cement could be weaker than control
Zone 4	specialty cement considered weaker than control
cov	coefficient of variation
cov_f	coefficient of variation of field mixed material strength in decimal form
n	number of data points
$\mathrm{pH}_{\mathrm{pre\text{-}cement}}$	pH of soil slurry prior to cement addition
$\mathrm{pH}_{\mathrm{post\text{-}cement}}$	pH of cementitiously stabilized soil slurry, i.e. after cement addition
q_u	compressive stress at failure (kg/cm^2)
q_{ud}	unconfined compression design strength
q_{ul}	average unconfined compressive strength from laboratory
s/c	soil to cement ratio on a mass basis
s_{ud}	laboratory mix design shear strength
s_u	shear stress at failure (kg/cm^2)
t	time expressed in hours
t_c	curing temperature
$t_{\Delta T}$	time after cement addition where ΔT_{max} occurs (hr)
$w_{\%}$	moisture content (water mass divided by soil mass) expressed as a percentage
$w_{\%\ mixed}$	moisture content during mixing of select specimens
$w_{as(\%)}$	measured moisture content of soil slurry expressed as a percentage
w_b	moisture content of processed soil contained in barrel in decimal form
w/c	water to portland cement ratio on a mass basis
w/cm	water to total cementitious material ratio on a mass basis
w_{ts}	target moisture content of soil slurry in decimal form
$w_{ts(\%)}$	target moisture content of soil slurry expressed as a percentage
ε	axial strain
ε_{max}	strain at P_{max}
ε_{ult}	strain at σ_{ult}
σ	normal stress
σ_d	deviator stress
σ_1	major principal stress
σ_3	minor principal stress
σ_{max}	stress at P_{max}

σ_{ult}	maximum stress after applying area correction
γ	factor accounting for field strength variability
γ_T	total unit weight of stabilized soil slurry using standard measure (g/cm^3)
γ_w	bulk unit weight considering water and solid weight (g/cm^3)
γ_d	maximum dry density from Proctor compaction test
μm	micrometer
ΔL	length change in axial direction (cm)
ΔpH	$pH_{post-cement} - pH_{pre-cement}$
ΔT	T_I - T_a
ΔT_{avg}	average value of T_I - T_a measured over an extended period (typ. 168 hr)
ΔT_{stdev}	standard deviation of T_I - T_a measured over an extended period (typ. 168 hr)
ΔT_{max}	maximum value of T_I - T_a measured over an extended period (typ. 168 hr)
λ	factor accounting for difference between laboratory and field strength
τ	shear stress
ρ_s	density of a stabilized slab or an unconfined compression specimen (g/cm^3)

ACRONYNMS

DHS	Department of Homeland Security
SERRI	Southeast Region Research Initiative
ORNL	Oak Ridge National Laboratory
CEE	Civil and Environmental Engineering
MSU	Mississippi State University
NRF	National Response Framework
NIMS	National Incident Management System
HLT	Hurricane Liaison Team
ICS	Incident Command System
EOC	Emergency Operations Center
ESF	Emergency Support Functions
FEMA	Federal Emergency Management Agency
USACE	United States Army Corps of Engineers
DoD	Department of Defense
CIKR	Critical Infrastructure and Key Resources
ASTM	American Society for Testing and Materials
HBI	Hayward Baker Incorporated
FDOT	Florida Department of Transportation
IHNC	Inner Harbor Navigation Channel
USCS	Unified Soil Classification System
AASHTO	American Association of State Highway and Transportation Officials

EXECUTIVE SUMMARY

The research discussed in this report was undertaken to develop an emergency construction material for use in a disaster due to flooding. Approximately 3,300 unconfined compression tests were performed alongside approximately 5,500 readings with each of 3 hand held gages. In total, nearly 20,000 strength readings were taken. The majority of the testing was performed on 3 soils at 3 moisture contents using 14 stabilization materials. This report revealed many suitable attributes that could be immediately useful in disaster recovery. Other techniques in this report are ready for a full scale demonstration, and provided it is successful, they should be used in a disaster environment.

A primary objective of this report was to develop strength, modulus, and ductility trends for a variety of soil types, cementitious materials, cementitious material contents, and moisture contents. Another primary objective of this report was to test specialty cements (either specialty grind portland cements or specialty blended calcium sulfoaluminate cements) and compare their characteristics to commercially and readily available products in the cements market with the intention of achieving better properties with the specialty cements.

Conventional lower moisture content uses of cementitiously stabilized soils typically have shear strengths of 1.5 to 10 kg/cm^2. Very high moisture content blends were capable of producing strengths comparable to conventional materials. Commercially available and specialty grind portland cements were the most universally applicable stabilization material. Specialty grind portland cements produced specifically for this research were useful in many applications. Calcium sulfoaluminate cements were more applicable for the higher end of moisture contents tested at dosages of 15 to 20% in relatively low organic content soils with a moderate liquid limit. Ground-granulated blast furnace slag could prove useful for applications with high strength requirements after a few days of curing with some soils but not others. Combining portland cement and fibers improves ductility tremendously.

The overall conclusion of the research is that the high moisture content cementitiously stabilized slurries are a viable emergency construction material for use on a short-term basis. The overall recommendation of this research is to use high moisture content cementitiously stabilized slurries as an emergency construction material on a short-term basis. The report provides results of all testing alongside design guidance. The emergency construction material developed in this report is intended to be used with the construction guidance provided in *SERRI Report 70015-008* performed under the same task order by the same principal investigator.

CHAPTER 1 - INTRODUCTION

1.1 General and Background Information

The work presented in this report was developed in partial fulfillment of the requirements of Task Order 4000064719 sponsored by the *Department of Homeland Security (DHS)* through its *Southeast Region Research Initiative (SERRI)* program administered by *UT-Battelle* at the *Oak Ridge National Laboratory (ORNL)* in Oak Ridge, Tennessee. The research was proposed by members of the *Department of Civil and Environmental Engineering (CEE)* at *Mississippi State University (MSU)* to *SERRI* in a document dated 1 June 2007. The proposed research was authorized by *UT-Battelle* in its task order dated 10 December 2007. This task order included a scope of work defined through joint discussions between *MSU* and *SERRI*. Work on the project was initiated on 1 January 2008. A modification of Task Order 4000064719 was proposed on 9 September 2008 and agreed upon on 29 September 2008. A second Task Order modification dated 22 June 2010 was also performed, which is the Task Order used to generate this report.

The scope of work associated with Task Order 4000064719 included several related components. The general objectives of the project were to investigate means for rapidly using on-site materials and methods in ways that would most effectively enable local communities to rebuild in the wake of a flooding disaster. Within this general framework, several key work components were associated with Task Order 4000064719. Specifically, the scope of work dated 22 June 2010 includes research efforts in the following six task groups:

Task 1: Erosion Control-Erosion Protection for Earthen Levees.

Task 2: Bridge Stability-Lateral & Uplift Stability of Gravity-Supported Bridge Decks.

Task 3: Levee Breach Repair-Closure of Breaches in Flood Protection Systems.

Task 4: Pavement Characterization and Repair.

Task 5: Emergency Construction Material Development-Staging Platform Construction.

Task 6: Fresh Water Reservoir-Restoration of Fresh Water Supplies.

The division of the research effort allowed the work to be broken into manageable portions so that key components could be reported in separate volumes to allow readers to obtain only the work related to their needs. The work contained herein was associated with Task 5. The report of this work was the 6th deliverable of the research project, hence the designation of the report as *SERRI Report 70015-006* of Task Order 4000064719. Work related to Task 5 was also submitted in *SERRI Report 70015-003, SERRI Report 70015-007,* and *SERRI Report 70015-008;* these four reports represent full completion of Task 5.

1.2 Objectives

The general objective of Task Order 4000064719 was to investigate several specific means by which local communities may best use available resources in an effort to rapidly recover from a flooding disaster. In the wake of a flooding disaster, this broad objective would include rebuilding a community with the efforts of a variety of professionals practicing within the physical and social sciences. The research conducted was much more narrowly focused upon certain recovery efforts typically associated with Civil Engineering.

A key component of this research was to develop solutions which may be rapidly deployed to achieve maximum benefit for the community, typically through the use of on-site materials, pre-engineered components, and innovative construction materials and techniques. This research aimed to develop solutions for protecting and/or expeditiously reconstituting critical civil infrastructure components. The research emphasized rapid constructability where existing on-site materials are used to strengthen selected infrastructure components. In this context, the specific objective of the total effort of Task Order 4000064719 was to develop specialty materials and design and construction procedures which may be rapidly deployed to protect and restore selected key civil infrastructure components. Combinations of dredging equipment, small barges, excavating equipment, positive displacement pumps, and soil mixing devices were investigated in terms of their ability to assist in the construction of essential temporary infrastructure out of controlled low strength materials.

When areas are inundated with flood waters up to a few meters deep over an area covering many square kilometers, construction materials will be scarce during early recovery stages. Furthermore, any material of reasonable quality will have many uses, and the supply will almost certainly not meet the demand. A stable platform from which to launch emergency medical operations, modular housing, and supply operations is one example of a need for large material quantities of only modest strength. The objective of this report centers around developing said emergency material with subsequent reports focusing on the potential uses of the material, in particular a staging platform. Instead of importing large quantities of select material (e.g. crushed stone or sand) from remote sites, the overwhelming majority of the volume of the emergency construction material will be acquired by dredging or excavating material from the ground surface existing beneath the water.

1.3 Scope

For the specific research component described in this report (Task 5), the revised scope of work dated 22 June 2010 includes the nine items summarized below. These nine items are the full deliverable of Task 5; this report fully addresses all but items d), g), and i). *SERRI Report 70015-007* fully addresses item d), *SERRI Report 70015-008* fully addresses item g), and *SERRI Report 70015-003* fully addresses item i).

a) Acquire representative material for testing from locations that would be candidates for flooding (e.g. New Orleans and Mobile). The origin of the material will vary from dredging operations to native soils in these types of areas and will be used throughout testing. Where applicable in-situ moisture contents will be obtained to provide a baseline of properties. Large quantities of three soils will be obtained with varying plasticity and organic content.
b) Characterize basic properties of materials. Testing will be performed to measure: 1) Activity (ASTM D 422), 2) Organic Content (ASTM D 2974 or equivalent), 3) Atterberg Limits (ASTM D 4318), 4) Specific Gravity (ASTM D 854), 5) USCS Classification (ASTM D 2487), 6) Particle Size Distribution (ASTM D 422), 7) XRF, and 8) pH.
c) Develop a comprehensive suite of load response properties with time for the soils

described in item a) using bench scale testing. The testing protocol will consist of shear strength testing of prepared stabilized slurry slabs and unconfined compression testing as appropriate. Very thin membranes will also be tested in conjunction with the materials. Both types of testing will be intended to simulate shear strength of the stabilized slurries with time over a period of seven days. The aforementioned test protocol was selected for two reasons. The slab testing method will be developed in a manner that will be applicable to on site responders, which makes it highly desirable. The stabilization materials to be blended with the candidate soils include: 1) *Type I* portland cement from both the major types of cement plants, 2) *Type III* portland cement from both major types of cement plants, 3) commercially available rapid set cement, 4) six specialty cements produced specifically from this research (four by interrupting normal production at both major types of portland cement plants and two blended calcium sulfoaluminate cements), 5) ground granulated blast furnace slag, and 6) two types of polymer fibers. This materials protocol includes 14 different stabilization additives encompassing a wide variety of properties. Development of the specialty cements will be performed using laboratory testing including semi-adiabatic calorimetry.

d) Investigate dewatering equipment and materials for applicability in disaster environments, in particular, to assist in the development of emergency construction materials with secondary emphasis in handling contaminated sediments. The investigation will a focus on the use of polymers for dewatering a soil mass and also investigate geotextile tubes. A test environment will be developed where a series of potentially applicable polymers will be tested (in conjunction with scaled geotextile tubes in some instances as appropriate) to determine if the technology can produce sufficient material at an acceptable moisture content for large scale emergency construction material needs. Moisture content variability conditions will also be investigated in the context of dewatering. The effect of dewatering polymers on shear strength in the presence of multiple cements will also be investigated via slab and unconfined compression techniques.

e) Select cementitious materials investigated in the bench scale study in item c) will be further investigated in a mixing (or blending) study to evaluate the effect of key parameters. Examples of key parameters would be cementitious sulfate content and its effect on shear strength and the effect of blending ground granulated blast furnace slag with portland cement in high moisture content fine grained soils.

f) Test the behavior of multiple cement blends (selected from the 14 original blends previously mentioned) in the presence of brackish water and seawater. Testing will be performed via slab and unconfined compression techniques. The bench and mixing studies only incorporate fresh (tap) water. A final blend will be selected for each soil type and set of conditions at the conclusion of this subtask considering all knowledge gained from subtasks a) to f).

g) Develop design and construction guidance (e.g. identifying suitable applications and providing placement and mixing approach) for using the emergency construction material blends developed at the conclusion of subtask f). Use of the material for the purpose of developing a staging platform will be highlighted. Strength and stiffness of the materials developed will be incorporated into the staging platform guidance (e.g. ability of staging platform to support helicopter loads and/or support freight lowered onto platform from a helicopter).

h) Design and construction procedures using the emergency material will be highly dependent upon the stabilized soil blend achieving a given set of properties with time. For this reason, hand held field shear strength measurement devices will be evaluated statistically for the purpose of assessing risk associated with strength gain measurement over time. (Precision, accuracy, and repeatability are envisioned to be the focus of the assessment.) The results of the hand held gage assessment could be used on site to quantify the impacts of equipment malfunctions, lack of personnel, or other events on the stability of the constructed platform or other structure.

i) Test material obtained from construction site visits in unconfined compression to provide a comparison of the properties of the stabilized blends made from materials obtained in subtask a). It is anticipated that test results will be obtained from three to five sites.

This document (*SERRI Report 70015-006*) is the fundamental concept of Task 5. This report aims to develop an emergency construction material from soil slurry at a given moisture condition. In some applications the soil slurry may require dewatering, which is addressed in *SERRI Report 70015-007*. Once the construction material has been developed, design and construction guidance is provided in *SERRI Report 70015-008*; the example use of a staging platform is the focus. This report includes how Task 5 as a whole fits into disaster recovery (i.e. *National Response Framework*).

1.4 Incorporation Into the *National Response Framework*

The *National Response Framework* (*NRF*) is a document that guides the United States when conducting all-hazards response. (The *NRF* refers to response as immediate actions to save lives, protect property and the environment, and meet basic human needs.) This framework is entailed in NRF (2008), which has complimentary material found in print and online. The *NRF* is a continuation of previous federal level planning documents (e.g. Federal Response Plan of 1992), and serves as the state of the art in responding to disaster events. The following paragraphs summarize how the research conducted in Task 5 could be applicable to the *NRF* and in what manner. The tone of the paragraphs assumes the reader is at least casually familiar with the *NRF* and supporting documentation.

According to NRF (2008), "Resilient communities begin with prepared individuals and depend on the leadership and engagement of local government, nongovernmental organizations, and the private sector." The word "prepared" in the previous sentence is very powerful and could refer to numerous components. The current state of practice in emergency construction after a water based catastrophe is an area where the authors feel the United States is not fully "prepared." To approach a state of readiness where the United States is "prepared" for these events, concepts need to be developed that are studied to reasonable resolution where design methods and materials are developed (primarily laboratory scale and analytical studies). These methods and materials then need to be demonstrated at full scale, and thereafter, training needs to be performed to ensure construction responders can perform the needed tasks. In present day, this level of preparedness does not exist.

The *NRF* is primarily oriented toward implementing nationwide response policy and operational coordination for any domestic event. NRF (2008) focuses on responding to and recovering from incidents that do occur, which is one of four major parts of a larger *National Strategy for Homeland Security*. NRF (2008) states that although some risk may be unavoidable, first responders can effectively anticipate and manage risk through proper training and planning. An entire chapter of NRF (2008) addresses planning. One of the three principal benefits that is listed for planning is "it contributes to unity of effort by providing a common blueprint for activity in the event of an emergency. Planning is a foundational element of preparedness and response and thus is an essential homeland security activity.

Neither training nor planning appears to be performed to any significant extent related to emergency design and construction for the purpose of rapidly repairing civil infrastructure. Training programs that result in certifications to perform certain activities would expedite the selection of qualified groups in the highly time sensitive environment of a disaster. Having known quantities of certified contractors in place capable of effectively using the emergency construction material would likely be needed.

The response structure of NRF (2008) is based on the *National Incident Management System* (*NIMS*). Several key concepts are presented in the *NIMS*. First, leaders and staff are said to require initial and ongoing training on response principles. Second, classifying types of resources is said to be essential to ensure effectiveness. During a crisis it is stated that there will not be time to determine staff qualifications, and that all stakeholders should regularly exercise incident management and response capabilities. A similar system for emergency construction activities using the materials developed herein could prove useful.

The goals of the research conducted under Task Order 4000064719 align with the needs of the *Hurricane Liaison Team (HLT)*, whose goal is to enhance hurricane disaster response. Task 5 is directly aligned with the stated mission of the *HLT*. All the aforementioned discussion also aligns with *Scenario 10: National Disaster-Major Hurricane* of the National Planning Scenarios that have been established in NRF (2008).

Response at the local level is organized within an *Incident Command System* (*ICS*). At the field level local responders use the *ICS*, which is led by an Incident Commander who has overall authority and responsibility at the incident site. An *Emergency Operations Center* (*EOC*) is a physical location established at the incident site. They can be organized by discipline (e.g. transportation), jurisdiction (e.g. city), Emergency Support Function (e.g. engineering), or a combination. A key *EOC* function is to ensure on scene responders have needed resources. The materials and construction methods produced from Task 5 would be needed resources and could be provided through the Incident Commander.

Preparedness is repeatedly stated (directly or indirectly) as an essential precursor to response. The *RESPONSE ACTIONS* chapter of NRF (2008) shows a circular preparedness cycle consisting of the following four categories: 1) plan; 2) organize, train, and equip; 3) exercise; and 4) evaluate and improve. Under the organize category, assembling well-qualified teams of paid and volunteer staff for essential response and recovery tasks is listed. Also under the organize category is discussion of *Pre-Scripted Mission Assignments*. (They are used to assist in planning for and reduction in time necessary to deploy resources and can be tailored for training, development, and to exercise rosters of deployable resources.) These assignments would be needed for the effective use of an emergency construction material. *Advanced Readiness Contracting* is used to ensure contracts are in place before an incident for often needed commodities (a list is provided that does not include construction materials). Advanced contracts would expedite availability of the materials tested in this report.

Under the *RESPOND* heading of the *RESPONSE ACTIONS* chapter of NRF (2008), the process of response is broken into three categories: 1) gain and maintain situational awareness; 2) activate and deploy resources and capabilities; and 3) coordinate response actions. Providing the right information at the right time is critical to gaining and maintaining situational awareness. With regard to activating and deploying resources, the text in the following paragraph is included in NRF (2008).

"**Identifying needs and pre-positioning resources**. When planning for heightened threats or in anticipation of large-scale incidents, local or tribal jurisdictions, states, or the Federal Government should anticipate resources and capabilities that may be needed. Based on asset availability, resources should be pre-positioned and resource teams and other support resources may be placed on alert or deployed to a staging area. As noted above, mobilization and deployment will be most effective when supported by planning that includes pre-scripted mission assignments, advance readiness contracting, and staged resources." This level of detail would be appropriate for the methods of this report, but currently they are not in place.

As stated in NRF (2008), the emphasis on response will gradually transition to an emphasis on recovery. Short-term recovery is defined as immediate, overlaps with response, and lasts up to a few weeks. The majority of this research is on short-term recovery.

Fifteen Emergency Support Functions (ESF's) have been established under *FEMA*. *ESF #3-Public Works and Engineering* is applicable to this report. (*USACE* acts as a primary coordinator of *ESF #3*.) *ESF #3* includes conducting post-incident public works and infrastructure assessments, providing technical and engineering expertise including the repair of damaged public infrastructure, and construction management.

State, tribal, and local governments are responsible for their public works and infrastructure. Private sector entities, though, either own or operate a significant portion of the nation's infrastructure. *DHS/FEMA* are the leads for *ESF #3* recovery resources, which includes assistance under the *Stafford Act Public Assistance Program*. The *USACE* and *DOD* are *ESF #3* coordinators, and are the primary response agencies. Response and short-term recovery overlap in very early stages; thereafter, recovery is an extension of response.

Responsibility to respond to natural events is initiated at the local level, particularly with elected officials. Key responsibilities of these officials include: establishing relationships with vital public and private sector entities; training with local partners in advance of an incident; and leading and encouraging local leaders to focus on preparedness by participating in planning, training, and exercises. With regard to coordinating response actions, catastrophic events with little to no notice are a precedent for state and federal governments to take proactive measures to mobilize/deploy assets in anticipation of formal assistance requests. During this period, specialty cement production could begin alongside preparation to mobilize construction equipment (extent would depend on the situation).

As stated in NRF (2008), government works with private sector groups as partners in emergency management; an example is businesses involved in civil infrastructure. *Critical Infrastructure and Key Resources* (*CIKR*) are grouped into 17 sections that provide essential functions and services. The research team consisted of many private sector groups to ensure they were adequately represented and involved in the research. Task 5 involved members of the private sector (e.g. *Ciba*, *CTS Cement*, *Hayward Baker*, and *Holcim Cement*).

CHAPTER 2-LITERATURE AND PRACTICE REVIEW

2.1 Overview of Literature and Practice Review

A review of literature and current practice was performed to: 1) identify useful information for conducting the experimental portion of the research; 2) identify commercially available stabilization materials; 3) assess pertinent properties of unstabilized soils; 4) assess properties of stabilized soils (primarily uncompacted soils or stabilized soil slurries) found in either literature or current practice; and 5) investigate portable soil strength measurement techniques. Other information found that was pertinent to the research was also summarized. The information is provided in the remainder of this chapter.

2.2 Stabilization Materials

A review of commercially available stabilization materials was performed with emphasis on materials that would be readily available. A variety of specialty application materials have been produced that are not discussed in this section as availability after a disaster would be questionable. Commercially available stabilization materials that are routinely used are summarized in the remainder of this section.

2.2.1 Portland Cement

Table 2.1 contains terminology used by cement chemists related to clinker. Clinker typically makes up approximately 90% of the mass of portland cement (Kosmatka et al. 2006). The remaining material usually consists of a calcium sulfate source and grinding aids that are added after clinker leaves the kiln. Sulfur trioxide (SO_3) is, in general, the compound of primary interest within the calcium sulfate.

Table 2.1. Cement Chemistry Terminology Related to Clinker

Shorthand	Chemical Formula	Compound's Existence
A	Al_2O_3	Pre Kiln
C	CaO	
F	Fe_2O_3	
H	H_2O	
M	MgO	
S	SiO_2	
C_3S	$3CaO{\cdot}SiO_2$	Developed in Kiln
C_2S	$2CaO{\cdot}SiO_2$	
C_3A	$3CaO{\cdot}Al_2O_3$	
C_4AF	$4CaO{\cdot}Al_2O_3{\cdot}Fe_2O_3$	

Calcium sulfate exists primarily in three forms depending on the extent water molecules are attached; in general, the more water molecules attached the more water soluble the form of calcium sulfate. The forms of calcium sulfate in order of water solubility beginning with the highest are: 1) dihydrate with chemical formula $CaO{\cdot}SO_3{\cdot}2H_2O$; 2) hemihydrate with chemical formula $CaO\cdot SO_3\cdot 0.5H_2O$; and 3) anhydrite with chemical formula $CaO\cdot SO_3$. There are no pure sources of calcium sulfate. Most sources contain more than one of the forms listed previously, or contain other forms in some cases. Gypsum is the predominant source of calcium sulfate used in portland cement, though gypsum refers to a material with a range of properties that can vary considerably. Most gypsum sources used in the manufacture of portland cement would contain 55 to 75% dihydrate, though anhydrite is the predominant form in some gypsum sources used in portland cement. Sources with high anhydrite levels are used to control the false set of portland cement where excessive hemihydrate calcium sulfate is present. Total SO_3 content using raw materials at a given cement plant cannot necessarily be directly compared to total SO_3 content using different raw materials at another cement plant, as solubility of the materials can differ.

Portland cement is normally governed by *ASTM C 150;* eight types are identified therein. Table 2.2 summarizes properties of the most commonly used portland cement (*Type I*) alongside properties of the most rapid strength gaining portland cement (*Type III*). As seen, the mean chemistries are similar. The primary difference in *Type I* and *Type III* cements is the final grinding size. The Blaine Fineness (*ASTM C 204*) of *Type I* cement is typically on the order of 370 m^2/kg, while *Type III* cement is on the order of 550 m^2/kg.

Table 2.2. Chemical Composition of Portland Cements from Kosmatka et al. (2006)

	Chemical Composition (%) range over mean						**Compound Composition (%)**			
Type	SiO_2	Al_2O_3	Fe_2O_3	***CaO***	***MgO***	SO_3	C_3S	C_2S	C_3A	C_4AF
I	18.7-22.0	4.7-6.3	1.6-4.4	60.6-66.3	0.7-4.2	1.8-4.6	40-63	9-31	6-14	5-13
	20.5	*5.4*	*2.6*	*63.9*	*2.1*	*3.0*	*54*	*18*	*10*	*8*
III	18.6-22.2	2.8-6.3	1.3-4.9	60.6-65.9	0.6-4.6	2.5-4.6	46-71	4-27	0-13	4-14
	20.6	*4.9*	*2.8*	*63.4*	2.2	*3.5*	*55*	*17*	*9*	*8*

SO_3 is of key interest to this research. Cost (2006) provides a detailed description of incompatibility for conventional concrete applications, and a summary of that work alongside other information provided by the author is provided in the remainder of this section. Incompatibility of materials becomes more common in concrete with complex mixtures and can result in poor early strength performance.

Under sulfated cements have been cited as a major contributor to incompatibility, especially if certain admixtures are present, supplementary cementitious materials (e.g. *Class C* fly ash) are used, and/or if the material is placed in hot weather. (Sulfate solubility increases as temperature decreases.) SO_3 helps control cement set time by reacting with clinker aluminate compounds (especially C_3A, tricalcium aluminate) to produce certain byproducts that coat calcium silicate particles in solution and delay their hydration for a few hours. All three of the aforementioned factors tend to accelerate C_3A hydration, thus requiring more SO_3 for proper control as C_3A can inhibit *C-S-H* hydration and result in low early strengths. Aluminate reactions can result in flash set. SO_3 content is not usually highly sensitive, as long as it is adequate for control of the initial C_3A hydration. Should it be inadequate, though, resulting strength development can be very poor and setting performance may become abnormal as well. Incompatibility can be close to occurring without showing signs of abnormal behavior; a small change in conditions can lead to considerably different results. While too little SO_3 can lead to incompatibility, too much can lead to expansion. These factors have cumulatively resulted in market cements that contain higher levels of calcium sulfate (especially the more soluble phases) than would be needed for optimum performance of cement-only mortar, paste, or slurry, especially at mild or cool temperatures.

Cost (2006) performed paste calorimetry and tested mortar cubes to investigate incompatibility effects. The research focused on materials that have performed well with other combinations of materials, but suffered severely abnormal setting tendencies and poor early strength development in other conditions. For normal hydration kinetics, the aluminate hydration effects of interest are captured during the first few minutes, and thereafter the effects moderate as sulfates and aluminates interact. Dormancy for a few hours typically occurs next followed by hydration of calcium silicates. Thermal profile curves of the abnormal strength specimens were compared to normal thermal profile curves to diagnose the cause of the low early strengths. A reverse approach could be applied to reduce SO_3 levels below those typically used in cements optimized for general use for disaster applications where early set time and early strength are of paramount importance.

2.2.2 Ground Granulated Blast Furnace Slag (*GGBFS*)

Ground Granulated Blast Furnace Slag *(GGBFS)*, aka slag cement, is classified by *ASTM C 989*. Grade 120 is the highest level of reactivity. *GGBFS* tends to retard concrete setting time. *GGBFS* is used in soil stabilization (especially organic clays) in combination with portland cement to improve performance. *GGBFS* has combined hydraulic and pozzolanic properties, and is available in most of the US except parts of the west. *ASTM C 989* reference cement has considerable leeway for alkali content and strength; the grading system does not capture set time and early age strength gain. Of significantly more importance is the granule chemistry (affected substantially by the source of iron ore).

2.2.3 Calcium Sulfoaluminate Cements

Calcium sulfoaluminate (*CS*) cements are typically used with aggregate to achieve compressive strengths in excess of 14 MPa within two hours. They are available in bulk. *CS* cements are somewhat similar to portland cement in terms of mineralogical composition.

2.2.4 Discrete Polymer Fibers

Fiber reinforcement is used primarily in concrete and to a lesser extent for soil stabilization. Typical fiber lengths are 25 to 100 mm (Koerner 1998). Denier is defined as the mass in grams of 9,000 m of fiber, with smaller deniers being smaller in diameter. Fibers differ in terms of raw materials (polypropylene most common) and manufacturing (fibrillated and monofilament). Fibrillated fibers are relatively flat and rectangular, and they open into net, grid, and fiber configurations. Monofilament fibers are cylindrical, solid, and continuous. Some fibers are packaged in small bags (e.g. 0.23 to 0.46 kg) that can be thrown into a concrete drum or pugmill for five minutes of mixing as bags dissolve.

2.2.5 Bitumen

Generally, soils suitable for bituminous stabilization are silty sand and granular materials. The coating of particles is key to successful bituminous stabilization. Bituminous stabilization is accepted as a candidate, in general, for materials with a *PI* less than 10 and less than 30% fines. The soils and conditions investigated in this report would not lend themselves well to stabilization with bitumen materials.

2.2.6 Pozzolans

ASTM C 618 describes commonly used pozzolans *Class C* and *Class F* fly ash. *Class F* is predominant in the eastern US, while *Class C* is available in most of the US. *Class C* can contain up to 30% *CaO*, and as little as 2% carbon (Kosmatka et al. 2006). *Class F* is almost a pure pozzolan, but its properties do not lend themselves well to the current application. Fly ash also tends to retard the setting time in standard concrete operations.

Many pozzolanic materials are used in practice to reduce water requirements, improve workability, control segregation, control air content, and reduce total mixture heat. None of these behaviors are of particular interest to the current research. Pozzolan materials with rapid set properties are rare and cannot be incorporated into the recipe philosophy of this project. Beneficiated pozzolans (e.g. beneficiated *Class F* fly ash) could theoretically be an option but are not commonplace.

Silica fume is another commercially available pozzolan that is very reactive; it is governed by *ASTM C 1240*. It is a bi-product of producing silicon metal or ferrosilicon alloys that is commonly supplied in liquid form and aids in early strength development of other products. Silica fume is usually available (moderate quantities in many instances).

2.2.7 Silicate Grouts

Silicate grouts are primarily used in large scale construction for extreme cases when rapid set (set in this case refers to formation of a gel rather than a hardened product) is necessary due to moving water requiring a quick gel time (≈ 10 min). When used, sodium silicate grouts complement cement slurries and are handled as slurry. Silicate grouts are nationally available, non-hazardous, and safe, but do not work well in dry pneumatic lines. A common practice is to pump cement and sodium silicate grouts separately (requires two different sets of equipment) and introduce the products at the final destination.

2.3 Practice Review

All information in this section was obtained from Hayward Baker Inc (*HBI*) and pertains to their practices. Soils encountered, stabilization materials/quantities used, and laboratory practices are presented. Soil mixing as described in this section, in general, is not performed on soil slurries; rather, it is performed on in situ masses or columns of soil. Soil mixing is conducted by adding cementitious material via: 1) dry addition or dry soil mixing (*DSM*); and 2) wet addition in the form of grout referred to as wet soil mixing (*WSM*).

2.3.1 Materials Used

As of April 2009 estimated soil conditions encountered along the Gulf Coast were: sand 5% of the time; peat 15% of the time; lean clay with various organic contents 20% of the time; and fat clay with various organic contents 60% of the time. (Some peat may be present.) Approximately 80% of the jobs use approximately 75% *GGBFS* and 25% *Type I* portland cement, while approximately 20% use exclusively *Type I* portland cement. Fly ash and lime have been used in past work but are not typically used in current practice.

Soil type can have an effect on strength, with the sand, silt, clay, and organic contents used as primary strength indicators in current practice. In general, sand has the highest strength and peat has the lowest strength when mixed with various stabilization agents. Portland cement and clay particles (smaller than 2 μm) are not typically considered to react well with one another in many cases; organic clays in particular. Increases in clay content are often associated with decreases in strength if all other variables are held constant and portland cement is the stabilizing agent. *GGBFS* and portland cement are blended to improve performance of high clay content, especially organic, soils. *GGBFS* and portland cement are commonly used with fat clay, organic silts, and organic clays. *Type I* portland cement is typically used to stabilize *CL*, *ML* (no organics), sand, and granular material.

2.3.2 Dosage Rates

DSM dosage rates are expressed as the mass of cementitious binder per volume of in situ material (soil, air, water) to be stabilized. *WSM* dosage rates are expressed as the mass of cementitious binder per sum of volume of in situ material plus volume of grout (soil, air, water, and grout). Dosage rate (D) represents *DSM* (or any approach with pre-treated volume as a reference) while D_w represents the dosage rate for *WSM*; both have kg/m^3 units.

The lowest practical dosage rate used by *HBI* was estimated at 100 kg/m^3 for essentially all operations. Soil mixing in Sweden has been performed with lower dosage rates on the order of 60 to 80 kg/m^3. Large dosage rates (e.g. 400 kg/m^3) can and have been used in soil mixing applications.

2.3.3 Laboratory Test Methods

Standard laboratory test methods have been used for several years, in general, by soil mixing contractors that typically provide strengths that can be correlated to field conditions. Laboratory bench scale testing protocols of most soil mixing contractors rely on *UC* testing. Recommendations provided by the *Swedish Geotechnical Society* are often incorporated into test methods.

HBI acquires samples from field projects and sends them to externally contracted laboratories for evaluation. The material is sent in sealed 18.9 liter plastic containers and is not dried prior to testing. Laboratory personnel determine Atterberg limits; classify the soil; determine organic content; and test material properties with the 3 cementitious dosage rates of low, medium, and high. Cementitious blends are developed with the aforementioned properties and field experience. The samples are: 1) mixed at in situ moisture and density with cementitious materials added (either dry cementitious material or grout); 2) fabricated into 75 mm diameter by 150 mm tall specimens; 3) cured in 100% humidity; and 4) tested in unconfined compression.

Mixing energy is a key parameter where *HBI* has adjusted protocols over time to better replicate observed in situ properties. In general, the longer material is mixed (i.e. higher mixing energy), the higher the strength. Wet soil mixing does not provide as much mixing energy as dry soil mixing. The wet soil and cementitious materials are mixed with a

Toastmaster GP620B (or equivalent industrial mixer) using a dough hook (Figure 2.1 shows common attachments) within an 11.3 liter mixing bowl on low speed for 3.5 minutes. After the mixing period, the material is further blended by hand for a brief period. Revolutions per minute (RPM's) are not considered to be a significant parameter within a reasonable range (e.g. low speed from one industrial mixer to another).

Figure 2.1. Attachments for Industrial Mixers

Fabrication steps used by Sehn (1998) are provided as a guide. When in situ moisture exceeded the *LL*, the material was placed into the mold in several increments; each increment placed an ellipsoidal mass along the inside of the mold that was moved into place by heavy tamping of the mold on the table top. Approximately 5 to 10 increments were used to fill each mold. When in situ moisture was less than the *LL*, the material was rodded into the molds using a blunt-ended shaft with a diameter on the order of 37 mm. All stabilized soil specimens were tested according to *ASTM C 39-94* at a loading rate of 0.51 mm/min after being capped immediately prior to testing using a quick set hydraulic cement.

2.3.4 Laboratory Test Data

Tables 2.3 and 2.4 provide laboratory test data from *DSM* and *WSM* applications in multiple states across the US. The data was obtained with the methods discussed previously in this section. A variety of materials and dosage rates were included. The data from Mississippi (MS) provided in Table 2.4 was taken from Sehn (1998). Therein, 18 grout-soil combinations were tested with coastal materials for a project to stabilize 8,750 m^2 in plan view up to variable depths reaching as far as 15 m. The grout was prepared with a laboratory scale grout mixer, with select combinations provided in Table 2.5.

The *SC* soil had in situ moisture in excess of its *LL*, which resulted in relatively small in situ stiffness that reduced the need for additional water from the grout to facilitate mixing. The pattern of strength of the *SC* stabilized soils and the neat grouts were very similar; strength decreased with increased moisture.

Table 2.3. ***DSM*** **Data From** ***HBI*** **Projects**

Soil	State	γ_w (g/cm^3)	$w_{\%}$ (%)	D (kg/m^3)	Additive[a]	Time (hr)	s_u (kg/cm^2)
CH	LA	1.44	80	125	*Type I* (25) *GGBFS* (75)	72 168	--- 1.70
CH	LA	1.44	80	150	*Type I* (25) *GGBFS* (75)	72 168	--- 2.54
CH	LA	1.44	80	175	*Type I* (25) *GGBFS* (75)	72 168	--- 2.89
Sandy Peat	FL	---	---	8[b]	*Type I* (100)	72 168	0.71 1.09
Sandy Peat	FL	---	---	12[b]	*Type I* (100)	72 168	4.12 6.09
Sandy Peat	FL	---	---	15[b]	*Type I* (100)	72 168	5.35 7.75
ML to MH[c]	FL		62 to 78	100 to 250	*Type I* (25) *GGBFS* (75)	72 168	0.56 to 1.30 0.63 to 1.41
Peat	LA	---	600	350	*Type I* (25) *GGBFS* (75)	72 168	1.41 1.80
Peat	LA	---	600	400	*Type I* (25) *GGBFS* (75)	72 168	1.76 2.54
Peat	LA	---	600	175	*Type I* (25) *GGBFS* (75)	72 168	--- 0.46
OH	LA	---	80	150	*Type I* (25) *GGBFS* (75)	72 168	--- 0.47
OH	LA	---	80	175	*Type I* (25) *GGBFS* (75)	72 168	--- 0.77
OH	LA	---	80	200	*Type I* (25) *GGBFS* (75)	72 168	0.95 2.01
CL	LA	---	---	150	*Type I* (25) *GGBFS* (75)	72 168	0.46 0.47
CL	LA	---	---	175	*Type I* (25) *GGBFS* (75)	72 168	1.09 1.83
CL	LA	---	---	200	*Type I* (25) *GGBFS* (75)	72 168	1.23 1.34

a: Cementitious material and its respective percentage of the total dosage rate.
b: Percent binder to dry unit weight.
c: LL ranged from non-plastic to 65, PL ranged from non-plastic to 52, fines content was 78 to 93%, organic contents from 18 to 44%, pH of approximately 6, and this soil was noted to be somewhat rare.

For the *CL* soil, neat grout strength (Table 2.5) and stabilized soil strength were reported to be somewhat erratic. The *CL* soil had in situ moisture below its liquid limit. Erratic behavior was attributed to the soil stiffness and the influence of water in the grout that was believed to contribute to mixing effectiveness. Keeping the cementitious material constant per unit of in situ bulk unit weight (γ_w) while increasing the *w/cm* ratio of the neat grout resulted in more total water to soften the soil and facilitate mixing. This is a key observation for the current research since it indicates that increased mixing effectiveness due to increased water can outweigh the negative influence of additional water in some cases.

Table 2.4. *WSM* Data From *HBI* Projects

Soil	State[b]	γ_w (g/cm^3)	$w_{\%}$ (%)	D_w (kg/m^3)	Additive[a]	Time (hr)	s_u (kg/cm^2)
CL	LA	1.60	58	166	*Type I* (25)	72	0.46
					GGBFS (75)	168	2.73
CL	LA	1.60	58	192	*Type I* (25)	72	1.18
					GGBFS (75)	168	4.01
CL	LA	1.60	58	215	*Type I* (25)	72	2.83
					GGBFS (75)	168	4.61
Peat	LA	0.96	218	192	*Type I* (25)	72	---
					GGBFS (75)	168	1.61
CL[c]	MS	2.08	21	163	*Type I* (25)	72	1.55
					Fly ash-F (75)	168	2.18
CL[c]	MS	2.08	21	224	*Type I* (25)	72	2.15
					Fly ash-F (75)	168	2.85
CL[c]	MS	2.08	21	146	*Type I* (25)	72	4.79
					GGBFS (75)	168	7.39
CL[c]	MS	2.08	21	131	*Type I* (25)	72	3.10
					GGBFS (75)	168	3.63
CL[c]	MS	2.08	21	176	*Type I* (25)	72	5.35
					GGBFS (75)	168	9.51
CL[c]	MS	2.08	21	166	*Type I* (25)	72	4.89
					GGBFS (75)	168	7.78
SC[d]	MS	1.81	34	163	*Type I* (25)	72	0.63
					Fly ash-F (75)	168	0.99
SC[d]	MS	1.81	34	224	*Type I* (25)	72	1.06
					Fly ash-F (75)	168	1.69
SC[d]	MS	1.81	34	146	*Type I* (25)	72	4.72
					GGBFS (75)	168	8.91
SC[d]	MS	1.81	34	131	*Type I* (25)	72	3.94
					GGBFS (75)	168	8.13
SC[d]	MS	1.81	34	176	*Type I* (25)	72	3.31
					GGBFS (75)	168	6.55
SC[d]	MS	1.81	34	166	*Type I* (25)	72	2.47
					GGBFS (75)	168	5.35

a: Additive and its respective percent of the total dosage rate.
b: All data from MS was from Sehn (1998). Field specimens taken resulted in average 28 day shear strengths on the order of 6 kg/cm^2.
c: The soil had a LL of 28, 53% fines, and 24% clay.
d: The soil had a LL of 31, 46% fines, and 18% clay.

Table 2.5. Candidate Grouts for Use in Soil Mixing in Mississippi (Sehn 1998)

w/cm[a]	0.47	0.57	0.94	1.24	1.55
Grout[b]	**Fly ash-F[c]**	***GGBFS*-120**	***GGBFS*-120**	***GGBFS*-120**	***GGBFS*-120**
s_u	kg/cm^2	kg/cm^2	kg/cm^2	kg/cm^2	kg/cm^2
72 hr	8.73	52.68	11.62	9.01	6.69
168 hr	13.24	98.59	28.94	15.70	11.62

a: Water to cementitious ratio as mixed.
b: 75% of the material shown blended with 25% Type I portland cement.
c: Data shown are the average of 2 sets of testing.

2.4 Controlled Low-Strength Material (*CLSM*)

The 2005 edition of the *ACI Manual of Concrete Practice* section 229R-10 discusses controlled low-strength material (*CLSM*) and provides estimates of flowable fill batch quantities. Table 2.6 provides mix proportions used in various US states incorporating only fine aggregate. Flowable fill and similar materials with sand are established technologies.

Table 2.6. Select Flowable Fill Mixture Proportions from 229R-10

Cement (% by wt)	Fly ash (% by weight)	Aggregate (% by wt)	Water (% by wt)	*w* (%)	*w/cm* (---)
2.8	8.4	72.5	16.3	23	1.46
1.6	8.8	76.0	13.6	18	1.31
1.4	6.7	78.4	13.5	17	1.67
2.7	6.7	77.0	13.6	18	1.43

CLSM is a product used for applications such as backfill that according to ACI 229R-99 and ASTM D 4832-02 has a shear strength (s_u) less than 42.5 kg/cm^2. ASTM D 4832-02 indicates typical shear strengths are 1.75 to 3.5 kg/cm^2 at 28 days. Conventional *CLSM* can contain up to 25% non-plastic or slightly plastic fines. The presence of fines can keep sands in suspension for easier flowability. Low shear strength materials (up to approximately 5 kg/cm^2) can be excavated using heavy equipment, which is beneficial in some applications (disaster recovery being one). *CLSM* investigated by Tripathi et al. (2004) was deemed ready for load between 44 and 92 hr(+).

2.5 Soil Properties

Terzaghi et al. (1996) described clay materials with the following categories: *Very Soft* clay has an s_u of 0 to 0.13 kg/cm^2; *Soft* clay has an s_u of 0.13 to 0.25 kg/cm^2; *Medium* clay has an s_u of 0.25 to 0.5 kg/cm^2; *Stiff* clay has an s_u of 0.5 to 1.0 kg/cm^2; *Very Stiff* clay has an s_u of 1.0 to 2.0 kg/cm^2; and *Hard* clay has an s_u in excess of 2.0 kg/cm^2. *Hard* clay is an excellent bearing material. A Terzaghi rule of thumb is that the shear strength of a soil near the liquid limit is typically on the order of 0.03 kg/cm^2. Landfill shear strength disposal requirements are typically on the order of 0.10 to 0.20 kg/cm^2. Low ground pressure heavy equipment typically requires supporting soil with a shear strength of at least 0.20 kg/cm^2. Note 1 ton per square foot (TSF) is equivalent to 0.98 kg/cm^2.

The pH of soils is low in many cases; pH of 9 has been said to be a notable threshold for soil stabilization. Water has a pH of approximately 7, acids have a pH less than 7, and bases have a pH greater than 7. The scale is logarithmic, which means there is a difference of 100 in a pH of 7 and a pH of 9.

Peats and organic soils routinely have in situ moisture contents noticeably higher than corresponding inorganic materials. Edil and Wang (2000) tested amorphous peats, fibrous peats, and organic clays. Organic clays were classified as having 25% or less organic content, while peats were between 31 to 95% organic. G_s of the organic soil was 2.29 to 2.63, while G_s of the peats ranged from 1.40 to 2.23. Review of the strength data showed a shift in behavior as organic content increased beyond a 20 to 25% threshold. Therefore, the upper limit of organic content of soils (often clays) was proposed to be 20 to 25%.

Skempton (1953) developed the concept known as *Activity* of clays. The concept is shown in Eq. 2.1. Skempton (1953) tested 4 soil deposits and observed an approximately linear relationship to develop Eq. 2.1. Corresponding qualitative observations of *Activity (A)* were: 1) 0.75 or less is *inactive*; 2) 0.75 to 1.25 is *normal*; and 3) greater than 1.25 is *active*.

$$A = \frac{PI}{CF} \tag{2.1}$$

Where,

A = Activity
PI = Plasticity Index (%)
CF = clay fraction, or percent clay; particles smaller than 2 μm

Specific Surface Area (*SSA*) and Cation Exchange Capacity *(CEC)* are fundamental soil properties believed by some researchers to be more appropriate than (A) in some conditions (e.g. Cerato and Lutenegger 2005). Table 2.7 summarizes activities of the most common clay minerals.

Table 2.7. Activity of Common Clay Minerals

Mineral	Activity
Kaolinite	< 0.5
Illite	0.9
Montmorillonite-Ca	1.5
Montmorillonite-Na	7.2

Howard (2006) tested a construction quality soil in eastern Arkansas near the *Mississippi River* that would have similar characteristics of some fine grained soils in high risk flood zones. In general, the material classified as a *CH*. Additional results of testing have been provided as a reference of typical variability of fine grained soils and are: 1) *LL* of 55 to 73; 2) *PL* of 17 to 22; 3) *PI* of 38 to 54; 4) G_s of 2.67 to 2.72; 5) Percent Fines of 77 to 92; 6) Percent Clay of 34 to 50; and 7) Activity of 0.86 to 1.19 (normally active).

2.6 Portable Soil Strength Measurement Techniques

Portable shear strength measurement devices have been in existence for many years. Hand held gages, for example pocket penetrometer and miniature vane shear devices, are portable shear strength measurement devices that are lightweight and easily handled. Hand held gages historically have been most commonly employed to evaluate the consistency of unstabilized cohesive soils. FDOT (2000) indicates miniature vane shear (torvane) and pocket penetrometer tests should only be used as an index of s_u for clay samples.

Test methods for penetrometers and different types of shear devices are in various stages of development. As of April 2010, ASTM committee D18 had an open work item (WK27337) involving evaluation of the consistency and appropriate *UC* strength of soils using a pocket penetrometer. No standards are currently in place in this regard. Standards exist for some portable shear measurement devices, but no standard was found regarding the use of a hand held and hand operated miniature vane shear device. *ASTM D 2573* describes a test method for field vane shear testing in cohesive soils, while *ASTM D 4648* describes a laboratory miniature vane shear test for soft to stiff saturated fine-grained clayey soils with an undrained shear strength of less than 1 kg/cm^2. The device is motorized and has a 4 vane head with diameter to height aspect ratios of 1:1 and 1:2.

Literature documents the use of hand held gages, though specific details of how the gages were used alongside calibration of readings using widely accepted shear strength measurement techniques for the type of material under investigation are not prevalent. Emery (1980) and MacKay and Emery (1994) tested stabilized lake bottom sediments with a *Soiltest CL-700* penetrometer, while Vaghar et al. (1997) tested stabilized harbor bottom sediments/ organic deposits with a torvane shear and pocket penetrometer; none of the studies provided specific details related to calibration and use of the gages.

A model H-60 *Geonor Inspection Vane Tester* was used by Przewlocki (2000) to evaluate the 2D random field variability of clay shear strength. The H-60 is equipped with 3 vanes allowing shear strengths up to approximately 0.6, 1.3, and 2.6 kg/cm^2 to be measured. Kelly and Diethem (1996) used a pocket penetrometer to test stabilized sludge containing oil and grease after curing for 7, 14, or 28 days at ambient temperature in a sealed container. Testing was also performed via unconfined compression as per *ASTM D 2166*. Cementitious additives incorporated were lime, fly ash, and *Type II* portland cement; additive contents were over 50% of the total sludge weight in some instances and were over 100% of the total sludge weight in other instances. No performance issues related to the penetrometer were provided in either of these studies.

Gassman et al. (2001) used a pocket penetrometer to determine time of set of *CLSM*. Tripathi et al. (2004) compared the pocket penetrometer and torvane shear measurements for *CLSM* to Kelly Ball testing according to *ASTM D 6024* for estimation of bearing capacity. *UC* testing via *ASTM D 4832* in 75 by 150 mm cylinders was also performed. Specimens were produced at *w/cm* ratios of 0.69 to 1.14 containing 67.9 to 69.4% sand that used mostly fly ash with small amounts of portland cement as the cementitious blend. Shear strengths were on the order of 0.5 kg/cm^2 at 72 hr and up to 1.7 kg/cm^2 at 168 hr. The penetrometer could measure readings allowing shear strength calculations up to 2 kg/cm^2, and the torvane could measure shear strengths up to 1 kg/cm^2. More scatter was reported with the torvane than the penetrometer, but few other details were reported.

2.7 Soil Stabilization Using Fibers

2.7.1 Soils Stabilized with Fibers Only

Fibers are often used for soil stabilization without chemical additives in non-cohesive soils such as sands or non-plastic silts, but have also been used in cohesive silts and clays. A number of laboratory studies have been conducted and have shown strength increases as a result of randomly oriented polymer fiber inclusions for both cohesive and non-cohesive compacted soils; increases were observed in the cohesion and angle of internal friction properties. The general consensus is that there is an optimum (or limiting) fiber content (typically referenced to as dry soil weight) for a given set of conditions that varies with soil type and application.

Gray and Ohashi (1983), Ranjan et al. (1996), and Santoni et al. (2001), reported strength increases (up to limiting value) in sand as the fiber content increased. In a similar fashion, Puppala and Musenda (2000) performed unconfined compression tests on expansive clays with a *PI* of 46 to 55 and observed strengths of 113 to 144% in excess of control specimens. The stiffness of the clays was essentially unaffected. Fletcher and Humphries (1991) conducted *CBR* tests on silts (MH) and observed improvement due to fiber inclusions.

Santoni and Webster (2001) conducted full scale testing of polypropylene fibers (51 mm length and 50 denier) within sands (SP) to support military air and ground traffic. Compacted specimens were tested in unconfined compression with fiber contents up to 2% of specimen dry weight to determine the fiber content (1%) for full scale testing. Compacted fiber reinforced sands were able to support over 1,000 passes of C-130 aircraft loads.

Polypropylene fibers 25 to 51 mm long were observed to be the most prominent in literature. A length of 51 mm was used in Santoni and Webster (2001), as well as recommended in Santoni et al. (2001). Therein, fiber lengths below 25 mm did not significantly improve fiber performance, while fibers longer than 51 mm proved difficult to mix and provided only slight differences relative to the 51 mm fibers.

Gray and Ohashi (1983) tested 1 to 2 mm diameter fibers that were 20 to 250 mm long. Fletcher and Humphries (1991) determined that aspect ratios of 33-50 could improve the *CBR* of silty material. Ranjan et al. (1996) tested sands with fiber aspect ratios of 50 to 125 and reported optimum results from the 125 ratio. Santoni et al. (2001) reported a 14% unconfined compression strength increase by decreasing the fiber denier from 20 to 4. The lengths evaluated in the study ranged from 13 to 76 mm, with fiber deniers of 4, 15, and 20.

Puppala and Musenda (2000) used fibrillated fibers to improve the volume stability of clays. Fletcher and Humphries (1991) and Santoni et al. (2001) used fibrillated and monofilament fibers. Fibrillated fibers were said to disperse better into the soil (Fletcher and Humphries 1991).

Santoni et al. (2001) stated moisture control, mixing, and compaction/preparation to be critical aspects of specimen preparation. Ranjan et al. (1996) mixed fibers into poorly graded sands by hand. To improve laboratory fiber dispersion, Santoni and Webster (2001) punched holes in a 125 liter plastic bag, inserted fibers, and applied compressed air. The fibers exited the bag in a form resembling "cotton candy" where they could be uniformly placed into the small samples. Ang and Loehr (2003) performed extensive unconfined compression testing and reported that specimens in excess of 70 mm diameter should be reasonably representative of in-service fiber reinforced soils.

The highest optimum polypropylene fiber content was found to be 2% in poorly graded sands (Ranjan et al. 1996). Therein, triaxial testing showed linear strength increases of up to 2% fibers, with a subsequent tapering off thereafter. Also, a critical confining pressure of approximately 50 kPa was identified; below this value strength differences were not nearly as pronounced (likely due to fiber slippage and pullout). Santoni et al. (2001) reported the optimum sand fiber content to be between 0.6 and 1.0%. The authors tested fiber contents from 0.2 to 1.0% and found that contents less than 0.6% exhibited strain softening (decreasing unconfined compressive strength with increasing strains). Fletcher and Humphries (1991) reported that 1% was the optimum fiber content of 25 mm long fibers used for *CBR* improvement in silty material. To treat expansive clays, Puppala and Musenda (2000) determined the optimum fiber content was 0.3 to 0.6% depending on the fiber length (25 to 51 mm). Testing was performed on 0 to 0.9% fiber contents. Direct shear testing of sands showed optimum fiber contents near 1.7% (Gray and Ohashi 1983) alongside fiber slippage and pullout below a threshold confining pressure.

2.7.2 Soils Stabilized with Fibers and Chemical Additives

The use of fibers as a secondary stabilizer with a chemical additive (e.g. portland cement) as the primary stabilizer has been shown to improve performance in both cohesive and cohesionless soils. In sandy soils, the major reinforcing mechanism is the transfer of load from the soil to the fibers through the development of friction (Maher and Ho 1994). The friction develops tension in the fibers, which enables them to restrict movement and form a particle-fiber interlock (Tingle et al. 1999). Thus, longer fibers provide more surface area for more friction development and, therefore, higher strengths (Maher and Gray 1990). Cement not only improves soil strength, but also helps the fibers adhere to the soil for additional friction development (Newman et al. 2008).

In clay soils, fibers increase strength by crossing potential shear planes and preventing failure (Maher and Ho 1994). Consequently, shorter fibers have been shown to be more effective than longer fibers in cohesive soils since for the same fiber content there are more fibers which are able to prevent more potential shear planes (Maher and Gray 1990). In addition, if clay is treated with cement, the fibers may bond better to the clay, since bonding between fibers and cement has been observed (Rafalko et al. 2007). While an increase in strength due to the addition of fibers was reported by some research groups, others observed only a small increase or a decrease in strength. However, all literature reviewed reported a noticeable increase in ductility or toughness as a result of fiber inclusion.

Newman et al. (2008) studied the effects of fiber and cement addition on a silty sand, and the strength and ductility results of compacted specimens are summarized in Table 2.8. The fiber dosage rate was 0.5% by dry weight. A lower shear strength was observed when fibers were used, but ductility was noticeably greater as evidenced by a higher yield strain and toughness. Toughness is defined as the energy absorbed up to the yield point, or the point of maximum applied stress. Toughness is computed by finding the area under the stress-strain curve up to the yield point and dividing by the volume of the specimen.

Table 2.8. Select Test Results from Newman et al. (2008)

Stabilizer	s_u (kg/cm^2)	Yield Strain (%)	Toughness (kg/cm^2)
None	5.32	0.62	0.04
Type III (4% of dry soil mass)	13.00	0.83	0.16
Type III & Fibers (4% and 0.5% of dry soil mass)	9.97	1.72	0.24

Note: Results are average of three specimens. Fibers used were 19 mm monofilament polypropylene.

Rafalko et al. (2007) explored the effect of using fibers as a secondary stabilizer along with cement. Several fiber types were added alongside *Type III* cement to a clay soil. The cement dosage was 5% and the fiber dosage was 1%, both by dry weight of soil. Two types of poly(vinyl) (PVA) fibers were tested along with fibrillated polypropylene (FP) and nylon fibers. The PVA1 and PVA2 fibers were 8.4 and 12.7 mm long, respectively, while the FP and nylon fibers were 19 mm long. Shear strength and toughness results are displayed in Table 2.9. Shear strength was lower for all fibers tested except 1, and the shear strength appeared to decrease with increasing fiber length. However, as evidenced by the toughness results, ductility was improved by the addition of all types of fibers except nylon fibers.

Table 2.9. Select Test Results from Rafalko et al. (2007)

Stabilizer	s_u (kg/cm^2)	Toughness (kg/cm^2)
None	0.56	0.01
Type III Cement	9.37	0.22
Type III Cement, PVA1 Fibers	9.68	0.33
Type III Cement, PVA2 Fibers	8.28	0.25
Type III Cement, FP Fibers	7.47	0.23
Type III Cement, Nylon Fibers	6.69	0.18

Note: Soil was USCS classified CH with LL of 53, PL of 25, PI of 28, and 81% fines.

Inclusion of discrete fibrillated polypropylene fibers increased the elastic limit, strain energy, and the ductility at strain levels on the order of failure for non-reinforced chemically treated (hydrated lime and portland cement) soils (Austin et al. 1993). The focus of the work was to study 25 mm fibers in chemically stabilized pavement layers within an oval test track; 1 was a silty-sand (*SP-SM*) and the other was a clay (*CH*). Unconfined compression and triaxial testing was performed on laboratory molded and undisturbed field samples to quantify field conditions and determine design stabilization rates (test data was not reported). Laboratory specimens were prepared in a large commercial mixer with various whips and blades depending on soil characteristics. Mixing time was until the fibers filamentized. Design dosage rates were: 1) 5% hydrated lime and 0.3% fibers for *CH* soil; and 2) 5% portland cement and 0.5% fibers for *SP-SM* soil.

2.8 Design Equations

Historically, Japan has used the most wet soil mixed material (Japan dredges a large amount of soil.) and has limited design strengths to s_u of 2.5 kg/cm^2. Burke and Shen (2005) present the wet soil mixing design equations provided in Eq. 2.2 and 2.3.

$$q_{ud} = (\gamma)(\lambda)(q_{ul}) \tag{2.2}$$

$$\gamma = 1 - 1.3(cov_f) \tag{2.3}$$

Where,

q_{ud} = unconfined compression design strength; 90% of field strengths exceeding this value
q_{ul} = average unconfined compressive strength from laboratory
γ = factor accounting for field strength variability
λ = factor accounting for difference between laboratory and field strength
cov_f = coefficient of variation of field mixed material strength in decimal form

Historically, the Japanese have used 0.5 for γ and λ. A large study incorporating 3,700 data points supported the use of γ and λ equal to 0.5 for clay and organic soils, with a reported 50% *cov*. Another study reported γ of 0.62 but recommended using the conservative value of 0.5; the study reported 38% cov_f for cohesive soils. Data provided from eleven wet soil mixing projects from Burke and Shen (2005) resulted in cov_f values of 0.22 to 0.76, or γ values from 0.01 to 0.76. Averaged values from all the projects resulted in cov_f and γ values of 0.45 and 0.42, respectively.

Burke and Shen (2005) presented a case that γ of 0.5 may be too conservative for large areas as stress follows the path of greatest stiffness, the mean compressive strength exceeded the imposed stress by a factor of 1.5 (a ratio later deemed acceptable), and the eleven projects were all performing satisfactorily. A case was presented that over 10% of the test results falling below q_{ud} does not significantly impact large mixing project performance.

To investigate the effects of elastic modulus (E) and shear strength, a soil with *LL* of 84, 84% moisture, *PL* of 27, G_s of 2.68, 11% *LOI*, pH of 7.7, 30% clay, and 44% silt was tested by Hirabayashi et al. (2009). At a 28 day cure, 100, 200, and 300 kg/m^3 dosage rates were tested with three preparation methods (tapping, static compaction, and dynamic compaction). The relationship was approximated based on data provided as shown in Eq. 2.4. The relationship held over the three dosage rates and three preparation methods.

$$E \approx 25\ (s_u) \tag{2.4}$$

Where,

E = elastic modulus (MPa)
s_u = shear strength (kg/cm^2)

2.9 Maturity Methods

The maturity concept is to relate the effects of time and temperature. Conceptually, a higher curing temperature for the same period of time or the same curing temperature for a longer period of time provide a more mature test specimen when chemical additives are used. *ASTM C 1074* relates these two factors in the simplest form using Eq. 2.5.

$$M(t) = TTF = \sum (T_i - T_o)\,\Delta t \tag{2.5}$$

Where,

$M(t)$ = temperature-time factor accumulated up to time t (C-hr)
TTF = symbol to denote temperature-time factor (C-hr)
T_i = maturity temperature (C)
T_o = reference temperature (C)

Kitazume and Nishimura (2009) used a maturity function defined as per Eq. 2.6 to investigate a soil with 16% sand, 50% silt, 33% clay, *LL* of 53, and *PL* of 24 tested at 65% moisture with 5, 10, and 15% binder on a dry soil mass basis. The use of the maturity concept brought strength data points along unique lines with some scatter according to binder content. Strength at 3.8 days at 40 C aligned with strength at 28 days and 20 C, and the maturity predicted from Eq. 2.6 was the same. Curing at low temperature (7 C) exhibited a very small long-term strength gain, and test data indicated that predicting long-term strength at low temperature using short term testing at higher temperatures may be difficult.

$$M = \int_0^{T_c} 2\exp\left(\frac{t_c + 10}{10}\right) dT_c \tag{2.6}$$

Where,

M = maturity
t_c = curing temperature
T_c = curing period

To investigate the maturity concept, Hirabayashi et al. (2009) tested a clay soil with a 100 kg/m^3 stabilization dosage in conjunction with 20 C and 40 C curing between 1 to 28 days. Strength increase with time was gradual at 20 C but more rapid at 40 C. Eq. 2.6 was used to calculate maturity, and all data was brought broadly along a unique line, according to the authors, with maturity on a log scale and strength on an absolute scale.

Some time ago, the maturity concept was used by Anday (1963) to compare field and laboratory cured lime stabilized cohesive soils. Laboratory specimens were cured under accelerated conditions in an oven at 49 C. Cylindrical specimens (5.9 by 7.1 cm) cured under the same conditions for 2 days were found to have the same strength as field specimens at about 3,000 degree-days using a datum temperature of 0 C. Actual specimen temperatures were not recorded, and the curing temperature was taken to be the oven temperature.

Circeo et al. (1962) compiled over 500 sets of data for portland cement treated soils with varying curing times up to five years and was able to develop a relationship between curing time and unconfined compressive strength. The study concluded that the relationship was affected by cement content, curing temperatures, specimen density, moisture content, and chemical additives.

2.10 Properties of Chemically Stabilized Soils

2.10.1 Properties of Chemically Stabilized Soil Slurries

According to Burke and Shen (2005), wet soil mixing traditionally produces a strong, stiff, brittle material (2% strain or less). Shear strengths of 1.5 to 10 kg/cm^2 were stated to be the typical range for cohesive soils. The typical range is much higher for cohesionless soils.

MacKay and Emery (1994) detail 15 years of experience using cementitious materials for stabilization of many types of contaminated sludge, sediment, and waste. The systems were practical enough to implement as demonstrated by case studies. Many additives such as portland cement, slag cement, cement kiln dust, lime kiln dust, hydrated lime, and fly ash were incorporated in projects presented as case studies where a variety of soil and sludge types were investigated with moisture contents from near dry to near fluid.

The studies of MacKay and Emery (1994) date back to approximately the mid 1970’s. One of the case studies involved lake bottom sediments in Ontario, Canada. Emery (1980) and MacKay and Emery (1994) provide study details. The soil (industrial sludge) was very soft but was stabilized to adequately meet both environmental and bearing capacity requirements for industrial fill applications. The sediment was fairly consistent in appearance, contained organics (7 to 31% *LOI*), had a variable moisture content of 40 to 300%, and as a result of moisture the bulk density varied from 1.30 to 1.65 g/cm^3. Contaminated soils were stabilized to prevent contaminants from dissolving in the groundwater and to encapsulate the contaminants to limit exposure.

Laboratory testing involved mixing sludge and stabilization materials in a small Hobart mixer to uniformity, placing material into 10 cm diameter by 10 cm high sealed containers, curing at 20 C, and taking pocket penetrometer (*Soiltest CL-700*) readings to determine shear strength (s_u). Laboratory blends were: 1) 80% sludge and 20% cementitious material by mass; and 2) 75% sludge and 25% cementitious material by mass. Cementitious materials consisted of various fly ash combinations, waste cement kiln dust, waste lime kiln dust, steel slag fines, portland cement, and *GGBFS*. Test results are in Table 2.10.

Table 2.10. Stabilized Soil Laboratory Test Results of MacKay and Emery (1994)

Sediment (%)	Additive (%)	Additives (---)	s_u at 24 hr (kg/cm^2)	s_u at 168 hr (kg/cm^2)
80	15 5	Lime kiln dust-quick Portland cement	1.48	2.20
80	15 5	Quick lime kiln dust Slag cement	1.28	1.58
80	15 5	Cement kiln dust-bypass Portland cement	0.21	2.14
80	15 5	Lime kiln dust Portland cement	0.61	1.02
80	15 5	Fly ash-8% carbon Lime kiln dust-quick	0.36	0.61
80	15 10	Lime kiln dust Fly ash-8% carbon	0.82	1.38
75	20 5	Cement kiln dust Portland cement	0.11	1.02
75	15 5 5	Fly ash-8% carbon Lime kiln dust Portland cement	0.61	1.23
75	15 5 5	Fly ash-8% carbon Lime kiln dust Slag cement	0.21	0.61

Note: Testing performed with Soiltest CL-700 Pocket Penetrometer.

To investigate scale effects, 90 kg samples were prepared in a drum rotation mixer to simulate a pugmill. Laboratory results indicated that the high organic content did not stop pozzolanic or hydraulic reactions but probably inhibited them to some extent. The results of testing the 90 kg samples were similar to the 10 cm by 10 cm laboratory specimens.

Field trials incorporated a large drag line to mix cementitious materials into the in-situ sludge. The process required excessive time and left substantial pockets of unstabilized sludge. It was decided that the sludge would have to be placed through some type of stabilization plant; it was verified using a large grader that "proper" mixing resulted in the required stabilization. Additional field trials discussed by Emery (1980) resulted in a methodology that allowed multiple batches to be tested each day. A chute was constructed to allow excavated sludge to be fed into a standard ready mix truck, while a small conveyor belt incorporated stabilization additives. The system using a ready mix truck was reported to be so efficient that a large base stabilization plant was passed over in favor of the logistically simpler approach of multiple trucks. Table 2.11 provides typical field results reported by Emery (1980). MacKay and Emery (1994) reported that optimum stabilization agents were selected based on economics, availability, and performance. They were 8 to 12% quick (unslaked) lime kiln dust and 3 to 5% slag cement.

Table 2.11. Stabilized Sludge Field Test Results of Emery (1980)

Sludge (%)	Fly Ash (%)	Lime Kiln Dust (%)	Portland Cement (%)	s_u (kg/cm^2)
72.8	13.7	8.1	5.4	2.09 at 144 hr
83.6	12.9	0.0	3.5	1.43 at 120 hr
75.7	11.5	9.9	2.9	1.13 at 120 hr

Super Geo-Material (SGM) described by Nakai et al. (2009) has been used recently on large projects. The material has 0.8 to 1.3 g/cm^3 density (1.1 g/cm^3 is typical), which is much less than typical excavated soil at 1.8 to 1.9 g/cm^3. SGM has air foam treated soil where the stabilizing agent and the air foam are pre-mixed and then mixed with clay slurry at a moisture content above the *LL*; 30 to 35% air is typical. The pneumatic flow mixing (PFM) method described by Oota et al. (2009) has also been used recently on a large scale project.

Tables 2.12 and 2.13 show key properties of these materials and the corresponding construction projects. It should be noted that on site variability is considerable, which is the motivation for reducing the laboratory mix design shear strengths for use in structural design in the field (s_{ud} or $0.5q_{ud}$). Oota et al. (2009) considered an average field shear strength of 0.80 kg/cm^2 equivalent to 0.61 kg/cm^2 design. Oota et al. (2009) further reduced the field average strength by a factor of 2 to arrive at the total reduction of 2.6 as shown in Table 2.12. The same authors took re-molded specimens from samples taken from the placement vessel that averaged 2.19 kg/cm^2 with 33% *cov* and *n* of 381. Specimens obtained by boring 25 locations resulted in 1.86 kg/cm^2 with 28% *cov* in air and 1.44 kg/cm^2 with 38% *cov* in water at 28 days, which well exceeded the 0.80 kg/cm^2 field design requirement.

Table 2.12. Cementitiously Stabilized Slurry Applications Using SGM or PFM

Source	Project	Volume (m^3)	Thickness (m)	28 day Lab Mix Design s_u (kg/cm^2)	Production Rate	Table 2.2 Soil ID	w (%)	Cement (%)
Tanaka et al. (2009)	Tunnel Backfill[1]	6.8(10^4)	13.8	3.06	2,000 m^3/day	A	250	14.8 Slurry Wt.
Tanaka et al. (2009)	Tunnel Backfill[2]	8.2(10^4)	2.5 to 3.5	2.24	100 m^3/hr	B	142	8.7 Slurry Wt.
Oota et al. (2009)	Japan Airport[3]	8.6(10^6)	5 to 9	1.60	25,000 m^3/day	C	85	3.3 to 5.7 Slurry Wt.
Nakai et al. (2009)	Shield Tunnel[4]	9.6(10^4)	3.0 to 3.5	2.24	100 m^3/hr	D	145	8.7 Slurry Wt.

1: Site variability factor of 3 was used, so s_{ud} was 1.02 kg/cm^2. The equipment used had 360 m^3/hr capacity.
2: Field coring showed successful strengths. Site variability factor of 2.2 was used, so s_{ud} was 1.02 kg/cm^2.
3: Project constructed in 3 to 8 m deep water. Site variability factor of 2.6 was used, so s_{ud} was 0.61 kg/cm^2.
4: Laboratory shear strength was approximately 8.7 kg/cm^2 with a cement content of 20.7% of slurry weight. Site variability factor of 2.2 was used, so s_{ud} was 1.02 kg/cm^2.

Table 2.13. Soils Used in SGM or PFM Applications

Reference	Tanaka et al. (2009)	Tanaka et al. (2009)	Oota et al. (2009)	Nakai et al. (2009)
Soil ID	A	B	C	D
General	---	Sandy Cohesive	---	Clay
Sand (%)	2	---	---	---
Silt (%)	53	---	---	---
Clay (%)	45	---	---	---
Fines (%)	98	58	93	50 to 80
G_s	2.66	2.70	2.70	---
LL	91	58	76	---
PL	34	28	---	---
Organics (%)	3.6	2.5	7.0	---
pH	---	7.5	---	---

Horpibulsuk et al. (2005) tested 1 soil with *LL* of 120, *PL* of 57, G_s of 2.61, *pH* of 8.8, and natural water content of 130%. Soil was mixed to moisture contents from 120 to 250%, and cement contents were 8 to 33% of dry soil weight; three *w/c* ratios were maintained: 7.5, 10, and 15. Testing was conducted on 5 by 10 cm specimens. *UC* and isotropically consolidated triaxial tests (drained and undrained) were performed.

At 28 days, *UC* strain to failure was less than 2%. The shear strength of the cemented soft clay remained essentially consistent with the *w/c* ratio, with the strength increasing as the *w/c* ratio decreased. Shear strength increased to approximate values of 2, 6.6, and 10 kg/cm^2 as the *w/c* ratio decreased. Table 2.14 summarizes the results when the specimens were confined.

Table 2.14. Select 28 Day Test Results of Horpibulsuk et al. (2005)

Shearing Type	*w/c*	Effective Confining Pressure (kPa)	Approximate s_u (kg/cm^2)
Undrained	15	100	2.5
	7.5	400	10.0
Drained	15	100	3.5
	7.5	400	12.8

Lorenzo and Bergado (2006) studied the effect of moisture content and cement dosage on a soft clay with *LL* of 103, *PL* of 43, *A* of 0.87, 69% clay, 28% silt, and 3% sand. *Type I* portland cement was added as a slurry with a *w/c* ratio of 0.6 to provide a cement content of 10% (basis was not stated). The moisture contents tested were 80, 100, 130, and 160%, which were measured before the addition of cement slurry. The strength was observed to decrease with increasing moisture content at all test times, except at a moisture content of 80%, as shown in Table 2.15. Lorenzo and Bergado (2006) concluded that the trend of strength decreasing with increasing moisture content is only applicable for cases where the moisture content is equal to or above the Liquid Limit (*LL*).

Table 2.15. Lorenzo and Bergado (2006) Results

$w_{\%}$ (%)	7 day s_u (kg/cm^2)	14 day s_u (kg/cm^2)	28 day s_u (kg/cm^2)
80	0.97	1.28	1.63
100	2.09	2.65	3.47
130	1.02	1.53	2.30
160	0.61	0.82	1.43

The cement content effect was evaluated by plotting shear strength against total *w/c* ratio, which is the clay moisture content plus water added during mixing divided by cement content. A plot of shear strength versus total *w/c* ratio showed shear strength had some level of fit as a function of total *w/c* ratio; scatter was considerable (Lorenzo and Bergado 2006).

2.10.2 Soil Property Effects on Chemically Stabilized Soils

According to Tremblay (2002), organics slow, or inhibit, cement hydration by coating additive grains. Prusinski and Bhattachajra (1999), however, state that cement is mistakenly assumed effective only in soils with a *PI* of 20 or less, and that cement is at least as effective as lime in soil stabilization with *PI* values up to 50. Hernandez-Martinez and Al-Tabbaa (2009) studied the effect of organic matter in soil stabilization by testing strength properties of portland cement added to 2 soils with different organic contents. The 2 clays were created by adding kaolinite to Irish moss peat. The Irish moss peat had an organic content of 94%, a natural moisture content of 500%, and a wet density of 446 kg/m^3. The medium organic clay contained 50% kaolinite at 100% moisture content and 50% peat. The lower organic clay was produced by mixing 90% kaolinite at 75% moisture content with 10% peat. Properties of both soils tested are provided in Table 2.16. Grout with equal parts water and portland cement was added to the 2 clays. The clay with lower organics exhibited a higher shear strength at all testing times, and also continued to gain strength with time.

Table 2.16. Select Results from Hernandez-Martinez and Al-Tabbaa (2009)

Properties	Medium-Organic Clay	Low-Organic Clay
Organic Content (%)	30	4
Density (kg/m^3)	1219	1471
LL (%)	157	65
PL (%)	123	34
Total Moisture (%)	180	85
Cement Dosage by wt.	14.5	12.7
28-day s_u (kg/cm^2)	1.83	2.55
60-day s_u (kg/cm^2)	2.43	4.54
90-day s_u (kg/cm^2)	2.09	5.20

Li et al. (2008) studied organic content influence on strength in soft soils. A soil was selected, brought to 60% moisture content and stabilized with portland cement at a dosage rate of 14% by dry soil weight. Shear strength results are displayed in Table 2.17. Shear strength noticeably decreased with organic content, although the trend was less pronounced at organic contents above 12%.

Table 2.17. Select Shear Strengths from Li et al. (2008)

Organic Content (%)	7 day s_u (kg/cm^2)	14 day s_u (kg/cm^2)	28 day s_u (kg/cm^2)	60 day s_u (kg/cm^2)
3	1.73	2.29	3.62	5.35
6	1.17	1.27	1.94	2.65
9	1.02	1.12	1.63	2.09
12	0.92	1.02	1.22	1.32
15	0.82	0.92	0.92	1.02
18	0.82	0.92	0.92	0.92

The effects of untreated soil plasticity were investigated by Miller and Azad (2000). Three soil types were compacted to standard proctor density and *OMC* and stabilized with cement kiln dust (CKD), which reacts with soil in a way similar to portland cement. Properties of the soils tested are displayed in Table 2.18, along with the shear strengths at 7, 14, and 28 days. The dosage rate of CKD was 15% by dry weight. An inverse relationship was observed between s_u and untreated soil *PI*. Soil G had higher strengths than Soil E and Soil F and exhibited a much lower activity. All soils exhibited brittle failure with maximum strain values between 1 and 3% despite strength differences.

Table 2.18. Miller and Azad (2000) Results

	Soil		
Property	**E**	**F**	**G**
USCS Classification	CH	CL	ML
% Sand	2	6	48
% Silt	47	52	31
% Clay	51	42	21
LL (%)	55	48	23
PI (%)	40	33	6
Activity	0.78	0.79	0.29
G_s	2.82	2.72	2.67
OMC (%)	23.3	16.0	14.0
Organics (%)	1.62	0.86	0.36
7 day s_u (kg/cm^2)	2.87	3.19	3.19
14 day s_u (kg/cm^2)	3.83	4.46	4.46
28 day s_u (kg/cm^2)	7.33	14.03	15.94

Chang et al. (1996) tested the engineering applicability and stability of sludge from the *Keelung River* in Taiwan that was solidified with 4 products including portland cement. The treated sludge was tested after 7, 14, 28, and 60 days. The dosage rates (by dry sludge weight) were 5, 10, 15, and 20%. No significant property improvement was found when less than 10% of the stabilization materials were used.

In general, the sludge (pH of 6.3) in situ had moisture contents between 90 to 95%, was piled for air drying from 45 to 54% moisture, and during testing the moisture was 70%. (The stabilization additive was already included when the final moisture was measured.) *UC* specimens were 63.5 mm in diameter by 177.8 mm tall and tested in accordance with *ASTM D 2166-85*; load was applied at 0.35 mm/min.

Specimens tested by Chang et al. (1996) were mixed by slowly adding the agent and then mixing for 10 minutes. The material was poured in the mold and then tapped lightly to remove air bubbles. The specimen was sealed, cured at 22 $\pm$ 1C, and tested. Portland cement was reported to be inadequate for the high level of organic matter in the sludge. A shear strength threshold of 1.25 kg/cm^2 was selected for re-use of the material. It took between 15 to 20% of the additives other than portland cement to achieve 1.25 kg/cm^2; portland cement achieved near 1 kg/cm^2 shear strength at 20% concentration coupled with longer curing durations. The portland cement achieved just under 0.5 kg/cm^2 shear strength at 168 hours curing and 20% concentration.

Chew et al. (2004) investigated the effect of *Type I* portland cement on clay soil using X-Ray Diffraction (XRD) to identify the amounts of illite, quartz, and kaolinite in untreated soils and also to determine the change in quantities of these materials as the cement content varied. The soil tested had a *LL* of 87, *PL* of 35, Activity of 0.77, G_s of 2.67, 68% clay, 22% silt, 10% sand, and a cation exchange capacity of 33.30 meq/100g. Clay slurry was produced at 90 and 120% moisture and then 1:1 cement slurry was added at cement contents of 5 to 50% of solid weight. Specimens were cured under water in 5 by 10 cm PVC molds.

XRD testing suggested the rate of *C-S-H* and calcium aluminate silicate hydrate (*C-A-S-H*) increase was higher when cement content was below 10%. Data showed kaolinite is rapidly exhausted by the pozzolanic reaction based on XRD testing, while illite levels were reduced relative to the untreated soil but did not reduce more as the cement content increased indicating less pozzolanic involvement. Scanning electron microscope (SEM) images showed an overall effect of kaolinite removal and an addition of cementitious material between flocculated particles.

UC testing showed that axial strains were 3% or less for cement treated material. Seven day strength increases were observed to be almost proportionally associated with cement content, whereas 28 day strengths were reported to show a more complex set of behaviors. Seven day strength behaviors supported the notion that seven day strength is governed largely by the short term hydration reaction depending only on the cement content. Twenty-eight day strength, however, was stated to be more likely reliant upon the pozzolanic reaction. At cement contents above approximately 10%, the amount of pozzolanic reaction stabilized to a limiting level, believed to be due to kaolinite exhaustion.

2.10.3 Portland Cement Property Effects on Chemically Stabilized Soils

Rafalko et al. (2007) studied the effects of varying cement fineness during stabilization of 2 fat clays, referred to as Staunton Clay and Vicksburg Buckshot Clay (VBC). The 2 clay soils were stabilized with a *Type I/II* and *Type III* portland cement. Both cements were assumed to have come from the same production facility, but this information was not specifically stated. The 2 soils were tested at a moisture condition that allowed for an untreated California Bearing Ratio (*CBR*) of 2 to be achieved; this resulted in moisture contents of 34% and 44% for the Staunton Clay and VBC, respectively. A cement dosage rate of 5% on a dry weight basis was used. Table 2.19 shows select soil properties and the 3 day shear strengths of each soil and cement.

Table 2.19. Select Test Results from Rafalko et al. (2007)

Soil Name	*USCS*	*LL*	*PL*	*PI*	Fines (%)	3-Day s_u (kg/cm^2) *Type I/II* (kg/cm^2)	*Type III* (kg/cm^2)
Staunton Clay	CH	53	25	28	81	10.00	9.37
VBC	CH	84	35	49	> 95	3.34	3.94

Since *Type III* cement has a higher Blaine Fineness then *Type I/II* cement, it is expected to provide higher strengths. However, as shown, *Type I/II* cement slightly outperformed the *Type III* cement for the Staunton Clay, while the VBC was stronger with *Type III* cement. Rafalko et al. (2007) suggests *Type III* cement was likely more effective for the VBC since the montmorillonite content in the VBC combined with the large surface area of the *Type III* cement allows the calcium hydroxide created by cement hydration to have more access to silica or alumina for pozzolanic reactions.

Clare and Farrar (1956) evaluated the effects of varying cement fineness and SO_3 content on the strength of soil-cement mixtures. A silty clay (*LL* of 40, *PI* of 22) was tested with different cements, and the shear strength results, along with the properties of the cements tested, are shown in Table 2.20.

Table 2.20. Select Test Results from Clare and Farrar (1956)

Property	Cement 1	2	3	6
Specific Gravity	3.17	3.15	3.16	3.10
Specific Surface (m^2/kg)	584	494	293	538
SO_3 (%)	1.7	1.8	1.8	2.6
1-Day s_u (kg/cm^2)	14.79	14.79	11.98	23.25
3-Day s_u (kg/cm^2)	19.73	19.37	18.32	29.59
7-Day s_u (kg/cm^2)	26.42	23.60	21.14	37.34
10-Day s_u (kg/cm^2)	27.48	25.36	23.95	37.69
14-Day s_u (kg/cm^2)	28.53	---	25.36	---
21-Day s_u (kg/cm^2)	30.65	26.77	24.66	38.04
28-Day s_u (kg/cm^2)	32.05	28.18	24.66	39.45

Note: 17% moisture in the compacted specimens along with 10% cement by dry soil weight.

UC tests results showed shear strength increased with cement fineness at all test times, except at 1 day where Cement 1 and Cement 2 produced the same results. However, despite a slightly lower fineness than Cement 1, Cement 6 noticeably outperformed all other cements at all testing times. Since Cement 6 exhibited a higher SO_3 content, additional testing commenced to investigate the effect of SO_3 content on shear strength of the soil-cement mixture. Various SO_3 contents were tested with a cement used elsewhere in the study, referred to as Cement 4. *UC* specimens were tested after 28 days of curing, and the shear strength results are displayed in Table 2.21, along with the properties of Cement 4. As shown, shear strength increased as the SO_3 content increased from 0.3 to 2.3%, but then began to decrease with progressively increasing SO_3 contents. Clare and Farrar (1956) concluded that fineness and SO_3 content appeared to independently affect shear strength and suggested that an optimum SO_3 content should be obtained for soil-cement mixes.

Table 2.21. Cement 4 Shear Strengths from Clare and Farrar (1956)

Property/Test Result	Value
Specific Gravity	3.15
Specific Surface (m^2/kg)	297
s_u (kg/cm^2)-0.3% SO_3	24.31
s_u (kg/cm^2)-1.3% SO_3	28.88
s_u (kg/cm^2)-2.3% SO_3	29.06
s_u (kg/cm^2)-3.3% SO_3	27.48
s_u (kg/cm^2)-4.3% SO_3	25.71
s_u (kg/cm^2)-5.3% SO_3	26.42
s_u (kg/cm^2)-6.3% SO_3	23.25
s_u (kg/cm^2)-7.3% SO_3	23.95

2.10.4 Properties of Supplementary Cementitious Materials

Zheng and Qin (2003) tested 2 soils in 7.1 cm cube molds stabilized with portland cement for comparison to the same soils stabilized with industry waste binder including *GGBFS*. Binder content was defined relative to wet soil mass, and the moisture contents of the soils were 30 and 50% as tested, which was just above the soils' natural moisture content. Between 40 and 80% of the industrial waste binder was *GGBFS*, and total binder contents up to 25% were tested. Binder contents below 5% produced low unconfined compressive strengths. At 15% binder content, portland cement produced a 7 day strength from 20 C curing in Soil H of 10.2 kg/cm^2, while industrial waste binders were 6.1 to 40.8 kg/cm^2 in the same conditions. For the same conditions, portland cement produced a 5.1 kg/cm^2 strength in Soil I, while the 1 industrial blend attempted was 12.3 kg/cm^2. The optimized industry waste binder was concluded to produce higher strengths than portland cement.

Sing et al. (2008) tested peat with a pH of 3.51, moisture content of 670%, and bulk density of 1,038 kg/m^3 after mixing with 950 kg/m^3 sand. *UC* testing of 50 by 100 mm specimens was performed after underwater curing for 28 days. Dosage rates were 200, 250, and 300 kg/m^3 relative to peat at in situ moisture, and the blends tested included: 25% portland and 75% *GGBFS*, 50% portland and 50% *GGBFS*, and 75% portland and 25% *GGBFS*. The pH of the stabilized specimens exceeded 9, with the highest value being 9.97 with a 300 kg/m^3 dosage of 75% portland and 25% *GGBFS*. It was stated a that pH above 9 demonstrated the formation of cementing products that was not being retarded by organic matter due to sufficient binder to neutralize organic acids. It was suggested that a pH lower than 9 in the stabilized specimens indicates prevention of cement product development. The highest shear strength was produced with 75% cement and 25% *GGBFS*, which produced strengths of 0.66, 0.73, and 0.91 kg/cm^2 with dosages of 200, 250, and 300 kg/m^3, respectively. High portland cement content was said to be required to stabilize the peat and produce an alkaline condition to activate the slag.

2.10.5 Additional Properties of Cement Stabilized Soils

Lee et al. (2005) suggests the influence of soil-cement (*s/c*) ratio (defined as soil to cement mass) should be included along with the customary *w/c* ratio. Two marine clays were stabilized with portland cement at various *w/c* and *s/c* ratios. Results showed that for a given *w/c* ratio, shear strength of *UC* specimens increased with *s/c* ratio. Higher strength could be attributed to the fact that a higher soil-cement ratio would result in a smaller void ratio, leading to less flocculation of the soil-cement mixture.

Kitazume and Nishimura (2009) tested a soil with 16% sand, 50% silt, 33% clay, *LL* of 53, and *PL* of 24 at 65% moisture with 5, 10, and 15% binder on a dry soil mass basis. Cement slurry on a 1:1 weight basis was mixed and thereafter left to sit for up to 180 min. Results showed little influence of the cement slurry preparation time or the time between the end of soil and cement slurry mixing, provided the soil and cement slurry mixing occurred within 30 minutes of cement slurry mixing.

Lin et al. (1996) investigated the stabilization of lead contaminated soils from Georgia. Sulfur was the primary stabilization material under investigation, but *Type I* portland cement stabilized specimens were used as a control. The material tested was reddish soil, 66% of particles passed the No 10 sieve, and the moisture content was 17%. Only particles passing the No 10 sieve were used during testing. A portion of the soil samples were spiked with lubricating oil (up to 4% by weight). Unconfined compression tests were performed with portland cement stabilized specimens after curing under standard moisture conditions for 168 hours. Table 2.22 summarizes the portland cement test data. Lubricating oil did not appear to interfere with the portland cement stabilization process.

Table 2.22. Test Results of Lin et al. (1996)

Portland Cement (% by weight)	γ_w (g/cm^3)	s_u (kg/cm^2)	Oil Spiked ---	$w_{\%}$ ---
17.6	1.94	7.87	N	17.0
25.0	2.08	20.78	N	17.0
33.3	2.12	22.67	N	17.0
15.0	2.11	24.41	Y	13.5
20.0	2.11	38.14	Y	13.5
25.0	2.11	44.63	Y	13.5
30.0	2.11	51.06	Y	13.5

2.11 Re-use of Compacted Chemically Stabilized Soils

Vaghar et al. (1997) tested harbor bottom sediments/organic deposits and marine clays from the *Boston Inner Harbor* leading to the *Ted Williams Tunnel* in Boston. The material dredged was used for construction on Spectacle Island after drying and compaction. The end criterion was an undrained shear strength on the order of 0.75 kg/cm^2. Chemical stabilization was performed with multiple materials by referencing wet sediment weight. Sediment properties were a wet bulk density of 1.18 to 1.25 g/cm^3, moisture content of 260 to 270%, G_s of 2.26, and classification as *ML* to *OL*. Test specimens were remolded to in situ moisture, mixed with stabilization additives, dried under ambient air flow when spread in 75 to 100 mm lifts, and tested every 24 hours with a torvane shear and a pocket penetrometer until an average torvane reading of 0.20 kg/cm^2 was obtained since this was the minimum value to support low ground pressure heavy equipment. This took approximately 40 hours. The untreated material achieved a shear strength of 1.71 kg/cm^2 when compacted to 93% maximum dry density (γ_d) and 21% moisture. When treated the material achieved 1.81 to 2.54 kg/cm^2 at densities from 83 to 92% of γ_d and moisture contents of 37 to 49%.

The state of New Jersey committed to allocate approximately 1.39 million cubic meters of material for beneficial use rather than disposal. To this end Bennert et al. (2000)

performed laboratory testing of Newark Harbor dredged sediments stabilized with 8% portland cement by total weight. The sediment and portland cement were combined in the laboratory using a blade mixer, and then allowed to dry in ambient laboratory conditions. The stabilized dredged sediment was stockpiled for future use as a compacted material. Initially, the material had 195% moisture in situ, which immediately reduced to 143% when 8% portland cement was added in the laboratory. At 6.5 and 30 days the moisture had reduced to on the order of 100% and 50%, respectively. After mixing with cement the material classified as *MH* with on the order of 10% sand, 87% silt, and 3% clay. Shear strength and *CBR* were found to be 4 kg/cm^2 and 17, respectively.

CHAPTER 3-EXPERIMENTAL PROGRAM

3.1 General Information and Terminology

A key component of the research was the development of a suitable experimental program, as a project of this nature could not be identified in literature. Useful information was found in literature that pertained to the study, but most of the information applied to testing of materials with less moisture than that of the current project. As a result, the research team developed an experimental program suitable to meet project objectives.

The experimental program was focused on testing the properties of stabilized soil slurries as they matured with time. Shear strength (s_u) and elastic modulus (E) were key properties measured during the experimental program and which were measured with unconfined compression (*UC*) suites and sets, alongside slab trials and variability slabs tested with hand held gages. When possible, standard terminology was used, though instances occurred where project specific terminology was required.

Test variables included soil type (Soils 1 to 5), target moisture content (100% to 233%) denoted $w_{ts(\%)}$ where the extreme values used are equivalent to $TS_{\%}$ of 30 and 50, stabilization additive or additives (17 different additives), total stabilization additive content (C_T) of 5 to 20%, water type (fresh/tap, brackish, or saltwater), curing temperature (cold, room, and warm), and soil processing (processed or unprocessed). The terms C_T and $w_{ts(\%)}$ are routinely used to categorize specimens in this report and are often referred to herein in the shorthand form (C_T, $w_{ts(\%)}$). For example, (5, 100) indicates a specimen with 5% total stabilization additive content by slurry weight and a target moisture content of 100%.

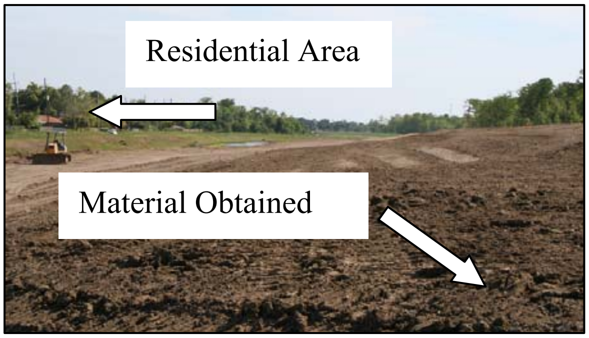
The experimental program was built on information contained in complimentary work performed under the same research effort (*SERRI Report 70015-007*). Methods to acquire and dewater the material needed were developed and discussed. The current report begins with material that has been obtained and dewatered to a given moisture content. Testing of soils dewatered using polymers was provided in *SERRI Report 70015-007*.

3.2 Materials Tested

3.2.1 Soils Tested

Soil used for the research was obtained from areas that would be likely candidates for flooding and from select construction sites. The soils were intended to cover a wide range of materials. Three primary samples were obtained from: 1) a levee construction site in New Orleans, LA (*Soil 1*); 2) *River Birch Landfill-Avondale Spoil Pit* in New Orleans, LA (*Soil 2*); 3) *North Pinto Disposal Facility* at *Mobile Harbor* in Mobile, AL that originated up to 1.6 km upstream in the Mobile River (*Soil 3*). Figure 3.1 provides photos of these locations.

Soils 2 and 3 were obtained in a condition that could be encountered in a disaster environment and in situ moisture contents were obtained; *Soil 1* had been processed and was relatively dry upon acquisition. Engineers of the *USACE* indicated *Soil 3* can change state considerably over time. They indicated that the material initially classifies as OH in most instances and has the consistency of dark axle grease. As organics rot, they often observe classification changes to CH or in some instances CL. Disposal facilities often crust over leaving wetter material at shallow depths; material obtained was below the crust.



(a) Soil 1 Site Location

(b) Soil 1 Close Up View

(c) Soil 2 Site Location

(d) Soil 2 Close Up View

(e) North End of Soil 3 Site Location

(f) South End of Soil 3 Site Location

(g) Soil 3 Distant View

(h) Soil 3 Close Up View

Figure 3.1. Photos of Acquisition of Soils 1 to 3

Soils 4 and 5 were obtained from construction sites. *Soil 4* was obtained from the *Inner Harbor Navigation Channel* (*IHNC*) in New Orleans, LA. *Soil 5* was obtained from a sand and gravel deposit of *Wissota Sand and Gravel* in Richfield, WI. Figure 3.2 contains photos of soils 4 and 5.

(a) Soil 4

(b) Soil 5

Figure 3.2. Photos of Soils 4 and 5

3.2.2 Water Sources Tested

Three water sources were used during testing: 1) fresh/tap water from laboratory; 2) brackish water; and 3) saltwater. Brackish water was sampled from the north-east shore of Lake Ponchartrain in June of 2009 at a location between the U.S. Hwy 11 bridge and the adjacent railroad bridge in Slidell, LA. Figure 3.3a was taken looking south-west on the immediate lake shore at the intersection of a canal and Lake Ponchartrain proper; the U.S. Hwy 11 Bridge is on the left, and the railroad bridge is on the right. The sampling location was a boat ramp entering the canal approximately 20 m inland and directly behind the vantage point of Figure 3.3a. Saltwater samples were obtained in July of 2009 from Okaloosa Island Fishing Pier on Santa Rosa Island near Destin, FL. Figure 3.3b shows the saltwater sampling location.

(a) Brackish Water Sampling: Lake Ponchartrain

(b) Saltwater Sampling: Destin, FL

Figure 3.3. Locations of Brackish and Salt Water Sampling

3.2.3 Stabilization Materials Tested

Seventeen stabilization materials were tested. Nine of the materials were portland cements, 3 were calcium sulfoaluminate cements, 2 were ground granulated blast furnace slag, 2 were chopped polymer fibers, and 1 was Plaster of Paris (PoP). These materials are described in a standalone fashion in Chapter 4.

3.3 Soil Processing

Handling the materials was considered prior to commencing the large scale processing operations, in particular considering in situ conditions at the time of soil acquisition while also balancing testing efficiency and consistency. Particle chemistry has been observed to change with varying conditions. To fully re-constitute an in situ condition existing at a particular location and time, the majority of the moisture should not be removed from the soil, and all water added should come from the site. Provided this was a goal of the research, additional care related to drying, soil breakdown, etc. would have been taken. However, the goals of the current research were to test a series of combinations since the in situ condition present when the materials were sampled for testing will not be present at a future disaster environment. Key variables such as soil type and water properties were considered in the experimental program but were recombined at a later stage in the research in the majority of cases.

A large volume of soil (several metric tons) was needed for the experimental program. The soils were acquired in different fashions at different moisture contents depending on site conditions. *Soil 1* was acquired near *OMC*, which is significantly drier than the lowest moisture content investigated of 100%. *Soil 2*, on the other hand, had in situ moisture contents ranging from 65 to 141%. At a minimum, portions of this material would have required air drying and/or some form of handling for 100% moisture testing. The researchers elected to process the material as described in the remainder of this section unless specifically stated. Processing allowed more uniform soil and water conditions between tests and for more consistent laboratory mixing and testing. A portion of soils 2 and 3 were left unprocessed (i.e. in the same condition found in the field) and stored in sealed containers for comparison of index and strength properties relative to the processed materials.

A detailed procedure was used to process the soil. (Figure 3.4a is an example.) The procedure used is believed by the research team to be efficient and an advancement to previous approaches used for large scale soil processing as long as specific in situ properties are not the objective (method is for preparing consistent material for comparison studies). The procedure is detailed as follows and was applied only to soils 1 to 3. Soils 4 and 5 were obtained in relatively small quantities, and they were processed in conventional fashions.

Material was first passed through a No. 4 sieve. Small amounts of debris remained on the sieve that was discarded. The materials tested were such that their individual particle sizes were smaller than a No. 4 sieve. The methods employed were not intended to break particles but rather separate them for more efficient and accurate testing, (*PSD* testing was performed before and after processing to investigate changes in particle size and revealed some particle breakage.) To pass the cohesive soil through the sieve, drying was necessary. The soil was spread onto tarps and placed under fans. Multiple barrels were typically dried simultaneously using staggered drying phases to provide a steady supply of adequately dry material. In some cases, drying was accelerated by cutting soil into 10 cm cubes using a wood framed wire press (cutting of soil must be performed while at a moisture content significantly in excess of *OMC*). An overall view of the drying cubes can be seen in Figure 3.4b, while the wood framed wire press can be seen in Figure 3.4c.

Handling was designed to ensure that all raw material on a tarp was collected, as it was processed and placed, in an isolated area so as not to mix material from different tarps until a later stage. This allowed for the full processing of each tarp prior to mixing into drums with other tarps. The intent was to avoid segregation of the different particle types during handling and processing. Note that different materials in a given sample (i.e. sand, silt, and clay) respond differently to the various processing stages, and keeping the tarps together until the entire process was finished allowed re-mixing of each tarp to uniformity prior to combining with the rest of the soil type and placing in a barrel.

Soil was first broken down using concrete mixers. The soil was placed inside the concrete mixer along with pieces of scrap metal and allowed to run for periods of 15 to 45 minutes depending on the size of the clumps and apparent wetness of material. An example mixer with scrap metal can be seen in Figure 3.4d. A large No. 4 sieve was placed over a rolling bin with the same dimensions as the sieve (Figure 3.4e). After the aforementioned time in the mixer, the soil was emptied over the sieve; the material broken up sufficiently passed through the sieve, while the material requiring more processing was retained by the sieve. These clumps of material were placed back into the mixer alongside sufficient material from the tarp to fill the mixer, and the process was repeated.

Once all material from a tarp was broken down into small sizes (roughly 37.5 mm or less), it was placed into an LA Abrasion machine (Figure 3.4.f) with the steel spheres of the equipment. In many cases the material was sufficiently dry, but in other cases further room temperature air drying prior to this step was required to prevent coating of the drum. The material was removed from the LA Abrasion machine and emptied onto a No. 4 sieve. If any material was retained, it was placed in the machine again, and the process was repeated.

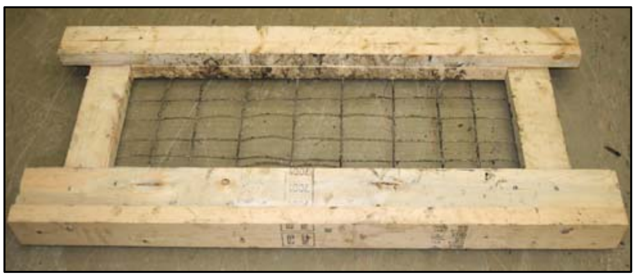
Once all the soil passed a No. 4 sieve and had been completely re-mixed, it was placed in a barrel. Care was taken to preserve randomness by placing random locations into the barrel, and the barrel was labeled with information regarding soil origin, date of processing, and soil properties such as average moisture content remaining after processing. Multiple moisture contents were collected from the top, middle, and bottom thirds of the barrel, and recorded for batching purposes (consistent moisture was observed). The barrels were then secured with a lid and a sealing ring for storage prior to use (Figure 3.4g). The barrels were identified with a letter (e.g. barrel A) which is used in discussion as appropriate.

The processing methods were developed considering the soils tested and ultimate use of the data. The organic materials tested would be considered organic soils and not peat due to the magnitude of organics contained. The materials tested contained less organics than would be present in peat materials. As a result, air drying and tumbling processes were deemed acceptable to process the material since no excess heat from ovens would be present to drive off the organic matter. Also, the goal of the research was not to replicate any specific condition, but rather evaluate the overall properties of the categories.

The aforementioned method was developed after performing several iterations using different approaches. Originally, a wet method was used to process the material across a No. 4 sieve. This was performed by wetting the material to the point where it could be pushed (i.e. grated) through the sieve. The resultant material was then dried and crushed by hand until it passed the sieve. This method was determined to be grossly inefficient for the quantity of material needed. This method may be reasonably efficient for very small samples that are negligible in comparison to the sizes required for this research, but it was not suitable for current needs.

(a) Soil Drums Obtained From Field (Soil 1)

(b) Drying of Soil 1 Cubes



(c) Wood Framed Wire Press

(d) Concrete Mixer and Scrap Metal

(e) Rolling Bin and No. 4 Sieve

(f) Soil 1 at Completion of Processing

(g) LA Abrasion Machine

(f) Final Barrels of Processed Soil

Figure 3.4. Photos of Material Processing Protocol

3.4 Properties of Materials Tested

3.4.1 Soil Properties

Fundamental properties were measured for the soils tested to allow the material to be classified for conveyance of properties between responders and to provide data for further testing and analysis. The *Unified Soil Classification System (USCS)* and the *American Association of State Highway and Transportation Officials (AASHTO)* methods were utilized to classify the soils. The methods are discussed in and were performed according to *ASTM D 2487* and *AASHTO M145*, respectively. For select clay soils *ASTM D 422* was used to perform particle size analysis and to determine activity using a 152H hydrometer. For other soils, washed gradations were performed according to *ASTM C 117* and *ASTM C 136*; percent fines were determined according to *ASTM C 117*. Organics/volatiles content was determined according to *ASTM D 2974* using a muffle furnace at 750 C. Atterberg Limits were performed according to *ASTM D 4318,* slump was performed according to *ASTM C 143*, and specific gravity of soil solids (G_s) was performed according to *ASTM D 854*. Maximum dry density (γ_d) and optimum moisture content (*OMC*) were determined with *ASTM D 698-00a Method A Standard* and *ASTM D 1557-00 Method A Modified*. The California Bearing Ratio (*CBR*) test was conducted via *ASTM D 1883-99*. Where appropriate soil properties were measured before and after processing (See Section 3.3).

X-ray fluorescence (*XRF*) was used to identify elemental oxides of processed samples. *XRF* excites the materials using gamma rays and reports any of the approximately 150 elements of interest provided 0.01% or more is present. When calibrated for cement and used for soil as was the case herein, the accuracy should not be interpreted to be more than $\pm$ 10%; i.e. 1% of an element is to be interpreted 0.9 to 1.1% of the element. Also, the lower the concentration of the oxide the lower the accuracy of the scan. Any known information that can be entered into the *XRF* the more accurate the results. For the analysis conducted in this report, loss on ignition (*LOI*) was conducted on oven dry specimens by placing them in a 950 C muffle furnace for 10 minutes prior to the *XRF* scan.

Soil pH was measured on the processed material in the barrels from Section 3.3 in the following manner. Two random soil samples of 10 g were taken and added to 20 ml of deionized water. The mixture was allowed to sit for 15 to 30 minutes, stirred to uniformity, and the pH measured using the meter discussed in the following paragraph. The approach used is common among soil scientists.

All pH testing was performed using the hand held *Hannah Checker Model 98103* with electrode *Model HI 1270*. A 7.01 buffer solution *Model 7007M* and a 10.01 buffer solution *Model 7010M* were used for calibration. The electrode was conditioned by soaking in 7.01 buffer solution for 2 hours. The meter was calibrated before a large block of pH testing was to take place. First, the meter was turned on and the tip of the electrode dipped approximately 2.5 cm into a small beaker of 7.01 buffer solution. The reading was allowed to stabilize, and then a screwdriver was used to turn the pH 7 trimmer until the display read 7.01. The electrode was then rinsed with water, and the same procedure used with 10.01 buffer solution. The 4/10 pH trimmer was used to adjust the display to 10.01. The pH meter was stored with a few drops of storage solution *Model 70300M* added to the protective cap.

3.4.2 Water Properties

Brackish, fresh (tap), and sea (salt) water were tested. Water pH was measured using the same meter as was used for soil pH. Random samples were obtained and tested. Salinity was tested using the *HACH® Session™156 Portable Multiparameter Meter*. Salinity is a relative scale based on a potassium chloride (KCl) solution containing 32.4356 g KCl in 1 kg of solution at 15 C. The meter determines salinity based on the Extended Practical Salinity Scale of 1978, as referenced in the 17th edition of *Standard Methods,* 25200 B. The meter's temperature range is -2 to 35 C, and the salinity is reported in parts per thousand (ppt).

A sample was taken from each brackish or seawater bucket to be tested for salinity; fresh water was taken from the laboratory faucet. The bucket was first gently agitated to obtain a representative sample, and the probe was placed in the sample to measure salinity. Once the salinity measurement reached equilibrium, the reading was recorded. The meter displays the temperature of the sample, which was also recorded to ensure that the temperature of the water sample fell within the applicable temperature range of the meter (testing occurred at approximately 24 C). As a reference, ocean salinity is on average 33.5 to 34 ppt worldwide and can reach 60 ppt in some areas due to evaporation.

3.4.3 Stabilization Material Properties

Chapter 4 has been devoted to properties of stabilization materials. The materials incorporated were a combination of commercially available products and products developed for this research effort. A significant amount of information was provided in this regard and is best represented as a chapter.

3.5 Batching and Blending of Soil Slurries

Unless stated otherwise, soil slurries were prepared in the manner presented in this section; these procedures were applied primarily to soils 1 to 3. Soil slurries contain no stabilization additives. Once stabilization additives are incorporated (e.g. portland cement), the mixture is referred to as a stabilized soil slurry; discussion of this topic is left for Section 3.6. Soil slurries were mixed in 18.9 liter plastic containers. Mixing was performed with a hand held drill and off-the-shelf standard attachment (Figure 3.5a).

To batch the material, the known moisture contents of the barrels of Figure 3.4g were used to calculate the amount of material required from the barrel and the amount of additional water to achieve the desired moisture content. Materials were placed in 18.9 liter containers prior to mixing (Figure 3.5b). The procedure is provided in Eq. 3.1 through 3.3.

$$w_{ts} = \frac{W_{wts}}{W_{sts}} \tag{3.1}$$

$$W_t = W_{wts} + W_{sts} = W_{ws} + W_{wb} + W_{sts} = W_{sts}\,(1 + w_{ts}) \tag{3.2}$$

$$W_{sts} = \frac{W_{ws}}{1 + w_b} \tag{3.3}$$

(a) Mixing Attachment

(b) Pre-batched Soil and Water

(c) Adding Processed Soil to Water

(d) Agitation of Soil Slurry

(e) Soil Slurry Prior to Aging

(f) Segregated Soil Slurry at 24 Hours

Figure 3.5. Mixing of Soil Slurries

Where,

w_{ts} = target moisture content of soil slurry in decimal form
W_{wts} = weight of water in final soil slurry
W_{sts} = weight of soil solids in final soil slurry

W_t = total weight of soil slurry to be mixed
W_{ws} = weight of soil in barrel including residual moisture
W_{wb} = weight of water to be batched or added into the soil slurry
w_b = moisture content of processed soil contained in barrel in decimal form

A typical batching operation used the known value of w_b, selected w_{ts} and W_t, and calculated $W_{wb.}$ Processed soil was added to the water (Figure 3.5c) at a constant rate on the order of 2.5 kg/min under vigorous agitation. To achieve desirable results, adding soil to the water is more efficient than the reverse. Adding the water to the soil causes large clumps to form that must be stirred for longer periods. Once all the soil was in the slurry, it was agitated for 3 additional minutes (Figure 3.5d). The final product is seen in Figure 3.5e.

The bucket containing soil slurry was labeled and allowed a minimum of 24 hours of marinating time to allow full dispersion of water and interaction of water with the soil. This step was performed to ensure that the soil (clay in particular) would be in a state representative of that found at a disaster area that was flooded. A noticeable decrease in volume was observed during the aging process, and the soil and water segregated to some extent in many cases over the 24 hour period (Figure 3.5f). The actual moisture content of the soil slurry was not measured until later in the process. During this 24 hour period the soil was left in the room adjacent to the environmental chamber so that the initial temperatures would be similar.

3.6 Preparation and Mixing of Stabilized Soil Slurries

Mixing is known to be a factor of significance for controlled laboratory testing and field performance; mixing energy is directly related to performance. Moisture content affects the equipment, mixing protocol, and effectiveness of the stabilization. Depending on the aforementioned parameters, wet soil and dry soil become relative terms. For this research, the assumption is that all soils have at or above the moisture in their native state relative to when they will be mixed with stabilization materials. In other words, no water would be added to the soil in the field. The mixing protocols used for the components of this research are discussed individually in the remainder of this chapter. Samples prepared in accordance with Section 3.5 were used in most cases, with exceptions noted where they occurred. All stabilization materials (cementitious materials and fibers) were specified as a percentage by total slurry weight (i.e. soil and water). Some stabilized soil blends had only 1 additive, while others contained multiple additives.

3.6.1 Preparation of Stabilized Soil Slurries for *Slab* Testing

The soil slurries were re-mixed with vigorous agitation for 5 minutes to restore uniformity (Figure 3.6a). The material in the individual plastic buckets was then combined into a large mixing barrel (Figures 3.6b and 3.6c). Typically, the slurry mass was 40 to 140 kg. The mass of material was selected based on the number of slabs to be fabricated.

Moisture content of the final soil slurry ($w_{as(\%)}$) was taken randomly to compare with the target soil slurry moisture content ($w_{ts(\%)}$). $w_{as(\%)}$ was used as a quality control mechanism to, for example, verify that batching was conducted properly. Batching tolerances were set to have $TS_{\%}$ within 1% of the target value when rounded to the nearest whole number; $w_{ts(\%)}$ of 100 and 233 were considered acceptable when $TS_{\%}$ values were 49 to 51 and 29 to 31, respectively. Note that repeatable $w_{as(\%)}$ values at $w_{ts(\%)}$ of 233 was difficult due to sampling; $w_{as(\%)}$ measurement errors on the order of 5% were believed to be due to sampling.

(a) Re-agitation of Soil Slurries

(b) Combining Soil Slurries (1 of 2)

(c) Combining Soil Slurries (2 of 2)

(d) Addition of Cementitious Materials

(e) Mixing of Stabilized Slurry (1 of 2)

(f) Mixing of Stabilized Slurry (2 of 2)

Figure 3.6. Mixing of Stabilized Slurries (Soil 1 Shown)

Cementitious materials were added to the slurries under vigorous agitation (Figure 3.6d) at a rate of approximately 0.5 kg per minute and mixed to uniformity (Figures 3.6e and 3.6f). A stopwatch was used to record when cement was first introduced into the soil slurry. When incorporated, fibers were prepared using a modified plastic bucket. A hole was drilled in the bottom of the bucket and a standard air pressure attachment inserted. Four metal screws were drilled into the bucket to separate the fibers as they were forced from the bottom with the air pressure (Figure 3.7a). The lid of the bucket contained a spout, and a wire mesh was secured around the spout to allow air flow but restrain fibers (Figure 3.7b). Fibers were placed in the bucket, and air pressure was used to agitate and disperse the fibers so they could be mixed into the soil slurries (Figures 3.7c and 3.7d). Figures 3.7e and 3.7f show the fibers being mixed into the stabilized soil slurries; mixing was performed to uniformity.

(a) Top View of Screws *(b) Spout Covered With Mesh* *(c) F70 Fibers*

(d) F20 Fibers *(e) Mixing F70 Fibers* *(f) Mixing F20 Fibers*

Figure 3.7. Mixing of Polymer Fibers

3.6.2 Preparation of Stabilized Soil Slurries for *UC* Testing

Some *UC* material was prepared according to Section 3.6.1. When only *UC* specimens were produced, the same fundamental procedure and tolerances of Section 3.6.1

were used except smaller quantities were produced. The material was mixed with the same off the shelf attachment. These procedures were applied primarily to soils 1 to 3. Soils 4 and 5 were not processed, batched, and/or blended according to Sections 3.3 and 3.5. *Soil 4* was prepared in the same manner as soils 1 through 3 except for processing. The material was air dried, and tap water was used to produce the desired moisture content. *Soil 5* was prepared with material at the in situ moisture condition of 100.5% and mixing cementitious material with the same procedures as the other soils.

3.7 Preparation of Specimens for Strength Testing

3.7.1 Preparation of *Slab* Specimens

Stabilized slurry mixed as per Figures 3.6 and 3.7 was made into slabs using the molds shown in Figures 3.8 and 3.9. Molds were fabricated and replaced as necessary. Preliminary testing incorporated larger molds which worked well from a data collection perspective, but were too heavy to be efficiently handled in the laboratory; so, dimensions were reduced to those of Figure 3.8. Originally, molds were on the order of 0.03 m^3 with the final molds being on the order of 0.02 m^3 and weighing on the order of 12 kg when empty.

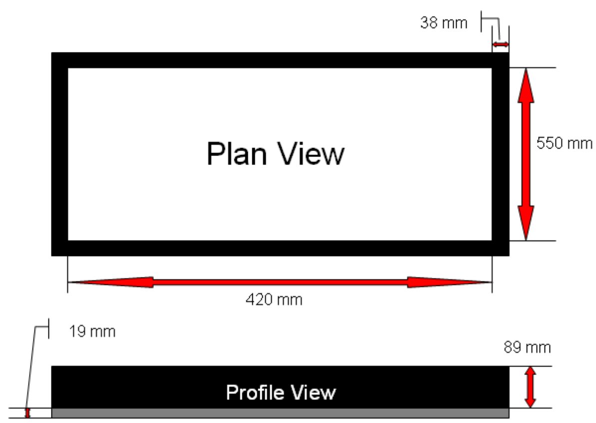


Figure 3.8. Schematic of Molds Used for Slab Testing

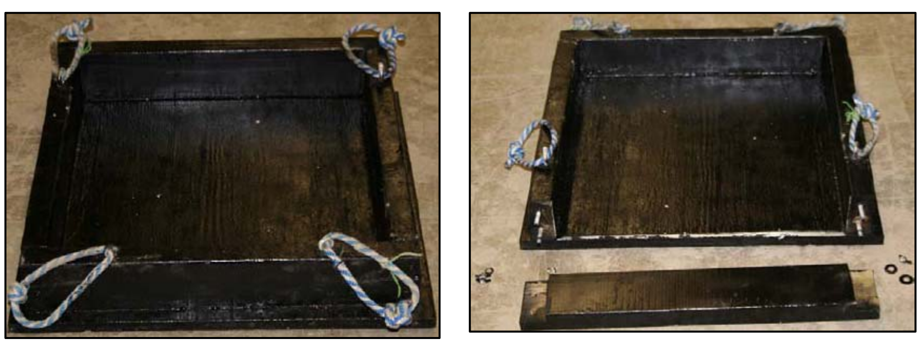


Figure 3.9. Photos of Molds Used for Slab Testing

The molds were pre-treated wood with latex paint coating. Composite board of 19 mm thickness with metal strip reinforcement was used for the base. The mold was sealed with latex spray paint to minimize volume change during sustained periods of 100% humidity. The volume of each mold was determined using water at 20 C. Mold volumes were re-measured when alterations to the volume were believed to occur (e.g. when re-painted, dropped, or similar). Mold volumes were used to calculate slab density (ρ_s) to an accuracy on the order of 5% for use primarily as a quality check.

The total wet unit weight of the stabilized soil slurry (γ_T) was determined using a standard measure and vibrating table as shown in Figure 3.10, which is more accurate than ρ_s. Vibration was performed on the highest setting using a *Syntron Type VP 200-Style 1653* table with a *VC 200-Style 4471* controller. The materials were vibrated in 3 lifts with each lift vibrated for 1 to 2 minutes or until air ceased to be dislodged from the material (Figure 3.10a). A small amount of additional material was placed to top off the sample, and this material was massaged and the measure tamped to dislodge any remaining air. The unit weight measure was struck off (Figure 3.10b).

(a) Vibration of Sample

(b) Striking off Surface

Figure 3.10. Determination of γ_T Using Vibration Table

A synthetic liner or petroleum jelly was placed inside the mold to allow for easy removal of the slab at the conclusion of testing (Figure 3.11a). Note that the volume of the mold was determined with the liner in place. The stabilized slurry was placed into the mold by pouring and shoveling (Figures 3.11b and 3.11c) or for less fluid slurries it was placed by hand. Once sufficient material was in the mold, it was gently massaged by hand for a brief period to ensure no large air pockets were present (Figure 3.11d). The material was struck off with a straightedge (Figure 3.11e). The density of each slab was calculated at this juncture using the known volume of the mold and the total mass of the stabilized slurry. The final product ready for curing is shown in Figure 3.11f.

(a) Synthetic Liner

(b) Pouring Stabilized Slurry Into Mold

(c) Shoveling Stabilized Slurry Into Mold

(d) Massaging Out Large Air Pockets

(e) Striking Off Surface

(f) Final Product: Stabilized Slurry Slab

Figure 3.11. Molding of Stabilized Slurry Slabs (Soil 1 Shown)

3.7.2 Preparation of *UC* Specimens

After considering available options, cylindrical *UC* molds were fabricated from 7.6 cm diameter PVC pipe that was cut into 16.5 cm lengths in a manner to ensure the ends of the specimen were parallel with one another. The molds were cut to allow samples with a 2:1 length to diameter aspect ratio to be tested while allowing room for a 6.4 mm thick porous stone to be placed within the mold on each end of the specimen. Each mold was split down its length to allow specimen extraction without damage. The molds were clamped during curing. The inside of each mold was lubricated with petroleum jelly to minimize surface friction during extraction. Filter paper was placed between the specimen and porous stones to minimize contamination. The porous stones were soaked prior to use to avoid them taking moisture from the test specimen. Figure 3.12 shows key items related to the mold and its accessories. Approximately 80 molds were fabricated for use and replaced as needed.

Figure 3.12. Unconfined Compression (UC) Specimen Molds

The mold, soaked porous stones, clamps, and filter paper were weighed, and immediately thereafter the stabilized slurry was added in 3 separate lifts using a delivery apparatus resembling a large syringe or a spoon. The syringe like apparatus was used for highly fluid material (e.g. *Soil 1*) to minimize air entrapment. After each lift, the mold contents were consolidated by either hitting the bottom of the mold against a table or by tamping the side of the mold. Extremely stiff samples (e.g. *Soil 2* at 100% moisture) were also consolidated by pressing a steel cylinder with approximately the same diameter into the mold. After the third lift, material was added until the top of the mold was reached, and then the top porous stone was added. A second larger porous stone was subsequently placed on top of the mold, and pressure was applied until the smaller porous stone became flush with the top of the mold. Some material was typically expelled near the top of the mold during this process. Pressure was maintained on the top of the mold while the top clamp was tightened. Note that the bottom porous stone and clamp would already be in place with the bottom porous stone flush with the bottom of the mold. The mold containing the specimen was then weighed in order to determine density using the known mold volume. The specimen was ready for curing at this juncture. Determination of γ_T for the stabilized slurry was performed as with slabs (Section 3.7.1), except a smaller container was used.

3.7.3 Preparation of *Combined* Specimens

Preparation of combined specimens involved portions of Section 3.7.1 and Section 3.7.2. Specimen preparation occurred in phases and was performed in coordination with testing. The majority of the details related to the preparation and testing are discussed in Section 3.9.3 to allow the reader continuity through the process.

3.7.4 Preparation of Density Correction Specimens

Two standard plastic buckets were stacked on one another creating a 5 cm void between the bottom of the buckets. A hole was drilled into the side of this void to accommodate an air hose assembly. Next, approximately 25 holes were punched, evenly spaced, in the bottom of the first bucket having a diameter on the order of 0.5 cm. Screws were inserted where the 2 buckets overlapped to prevent separation under pressure.

The void chamber between the buckets was pressurized using compressed air until air could be felt escaping through the holes made in the bottom of the first bucket. Stabilized soil slurry prepared according to Section 3.6.2 was then added to the bucket slowly as to avoid splatter and mixed under vigorous agitation for 5 minutes. The soil slurry was then returned to the original mixing receptacle. *UC* specimens were prepared from material prior to any vibration. The batch was then vibrated for 30 seconds and subsequently additional *UC* specimens were prepared. The stabilized slurry was then vibrated until there was no further visible air escaping and subsequently additional *UC* specimens were prepared. Preparation of the specimens was the same as described in Section 3.7.3.

3.7.5 Preparation of Membrane Correction Specimens

Very thin membranes (PVC soap mold liners manufactured by Chestnut Farms) were evaluated to a limited extent after it was observed that a 2 hour test could be performed in absence of a membrane. The membranes are very thin and do not possess significant strength or ductility. Figure 3.13 shows the mold fabricated to allow the specimens with membranes to be prepared. (The membrane is in place in the figure.) The apparatus is similar to a triaxial mold with attached vacuum lines. The material placed into the mold/membrane assembly was prepared according to Section 3.6.2. Use of porous stones, curing, and mold removal are the same as in Section 3.7.2.

Figure 3.13. Mold With Thin Membrane Inserted

3.8 Curing of Specimens for Strength Testing

A stopwatch was used to record when cement was first introduced into the soil slurry, which was the absolute reference (t = 0) used for the entire testing period of any strength specimen produced with this material. The time and date of first cement introduction were recorded to the nearest minute for synchronization with the *SPER Scientific Model 800024* data logger used to continuously monitor temperature for all strength specimens during this research. The data logger recorded temperature every 4 minutes continuously throughout curing of a strength specimen. All curing in this research occurred in 99.5 to 100% humidity, which is referred to as 100% humidity for the remainder of the report.

3.8.1 Curing of *Slab* Specimens

All *Slab* specimens were cured at 18 to 25 C in the Figure 3.14 environmental chamber (referred to as T_a). The internal temperature of the stabilized slurry (T_I) was taken during curing within 1 slab of a particular soil, moisture, and additive combination to evaluate internal heat generation (ΔT). For most slabs, the temperature of the slurry just prior to cement addition was measured to allow more efficient heat generation calculations. All materials were kept at room temperature adjacent to the environmental chamber to minimize temperature differences during the first few minutes of curing. The stabilized slurry where internal temperature was measured was, in general, left to cure for 168 hours. Multiple slabs were required to complete testing of a trial. Testing of these slabs occurred at convenient times during laboratory operations, and the data was superimposed into a single trial.

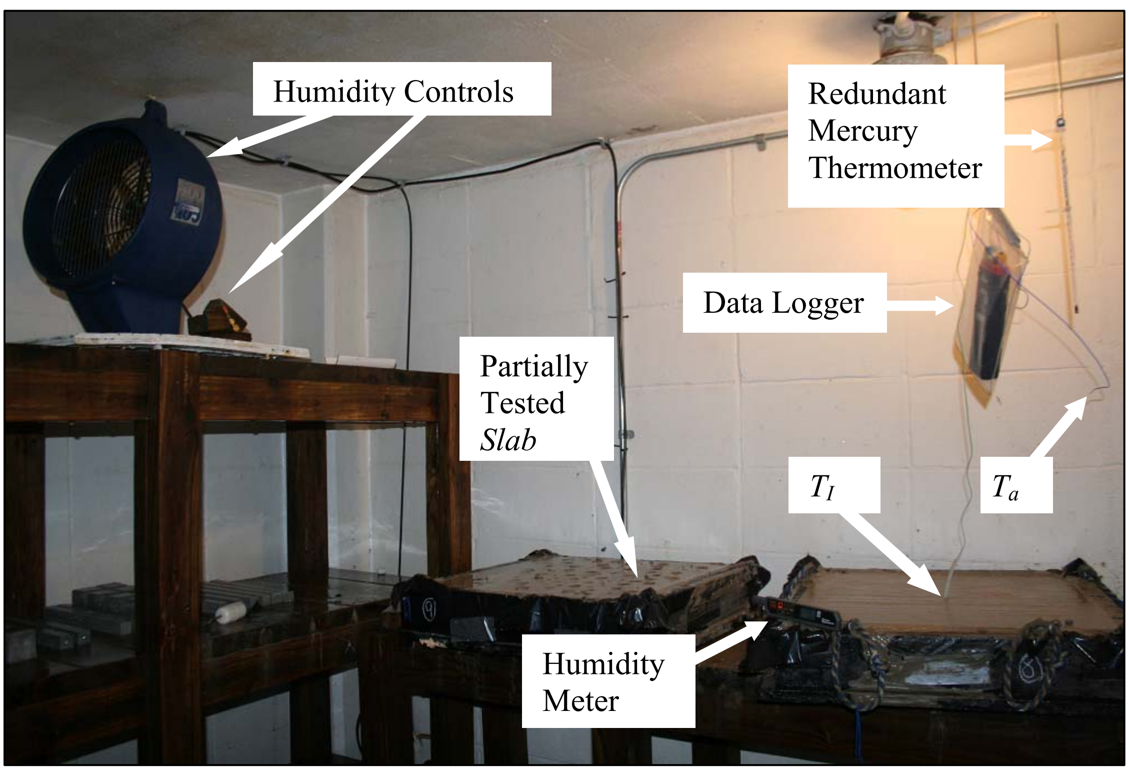


Figure 3.14. Environmental Chamber With Curing Slab Specimens

3.8.2 Curing of *UC* Specimens

UC specimens were cured at 3 temperatures: cold (≈ 4 to 6 C), room (≈ 18 to 24 C), and warm (≈ 33 to 37 C). If not specifically stated, curing occurred at room temperature. A small percentage of the testing occurred at cold or warm temperatures. All specimens were cured in a plastic bucket containing water that had been conditioned to the curing temperature prior to addition of the specimen. Specimens were submerged approximately 5 to 15 cm in conditioned water during the entire curing process. Curing at the cold and warm temperatures occurred inside a refrigerator and a forced draft oven, respectively, where temperature of the conditioned water was measured at a depth equal to the middle of the specimen. Curing at room temperature took place mostly with the conditioned water buckets in the environmental chamber and on a few occasions within covered conditioned water buckets on the lab bench. When curing at room temperature, temperature measurement was of the air directly above the conditioned water for convenience, as this value was already being measured for *Slab* testing in most cases.

3.9 Testing of Strength Specimens

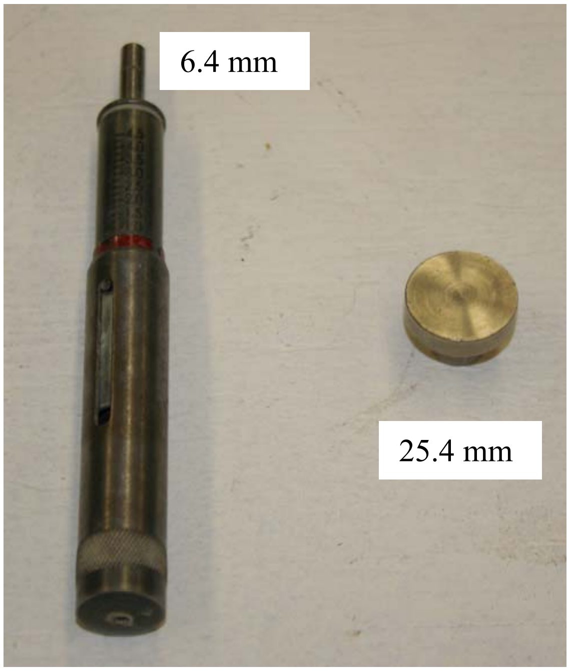
Two approaches were utilized for shear strength testing: 1) testing of stabilized slurry slabs using hand held devices that could be used by responders during the disaster event (*Slab*); and 2) unconfined compression specimens tested in a standard load frame as would be performed in routine engineering practice (*UC*). These types of testing complement each other in that the *Slab* testing is somewhat unique and *UC* testing is standard.

3.9.1 Testing of *Slab* Specimens

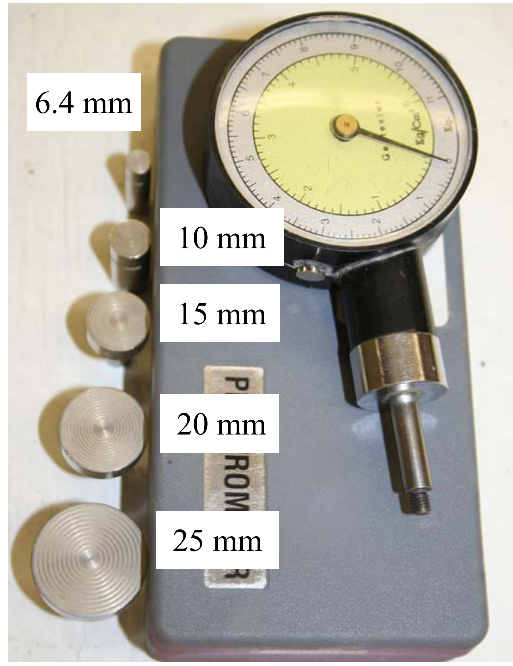
3.9.1.1 Testing of *Slab* Specimens-General Information

Testing was performed with 3 hand held devices available from test equipment distributors. The test devices are: 1) *Pocket Penetrometer (Ring)* part number HM-500 from Gilson Company*;* 2) *Pocket Geotester (Dial)* of Geotest Instrument Corporation and part number HM-502 from Gilson Company, and 3) *Pocket Vane Shear Set (Shear)* Model E-285 and part number HM-504A from Gilson Company. For the remainder of the report they are referred to using the terms in parenthesis which were selected because they identify key features of the devices. The *Dial* and *Ring* devices measure unconfined compressive strength, while the *Shear* device directly measures shear strength. Figure 3.15 shows each of the 3 devices alongside their various attachments. The devices are very robust, an operator can learn to use them effectively in a matter of minutes, and they are sent in a case that can be directly attached to clothing. Instructions on use of the devices are provided by the manufacturers and were followed in this research.

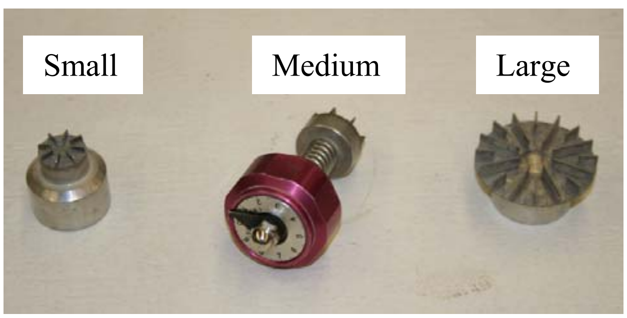
To allow a reasonable area to be tested while not exceeding gage capacity, different attachments were used as testing progressed. Initially, all devices were equipped with the largest attachment to maximize area tested, while this attachment did not exceed gage capacity. Once the stabilized slurry had matured to a stage where the initial attachments nearly exceeded the capacity of the devices, a smaller attachment was used, and so forth. Table 3.1 provides details of the attachments and corresponding device adjustments.



(a) Pocket Penetronometer (Ring)



(b) Pocket Geotester (Dial)



(c) Pocket Vane Shear Set (Shear)

(d) All Test Devices

Figure 3.15. Hand Held Devices Used for Strength Testing of Slabs

Table 3.1. Test Device Attachment Adjustment Factors

Gage	Attachment	Adjustment Factor (A_F)[1]
Dial	6.4 mm Foot	1.00
	10.0 mm Foot	2.48
	15.0 mm Foot	5.58
	20.0 mm Foot	9.92
	25.0 mm Foot	15.50
Ring	6.4 mm Foot	1
	25.4 mm Foot	16
Shear	Small Vane	4
	Medium Vane	10
	Large Vane	50

1: Divide Reading by Adjustment.

Shear stress is calculated using Eq. 3.4 or 3.5 depending on the gage used. The maximum possible reading (R) and shear stress (s_u) for the devices, respectively, are: *Dial* is 6.0 and 3 kg/cm^2; *Ring* is 4.5 and 2.25 kg/cm^2; and *Shear* is 10.0 and 2.5 kg/cm^2. Note that manufacturer instructions with the *Shear* device indicate that 1 full revolution of the device corresponds to 1 kg/cm^2 when using the medium vane and that the device is graduated in whole numbers. Readings in this research were taken in whole numbers as indicated on the device rather than in tenths as implied by the manufacturer, and the adjustment factors were determined accordingly. Either approach results in the same shear stress, though the approach taken in this research is believed to be easier on the user of the device.

$$s_u = \frac{R}{A_F(2)} \qquad \text{[\textit{Dial} or \textit{Ring} gage]} \tag{3.4}$$

$$s_u = \frac{R}{A_F} \qquad \text{[\textit{Shear} gage]} \tag{3.5}$$

Where,

s_u = shear stress at failure (kg/cm^2)
R = hand held gage reading
A_F = attachment adjustment factor

The hand held devices were periodically checked for basic functionality. The *Ring* and *Dial* gages were pressed into each other as well as onto a calibrated scale to verify appropriate readings. All 3 devices were inspected for damage, wear, signs of corrosion, etc. Unsatisfactory performance resulted in the gage being repaired or discarded.

Four types of data were taken during testing of the slabs: *Surface*, *Bottom, Horizontal-Perimeter,* and *Horizontal-Internal*. The S*urface* of the prepared slabs provided the majority of the data, was always performed first, and occurred entirely in the environmental chamber. To optimize the data that could be obtained from the samples that had been prepared, *Bottom* readings were taken from a slab at the conclusion of *Surface* testing, with *Horizontal* data taken thereafter. *Bottom, Horizontal-Perimeter,* and *Horizontal-Internal* readings were taken outside of the environmental chamber in the main laboratory. This entire portion of testing occurred within, at most, 1 hour from when the slabs were removed from the environmental chamber. Further details of all 3 types of testing are provided in the following sub-sections.

3.9.1.2 Testing of ***Slab*** Specimens-***Surface*** Testing

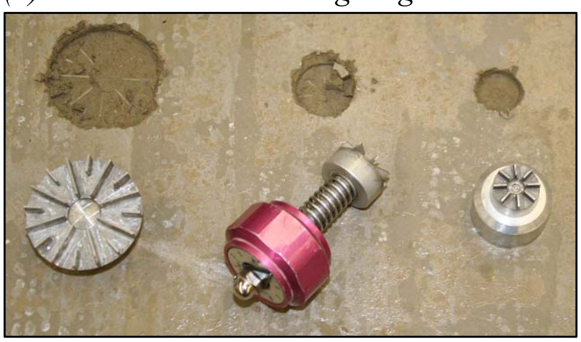
All hand held gages were used within (at most) 1 minute of each other. The initial time was recorded and used as the reference. The readings were held by follow up pointers and recorded after all 3 devices had been used to maximize the proximity of readings to one another. The order of testing was *Ring*, *Dial*, and *Shear* gages. This order was selected since it optimized user efficiency based on the way the devices can be held by the operator.

Locations of testing were completely random, even between gages. This was to eliminate bias due to testing specific locations within a given mold at a given spacing of time intervals.

Within each mold, approximately 20 to 40 groups of *Ring*, *Dial*, and *Shear* readings could be performed. The rate of strength gain affected the number of slabs required. Slower strength gain required considerably more test area (more slabs) to accommodate the appropriate attachments summarized in Table 3.1. Figure 3.16 shows the approximate area disturbed by each of the attachments for each of the 3 test devices. No readings were taken within 25 mm of each other or within 25 mm of the mold to eliminate edge effects.

(a) Tested Areas With Ring Gage

(b) Tested Areas With Dial Gage



(c) Tested Areas With Shear Gage

(d) Surface Completely Tested

Figure 3.16. Schematic of Areas Disturbed During Slab Testing

A group of *Slab* tests where more than 1 mold of the same soil, cementitious materials, cementitious content, moisture content and any other features (e.g. brackish water) was tested over time was referred to as a trial. Trials were performed in two manners; Table 3.2 summarizes test frequencies, and readings were uniformly spaced within the intervals. For example, during the first hour of protocol 1, readings were taken at 0.17, 0.33, 0.50, 0.67, 0.83, and 1.00 hours. Zero strength was entered for all scheduled readings where preparation was not complete; 0.17 hr and 0.33 hr were in this condition in some test trials. Preliminary testing was used to develop testing frequencies (≈ 650 readings not used for analysis). Protocol 1 was developed to provide high resolution, and protocol 2 was developed to allow incorporation of additional factor-level combinations. Sixty trials were conducted and discussed in this report, requiring a total of 4,556 readings with each hand held gage.

Slab testing where a single mold of stabilized slurry with the same soil type, cementitious materials, cementitious content, and moisture content was tested at a specific time interval was referred to as a variability slab. The slab would be cured for the specified amount of time (e.g. 24 hours) and then the entire surface tested using the hand held gages. The purpose was to evaluate variability of the stabilized slurry and the gages themselves. Thirty-seven variability slabs were tested, requiring a total of 919 readings with each hand held gage (25 readings per slab per gage was typical).

Table 3.2. Reading Schedule of Slab Surface Testing for Each Device

Protocol	*Test Interval (hr)*	*Reading Schedule*	*Total Readings*
1	0 to 12	6 per hour	72
	12 to 24	4 per hour	48
	24 to 36	2 per hour	24
	36 to 48	1 per hour	12
	48 to 168	1 per 1.5 hours	80
		Total Number of Protocol 1 Readings	***236***
2	0 to 14	1 per hour	14
	14 to 24	2 per hour	20
	24 to 168	1 per 24 hours	6
		Total Number of Protocol 2 Readings	***40***

3.9.1.3 Testing of *Slab* Specimens-*Bottom* Testing

Once the *Surface* of the slab was tested, the slab was extracted by removing the end of the mold and then gently flipping it so that the bottom of the slab was facing upward (Figure 3.17a). The *Bottom* of the slab was subsequently tested with the *Ring, Dial,* and *Shear* gages. Two readings per test device were taken at random within each of the slabs 4 quadrants, providing 8 readings per test device as shown in Figure 3.17b. If a quadrant was damaged during mold removal, readings were not taken in the quadrant. Damage of this nature only occurred in a few instances.

(a) Bottom of Slab-Pre Testing

(b) Bottom of Slab-Post Testing

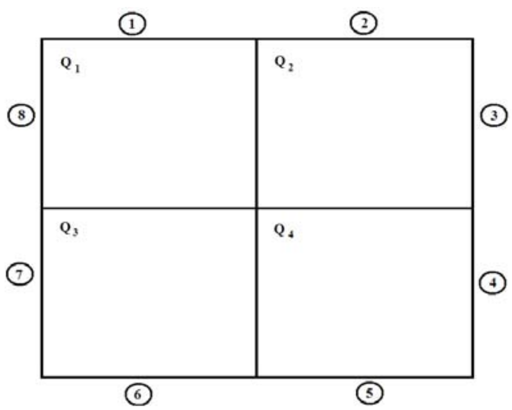
Figure 3.17. Testing of Bottom of Stabilized Slurry Slabs

The *Bottom* of the slab was tested immediately at the conclusion of *Surface* testing (See 3.9.1.2). *Surface* testing occurred in a 100% humidity environment, while the bottom testing occurred in laboratory conditions. The slab was removed from the 100% humidity environmental chamber, and testing commenced immediately on the bottom of the slab. A typical duration of testing would be just under 20 minutes.

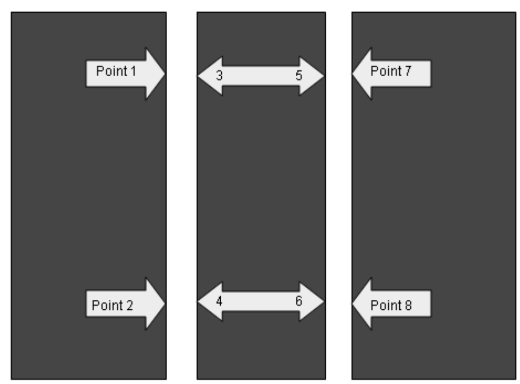
3.9.1.4 Testing of *Slab* Specimens-*Horizontal* Testing

Horizontal testing was performed immediately after *Bottom* testing (See 3.9.1.3). Typically, *Horizontal* testing commenced on the order of 20 minutes after the sample was removed from the 100% humidity environmental chamber and ended within 30 minutes thereafter. *Horizontal* readings were obtained by the *Ring* and *Dial* gages but were not obtained by the *Shear* gage since the thickness of the slabs was insufficient after both the *Surface* and *Bottom* had been tested. Note that these readings were being taken on vertical faces that would be perpendicular to the *Surface* and *Bottom* of the slabs.

Two types of *Horizontal* testing were performed: 1) *Perimeter* as seen in Figure 3.18a, and 2) *Internal* as seen in Figure 3.18b. *Perimeter* testing was performed immediately after *Bottom* testing, and *Internal* testing was performed thereafter. As seen, 8 readings were taken per test device around the sample perimeter and then internal to the sample. To perform *Internal* testing, samples were sawn horizontally (Figure 3.19a and 3.19b) using an off-the-shelf saw obtained from a local hardware store. Readings were taken on the vertical surfaces of the stabilized slab in the configuration shown in Figure 3.18b. Fibers used as stabilization additives prevented the samples from being sawn in many cases, making the only data available from these specimens the points on the perimeter of the slab. Figures 3.20a and 3.20c show a perimeter vertical face during and at the conclusion of testing. Figures 3.20b and 3.20d show an internal vertical face in the same state as the perimeter.



(a) Schematic of Perimeter Test Locations



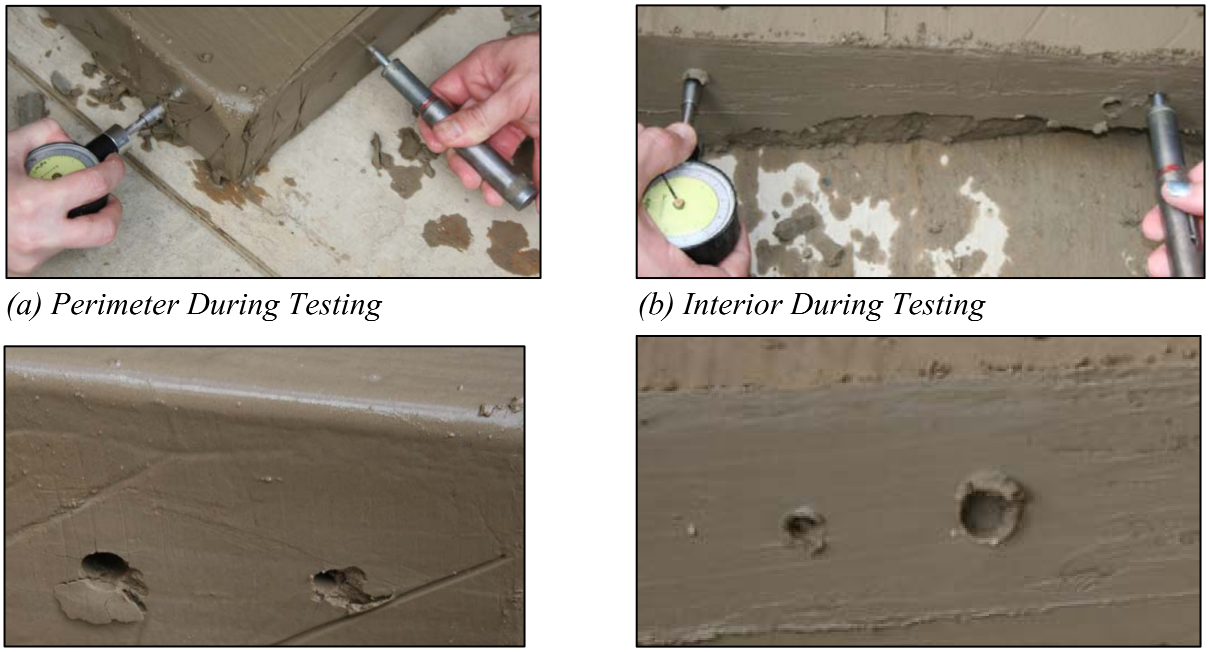
(b) Schematic of Internal Test Locations

Figure 3.18. Two Types of Horizontal Testing: Perimeter and Internal

(a) Sawing of Slabs

(b) Sawn Surface for Internal Testing

Figure 3.19. Testing of Horizontal Portions of Slabs



(a) *Perimeter During Testing* (b) *Interior During Testing*

(c) *Perimeter After Testing* (d) *Interior After Testing*

Figure 3.20. Vertical Stabilized Slurry Slab Faces During at After Testing

3.9.2 Testing of *UC* Specimens

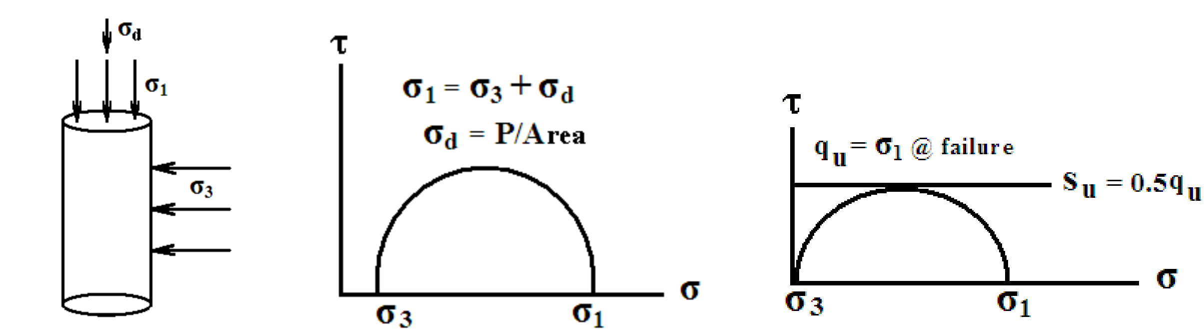
After curing was complete, *Unconfined Compression (UC)* testing was performed by referencing *ASTM D 2166-06* and *ASTM D 5102-04*. *ASTM D 2166* is the standard method for testing cohesive soils, and *ASTM D 5102* is the standard method for testing compacted lime treated soils. The tests being performed in this experimental program are perceived by the research team to fall between these standards, albeit closer to *D 2166* than to *D 5102*. *D 2166* requires load measurements that allow stress calculations to within 0.01 kg/cm^2, which is the stricter of the two standards and was followed in this test protocol. Both procedures require a height to diameter aspect ratio of 2:1 to 2.5:1, and an axial strain rate of 0.76 to 3.05 mm/min. The specimens tested had a 2:1 aspect ratio and were loaded at 2.29 mm/min. *D 2166* defines failure as the maximum stress or the stress at 15% strain, whichever comes first. *D 5102* uses 5% strain as the maximum allowable value; 15% was used in this experimental program though for most non-fiber reinforced specimens the maximum strain was below 5%. Readings were taken at 0.25% strain intervals.

Specimens were tested immediately after being taken from the conditioned water. To test the specimens they were first placed into the load frame (Figure 3.21a), or onto a tray adjacent to the load frame, where the mold was carefully removed by loosening the clamp (Figure 3.21b) and sliding the mold vertically off the specimen. A circular extruder was used to gently push some specimens from the bottom to facilitate mold removal; typically this occurred with stronger specimens. The porous stones and filter paper were not removed and were used as the loading platens to allow even pressure distribution during testing. (A metal load cap was placed on top of the top porous stone for protection.) Figure 3.21c shows a specimen ready for testing, and Figure 3.22 shows terminology associated with testing.

(a) Cured Specimen *(b) Removal of Mold* *(c) Ready for Testing*

Figure 3.21. Testing of Unconfined Compression (UC) Specimens

A group of *UC* tests with the same soil type, cementitious materials, cementitious content, moisture content and any other unique features (e.g. unprocessed soil, brackish water) tested as a function of age was referred to as a suite. *UC* suites were performed in four manners: 1) 1 room temperature cured specimen tested at 2, 4, 8, 16, 24, 48, 72, 96, 120, 144, and 168 hr for a total of 11 specimens and referred to hereafter as *Protocol 1*; 2) multiple room temperature cured specimens tested at 24, 72, and 168 hr for a typical total of 10 specimens and referred to hereafter as *Protocol 2*; 3) 4 warm temperature cured specimens tested at 14, 41, and 96 hr for a total of 12 specimens and referred to thereafter as *Protocol 3*; and 4) 4 cold temperature cured specimens tested at 96, 288, and 672 hr for a total of 12 specimens and referred to thereafter as *Protocol 4*. *Protocol 1* specimens were typically made at different times whereas *Protocol 2* specimens were typically mixed and created at 1 time. Two-hundred seventy-eight suites were conducted resulting in ≈2,775 *UC* data points. The term set was used when multiple tests were performed of a given combination at only 1 test time (3 to 5 replicates was typical). One-hundred forty-two sets were conducted resulting in ≈525 *UC* data points. The total number of *UC* tests was ≈3,300, consisting of ≈1,300 with *Soil 1*, ≈800 with *Soil 2*, and ≈ 1,200 with *Soil 3*.



(a) Test Schematic *(b) Typical Soil Stress State* *(c) Unconfined Compression*

Figure 3.22. Stress Terminology for Soils Testing

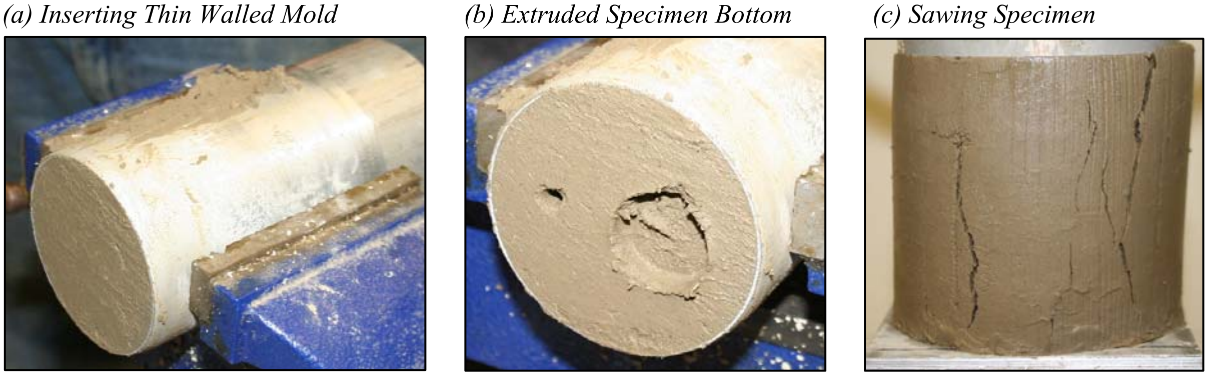
Unless specifically stated, the material was mixed and within molds within approximately 30 to 35 minutes after cement addition (referred to as normal placement or *NP*). Testing of rapid set products revealed that the elapsed time prior to having a fully prepared specimen could affect shear strength. As a result some testing was performed rapidly where all material was in molds and curing within 20 minutes of cement addition. Any time this occurred the designation rapid placement (*RP*) was indicted with the suite or set.

The elastic modulus (*E*) was determined using the *UC* stress-strain curve. The calculations considered as much of the stress-strain curve as possible to represent the modulus behavior. The average strain level below which modulus was calculated was 1.6%, 1.6%, and 1.4% for soils 1 to 3, respectively. The strain standard deviation below which the modulus values were taken were 0.7%, 0.6%, and 0.6% for soils 1 to 3, respectively.

3.9.3 Testing of *Combined* Specimens

Combined testing was performed (Figure 3.23) to provide an initial comparison of hand held gage and *UC* strength from the same mix. A slab was prepared and tested according to Section 3.7.1, and the S*urface* tested according to Section 3.9.1.2. A thin-walled lubricated metal tube with an inner diameter of 7.6 cm was then forced into the slab. The specimen within the tube was inspected for cracks and discarded if any were found. Spacers were used to extrude the specimen, and a thin portion was removed from the bottom with a wire saw. *Ring* and *Shear* testing was performed, and then all affected areas were trimmed and a *UC* test performed on the remaining material (1:1 aspect ratio). The 1:1 is not a conventional *UC* test (2:1 is standard), but to perform the testing on a material already tested for S*urface* properties, it was the deepest possible without fabricating additional molds.

(a) Inserting Thin Walled Mold *(b) Extruded Specimen Bottom* *(c) Sawing Specimen*



(d) Specimen Bottom After Sawing *(e) Specimen Bottom After Testing* *(f) Specimen After UC Test*

Figure 3.23. Photos of Combined Testing

3.9.4 Testing of Density Correction Specimens

Testing was performed as per Section 3.9.2. The only deviation was specimen preparation. Testing occurred between 72 and 72.5 hour for the 8 specimens tested.

3.9.5 Testing of Membrane Correction Specimens

Testing was performed as per Section 3.9.2. The only deviation was specimen preparation and the membrane in place during testing. Testing occurred after curing of 48 $\pm$ 1 hours, and the time of testing was recorded for each individual specimen.

3.10 Testing of pH Slurries

The pH of soil slurry ($pH_{pre\text{-}cement}$) was tested by first inserting the tip of the pH meter electrode (same meter as used throughout research) approximately 2.5 cm into the slurry. The meter was slightly agitated for a few seconds, and then the pH reading was allowed to stabilize, which typically took around 30 seconds. Once the reading had stabilized, the pH was recorded. The electrode was thoroughly cleaned using a squirt bottle containing tap water. After cement was added to the soil slurry making it a stabilized soil slurry, the pH of the slurry was measured again ($pH_{post\text{-}cement}$) within a minute of the end of mixing.

3.11 Semi-Adiabatic Calorimetry (*SAC*)

Semi-Adiabatic Calorimetry (*SAC*) is the term used herein to represent temperature profile testing of cementitious materials and/or cementitious stabilized slurries as they cure. *SAC* is commonly used to describe this type of testing, though from a technical standpoint other terms have been argued to be more technically correct. A draft *ASTM* standard refers to the approach as thermal measurements testing, while others refer to it as thermal profile testing. *SAC* was used in two manners in this report as described in the following sections.

3.11.1 *SAC* Testing of Cement Paste

Paste mixtures were prepared using a hand-held kitchen mixer with a single beater blade. Temperature data was collected using thermocouples and data loggers connected to a computer with specialty software. Cement was first introduced into the mixing bowl, and then mix water was added. When *PoP* was incorporated, it was pre-dissolved into the mix water by stirring for ≈ 15 seconds. The pure mix water or mix water-plaster solution was introduced into the cement (or soil and cement in some instances) and mixed for 60 seconds at low speed (≈ 350 rpm) so as not to generate foam or froth in the paste. Each mixed paste specimen was immediately poured into the mold (Figure 3.24) and covered with a plastic lid with a spacer (open-cell foam or wood) to keep the probe thermocouple tip centered in the paste during data collection. The thermocouple was inserted, and data collection initiated 90 seconds after the start of mixing which continued for several hours. Data collection consisted of the paste specimen temperature (T_{samp}) and reference specimen temperature (T_{ref}).

Two series of tests were performed with *SAC* on paste mixtures. The only noteworthy difference in their experimental programs was the Figure 3.24 mold used. *Series 1* used the 350 cc doubled paper cup molds where the cementitious content for batching was 500 g. *Series 2* used the 520 cc plastic 7.6 cm diameter and 11.4 cm tall molds where the cementitious content for batching was 700 g. *Series 2* also used a reference channel consisting of a specimen of sand and water.

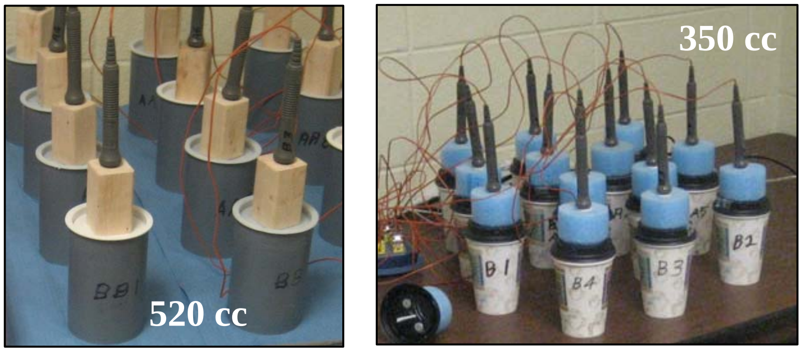


Figure 3.24. Molds and Probes Used for Paste Testing

3.11.2 *SAC* Testing of Soil Slurries

3.11.2.1 Equipment Used

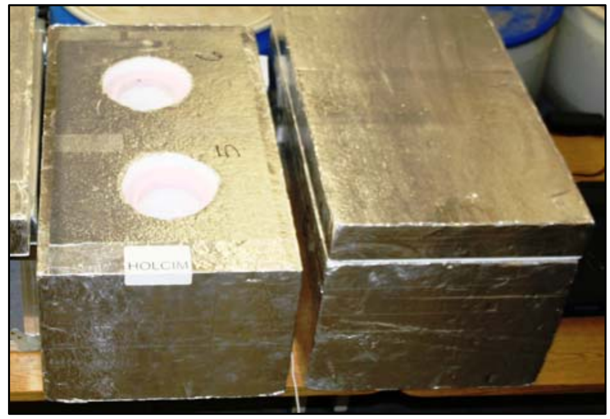
Figure 3.25 shows the 5 different equipment types used to test stabilized soil slurries, while Table 3.3 summarizes key characteristics of each device. All devices used contact temperature measurement rather than probes inserted into the specimens. Contact sensors provide many advantages including not damaging the specimen so it can be tested for strength at the conclusion of thermal profile testing. With exception of the *Thermos Jug,* the remaining devices use standard size plastic concrete cylinder molds.

The *AdiaCal Field Calorimeter* is commercially available and was designed to measure concrete maturity (Figure 3.25a). The contact sensors are located at the bottom of the specimens and use a metal plate to assist heat transfer. The passive data logger allows the device to operate on its own power (i.e. no computer needed) during testing, and data can be downloaded to a computer at the conclusion of testing. The device also features a protective case with locking lid and handle, which allows for easy transport from job site to job site.

The *Holcim Insulated Block* and *Holcim Convertible Block* are custom made devices designed specifically for *SAC* testing of concrete and paste mixtures (Figures 3.25b and 3.25e). The *Holcim Convertible Block* converts or adapts to different specimen sizes and insulation levels. Both devices are made from Styrofoam blocks with cavities cut out to accommodate the specimens. The *Holcim Insulated Block* is also wrapped in foil tape and has a Styrofoam lid for further insulation. The *Holcim Convertible Block* uses an active data logging system, meaning the data logger must be connected to a computer during testing and records temperature data in real time. Testing was performed using the *Holcim Convertible Block* with and without a Styrofoam lid, and the presence of the lid is denoted throughout testing.

The *Thermos Jug* and *Modified Ice Chest* (Figures 3.25c and 3.25d) were modifications of commercially available products. A hole was drilled through the top of the *Thermos Jug* to allow a probe thermistor to be inserted into the middle of the specimen. The *Modified Ice Chest* is an off-the-shelf product that has been filled with spray foam insulation. A hole was also drilled through the side of the ice chest and a probe type thermistor was inserted at the bottom of the specimen where it was utilized as a contact sensor.

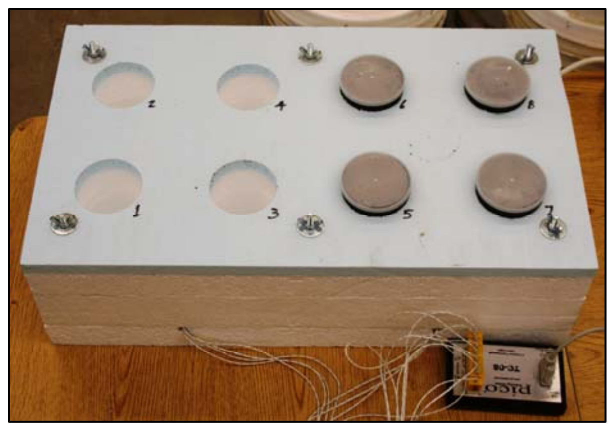
a) AdiaCal Field Calorimeter



b) Holcim Insulated Block

c) Thermos Jug

d) Modified Ice Chest



e) Holcim Convertible Block Without Lid

Figure 3.25. Calorimetry Equipment Used for Stabilized Soil Slurries

Table 3.3. Calorimetry Equipment Used for Stabilized Soil Slurries

Device	**Specimen Slots**	**Specimen Size (cm)**	**Data Acquisition**
1) AdiaCal Field Calorimeter	4	10.2 by 20.4	HOBO Thermistors (4 channel) with HOBOware Pro software
2) Holcim Insulated Blocks	2	10.2 by 20.4	HOBO Thermistors (4 channel) with HOBOware Pro software
3) Thermos Jug	1	7.1 by 14.0	LogTag Thermister Probe (1 channel) with LogTag Analyser software
4) Modified Ice Chest	1	10.2 by 20.4	LogTag Thermister Probe (1 channel) with LogTag Analyser software
5) Holcim Convertible Block	8	7.6 by 15.2	Pico TC-08 Wire Thermocouples (8 channel) with ThermoCal software

Figure 3.26 shows the curing chamber used during testing. The device is designed to cure concrete according to *ASTM C 31*, and is capable of cooling and heating. The chamber was filled with ≈ 13 cm of water and had a metal rack just above the water level. The equipment controls water temperature in the bottom of the device so a calibration curve should be developed to relate water temperature of the bath to the air temperature above the bath. Adjustments may also be needed if the temperature outside the environmental chamber changes considerably. When needed, the curing chamber was used to adjust the beginning temperature of the ingredients and the temperature surrounding the *SAC* equipment. The curing chamber was set to the desired temperature, and the *SAC* device placed inside along with the batched soil slurry and cement, which were stored separately in sealed containers. The chamber was closed for 24 hour allowing the *SAC* equipment and materials equalize before testing. Room temperature testing did not require the curing chamber in most instances.

Figure 3.26. Curing Chamber Used During Select SAC Testing

3.11.2.2 Specimen Preparation and Testing

If reference blank specimens are used (e.g. sand and water), they are inserted into the calorimetry device (Figure 3.27) before cementitious specimens to allow thermocouple readings to equalize. Stabilized specimen preparation begins with quickly removing the temperature conditioned pre-batched soil slurry and cement from the curing chamber for mixing but only removing the material required for the mix of interest. Opening and closing the curing chamber should be performed quickly and minimized to preserve the environmental conditions inside the chamber. Also, the time from when the materials are removed from the curing chamber to when the prepared specimens are placed into the calorimetry device should be minimized. The lid to the curing chamber is to be closed between the placement of each specimen, or between the removal of each mix. Temperatures reported herein are the conditioned temperatures of the pre-batch materials prior to mixing and placement into molds.

Before cement addition, the soil slurry was re-agitated using the same hand-held drill and mixing attachment described in Figure 3.5, and the initial slurry temperature was recorded using a digital thermometer. The pre-batched cement was then added (time of addition was recorded) and mixed with the hand-held drill for 2 minutes. After mixing, a spoon was used fill the specimen molds with stabilized soil slurry (Figure 3.27). Each mold was filled with 3 lifts, and each lift was consolidated by hitting the bottom of the mold against a table to remove any entrapped air pockets.

After the third lift, a small amount of extra material was placed on top to allow the surface to be struck off using a straight edge. Thereafter, a plastic lid was applied and the outside of the mold was wiped of excess material. Then, specimens were inserted into the calorimetry device, which was inside the curing chamber when needed. When the calorimetry device contained a lid, it was placed after all specimens were in the chambers and the lid was held in intimate contact by small weights on top of the lid. Immediately after placing a specimen in the calorimetry device, data collection began on that channel of the system, and the start time for each channel was recorded for reference. Data collection continued for the specified time (typically 24 hours) to complete the *SAC* procedure. Data collection consisted of measuring the stabilized specimen temperature (T_s) and when appropriate the reference specimen temperature (T_r).

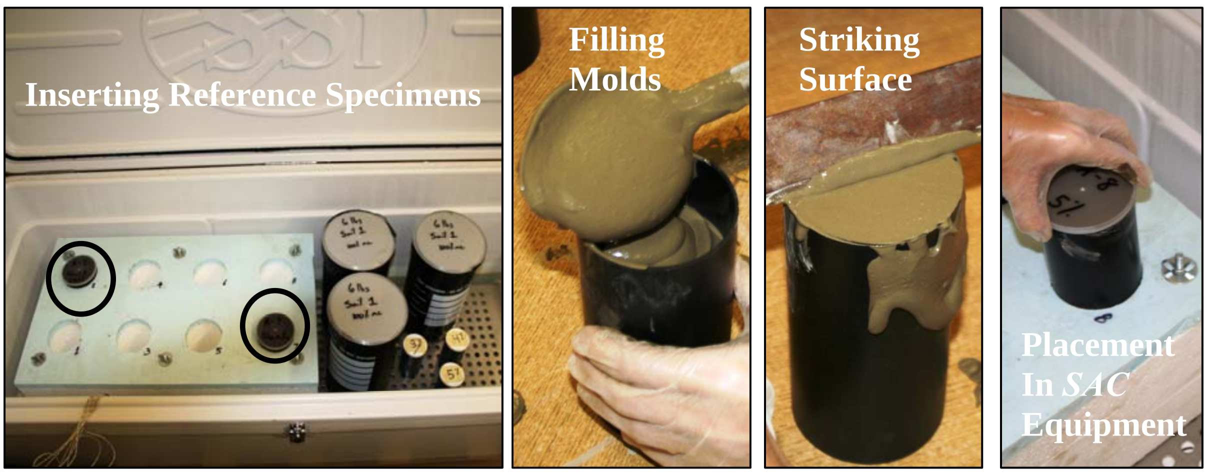


Figure 3.27. Preparation and Testing of SAC Specimens

CHAPTER 4 - STABILIZATION MATERIALS

4.1 Applicability Overview of Stabilization Materials

A wide range of stabilization materials were considered for the task of rapidly creating controlled low strength emergency construction material. Chapter 2 summarizes the classes of available materials. Four categories of stabilization materials were selected: 1) portland cements which are hydraulic; 2) calcium sulfoaluminate cements which are hydraulic; 3) ground-granulated blast furnace slag (*GGBFS*) which has hydraulic and pozzolanic properties; and 4) chopped polymer fibers. Plaster of Paris (*PoP*) was also used as a source of calcium slufate hemihydrate for select testing. Hydraulic cements are ideal materials for responding to a water-based disaster, and *GGBFS* has been shown advantageous in some soil mixing applications with clay soils. Chopped polymer fibers have the potential to compliment another stabilization material (e.g. hydraulic cement) provided they can be uniformly mixed into the soil slurry with the cement.

4.2 Portland Cement

Eight portland cements were tested as part of this research; 4 were commercially available, and 4 were products developed specifically for this research. The 4 specialty portland cements were manufactured by interrupting production at full scale plants. Cement chemistry terminology was explained in Chapter 2 and used in this section without explanation.

4.2.1 Modified Cement Specification Concept

A product optimized with special processes and materials could perhaps offer the most *ideal* characteristics for any given application, but it would be unlikely that cement plants would develop the capabilities for producing such a product since there would be no established and ongoing market. Sufficient storage for normal production and shipment of cement is one of the industry's greatest challenges; lack of consistent demand would inhibit stockpiling. Thus the development and marketing of an *ideal* application-specific cement for emergency repair and construction would not be likely.

A more realistic approach to product development was pursued in this research – to produce a special product with qualities as close as possible to *ideal* that can and will be immediately produced if needed using existing cement plants and on site materials. Essentially the idea is on-demand specialty cement, to be produced with little or no advanced preparation, and with only minor modification of traditional cement specifications.

The central concept of this research was to develop a special portland cement product that could be made available almost immediately for expedient disaster mitigation work anywhere in the United States. A modified specification was developed to provide simple guidance that could be implemented by any portland cement manufacturing plant for immediate production of a rapid repair product.

A modified specification approach would seem to be the most practical and useful solution for disaster-related needs. In order to refine the set and early strength characteristics of the proposed product for the application, the *ASTM C 150* cement specification with only minor modifications such as $CaSO_4$ (gypsum) content (component responsible for SO_3 oxide), and possibly fineness would be needed. These modifications were evaluated at two Holcim (US), Inc. plants (Artesia, MS and Theodore, AL). The trial grinds and testing provided data for *Type I* and *Type II* clinkers from wet and dry production processes.

4.2.2 Availability of Portland Cement

The concept of a modified cement specification is even more appealing when taken in context with the availability of portland cement. Using the plant's existing raw materials and normal processes, availability would be almost immediate. The emptying of a shipping silo to allow a location for the ground product to be stored just prior to loading into a shipment truck would likely be the biggest time consideration (could be done in a few hours, in most cases). Cement plants are located throughout the US, and one or more would likely be within a reasonable shipping distance of a site. For example as of January 2009, Holcim (US), Inc. operated 13 portland cement plants around the continental US, and the industry total was near 100. The plants are scattered geographically according to the availability of raw materials but also according to cement markets, as shipping of such massive products is quite expensive. A graphical indication of portland cement availability can be seen in Figure 4.1. As seen in Figure 4.1, it is very likely that there is a cement plant within approximately 300 km of the disaster location.

The 2008 production capacity of US portland cement plants ranged from around 272,000 to 2,720,000 metric tons (300,000 to 3,000,000 tons) per year, with an average capacity of around 727,000 metric tons (800,000 tons) per year. Cement quantities are referred to in either metric tons (1,000 kg) or in short tons (2,000 lb). The term ton in this report refers to a short ton, while a metric ton is referred to by its full description. The aforementioned capacities should be viewed as *average* capacity for *Type I* cement of around 2,000 metric tons (2,200 tons) per day. The values would be somewhat less for the finer grind materials considered, estimated at 1,450 metric tons (1,600 tons) per day. Note that a few thousand tons of silo storage should be available at any cement plant within hours or a couple of days, based on emergency needs, and the product could be ready for shipment for emergency construction essentially overnight, in most cases. Thereafter, a continuous supply stream could be achieved until the needs of the disaster were met.

Cement production revolves around two components: 1) kiln-clinker capacity; and 2) finish mill grinding capacity. Ideal conditions would have equal capacities of both. Realistic conditions typically dictate more finish mill grinding capacity since this component is more prone to malfunction which requires production downtime for repairs. Therefore, the key to cement production is the capability of the finish grinding mill(s). Finish grinding capacities are often the driving force behind the type of cement that can be produced under routine conditions (e.g. *Type I* vs. *Type III,* which requires more grinding time).

Controlled long term storage conditions are one of, if not the, biggest operational challenge faced by the cement industry. For this and other reasons already discussed, a just in time flow of cement to the disaster area would be optimal for the industry. This should not pose any problems at the disaster area since the production capacity at a nearby plant(s) will almost certainly exceed the ability to transport and/or use the material on site.

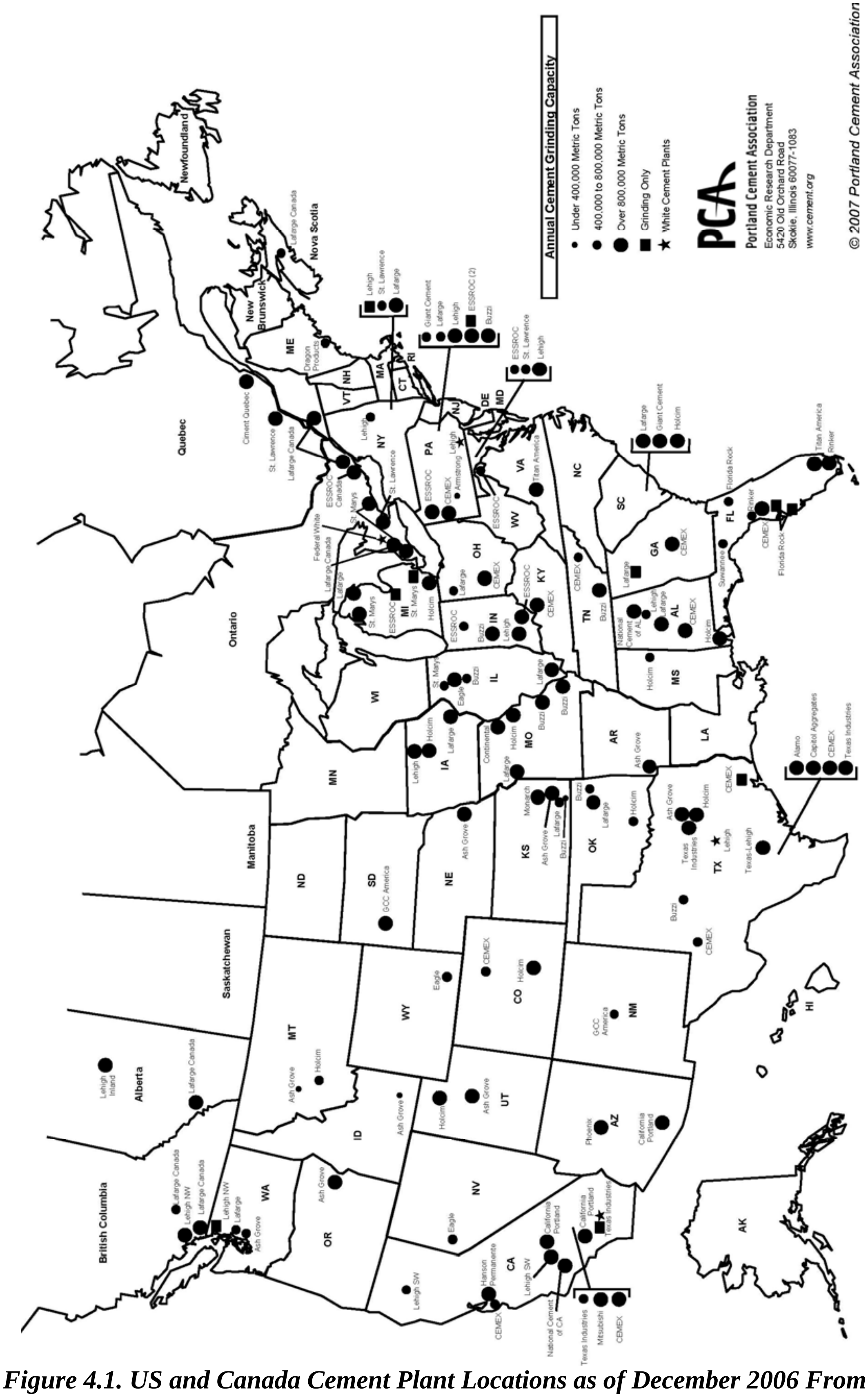


Figure 4.1. US and Canada Cement Plant Locations as of December 2006 From PCA

4.2.3 Economics of Portland Cement

As a point of reference, \$110/metric ton (\$100/ton) was used as the base cement price of *Type I* cement in absence of shipping costs. Shipping costs will be the same regardless of the chemistries of the cements. *Type III* cement typically costs 5 to 10% more due to extra grinding. At present the specialty cements are estimated to cost approximately 20% more than *Type I* cement. Using the modified specification approach, the need for material to be pre-produced is eliminated, providing economic advantage. Additionally, the just-in-time approach eliminates the need for packaging. The material can be sent in bulk directly from production to the disaster area.

4.2.4 Properties of Portland Cements Tested

Table 4.1 provides material properties of the portland cements tested. The specialty cement properties were developed based on the understanding that certain parameters are far easier than others to adjust. The overall process logistics, kiln operation, and raw material sources cannot be changed rapidly due to the need for equipment modifications, operator training, and similar. At any given disaster location, the cement plant of greatest logistical advantage could be either dry process or wet process, and any changes to clinker properties would also present additional material storage and handling issues, as most plants have limited capacity for segregating different clinkers. The most feasible process adjustments for rapid implementation would be those related to finish grinding, such as fineness, SO_3 targets (gypsum feed rate), and possibly mill temperatures.

C_3S and C_2S react with water to form *CaOH* and *C-S-H*. C_3A, $CaSO_4$, (assuming gypsum is used), and water combine to form ettringite, which controls setting (among other properties). In very general terms, gypsum coats the silicates and aluminates due to the formation of ettringite, limiting interaction with water and slowing initial hydration. In absence of this behavior, many common types of cement would set within minutes, though strength development would be adversely impacted. Modification of the traditional $CaSO_4$ "optimum", however, presents a possible opportunity to enhance performance for a specific application. Manipulation of the ettringite interaction and *C-S-H* formation was a focus when developing the specialty cement properties.

In May of 2008 2 specialty cement samples were produced at Artesia by interrupting normal production of *Type I* cement and modifying fineness and gypsum feed. As soon as consistency was achieved and maintained for a few minutes, test samples were obtained for property analysis and laboratory testing. The cements are referred to hereafter as *Specialty Cement 1 (SC1)* and *Specialty Cement 2 (SC2)*. Compared with "normal", sulfate-optimized *Type III* cement, *SC1* has a reduced SO_3 with a near-normal Blaine fineness and *SC2* has a reduced SO_3 with a somewhat higher Blaine fineness. Both are high in C_3A, characteristic of clinker made at Holcim's Artesia plant, some of the highest in the southeast.

In December of 2009 two specialty cements were produced at Theodore by targeting normal *Type III* Blaine fineness and finish mill temperatures while modifying the gypsum feed. As soon as consistency was achieved within the plant and maintained for a few minutes, test samples were obtained for property analysis and laboratory testing. The cements are referred to hereafter as *Specialty Cement 5 (SC5)* and *Specialty Cement 6 (SC6)*. These cements have reduced SO_3 levels.

Producing the 4 specialty cements through full-scale plant production was a major benefit to the research. Although similar samples could be produced using laboratory simulation methods, representative performance could not be assured. Particle size distribution of mill-ground cement cannot be easily replicated, and the *$CaSO_4$* forms and solubility would likely be different between laboratory simulation and full-scale production.

Table 4.1. Properties of Portland Cements Tested

Cement[1] **Source**[2]	***T I*** ***A***	***T III*** ***A***	***SC1*** ***A***	***SC2*** ***A***	***T I/II*** ***Th***	***T III*** ***Th***	***SC5*** ***Th***	***SC6*** ***Th***
SiO_2 (%)	19.29	19.36	20.26	20.39	19.28	19.43	20.66	19.93
Al_2O_3 (%)	5.42	5.48	5.91	6.01	4.51	4.60	5.01	4.89
Fe_2O_3 (%)	2.52	2.60	2.67	2.66	3.45	3.39	3.52	3.45
CaO (%)	65.30	64.60	65.73	65.92	64.65	63.57	64.90	64.17
MgO (%)	0.85	0.85	0.89	0.90	1.22	1.34	1.26	1.27
SO_3 (%)	3.24	4.02	1.76	1.56	3.16	4.73	2.19	3.52
Na_2O (%)	0.26	0.23	0.27	0.24	0.21	0.22	0.23	0.24
K_2O (%)	0.41	0.41	0.42	0.39	0.32	0.34	0.34	0.32
C_3S (%)	70.03	63.83	65.08	64.80	72.41	61.86	62.23	61.98
C_2S (%)	2.46	7.35	8.99	9.56	0.66	9.04	12.29	10.38
C_3A (%)	10.09	10.12	11.13	11.43	6.13	6.46	7.32	7.12
C_4AF (%)	7.67	7.92	8.13	8.08	10.49	10.31	10.72	10.49
Na_2O_{eq} (%)	0.52	0.50	0.54	0.50	0.43	0.44	0.45	0.45
Free Lime(%)	0.19	0.21	0.22	0.17	0.41	0.22	0.43	0.34
LOI (%)	1.75	1.63	1.12	1.18	2.86	2.62	1.48	1.65
Blaine (m^2/kg)	386	521	503	636	401	531	543	555
Pass 45 μm (%)	97.2	99.8	99.9	98.4	94.2	98.7	99.1	98.1
Vicat Initial (min)	105	73	80	10	110	80	80	80
Vicat Final (min)	265	173	160	20	270	170	155	155
Air (%)	9	7	8	10	6	7	6	7
NC (%)	25.7	30.0	31.7	39.2	25.5	27.6	28.9	28.8
pH[3]	12.70	12.75	12.74	12.78	12.67	12.72	12.95	12.87
1-day strength (MPa)[4]	19.1	26.0	24.0	31.8	16.1	26.2	22.5	23.5
3-day strength (MPa)[4]	27.3	35.8	36.4	44.6	27.9	37.3	36.5	31.3
7-day strength (MPa)[4]	37.4	42.1	47.4	50.9	36.1	44.8	46.3	39.4

1: T = ASTM C 150 Type; SC refers to specialty cement
2: A = Holcim Artesia, MS (wet process plant); Th = Holcim Theodore, AL (dry process plant)
3: 20 g cement and 40 ml deionized water were tested in the same manner as soil solids after 15 min hydration
4: 1, 3 and 7 day compressive strengths according to ASTM C 109 with exception of SC2 where flash set prevented preparation of specification mortar cubes. Normal mortar consistency was restored when terra alba was used at 2.33% replacement by mass to raise the SO_3 by 1%. Sulfate requirements typically increase with fineness and this sample had the highest fineness and lowest SO_3

Performance for this application is evaluated on the basis of rapid strength gain for slurry mixtures. To this end, it is vital to understand that this type of performance cannot be predicted based on laboratory documentation of cement properties. Even reporting of the content of the principal compounds in cement, as required in current specifications (via the Bogue Equations), is based on assumptions regarding kiln conditions and clinker cooling that may not prove valid. Actual chemistry and physical properties of both clinker and finished cement cannot be completely characterized in routine testing. Performance testing of cementitious combinations is critical to understanding the contributions of each variable.

Phase changes of $CaSO_4$ also occur in cement during the final grinding process, which also complicates the chemistry and necessitates performance testing. Typical final grinding occurs in a finish mill at temperatures that may vary seasonally, up to 120 C or higher. Specific mill temperatures tend to influence these $CaSO_4$ phase changes (relative states of hydration/dehydration), resulting in variations in $CaSO_4$ solubility that can impact performance in several ways; phase changes are somewhat difficult to quantify and control.

4.3 Calcium Sulfoaluminate Cements

Table 4.2 provides material properties of the calcium sulfoaluminate cements tested. The *CTS RS* cement is conventionally used with aggregate to achieve compressive strengths in excess of 14 MPa within 2 hours. *SC3* and *SC4* are specially blended cements based on high calcium sulfoaluminate (*CSA*) clinker and anhydrite that can absorb considerable water during hydration. *SC3* would be considered medium performance and *SC4* a high performance blend. These types of materials are used for highly specialized (*w/c* ratio up to 1.75:1) uses in underground mining. The mining cement blends were slightly modified for use in the current research. These products are available commercially, though to a limited market. As a result, they were referred to as specialty cements in this research.

Table 4.2. Calcium Sulfoaluminate Cement Properties

Cement[1] **Source**[2]	***RS*** ***CTS***	***SC3*** ***CTS***	***SC4*** ***CTS***
SiO_2 (%)	15.4	11.3	11.1
Al_2O_3 (%)	13.7	18.5	19.2
Fe_2O_3 (%)	2.4	1.3	1.2
CaO (%)	50.8	44.8	43.9
MgO (%)	1.3	3.8	4.0
SO_3 (%)	12.5	17.3	17.5
C_3A	0	0	0
Fluoride	Trace	Trace	Trace
$C_{11}A_7$	Trace	Trace	Trace
Free lime (%)	<1	1.8	1.2
LOI (%)	2.8	2.3	2.8
Blaine (m^2/kg)	610	893	808
Passing 45µm (%)	95.0	97.4	95.6
Initial Vicat (min.)	17	5	<5
Final Vicat (min)	30	15	<5
pH[3]	11.86	12.72	10.62
C_4A_3S (%)	26.0	30.4	28.6
CS Anhydrite (%)	12.0	18.5	18.3
1-day strength (MPa)[4]	30.8	46.6	42.7
3-day strength (MPa)[4]	39.2	51.2	47.4
7-day strength (MPa)[4]	43.1	55.8	55.7

1: SC = Specialty blended cement; RS = Rapid Set® Cement
2: CTS = CTS Cement in Cypress, CA
3: 20 g cement and 40 ml deionized water were tested in the same manner as soil solids after 15 min hydration
4: 1, 3 and 7 day compressive strengths according to ASTM C 109 with exceptions to method discussed in text

As indicated in Table 4.2, *ASTM C 109* was modified to measure mortar cube strengths. *CTS RS* was tested with a *w/c* ratio of 0.4 to produce a flow of 110 ± 5, whereas the standard *w/c* for testing is 0.485. *SC3* and *SC4* mortar cubes were proportioned differently than the standard 1 part cement and 2.75 parts sand by mass. Each contained 1 part cement and 1 part sand by mass. *SC4* also contained 0.75% citric acid by mass. The *w/c* ratio was 0.485, and the specimens were modified to achieve comparable flows for compressive strength tests.

Juarez, Mexico is the primary production facility of the products investigated, but the product is available in bulk. Company literature states that CO_2 emissions are 32 to 36% less than conventional portland cement. The products are somewhat similar to portland cement in mineralogical composition, while its main constituents are calcium sulfoaluminate, dicalcium silicate, and anhydrous calcium silicate. Cost of the product is typically on the order of 3 times that of portland cement.

4.4 Ground-Granulated Blast Furnace Slag (*GGBFS*)

GGBFS was obtained from the Holcim facility in Birmingham, AL. The material met requirements for Grade 100 according to *ASTM C 989* and *AASHTO M-302*. The results of testing can be found in Table 4.3.

Table 4.3. Properties of *GGBFS* Tested

Test Summary	Property	Result
GGBFS	Sulfide-S (%)	0.8
	Sulfate Ion-SO_3 (%)	2.02
	Blaine Fineness (m^2/kg)	585
	45 µm (+), No. 325 (%)	0.55
	Air Content (%)	4.51
	pH	11.53
Reference Cement[1]	Total Alkalies as Na_2O (%)	0.82
	C_3S	53
	C_2S	9
	Blaine Fineness (m^2/kg)	384
	Strength, 7-day (MPa)	31.1
	Strength, 28-day (MPa)	39.1
GGBFS-Reference Cement	Strength, 7-day (MPa)	26.0
	Strength, 28-day (MPa)	43.1
	Slag Activity Index (Avg 7 day)	93
	Slag Activity Index (Avg 28 day)	118

1: The data provided on the reference cement is that taken for periodic demonstration of compliance. It is not the data used to calculate the Slag Activity Index.

4.5 Blended Materials from Construction Project

A blend of 75% *GGBFS* and 25% *Type I* portland cement was taken from a construction project at the *Inner Harbor Navigation Channel* (*IHNC*) on May 27, 2009. The material has been labeled *Standard Blend-Hayward Baker* (*SB-HB*). The portland cement and *GGBFS* are from different sources than the other materials tested in this report. The only property measured on the material was the pH, which was 12.83.

4.6 Chopped Polymer Fibers

Two contrasting types of fibers were used in the research: *Geofibers® 3627BT* (referred to in this report as *F70*) and *Grace Microfiber™* (referred to in this report as *F20*). *F70* is particularly different than *F20* in terms of length (70 mm versus 20 mm) and mixing into the slurry. The *F20* fibers are used in the concrete industry to resist plastic shrinkage cracks, while the *F70* fibers were developed for and are used in soil stabilization. Standard dosage rates of *F70* and *F20* fibers are 3200 to 6400 g/m^3 and 300 to 600 g/m^3, respectively. Tables 4.4 and 4.5 provide properties of these fibers as obtained from the manufacturers.

Table 4.4. Properties of *F70* Fibers

Property	Test Method	Result/Requirement
Fiber Type	----	Discrete Tape
Reactivity	----	Inert
Moisture Absorption	----	None
Fiber Length	----	70 mm
Color	----	Black
Polypropylene	ASTM D 4101[1]	99%, minimum
Specific Gravity	ASTM D 792	0.91
Carbon Black Content	ASTM D 1603	0.6%, minimum
Tensile Strength	ASTM D 2256	276 MPa, minimum
Tensile Elongation	ASTM D 2256	15%, maximum
Elastic Modulus (*E*)	ASTM D 2101	4.14 GPa, minimum

1: Group 1/Class 1/Grade 2

Table 4.5. Properties of *F20* Fibers

Property	Result
Material	100% Polypropylene
Fiber Type	Microfilament
Color	White
Fiber Length	20 mm
Number of Fibers per kg	$110(10^6)$
Specific Gravity	0.91
Moisture Absorption	None
Elastic Modulus (*E*)	3.45 GPa
Melting Point	160 C
Ignition Point	590 C
Alkali, Acid, and Salt Resistance	High

4.7 Plaster of Paris

Plaster of Paris (*PoP*) contains calcium sulfate hemihydrates, calcium carbonate, and crystalline silica; the pH was measured to be 10.41. It was used as a supplementary cementitious material in select portions of this research to adjust SO_3 content to investigate sulfate balance and shear strength development. An *XRF* analysis was performed on the material and revealed approximately 35% SO_3. To adjust the composite SO_3 of the cementitious material, 2.86 g of *PoP* were added to 100 g of portland cement for every 1% increase in SO_3 that was desired. *PoP* was added to the soil slurry in the same manner and at the same time as portland cement.

CHAPTER 5 - SOIL AND WATER PROPERTY TEST RESULTS

5.1 Soil Property Test Results

5.1.1 Atterberg Limits

Table 5.1 contains Atterberg Limit test results. Testing was performed on the soil prior to stabilization additive addition, which was the case for all properties presented in this chapter. Unprocessed samples were sealed after acquisition and remained sealed until testing. Processing affected Atterberg Limits, as results were higher when unprocessed.

Table 5.1. Atterberg Limits Test Results

Soil	Processed	Test	*LL*	LL_{OD}	*PL*	*PI*
1	Yes	1	61	55	17	44
		2	47	50	16	31
		3	48	45	16	32
		4	51	54	17	34
		5	51	52	17	34
		6	68	64	20	48
		7	53	52	18	35
		8	55	53	18	37
		9	62	58	19	43
		10	63	59	19	44
		11	50	50	18	40
		12	50	50	17	29
		Avg	***55***	***54***	***18***	***38***
2	Yes	1	65	56	28	37
		2	108	101	46	62
		3	105	99	44	61
		4	101	92	42	59
		5	101	92	42	59
		6[a]	123	112	53	70
		Avg	***101***	***92***	***43***	***58***
2	No	1	145	98	50	95
		2	115	90	42	73
		3	196	108	105	91
		4	164	102	71	93
		5	139	92	55	84
		6[a]	178	121	58	120
		Avg	***156***	***102***	***64***	***93***
3	Yes	1	94	67	31	63
		2	86	64	36	50
		3	75	75	32	43
		4	73	75	30	43
		5	76	69	29	47
		6	69	70	32	37
		7	69	63	30	39
		Avg	***77***	***69***	***31***	***46***
3	No	1	100	70	32	68
		2	105	76	33	72
		3	106	74	33	73
		4	104	72	32	72
		5	113	78	38	75
		Avg	***106***	***74***	***34***	***72***
4	No	1	54	56	19	35

a: Split sample; half was processed and half was not.

5.1.2 Particle Size Distribution

Table 5.2 and Figures 5.1 to 5.6 provide particle size distribution (*PSD*) results. To calculate the sand, silt, and clay fractions the following breakpoints were used: 1) sand was material larger than 0.075 mm (75 µm); 2) silt was material smaller than 0.075 mm (75 µm) but larger than 0.002 mm (2 µm); and 3) clay was material smaller than 0.002 mm (2 µm). The measured sample G_s was used to calculate percent passing for the hydrometer.

Hydrometer calculations assume spherical particles. Sodium hexa metaphosphate may affect organic particles since they are more flakey. The noticeable *PSD* curve drops are believed to be due to G_s related issues used in calculations (*Soil 2* in particular) and not actual phenomena. Tests 2, 3, and 6 for processed *Soil 2* were examined for necessary G_s adjustments to smooth the *PSD* curve discontinuities. Adjusted G_s values for tests 2, 3, and 6 were 2.15, 2.25, and 2.15, where measured values were 2.37, 2.36, and 2.36, respectively. *Soil 2* data in terms of percent sand, silt, and clay should be considered in the aforementioned context. Activity values calculated for *PSD* curves with noticeable drops are questionable.

Table 5.2. Particle Size Distribution Test Results

Soil	Processed	Test	Sand (%)	Silt (%)	Clay (%)	Activity
1	Yes	1	8.9	53.1	38.0	1.16
		2	13.0	53.0	34.0	0.91
		3	8.7	61.1	30.2	1.06
		4	4.0	55.0	41.0	0.83
		5	3.5	54.0	42.5	0.80
		6	3.4	36.5	60.1	0.80
		7	5.1	47.7	47.2	0.74
		8	3.0	48.2	48.8	0.76
		9	5.0	38.0	57.0	0.75
		10	3.9	39.4	56.7	0.78
		11	6.0	41.0	53.0	0.75
		12	5.5	52.2	42.3	0.69
2	Yes	1	11.2	36.8	52.0	0.71
		2	0.0	37.0	63.0	0.98
		3	0.0	34.0	66.0	0.97
		4	2.6	35.9	61.5	0.96
		5	0.0	36.0	64.0	0.92
		6	0.0	37.0	63.0	1.11
2	No	1	1.9	27.1	71.0	1.33
		2	10.4	35.6	54.0	1.35
		3	10.1	71.9	18.0	5.06
		4	3.6	56.4	40.0	2.33
		5	6.5	51.5	42.0	2.00
		6	2.0	26.0	73.0	1.67
3	Yes	1	6.6	41.8	51.6	1.22
		2	9.1	40.1	50.8	0.98
		3	3.1	36.9	60.0	0.72
		4	3.3	40.7	56.0	0.77
		5	2.9	41.1	56.0	0.84
		6	3.0	31.7	65.3	0.57
		7	3.2	31.3	65.5	0.60
3	No	1	6.6	45.4	48.0	1.42
		2	3.1	49.0	47.9	1.50
		3	2.5	52.0	45.5	1.60
		4	4.7	49.0	46.3	1.56
		5	2.5	52.0	45.5	1.65
4	No	1	21[a]	79[a] (Fines-silt and clay)		
5	No	1	11	89 (Fines-silt and clay)		

a: Sample size was 110 g, which is less than allowable value in ASTM C 117.

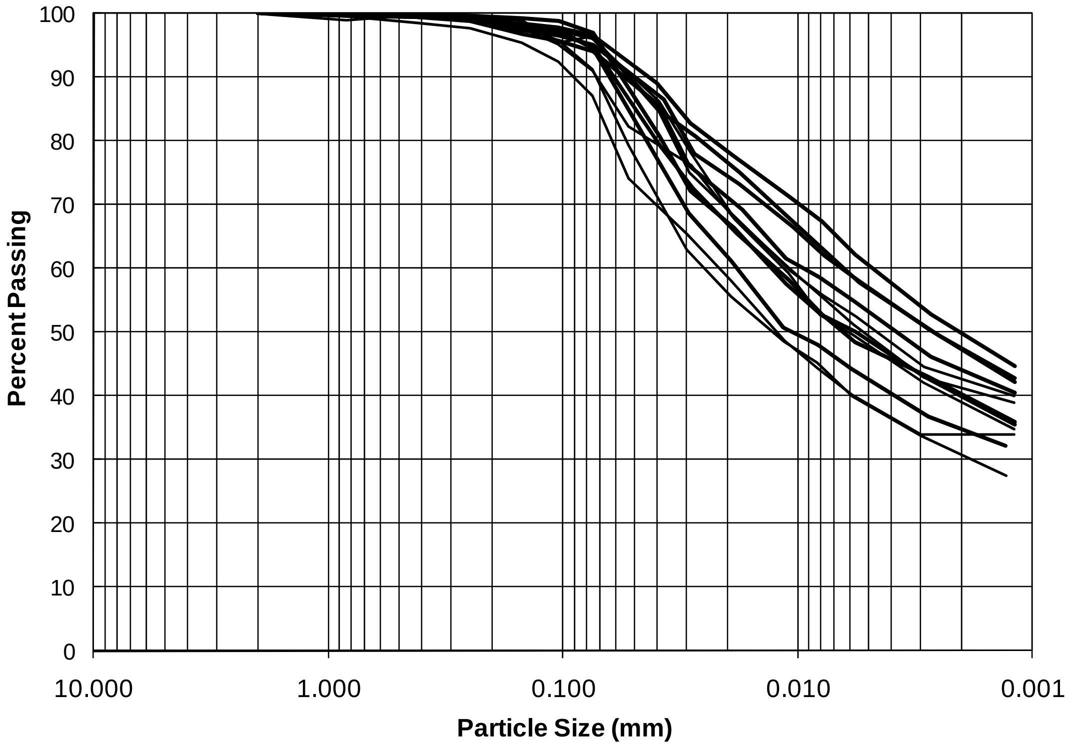


Figure 5.1. Particle Size Distribution of Soil 1-Processed

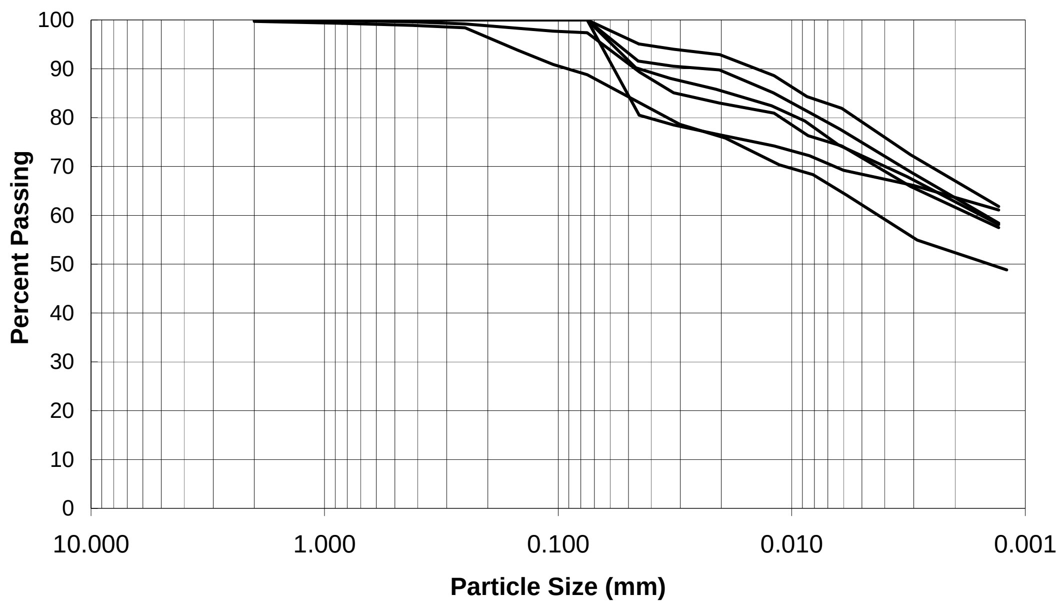


Figure 5.2. Particle Size Distribution of Soil 2-Processed

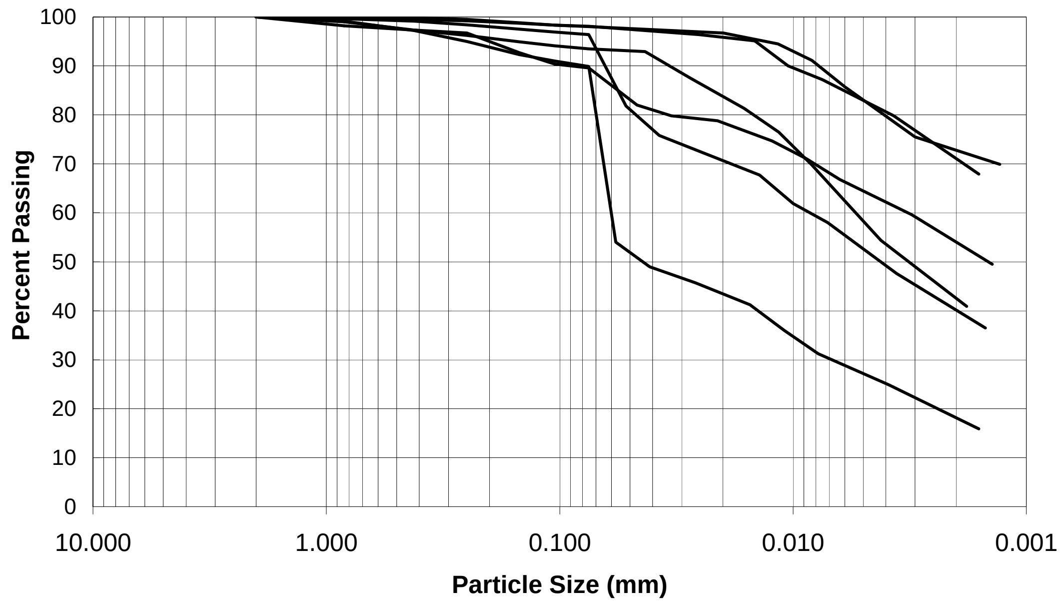


Figure 5.3. Particle Size Distribution of Soil 2-Unprocessed

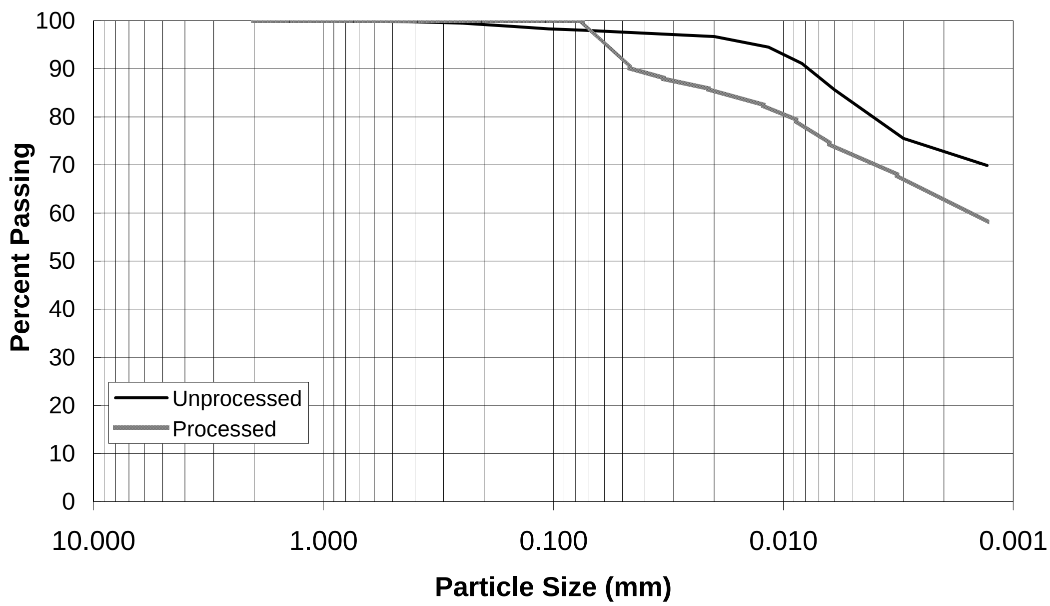


Figure 5.4. Particle Size Distribution of Soil 2-Split Sample Test 6

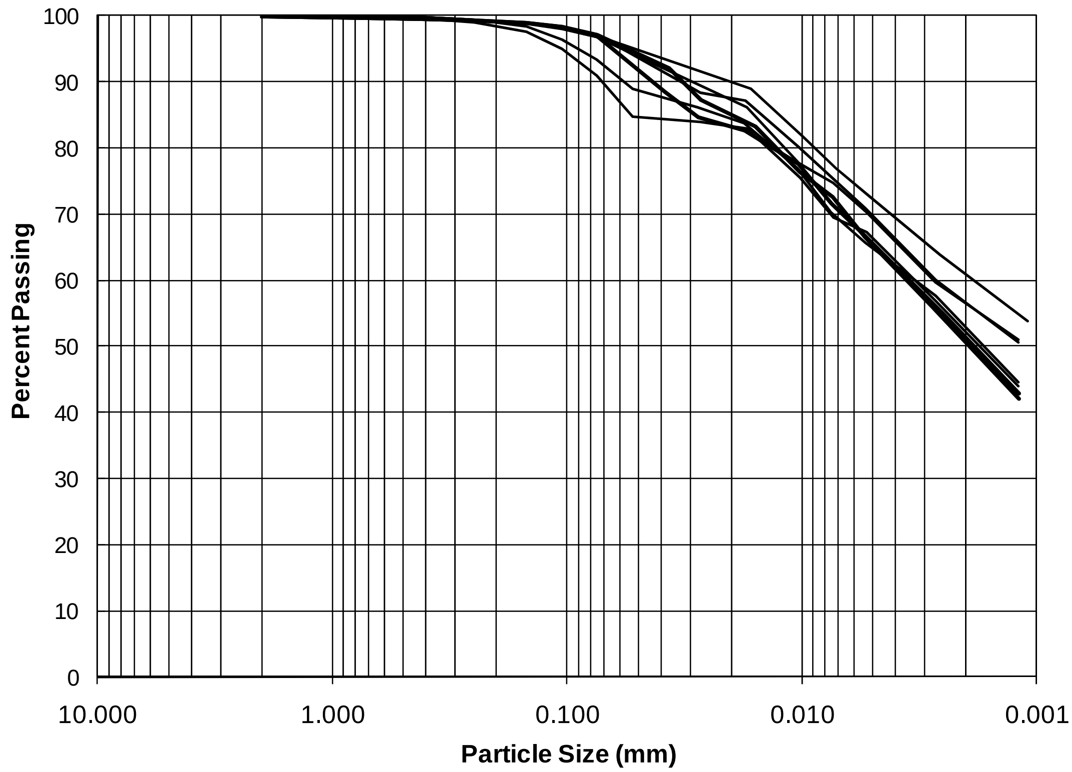


Figure 5.5. Particle Size Distribution of Soil 3-Processed

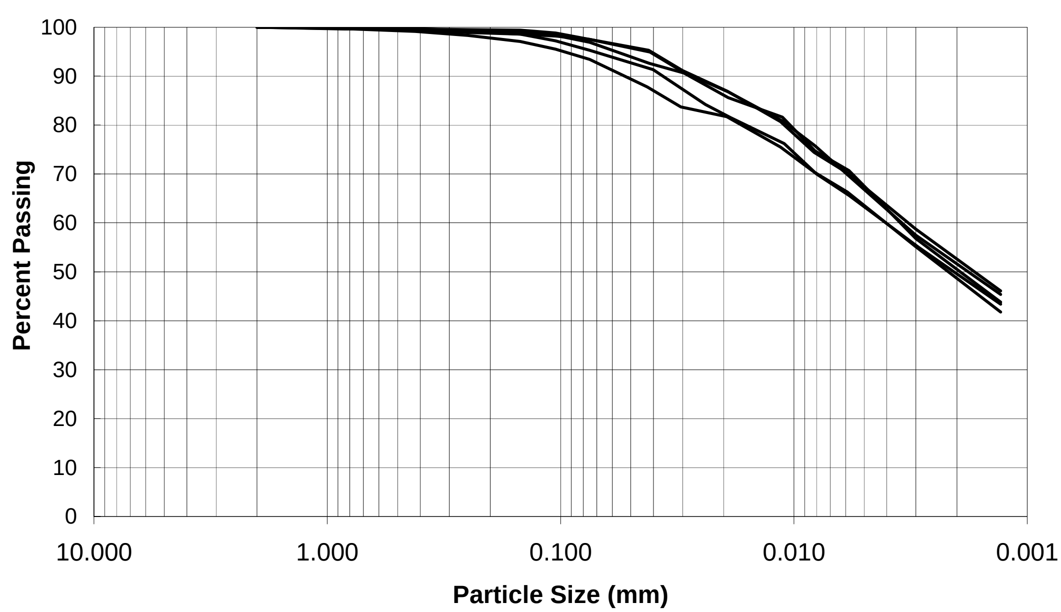


Figure 5.6. Particle Size Distribution of Soil 3-Unprocessed

5.1.3 Soil Classification

Table 5.3 provides soil classification results. Processed *Soil 1* classified as a CL or CH with the *LL* being the primary term differentiating the classification. *Soil 1* classified as an A-7-6 according to the *AASHTO* system. When unprocessed *Soil 2* classified as an OH and an A-8, though after processing the *USCS* classification changed to CH or MH. *Soil 3* classified as an A-7-5 using the *AASHTO* system before and after processing. Before processing *Soil 3* was classified as an OH, which changed to CH or OH after processing.

Table 5.3. Soil Classification Results

Soil	Processed	Test	USCS	AASHTO
1	Yes	1	CH	A-7-6 (43)
		2	CL	A-7-6 (27)
		3	CL	A-7-6 (30)
		4	CH	A-7-6 (35)
		5	CH	A-7-6 (35)
		6	CH	A-7-6 (52)
		7	CH	A-7-6 (36)
		8	CH	A-7-6 (39)
		9	CH	A-7-6 (45)
		10	CH	A-7-6 (47)
		11	CH	A-7-6 (41)
		12	CL	A-7-6 (29)
2	Yes	1	CH	A-7-6 (38)
		2	MH	A-8
		3	MH	A-8
		4	CH	A-8
		5	CH	A-8
		6	MH	A-8
2	No	1	OH	A-8
		2	OH	A-8
		3	OH	A-8
		4	OH	A-8
		5	OH	A-8
		6	OH	A-8
3	Yes	1	OH	A-7-5 (69)
		2	OH	A-7-5 (54)
		3	CH	A-7-5 (50)
		4	CH	A-7-5 (50)
		5	CH	A-7-6 (54)
		6	CH	A-7-5 (44)
		7	CH	A-7-5 (45)
3	No	1	OH	A-7-5 (74)
		2	OH	A-7-5 (83)
		3	OH	A-7-5 (86)
		4	OH	A-7-5 (81)
		5	OH	A-7-5 (90)
4	No	1	CH	---

5.1.4 Specific Gravity of Soil Solids

Specific gravity test results are provided in Table 5.4. *Soil 2* had the lowest processed specific gravity, while *Soil 1* and *Soil 3* were essentially the same when processed. The specific gravity of *Soil 2* decreased considerably when the soil was not processed, while the specific gravity of *Soil 3* decreased to a lesser extent when it was not processed. Processing could have resulted in loss of lighter volatile matter resulting in increased specific gravity, though insufficient information is available to make any detailed statements.

Table 5.4. Specific Gravity (G_s) of Solids Test Results

	Soil				
Test	**1-Processed**	**2-Processed**	**2-Unprocessed**	**3-Processed**	**3-Unprocessed**
1	2.66	2.58	1.96	2.62	2.62
2	2.67	2.37	2.40	2.67	2.61
3	2.57	2.36	2.11	2.70	2.61
4	2.73	2.45	2.17	2.72	2.60
5	2.71	2.47	1.86	2.69	2.61
6	2.60	2.36	2.45	2.71	---
7	2.71	---	---	2.70	---
8	2.71	---	---	---	---
9	2.72	---	---	---	---
10	2.72	---	---	---	---
11	2.60	---	---	---	---
12	2.66	---	---	---	---
Avg	***2.67***	***2.43***	***2.16***	***2.69***	***2.61***

5.1.5 Organics and Volatiles

Table 5.5 provides organic content results where each test was from a representative portion of a 1 kg bulk sample that had been homogenized. Table 5.6 shows organic content results where small selected portions of *Soil 2* were taken on site during acquisition of the bulk material. The purpose of the testing in Table 5.6 was to identify the variability of small portions within individual samples, especially the range of values that could occur due to variability. The data in Table 5.6 is from unprocessed material.

Table 5.5. Organics and Volatiles Test Results on Bulk Samples (ASTM D 2974)

	Soil					
Test	**1 Processed**	**2 Processed**	**2 Unprocessed**	**3 Processed**	**3 Unprocessed**	**4 Unprocessed**
1	3.8	11.4	16.8	10.2	11.3	6.3
2	4.6	31.3	14.9	11.4	12.0	---
3	3.6	31.1	32.3	10.6	11.9	---
4	4.5	27.2	26.5	10.5	11.6	---
5	4.5	20.1	19.7	10.5	13.2	---
6	9.9	32.2	20.0	7.8	---	---
7	5.9	---	---	12.0	---	---
8	5.3	---	---	---	---	---
9	6.6	---	---	---	---	---
10	6.4	---	---	---	---	---
11	4.3	---	---	---	---	---
12	4.4	---	---	---	---	---
Avg	***5.7***	***25.6***	***21.7***	***10.4***	***12.0***	***6.3***

Table 5.6. Organics and Volatiles Test Results on Select *Soil 2* Samples

Test	Organics and Volatiles	Statistics
1	25.2	n of 10
2	17.0	Mean of 22.6
3	36.2	Standard Deviation of 7.72
4	33.2	Maximum Value of 36.2
5	23.3	Minimum Value of 11.4
6	17.9	
7	25.3	
8	11.4	
9	19.9	
10	16.4	

Organic content of *Soil 2* was observed to vary widely. Organic content of *Soil 1* and *Soil 3* was within a much narrower band, though occasional tests did deviate somewhat from the mean value. The soils did, however, represent 3 distinct ranges of organic content as *Soil 1* had relatively low organics, *Soil 3* had moderate organics, and *Soil 2* had high organics. The extreme test results of each soil overlapped into the range of results for the soil with higher organics. Interestingly, the organic content of *Soil 2* when processed was higher than when unprocessed, which would indicate volatile material was not lost during processing. (G_s values indicated the opposite trend.) The organic content of *Soil 3* decreased due to processing, but not to a considerable level considering the amount of test replication.

5.1.6 In-Situ Moisture Conditions

In-situ moisture contents can provide an indication of a soil's affinity for moisture, index properties, and similar. Moisture contents of *Soil 1* were not taken in-situ. The soil had already been worked and dried prior to sampling, which made the data of no value. Moisture contents of *Soil 2* were obtained on 20 random samples, and the results were as follows: average of 99%; standard deviation of 21%; maximum of 141%; and minimum of 65%. Moisture contents of *Soil 3* were obtained on 20 random samples, and the results were as follows: average of 83%; standard deviation of 8%; maximum of 96%; and minimum of 65%. In situ moisture contents of 60% and greater are typical of organic clays, while peat soils can have in situ moisture contents on the order of 400%. *Soil 2* and *Soil 3* were obtained at in situ conditions representative of organic clays.

5.1.7 Slump Test Results

Slump tests were performed in accordance with *ASTM C 143*. *Soils 1*, *2*, and *3* were tested in duplicate at 100% moisture. The resulting slumps were: *Soil 1* of 29 cm (did not hold the shape of the cone), *Soil 2* of 0 cm, and *Soil 3* of 20 cm.

5.1.8 *XRF* Test Results

Table 5.7 provides *XRF* test results for 3 random samples of all 3 soils rounded to the nearest tenth. Elemental oxides where no soil had more than 0.5% were not shown. SiO_2 was the most prevalent oxide in all soils, though *Soil 2* had considerably less than *Soil 3*, which had considerably less than *Soil 1*. *Soil 3* had noticeably less *CaO* than the other soils, and also had the most SO_3.

Table 5.7. *XRF* Test Results of Random Samples from Soils 1 to 3

Oxide	*Soil 1*				*Soil 2*				*Soil 3*			
(%)	Rep 1	Rep 2	Rep 3	Avg	Rep 1	Rep 2	Rep 3	Avg	Rep 1	Rep 2	Rep 3	Avg
SiO_2	62.1	62.6	64.5	**63.1**	46.4	45.0	45.7	**45.7**	53.6	54.2	53.8	**53.8**
Al_2O_3	14.6	15.0	14.8	**14.8**	14.7	14.2	14.2	**14.4**	17.6	17.7	16.9	**17.4**
Fe_2O_3	6.1	6.0	6.1	**6.1**	7.0	6.7	6.8	**6.8**	8.2	7.9	8.0	**8.0**
LOI	5.5	5.8	5.0	**5.4**	22.5	26.2	25.4	**24.7**	10.5	10.6	10.8	**10.6**
CaO	3.7	2.5	2.3	**2.8**	3.8	2.5	2.6	**3.0**	0.6	0.7	0.9	**0.7**
K2O	2.6	2.7	2.5	**2.6**	1.7	1.6	1.6	**1.6**	1.9	2.0	1.9	**1.9**
MgO	1.8	1.9	1.8	**1.9**	1.7	1.6	1.6	**1.6**	1.5	1.5	1.5	**1.5**
Na_2O	0.9	1.0	0.8	**0.9**	0.2	0.3	0.3	**0.3**	1.8	1.7	1.7	**1.7**
SO_3	0.8	0.6	0.3	**0.6**	0.6	0.5	0.5	**0.5**	1.1	0.0	1.2	**0.8**
TiO_2	0.7	0.8	0.7	**0.8**	0.6	0.5	0.6	**0.6**	1.0	1.0	1.1	**1.0**
Cl	0.0	0.1	0.0	**0.0**	0.0	0.0	0.0	**0.0**	1.2	1.3	1.3	**1.3**

Table 5.8 provides test results from *Soil 1* taken from known storage barrels. Soil coming from *Group 1* had noticeably lower SiO_2 than soil coming from *Group 3*. The soil in *Group 3* was somewhat different between barrels. Additional information regarding *Soil 1* group classifications is provided later in the report.

Table 5.8. *XRF* Test Results of *Soil 1*-Known Sample Locations

Oxide	Group 1-Barrel E			Group 3-Barrel A			Group 3-Barrel B		
(%)	Rep 1	Rep 2	Avg	Rep 1	Rep 2	Avg	Rep 1	Rep 2	Avg
SiO_2	57.0	56.6	**56.8**	65.3	64.7	**65.0**	62.1	62.0	**62.1**
Al_2O_3	14.4	14.2	**14.3**	13.3	13.9	**13.6**	14.1	13.9	**14.0**
Fe_2O_3	14.1	15.5	**14.8**	8.4	8.8	**8.6**	10.7	11.3	**11.0**
LOI	5.8	5.4	**5.6**	5.3	4.8	**5.0**	5.3	5.2	**5.3**
CaO	2.6	2.4	**2.5**	2.6	2.7	**2.7**	2.3	2.6	**2.5**
K2O	2.0	1.8	**1.9**	1.4	1.5	**1.5**	1.7	1.7	**1.7**
MgO	1.8	1.7	**1.8**	1.5	1.4	**1.5**	1.5	1.4	**1.5**
Na_2O	0.8	0.8	**0.8**	0.8	0.8	**0.8**	0.8	0.8	**0.8**
SO_3	0.8	0.7	**0.8**	0.8	0.7	**0.8**	0.7	0.6	**0.7**
TiO_2	0.2	0.2	**0.2**	0.2	0.2	**0.2**	0.2	0.2	**0.2**
Cl	0.5	0.4	**0.5**	0.1	0.1	**0.1**	0.2	0.1	**0.2**

5.1.9 Comparison of Processed and Unprocessed Soil Properties

Soil 2 and *Soil 3* were tested in a processed and in an unprocessed condition. Processing affected the majority of the index properties, thus affecting classification (*USCS* more than *AASHTO*). Table 5.9 summarizes index property changes resulting from processing methods. Processed material had higher percent clay, indicating particle breakdown during processing. There was no noticeable difference in organic/volatile content between processed and unprocessed material. *Soil 2* organic contents were highly variable so a 4% difference should be considered within the source material variability (Table 5.6).

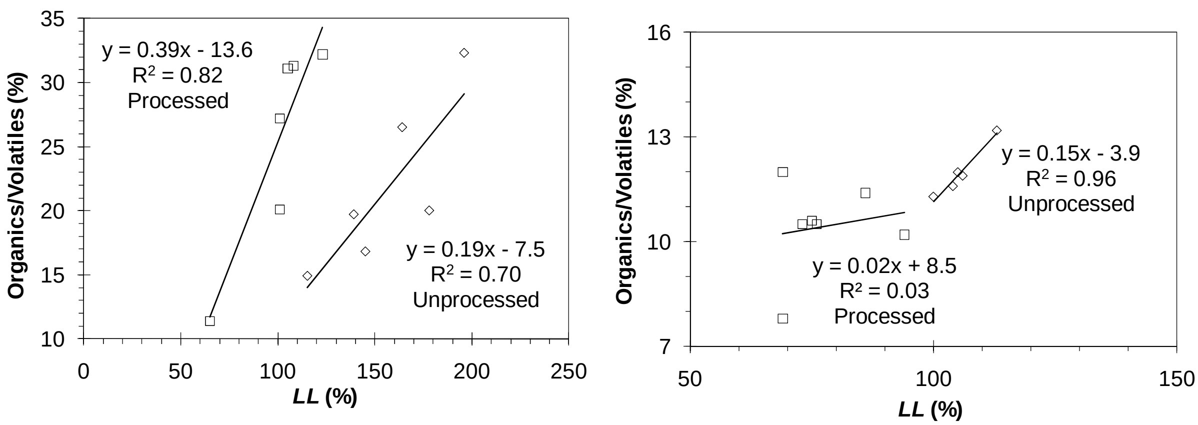
The *LL* was considerably reduced as a result of processing, while LL_{OD} was reduced but to a lesser extent. The *PL* was considerably reduced with *Soil 2* due to processing, though this was not the case with *Soil 3* as only a minor reduction was observed that would be within the range of material variability. Processed specific gravities (G_s) were higher than unprocessed specific gravities, possibly indicating loss of lighter volatile matter. Organic/volatile content test results, though, did not show property differences.

Table 5.9. Effect of Soil Processing on Index Properties

Soil	Processed	% Clay	% Organics	*LL*	LL_{OD}	*PL*	G_s
2	Yes	62	26	101	92	43	2.43
	No	50	22	156	102	64	2.16
3	Yes	58	10	77	69	31	2.69
	No	47	12	106	74	34	2.61

Note: Average values used.

The relationship between *LL* and organics/volatiles can be seen in Figure 5.7. Behavioral changes due to processing can be attributed more to *LL* than to organics/volatiles. In 3 of the 4 instances, there was a reasonable correlation between *LL* and organics/volatiles.



(a) Soil 2 *(b) Soil 3*

Figure 5.7. Comparison of Processed and Unprocessed LL and Organic Contents

5.1.10 Soil pH Test Results

Table 5.10 provides pH data for soils 1 through 3. Three distinct pH ranges were observed. *Soil 1* had the highest pH, followed by *Soil 3*, and then by *Soil 2*.

Table 5.10. pH Test Results of Soil Solids

Soil	1	2	3	4	5
Average	7.97	6.64	7.29	8.57	8.75
Standard Deviation	0.09	0.10	0.26	Average of 2 tests	
Range	0.44	0.31	0.79		
Maximum	8.18	6.77	7.69		
Minimum	7.74	6.46	6.90		

5.1.11 Untreated Shear Strength and Stability Test Results

Soil 2 was able to hold its shape prior to cement addition at 100% moisture so *UC* specimens were prepared to test its shear strength at this moisture condition. The other 2 soils had no shear strength at the moisture levels tested in this research. Six *Soil 2* specimens were produced. Three of the specimens were tested approximately 1 hour after fabrication; these specimens were placed on the lab bench, and when tested, shear strengths of 0.025, 0.030, and 0.030 kg/cm^2 were measured resulting in an average value of 0.03 kg/cm^2. The remaining 3 specimens were cured under water at room temperature for 24 hours; shear strengths of all 3 of these specimens were 0.02 kg/cm^2.

Soil 1 was used in construction of a levee so traditional compaction and bearing tests were conducted as a reference. Maximum dry densities from *D 698* and *D 1557* were 1.58 and 1.78 g/cm^3, respectively, while *OMC* values were 18.1 and 13.9%, respectively. *CBR* results with 25 blow compaction (90% of *D 698* density) were <1 when soaked and 8 when unsoaked. *CBR* results with 56 blow compaction (98% of *D 698* density) were 1 when soaked and 14 when unsoaked.

5.2 Water Property Test Results

Table 5.11 provides pH and salinity test results of the waters considered in this report. Results of pH testing showed brackish water to have the lowest pH, followed by fresh water and then salt water. Ocean salinity on average is 33.5 to 34 ppt, so the material obtained near the shore contains slightly more salt than would be expected in the open ocean which is desirable when testing the effect of salt content on shear strength.

Table 5.11. Water pH and Salinity Test Results

	pH			Salinity (ppt)		
Water	Tap (Fresh)	Brackish	Sea (Salt)	Tap (Fresh)	Brackish	Sea (Salt)
Average	7.99	7.73	8.19	0.0	4.6	39.9
St. Dev.	0.09	0.17	0.02	0.0	0.3	1.2
Range	0.20	0.69	0.12	0.0	1.8	4.9
Maximum	8.10	8.22	8.24	0.0	4.7	40.8
Minimum	7.90	7.53	8.12	0.0	2.9	35.9

CHAPTER 6 - SUPPORTING TEST RESULTS

6.1 Preliminary and Supporting Test Results Overview

This chapter contains the analysis conducted in preliminary stages of the work and test results used in support of other analysis or analysis techniques. Membrane correction test results were first evaluated to determine the necessity of their use in subsequent testing. Next, the need to account for moisture and density between specimens was considered, followed by methods to correct unconfined compression specimens for change in area during loading. The remainder of the chapter contains test results used multiple times in subsequent analyses presented in later chapters of the report, specifically unit weight, density, pH change associated with stabilized soil slurries, heat generation measured during *slab* testing, soil processing effects on shear strength, and curing temperature effects on shear strength.

6.2 Membrane Correction Results

Results of membrane correction testing are shown in Table 6.1. The effect of the membrane is evident. The variability of the test reduced, while the shear strength went up a measurable, but not excessive, amount. The specimens were able to be tested without a membrane at 2 hours, which ultimately made further use of the membrane for this research unnecessary.

Table 6.1. Membrane Correction *UC* Test Data

Membrane Used (---)	Time (hr)	s_u (kg/cm^2)	Statistics (---)	
Yes	48.45	0.19	n = 5	
	48.88	0.18	Mean	0.18 kg/cm^2
	48.05	0.17	Stdev	0.010 kg/cm^2
	47.50	0.17	Max	0.19 kg/cm^2
	47.31	0.19	Min	0.17 kg/cm^2
No	47.65	0.14	n = 5	
	47.17	0.15	Mean	0.13 kg/cm^2
	48.58	0.12	Stdev	0.019 kg/cm^2
	48.20	0.10	Max	0.15 kg/cm^2
	48.37	0.12	Min	0.10 kg/cm^2

Note: Soil 1 with A T I at (5,100) was tested.

6.3 Density Correction Results

Density correction test results are provided in Table 6.2. The results indicate the inherent variability within the range of densities that can be achieved through mixing and the varying levels of densification through measures such as vibration cannot be decoupled with reasonable amounts of effort. A plot of shear strength versus density showed no correlation (R^2 of 0.03). Performing specific gravity tests on very small soil samples, mixing them individually, and vibrating to varying levels could prove useful, but this would be a huge effort for each data point and was deemed too exhaustive an approach for the potential of removing the variability. Performing large numbers of tests was deemed more appropriate, and density correction was not performed for the remainder of the research.

Table 6.2. *UC* Density Correction Test Results

Density (g/cm^3)	Sample Vibration (---)	s_u (kg/cm^2)
1.567	None	0.81
1.544	None	0.76
1.583	None	0.76
1.547	30 sec	0.77
1.561	30 sec	0.74
1.534	30 sec	0.65
1.577	Fully	0.62
1.577	Fully	0.80

Note: Soil 1 at 72 hr with A T I cement in (5,100) condition.

6.4 Moisture Correction Results

Moisture correction was not performed on any test data after investigating batching tolerances and quality control measurements taken during testing. The majority of the quality control measurements ($w_{as(\%)}$) were within $\pm$ 0.5% in terms of $TS_{\%}$. Brackish and salt water testing had slightly higher $w_{as(\%)}$ variability than fresh water testing; salt content was accounted for via calculation and batch quantity adjustment to allow moisture contents within tolerance to be achieved.

The procedure used to process soil and ultimately batch soil slurry was deemed more accurate than the discrete measurements of moisture ($w_{as(\%)}$) taken as a quality control measure. This was verified by checking each step in the batching procedure and comparing the results to a group of randomly measured $w_{as(\%)}$ values from batched soil slurries. First, processed soil moisture contents were verified to be within at most 1% of the values used during batching. Next, soil slurry was produced with each soil at $w_{ts(\%)}$ of 233 and multiple $w_{as(\%)}$ measurements obtained. Each of the batches was subsequently dried to measure true moisture content in the batch containers. True moisture contents of the soil slurries varied on the order of $\pm$ 0.5% when presented in terms of $TS_{\%}$, while $w_{as(\%)}$ values varied on the order of $\pm$ 1%. No quantifiable and consistent correction for moisture could be performed under these circumstances, and the relatively tight moisture tolerances coupled with large amounts of testing are believed to offset the need for moisture correction entirely.

6.5 Unconfined Compression Area Corrections

The diameter of *UC* specimens was measured at the conclusion of testing to allow appropriate area correction for stress calculation. Additional detail can be found in Carruth and Howard (2011). The stress and strain at the maximum force (P_{max}) are referred to as σ_{max} and ε_{max}, respectively. For purposes of this report, ε_{max} values were taken at the maximum force reading upon first occurrence. The ultimate stress, σ_{ult}, was taken to be the maximum stress attained after a given area correction was implemented, with the ultimate force, P_{ult}, and ultimate strain, ε_{ult}, occurring at σ_{ult}. Eq. 6.1 to 6.5 are used in various manners in *ASTM D 2166-06* and/or *ASTM D 5102-04* to perform area correction and subsequent stress calculations. Eq. 6.6 was developed during this research after observing deformation characteristics of fiber reinforced specimens.

$$\varepsilon = \frac{\Delta L}{L_o}(100) \tag{6.1}$$

$$\sigma = \frac{P}{A_i} \tag{6.2}$$

$$A_1 = A_o \qquad \text{[Brittle Failure]} \tag{6.3}$$

$$A_2 = \frac{A_o}{(1-(0.6\varepsilon/100))} \qquad \text{[Barrel Failure]} \tag{6.4}$$

$$A_3 = \frac{A_o}{(1-(\varepsilon/100))} \qquad \text{[Cylindrical Failure]} \tag{6.5}$$

$$A_4 = \frac{A_o}{(1-(1.6\varepsilon/100))} \qquad \text{[Cylindrical plus Barrel Failure]} \tag{6.6}$$

Where,

ε = axial strain (%)
ΔL = change in length in axial direction (cm)
L_o = original length in axial direction (cm)
P = applied load (kg)
A_i = converted cross sectional area of specimen; A_1, A_2, A_3, or A_4 (cm^2)
A_o = original cross sectional area of specimen (cm^2)

Table 6.3 provides area correction results for all testing. Data was grouped: 0 to < 24 hr, 24 to < 72 hr, 72 to <168 hr, and 168 hr. Data within these intervals was combined and averaged. Note the number of data points (*n*) represented by each marker is shown. Table 6.3 was used to generate a series of figures that compared area predicted by the aforementioned equations to the measured area at the middle of the specimens. (See Carruth and Howard 2011 for more information.) Area corrections determined are as follows:

- Fiber Reinforced Specimens
  - All soil types: Use cylindrical plus barrel (Eq. 6.6) for all specimens.
- Non-Fiber Reinforced Specimens
  - *Soil 1*: For specimens tested before 24 hours use cylindrical correction (Eq. 6.5); for specimens tested after 24 hours use brittle correction (Eq. 6.3).
  - *Soil 2*: Use cylindrical correction for all specimens (Eq. 6.5).
  - *Soil 3*: For specimens tested before 24 hours use cylindrical correction (Eq. 6.5); for specimens tested after 24 hours, use brittle correction (Eq. 6.3).

Table 6.3. Area Correction Data

		Fibers					No Fibers				
				% Original Area					% Original Area		
Soil	Time (hr)	*n*	*ε*	*Top*	*Mid*	*Bot*	*n*	*ε*	*Top*	*Mid*	*Bot*
1	0 to <24	3	15	106	127	118	10	3.6	102	106	106
	24 to <72	10	15	111	131	118	87	2.8	102	100	100
	72 to <168	11	15	110	140	124	105	2.5	104	101	100
	168	14	15	106	130	114	116	2.5	102	100	99
2	0 to <24	6	15	121	126	107	35	5.2	105	109	108
	24 to <72	11	15	111	138	114	124	3.4	102	104	102
	72 to <168	17	15	112	139	120	136	3.0	103	103	103
	168	13	15	100	107	104	140	2.7	101	103	102
3	0 to <24	8	15	108	123	112	33	8.5	106	108	111
	24 to <72	13	15	115	127	118	117	3.1	102	101	101
	72 to <168	17	15	113	125	118	133	2.7	104	102	100
	168	14	15	106	134	119	148	2.9	103	101	100

Note: Top, Mid, and Bot represent the area measured at the specimen top, middle, and bottom, respectively.

6.6 Unit Weight and Density of Stabilized Slurries

G_s data provided in Table 5.4 was used alongside batch quantities to calculate maximum stabilized slurry density (i.e. no air) for comparison to measured slab density (ρ_s) and total unit weight (γ_T). Results of the calculations are provided in Table 6.4. The calculation was first performed on target batch quantities using the average G_s value and shown as the average value in Table 6.4. The low end of the range was calculated using the highest tolerable moisture content and lowest measured G_s, while the high end of the range was calculated using the lowest tolerable moisture content and highest measured G_s.

Table 6.4. Calculated Maximum Stabilized Slurry Density

C_T	$w_{ts(\%)}$	Soil	Average (g/cm^3)	Range (g/cm^3)
5	100	1	1.49	1.46 to 1.52
		2	1.46	1.43 to 1.50
		3	1.50	1.47 to 1.52
10	100	1	1.53	1.50 to 1.56
		2	1.49	1.47 to 1.53
		3	1.53	1.51 to 1.56
15	233	1	1.34	1.32 to 1.36
		2	1.32	1.30 to 1.35
		3	1.34	1.32 to 1.36

Note: Portland cement was the additive used for calculations.

Measured density and unit weights during *Slab* and *UC* testing are shown in Table 6.5. Measured *Slab* values with *Soil 1* at (5, 100) were higher than the range of values in Table 6.4; ρ_s was higher by 0.01, which could conceivably occur via a combination of rounding coupled with G_s values slightly higher than those found from sampling, while γ_T

being 0.03 higher would seem to indicate a testing error. At 100% moisture, *Soil 2* was below the range of values provided in Table 6.4 in all cases but 1, which was at the lower end of the range indicating some air remained in the specimens though the values were not excessively lower than the calculated range. *Soil 3 slab* density values were slightly lower than the range at (5, 100). *Soil 1* specimens were never below the Table 6.4 values.

Table 6.5. Measured Stabilized Slurry Unit Weight and Density

			***Slab* Data**		***UC* Data**	
C_T	$w_{ts(\%)}$	**Soil**	ρ_s (g/cm^3)	γ_T (g/cm^3)	ρ_s (g/cm^3)	γ_T (g/cm^3)
5	100	1	1.51 to 1.56	1.54 to 1.55	1.45 to 1.61	1.47 to 1.54
			1.53	**1.55**	**1.51**	**1.51**
		2	1.37 to 1.48	1.36 to 1.48	1.35 to 1.56	1.28 to 1.48
			1.42	**1.39**	**1.43**	**1.40**
		3	1.44 to 1.48	1.46 to 1.52	1.43 to 1.59	1.43 to 1.51
			1.46	**1.49**	**1.50**	**1.47**
10	100	1	1.52 to 1.52	1.53 to 1.54	1.46 to 1.65	1.53 to 1.55
			1.52	**1.54**	**1.54**	**1.54**
		2	1.46 to 1.47	1.39 to 1.39	1.25 to 1.57	1.30 to 1.48
			1.46	**1.39**	**1.43**	**1.43**
		3	1.49 to 1.49	1.51 to 1.54	1.44 to 1.62	1.35 to 1.57
			1.49	**1.53**	**1.52**	**1.51**
15	233	1	1.30 to 1.36	1.27 to 1.37	1.20 to 1.41	1.33 to 1.36
			1.34	**1.33**	**1.34**	**1.35**
		2	1.31 to 1.35	1.26 to 1.33	1.25 to 1.47	1.27 to 1.35
			1.33	**1.30**	**1.32**	**1.33**
		3	1.28 to 1.35	1.26 to 1.33	1.24 to 1.43	1.31 to 1.37
			1.32	**1.30**	**1.33**	**1.34**

Note: Portland cement was the only additive considered.

6.7 pH Changes Due to Stabilization

Table 6.6 summarizes pH test results. Prusinski and Bhattacharja (1999) indicate pH levels are about 12.5 in mortar and concrete systems. The majority of the pH change data ($pH_{post\text{-}cement}$ minus $pH_{pre\text{-}cement}$) was 4.0 to 5.0 with a fair amount of data between 5.0 and 5.5. No correlation was found between pH properties after cement addition and shear strength. Correlations were sought for pH change and pH after cement addition in relation to shear strength at all test durations. Data was investigated in smaller subsets [e.g. (5, 100) condition with *Soil 1*] in addition to as a whole. Cement addition appeared to converge the readings in such a manner than meaningful information was not obtained.

Table 6.6. pH Test Results

	$pH_{pre\text{-}cement}$		**Relative Frequency of $pH_{post\text{-}cement}$ Minus $pH_{pre\text{-}cement}$ (%)**						
Soil	***n***	**Mean**	**<3.0**	**3.0 to 3.5**	**3.5 to 4.0**	**4.0 to 4.5**	**4.5 to 5.0**	**5.0 to 5.5**	**>5.5**
1	53	7.6	4	4	9	40	30	13	0
2	49	6.8	8	2	12	33	20	18	6
3	75	7.0	5	1	8	31	33	21	0

Note: The standard deviation of $pH_{pre\text{-}cement}$ *of all 3 soils was 0.3.*

6.8 Stabilized Slurry Heat Generation

Stabilized slurry heat generation results are provided in Table 6.7. In general, results were measured over the entire 168 hour test duration. The method provided easily distinguishable temperature readings and peak values.

Table 6.7. Stabilized Slurry Heat Generation Results

Soil	Condition	Cement	ΔT_{avg} (C)	ΔT_{stdev} (C)	ΔT_{max} (C)	$t_{\Delta T}$ (C)
1	(5, 100)	*A T I*	0.6	1.1	2.6	4.0
		Th T I/II	1.0	1.0	3.9	3.2
		A T III	0.5	1.1	2.7	3.3
		Th T III	0.2	1.0	2.4	4.5
		A T I (GGBFS)	0.2	0.8	2.3	51.0
		CTS RS	0.9	1.0	3.5	3.5
		SC1	0.9	0.9	2.6	9.5
		SC1 (F70)	0.5	1.2	3.3	2.8
		SC1 (F20)	1.0	0.9	3.0	2.3
		SC2	2.9	1.7	4.7	36.0
		SC2 (PoP)	1.1	1.2	3.5	4.3
2	(5, 100)	*A T I*	---	---	---	---
		Th T I/II	0.8	1.1	4.2	2.0
		A T III	1.5	1.5	5.2	1.0
		Th T III	1.5	1.7	6.0	1.0
		A T I (GGBFS)	1.0	0.7	2.9	2.0
		CTS RS	0.6	1.0	3.3	4.0
		SC1	1.8	1.7	5.8	1.0
		SC1 (F70)	1.2	1.4	5.5	2.0
		SC1 (F20)	1.3	1.4	5.1	1.0
		SC2	1.5	1.7	5.0	3.0
		SC2 (PoP)	1.6	1.4	5.0	4.0
3	(5, 100)	*A T I*	1.6	1.2	4.4	2.0
		Th T I/II	1.3	0.8	3.8	21.0
		A T III	1.6	1.5	4.9	1.0
		Th T III	0.7	1.2	4.4	1.0
		A T I (GGBFS)	0.5	0.7	2.2	1.0
		CTS RS	0.8	0.7	2.8	4.0
		SC1	1.6	1.2	4.5	4.0
		SC1 (F70)	1.2	1.2	4.2	2.0
		SC1 (F20)	1.7	1.3	4.5	2.0
		SC2	1.7	1.4	5.0	1.0
		SC2 (PoP)	1.2	1.3	5.0	2.0
1	(10, 100)	*Th T III*	2.0	1.8	6.0	3.0
		SC1	2.8	1.7	4.0	6.1
2	(10, 100)	*Th T III*	1.8	1.7	5.3	1.0
		SC1	2.7	2.2	7.1	3.0
3	(10, 100)	*Th T III*	1.2	0.8	2.2	17.0
		SC1	2.4	1.9	6.0	4.0
1	(15, 233)	*Th T III*	3.1	2.2	6.6	11.0
		SC1	3.2	1.6	4.7	11.0
2	(15, 233)	*Th T III*	3.2	1.6	5.4	5.0
		SC1	2.8	1.7	5.7	4.0
3	(15, 233)	*Th T III*	2.3	1.6	4.3	15.0
		SC1	3.3	1.5	4.7	12.0

Figure 6.1 plots heat generation results with respect to shear strength measured by the *Dial* gage at a *TTF* of 500 C-hr. Slurries at (15, 233) had no correlation (R^2 = 0.01) and were not shown in Figure 6.1. Moderate correlations were observed for (5, 100) and (10, 100) in terms of the maximum temperature change measured and shear strength development. The slopes and amount of data scatter were similar between (5, 100) and (10, 100) test results, though considerably more data was available at (5, 100). Correlations were similar when plotted at 1,500 or 3,500 C-hr. The most interesting observation in Figure 6.1 is that an unrefined technique was capable of measuring a trend between heat generation within the slurry and shear strength. The data provides promise that more sophisticated thermal profile techniques would be capable of even better predictions. Another interesting observation was the magnitude of temperature increase was noticeable at 2 to 7 C.

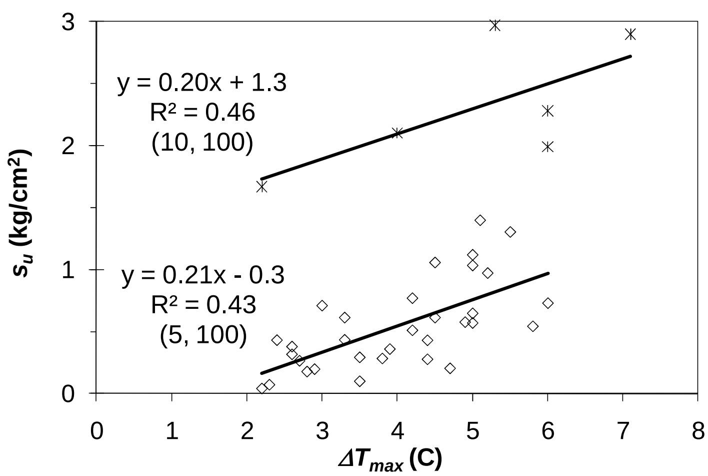


Figure 6.1. Slab Heat Generation Test Results

6.9 Maturity Methods

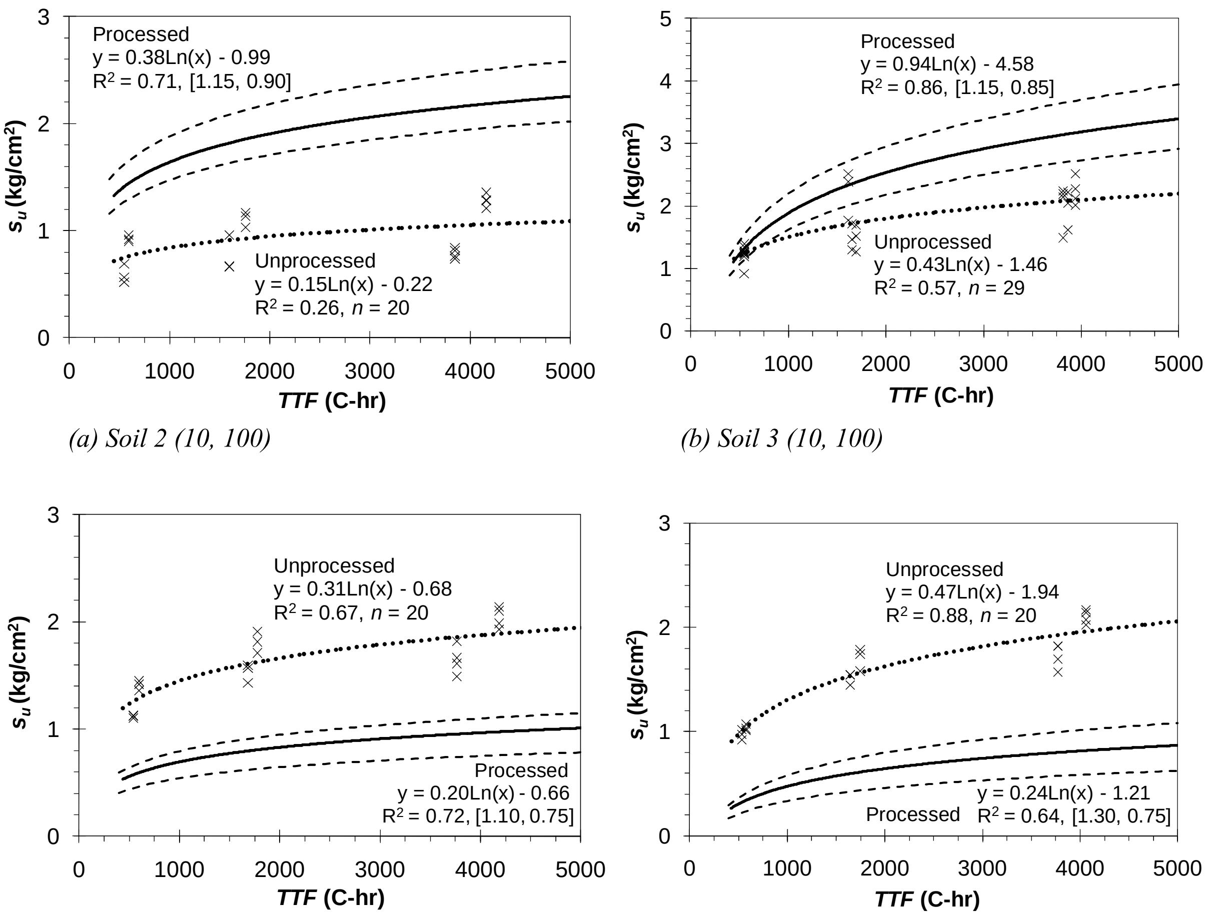
Equation 2.5 (pg 21) was used to characterize the time-temperature behavior of stabilized slurries. T_o of 0 C was selected as a reference. The selection of T_o was somewhat arbitrary, though it is not an uncommon reference and is recommended in *ASTM C 1074* for *Type I* cement. The maturity concept allows strength (*Y*-axis) to be plotted versus a temperature-time factor (*TTF*) with units of C-hr on the *X*-axis. For *TTF* calculations T_i was taken as the air or water temperature (T_a) surrounding the specimen during curing. It is uncommon to use the maturity concept in concrete where the internal temperature is not used. Use of the maturity concept with stabilized soil is not common. Using surrounding rather than internal temperature makes application of the concept more practical in a disaster environment.

6.10 Effect of Soil Processing on Shear Strength

Four *Soil 2* suites and five *Soil 3* suites were tested using unprocessed soil, and the properties were compared to processed soil test results. All testing was performed with *Th T III* cement. Shear strength test results are provided in Figure 6.2. Processed soil test results are represented by control bands with a solid line and 2 dashed lines as in the remainder of the report (See Chapter 8), while unprocessed soil results are represented by individual test results and a dotted line. Figure 6.2 clearly indicates that soil processing affected shear strength and that the effect was different between (10, 100) and (15, 233) slurries. The same general trend was observed in *Soil 2* and *Soil 3*. Processed soil was stronger than unprocessed soil at (10, 100), while unprocessed soil, was stronger than processed soil at (15, 233). *Soil 3* at (10, 100) did, however, have similar strengths between processed and unprocessed soil below a *TTF* of 1,000 C-hr.

Prior to cement addition laboratory personnel observed that the unprocessed soil appeared to be stiffer than the processed soil for *Soil 2* and *Soil 3* at 100 and 233% moisture. The reason is unknown but is believed to be a factor of shear strength differences. For (10, 100) testing, additional soil slurry stiffness might hinder mixing and result in lower strength; whereas, for (15, 233) additional soil slurry stiffness might be useful for an otherwise highly fluid material. Figure 6.2 agrees with this speculation in general terms.

Test repeatability was reasonable for the unprocessed materials. There were no considerable strength differences within or between suites of the same mixture. On average, processing soil increased the shear strength of (10, 100) specimens by a factor of 2.0 for *Soil 2* and 1.4 for *Soil 3*. On average, processing soil reduced the shear strength of (15, 233) specimens by a factor of 2.0 for *Soil 2* and 2.7 for *Soil 3*.



(a) Soil 2 (10, 100)

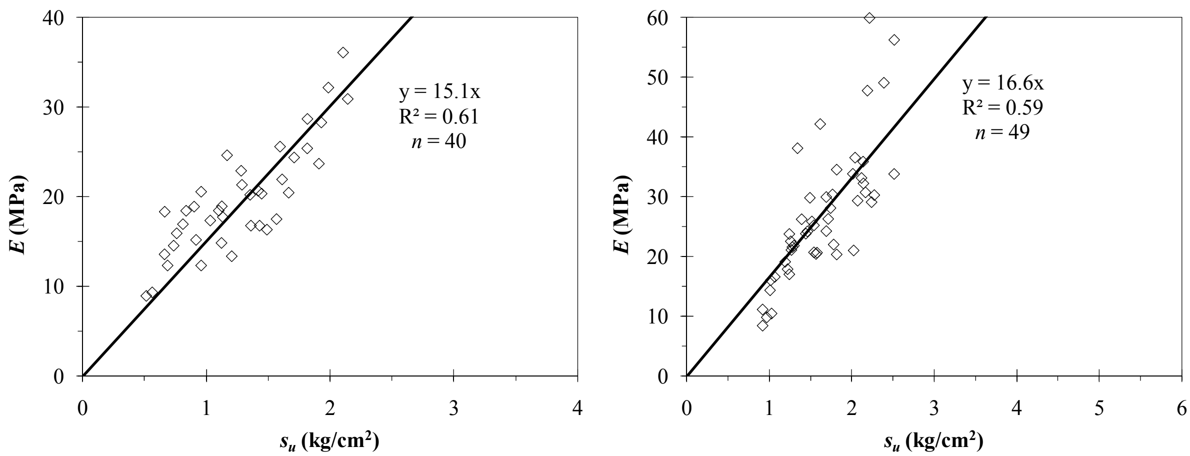
(b) Soil 3 (10, 100)

(c) Soil 2 (15, 233)

(d) Soil 3 (15, 233)

Figure 6.2. Effect of Soil Processing on Shear Strength

Figure 6.3 plots elastic modulus and shear strength correlations with unprocessed soil. The correlation was similar for both soils, as trendline slopes were 15.1 and 16.6. Correlations were 9.6 to 13.5 (*Soil 2*) and 6.9 to 14.1 (*Soil 3*) with processed soil under the same conditions. Maximum strain (ε_{max}) values for unprocessed *Soil 2* were 1.5 and 1.9% on average for (10, 100) and (15, 233), respectively with a standard deviation in each case on the order of 0.35%. Maximum strain (ε_{max}) values for unprocessed *Soil 3* were 1.4 and 1.6% on average for (10, 100) and (15, 233), respectively with a standard deviation in each case on the order of 0.26%. Processed *Soil 2* and *Soil 3* had ε_{max} values of 1.8 to 2.5% and 1.6 to 2.6%, respectively.



(a) Soil 2 *(b) Soil 3*

Figure 6.3. Correlation of Elastic Modulus and Shear Strength From Unprocessed Soil

6.11 Effect of Temperature on Strength Gain

Sulfate solubility is inversely proportional to temperature; i.e. solubility decreases as temperature increases. A given cement could experience sulfate starvation problems at higher temperatures where sulfates become less soluble, especially if the sulfate content of the cement has been reduced. To investigate this behavior, 3 suites were cured at cold temperatures, and 3 suites were cured at hot temperatures to investigate the effects on shear strength as a function of *TTF* for *SC5*, *SC6*, and *Th T III*. The data was compared to that collected at room temperature. Shear strength results are provided in Figures 6.4 to 6.6.

As seen in Figures 6.4 to 6.6, cold cured specimens were stronger than hot cured specimens at similar *TTF* values for all 3 cements. The strength difference between the hot and cold cured specimens was not drastic considering the large range of temperatures and that the differences largely lie within the strength band that could be produced with the room temperature data. The data suggests there was an inverse effect of temperature and strength gain, as would be suggested by sulfate solubility trends, but with the limited data collected there does not appear to be a major difference that would pose concern in using the materials in a range of temperature environments. There is no distinctive pattern between cements indicating that the behavior observed is not strongly tied to SO_3 content since the 3 cements each have different amounts of SO_3. In no case were specimens created that had strengths dramatically lower than would be expected from room temperature curing coupled with calculation of a *TTF*.

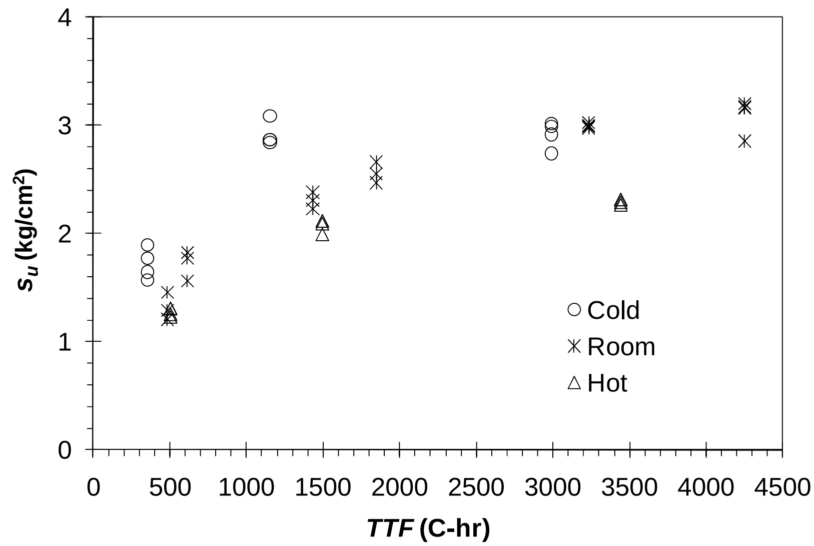


Figure 6.4. Temperature Effects on SC5 in Soil 3

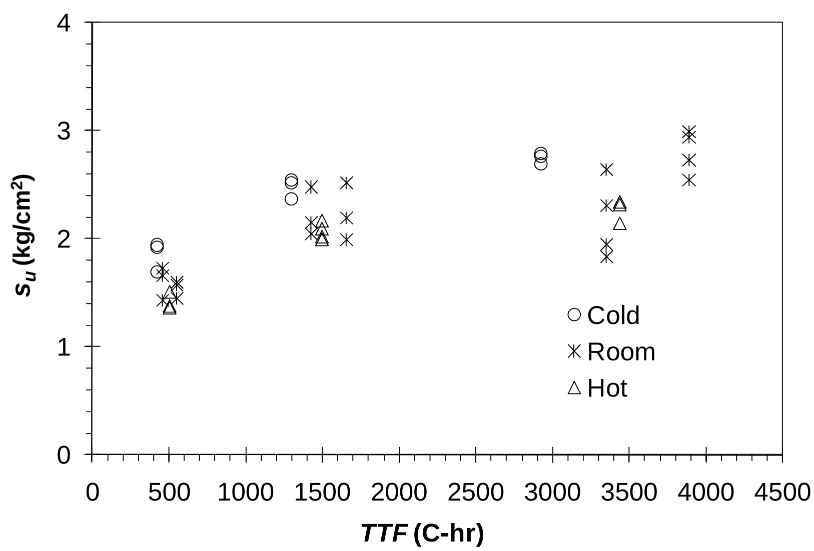


Figure 6.5. Temperature Effects on SC6 in Soil 3

Figure 6.7 plots the relationship between elastic modulus and shear strength. All data is somewhat grouped together, though the hot cured specimens generally occupy the top portion of the group and the room temperature cured specimens generally occupy the bottom portion of the group. There is not enough data to make definitive statements, especially with the large amount of scatter that is present in the cold cured specimens. It does appear that there may be some effect on the relationship of elastic modulus and shear strength attributable to the curing temperature, but it also appears that any affect is not dramatic. As with measured shear strengths, there does not appear to be a major problem using room temperature cured specimens and calculating *TTF* factors for assessment of material behavior. Maximum strains did not change appreciabially between cold, room, and hot curing temperatures as the average values were 1.5, 1.7, and 1.3%, respectively, and the standard deviations were 0.3, 0.2, and 0.3%, respectively.

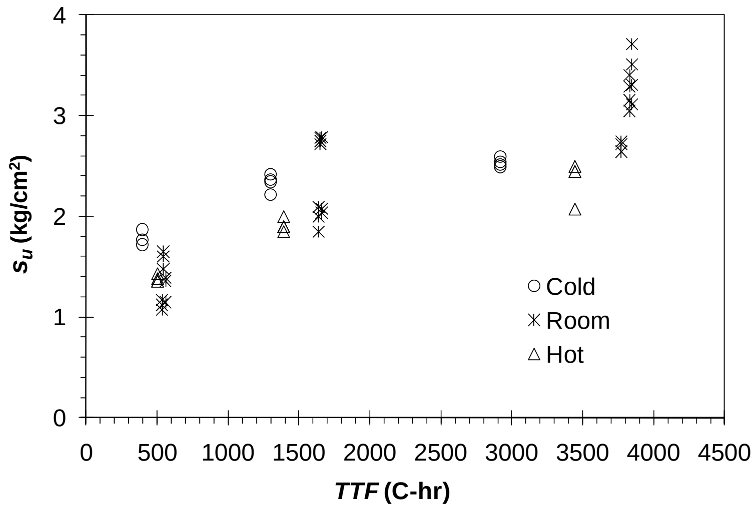


Figure 6.6. Temperature Effects on Th T III in Soil 3

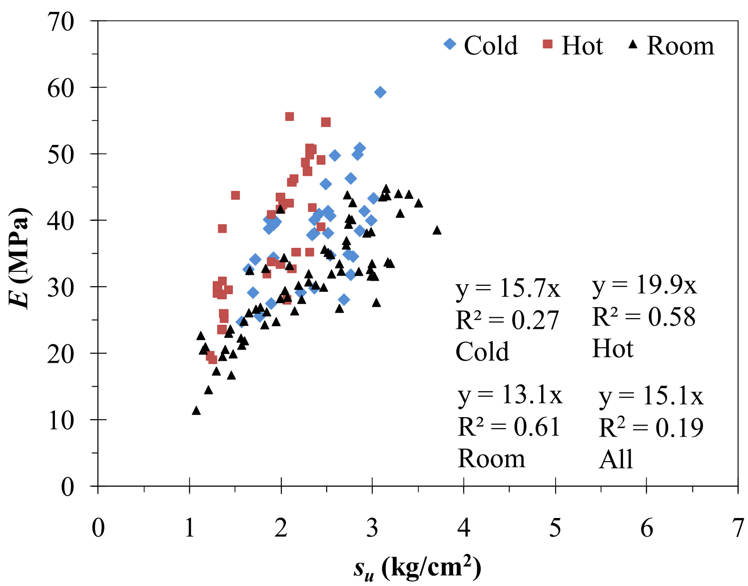
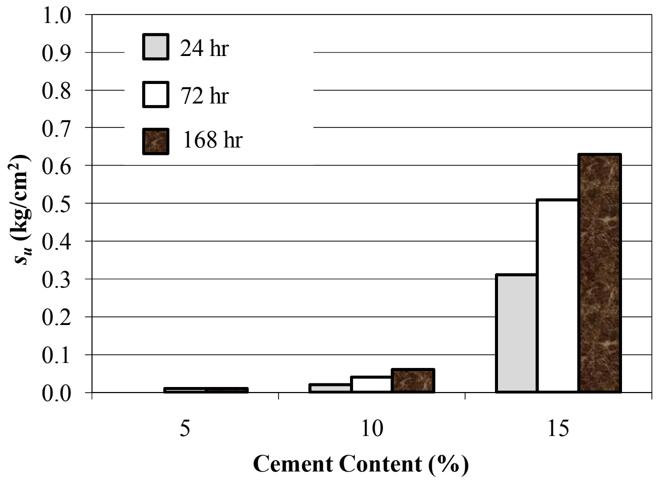


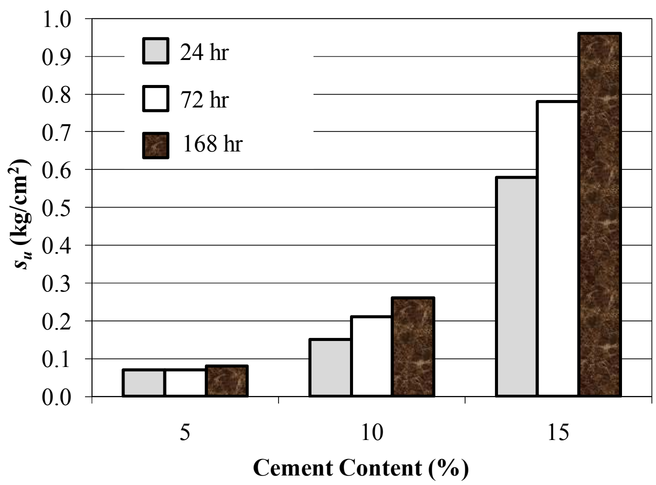
Figure 6.7. Temperature Effects on Elastic Modulus to Shear Strength Correlation

6.12 *UC* Testing To Establish Cement Dosage for 233% Moisture

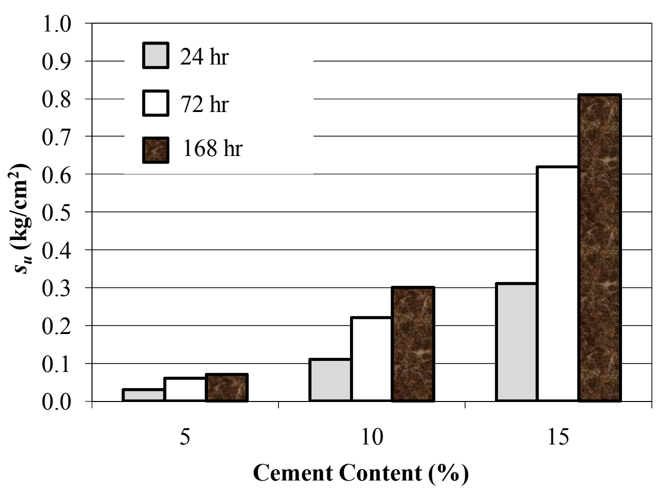
Figure 6.8 plots results using *Th T III* cement where all 3 soils were tested at 3 cement dosages at 233% moisture. Dosage rates for 100% moisture were more readily understood, but a reasonable dosage rate at 233% moisture required preliminary testing. The data presented in Figure 6.8 was used to select 15% cement by total slurry mass as the dosage rate to be used in the majority of *UC* testing at 233% moisture. A considerable strength increase was observed between 10 and 15% cement.



(a) Soil 1



(b) Soil 2



(c) Soil 3

Figure 6.8. Dosage Rate Investigation of Th T III at 233% Moisture

CHAPTER 7 - HAND HELD GAGE TEST RESULTS

7.1 Overview of Hand Held Gage Test Results

Sixty trials were conducted during the research resulting in 4,556 readings with each of the hand held gages (*Dial*, *Ring*, and *Shear*). Additionally, 37 variability slabs were tested resulting in 919 readings with each of the 3 hand held gages. The testing was performed to: 1) investigate trends within the soil and cementitious materials; 2) evaluate the hand held gages and their suitability for use in a disaster environment; and 3) investigate the effect that blending cements, fibers, and water type has on shear strength. This chapter focuses on soil and cementitious trends, which only requires a portion of the 60 trials and 37 variability slabs. The remaining data is used in subsequent chapters.

7.2 Control Cement Test Results

Th T III was used as the control cement for slab testing. Fifteen trials were conducted using this cement, and the trend lines generated from the testing can be seen in Figures 7.1 through 7.9, with all data provided in Appendix A Figures A.1 to A.8. Four variability slabs were tested using this cement, and their mean values have been plotted on Figures 7.1 through 7.9 alongside error bars at 1 standard deviation from the mean (error bars are not large enough to be visible in some instances). All data from the variability slabs has been provided in Appendix B Figures B.1 and B.2.

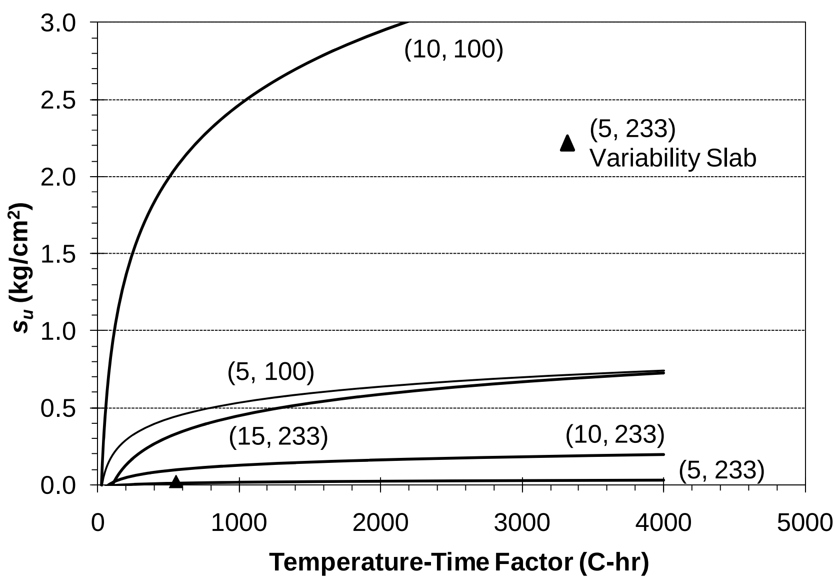


Figure 7.1. Th T III Soil 1 Dial Gage Test Results

Figures 7.1 through 7.9 were first used to select a cement content for slurries with 233% moisture for the remaining hand held gage testing. Five percent cement created too little strength in all soils as measured by all 3 gages. Ten percent cement did not create a reasonable strength in *Soil 1* with 233% moisture, though 10% cement was able to produce strength curves similar to (5,100) in *Soil 2* and *Soil 3* with 233% moisture. In *Soil 1*, it took

15% cement at 233% moisture to produce comparable strengths to (5,100). With *Soil 2* and *Soil 3*, (15, 233) was considerably stronger than (5,100). Fifteen percent cement was ultimately selected for use with 233% moisture since it was desired to have reasonable strengths from all 3 soils.

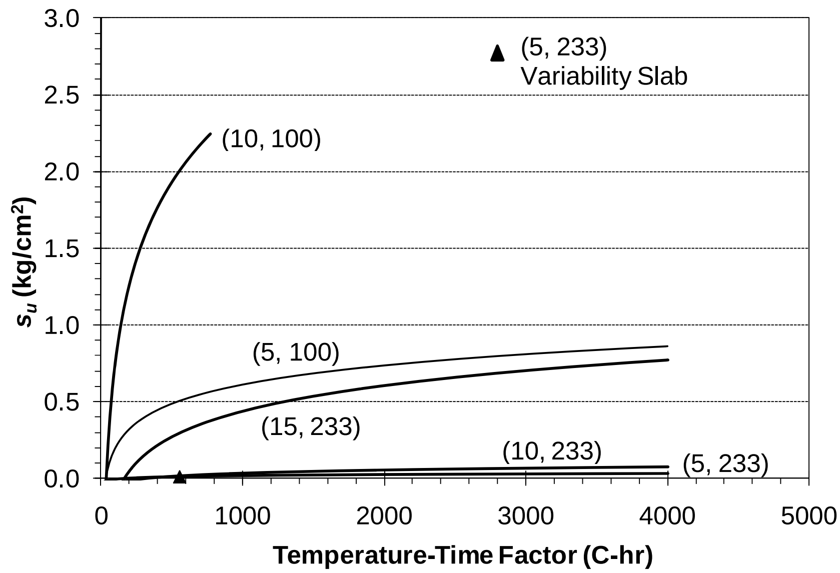


Figure 7.2. Th T III Soil 1 Ring Gage Test Results

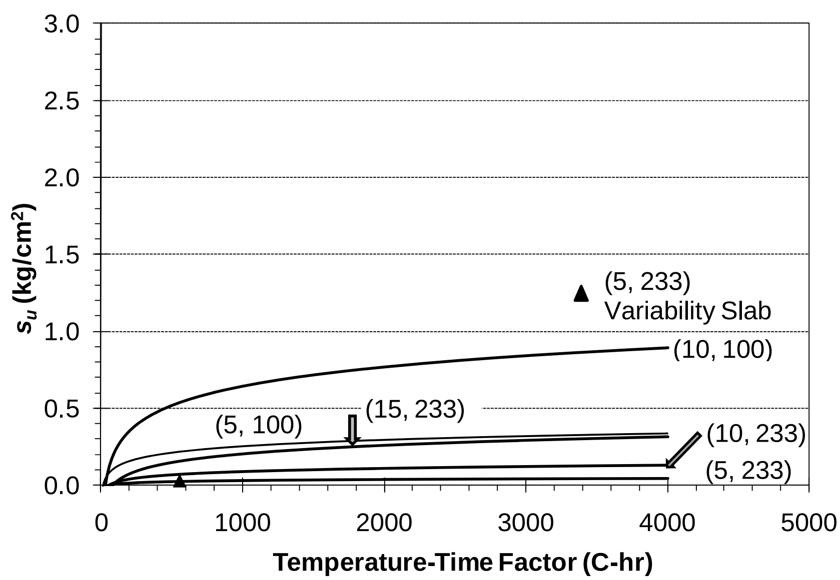


Figure 7.3. Th T III Soil 1 Shear Gage Test Results

The (10,100) condition resulted in peak strength readings from the *Dial* and *Ring* gages when testing *Soil 1* and *Soil 2*. Interestingly, *Soil 3* did not produce peak readings from the *Dial* gage, though it did peak the *Ring* gage. The *Dial* gage peaked at approximately 500 C-hr with *Soil 2* and approximately 2,100 C-hr with *Soil 1*. The *Ring* gage peaked at approximately 400 C-hr with *Soil 2*, 750 C-hr with *Soil 1*, and 1,500 C-hr with *Soil 3*. The *Shear* gage did not peak with any soil or test combination.

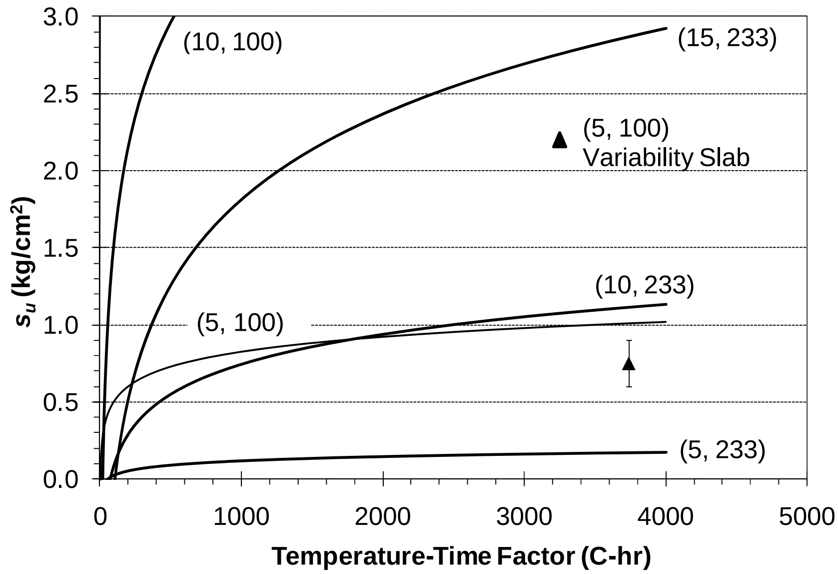


Figure 7.4. Th T III Soil 2 Dial Gage Test Results

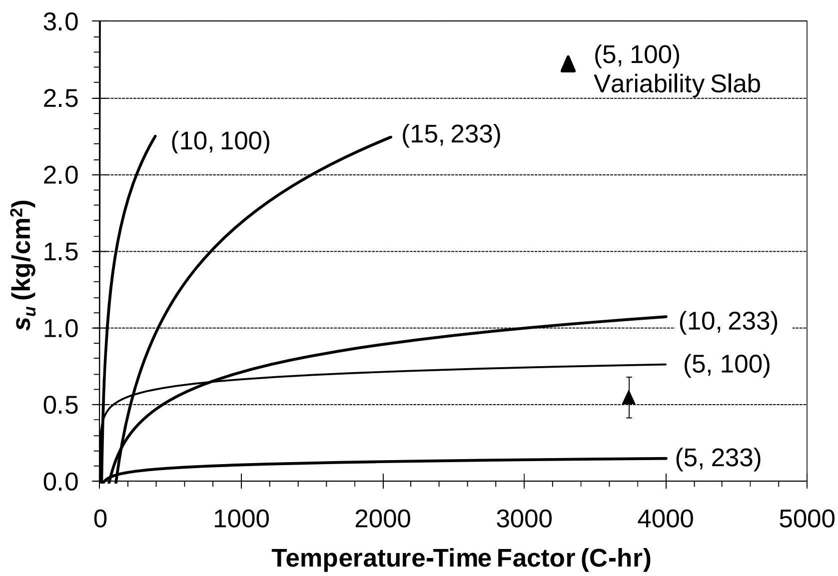


Figure 7.5. Th T III Soil 2 Ring Gage Test Results

The 1 variability slab with *Th T III* tested with *Soil 1* was at (5, 233) and showed agreement with the trendline from the corresponding trial, but the low strength of this combination makes this data of little practical value. The variability slab tested with *Soil 2* showed decent agreement with the corresponding (5, 100) trial. The largest difference was with the *Dial* gage where the trendline from the trial predicted a shear strength of 1.01 kg/cm^2 and the mean value of the variability slab was 0.75 kg/cm^2 when comparing the two at the same time-temperature factor. One standard deviation higher placed the variability slab at 0.90 kg/cm^2, and observing the scatter around the trendline in Figure A.5, it is observed that upper end values of 0.75 to 0.90 kg/cm^2 would easily overlap with the data. Additionally, *Perimeter*, *Bottom*, and *Internal* readings were measured at 0.72 to 0.76 kg/cm^2 at 3,750 to 3,900 C-hr.

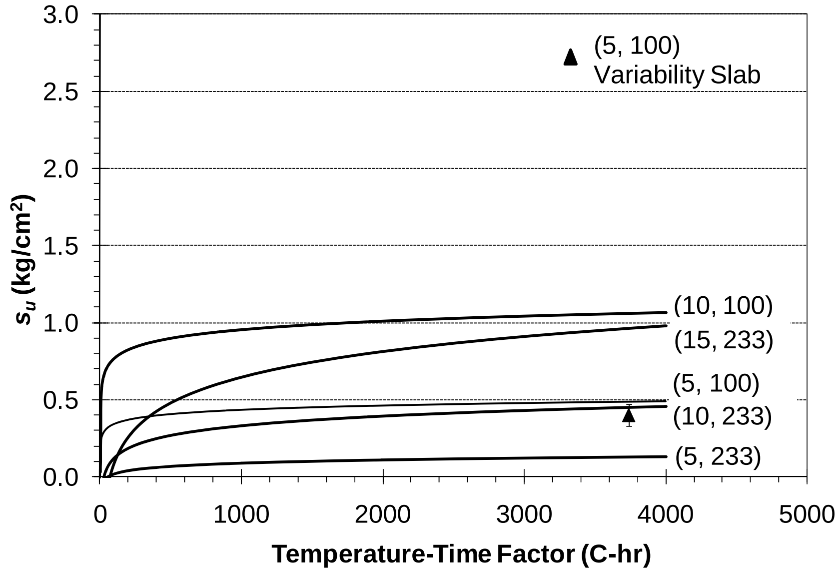


Figure 7.6. Th T III Soil 2 Shear Gage Test Results

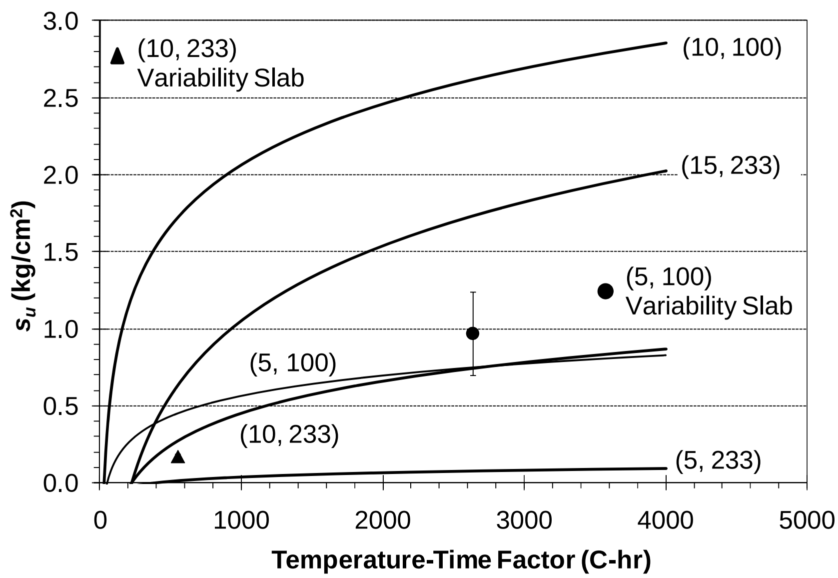


Figure 7.7. Th T III Soil 3 Dial Gage Test Results

Two variability slabs were tested with *Soil 3*. The slab tested at (10, 233) showed reasonable agreement with the trial from a practical perspective. The *Dial* gage trend line and variability slab mean values were 0.24 and 0.17 kg/cm^2, respectively. The *Ring* gage trend line and variability slab mean values were 0.22 and 0.11 kg/cm^2, respectively. The *Ring* gage trend line had measured values of 0.13 to 0.30 kg/cm^2 near the same time-temperature factor of the variability slab. The *Shear* gage trend line and variability slab mean values were 0.14 and 0.08 kg/cm^2, respectively. The *Shear* gage trend line had measured values of 0.11 to 0.18 kg/cm^2, which were near the same time-temperature factor of the variability slab. For a weak mixture in the early stages of curing, this is a reasonable agreement to 2 mixtures of the same material made at different times.

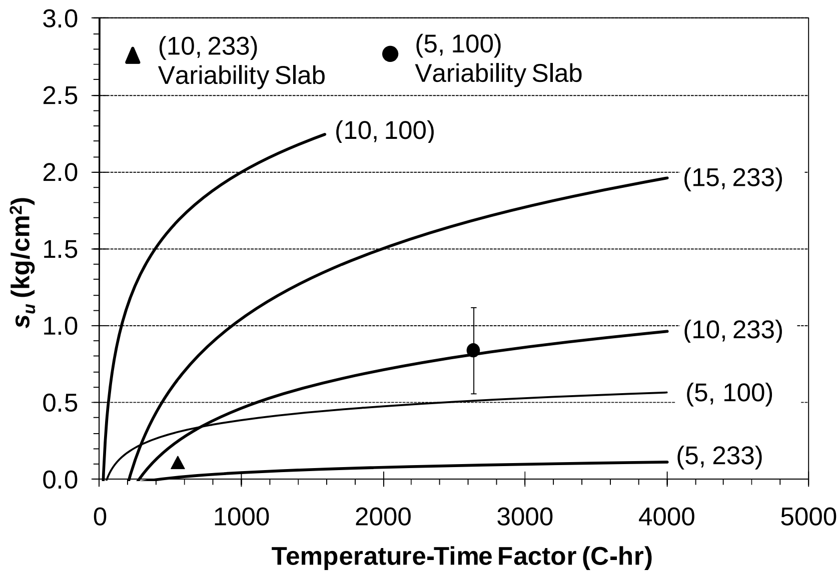


Figure 7.8. Th T III Soil 3 Ring Gage Test Results

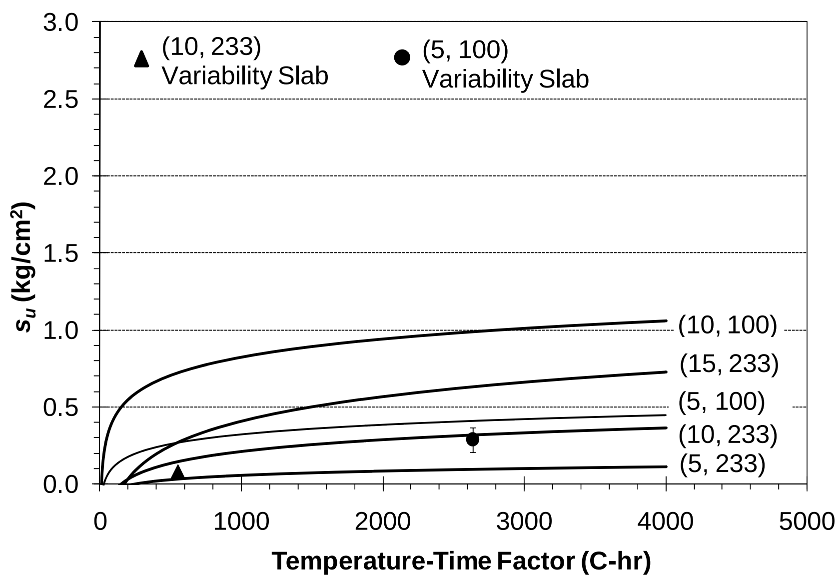


Figure 7.9. Th T III Soil 3 Shear Gage Test Results

The (5,100) variability slab had a mean value of 0.97 kg/cm^2 measured with the *Dial* gage, which is somewhat higher than the trendline prediction of 0.75 kg/cm^2, but measured values at 2,300 and 2,800 C-hr were 0.90 and 0.88 kg/cm^2, respectively which aligns very well with the mean value from the variability slab (Figure A.7). The *Ring* gage did not compare nearly as well between the trial and variability slab, with values of 0.51 and 0.84 kg/cm^2, respectively. The variability slab had a high standard deviation of 0.28 kg/cm^2, and readings measured during the trial of 0.63 to 0.68 kg/cm^2 at similar temperature-time factors (Figure A.7). The data from these cases cross each other, but barely. The *Shear* trial had a trendline predicted value of 0.41 kg/cm^2 with little scatter near the temperature-time factor of the variability slab. The mean value of the variability slab was 0.29 kg/cm^2 with a standard deviation of 0.08 kg/cm^2. These 2 data cases agree with each other to a reasonable extent.

7.3 Test Results at (5, 100) Condition

7.3.1 Trial Test Results at (5, 100) Condition

Twenty-three trials were tested at the (5,100) condition for comparison to the *Th T III* control data. Initially, 1 trial was performed using each of the 3 soils in conjunction with each of the 6 cements, and the results of these eighteen trials are plotted alongside *Th T III* in Figures 7.10 through 7.18. Raw data used to generate these plots has been provided in Figures A.9 to A.17.

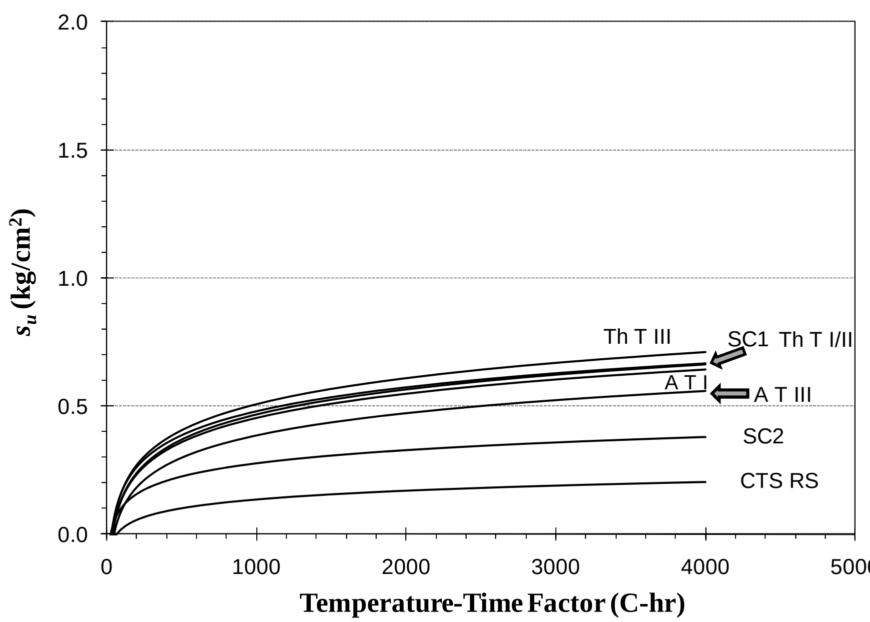


Figure 7.10. Soil 1 Dial Gage Test Results in (5,100) Condition

All 3 gages showed *CTS RS* to produce the lowest strength in all 3 soils. *SC2* did not perform well in *Soil 1* as it was the weakest of the 6 portland cements as measured by all 3 gages. Otherwise, few easily identifiable trends were observed in *Soil 1* with respect to cement performance. The remaining 5 cements exhibited no noticeable pattern other than that *A T III* had the lowest shear strength of the 5 cements measured by all 3 gages. *A T III* strength, however, was not appreciably lower than the other cements with the range of strengths between the 5 cements at 4,000 C-hr being 0.56 to 0.71 kg/cm^2, 0.63 to 0.84 kg/cm^2, and 0.29 to 0.43 kg/cm^2 for the *Dial*, *Ring*, and *Shear* gages, respectively. *Th T III* was the strongest by a narrow margin according to the *Dial* and *Ring* gages, but was an intermediate performer according to the *Shear* gage.

SC2 performed the best in *Soil 2* with all 3 gages, with *Dial* and *Ring* readings on the order of 1.5 kg/cm^2 at 4,000 C-hr and *Shear* readings on the order of 0.7 kg/cm^2 at 4,000 C-hr. The 4 Artesia cements out performed both Theodore cements as measured with all 3 gages for *Soil 2*. The relative performance of the 4 Artesia cements varied depending on the gage used for measurement.

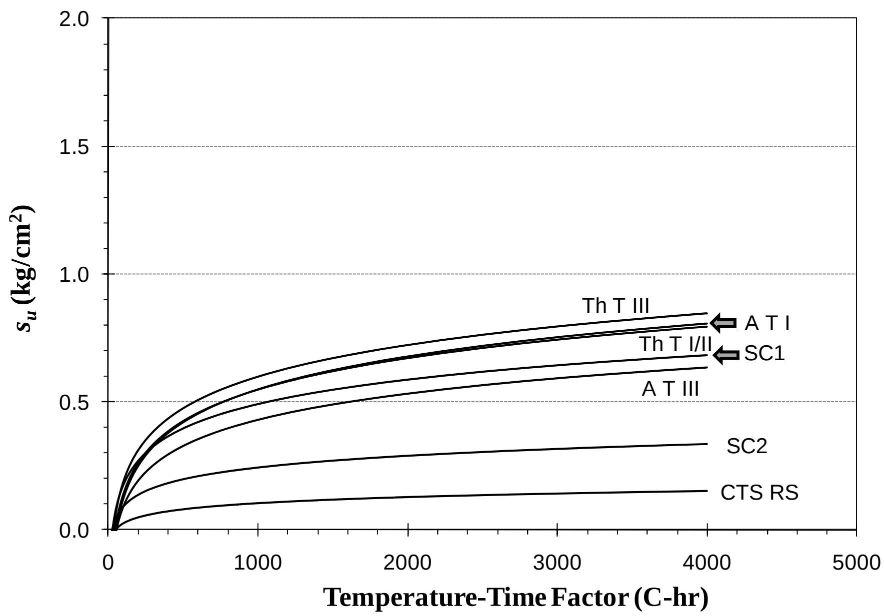


Figure 7.11. Soil 1 Ring Gage Test Results in (5,100) Condition

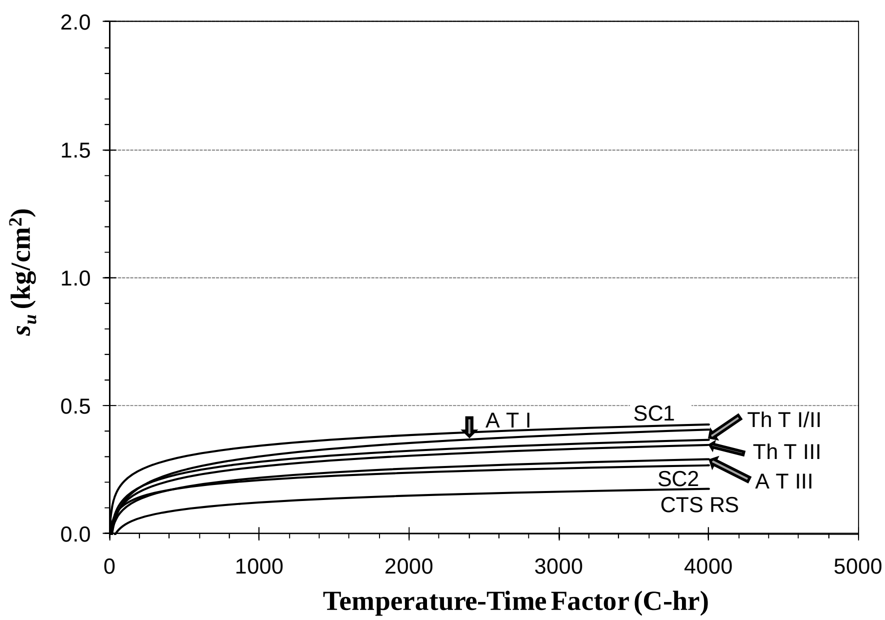


Figure 7.12. Soil 1 Shear Gage Test Results in (5,100) Condition

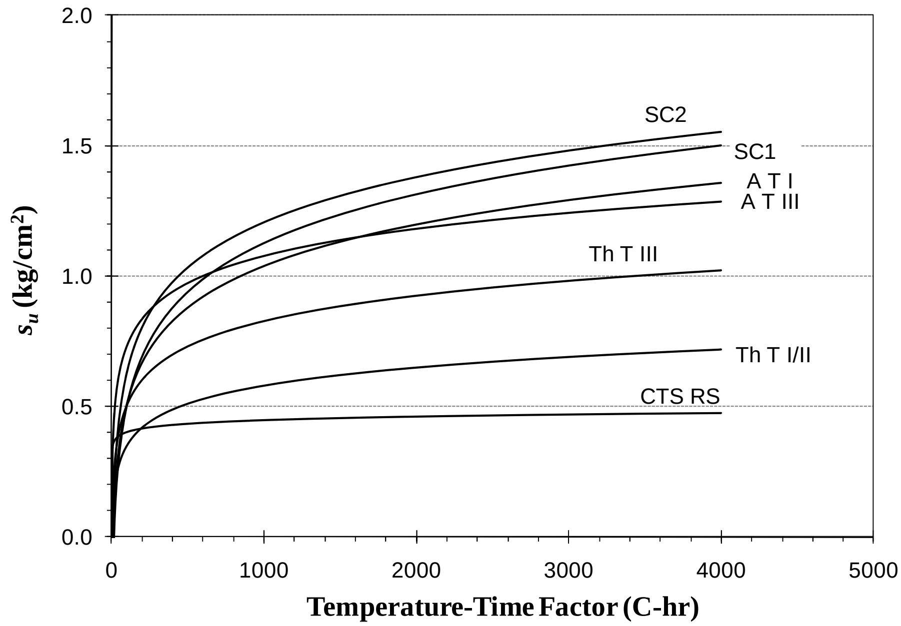


Figure 7.13. Soil 2 Dial Gage Test Results in (5,100) Condition

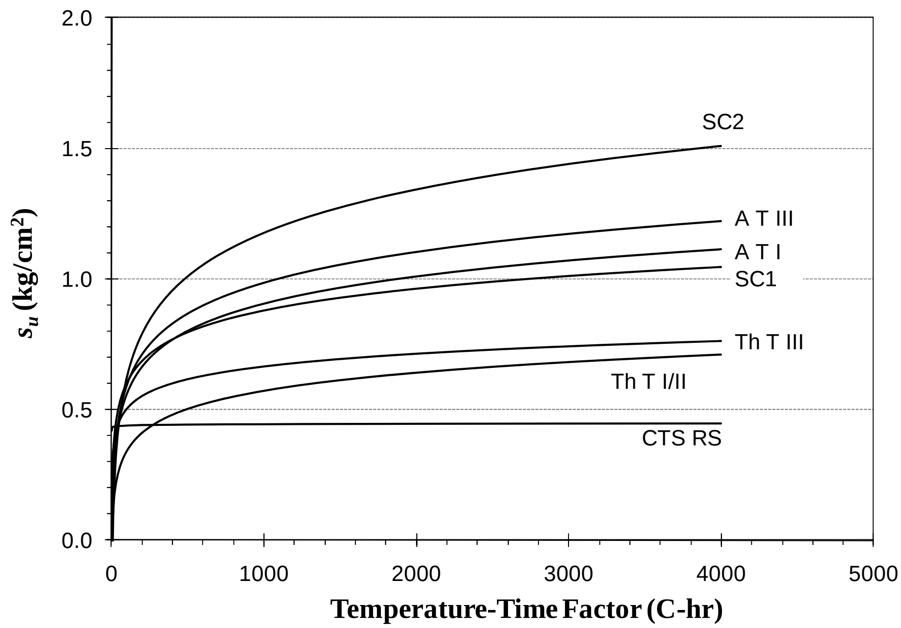


Figure 7.14. Soil 2 Ring Gage Test Results in (5,100) Condition

With *Soil 3*, *A T I* appears abnormally low relative to its performance in *Soil 1* and *Soil 2*. In *Soil 1* and *Soil 2*, the *Type I* cement was comparable to *A T III* (slightly stronger in some cases yet slightly weaker in other cases). In *Soil 3*, *A T I* was considerably weaker than the other 3 products from Artesia. The scatter around the trendline of *A T I* was on par with the other cements. R^2 values of 0.50 to 0.72 were recorded for *A T I* while the composite range for *Th T III*, *A T III*, *Th T I/II*, and *SC1* were 0.51 to 0.79. With exception of *A T I*, the remaining Artesia cements were stronger than Theodore cements measured by the *Dial* and *Ring* gages, while the *SC2* and *SC1* were stronger than other cements by a considerable margin when measured by the *Shear* gage. Interestingly, no Theodore cement tested was clearly the strongest cement with any of the 3 soils or hand held gages, which is a point of discussion for comparison of hand held gages to unconfined compression results.

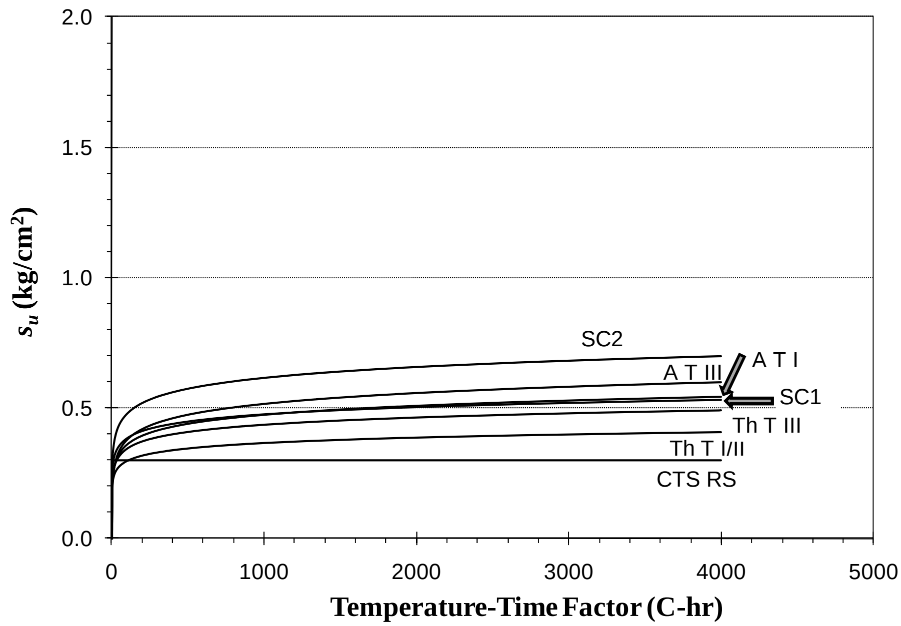


Figure 7.15. Soil 2 Shear Gage Test Results in (5,100) Condition

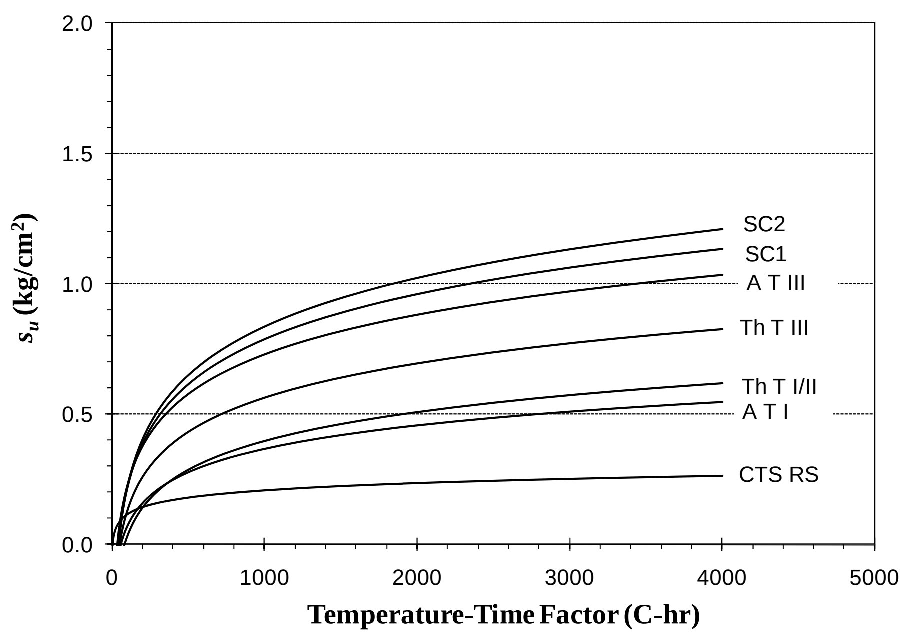


Figure 7.16. Soil 3 Dial Gage Test Results in (5,100) Condition

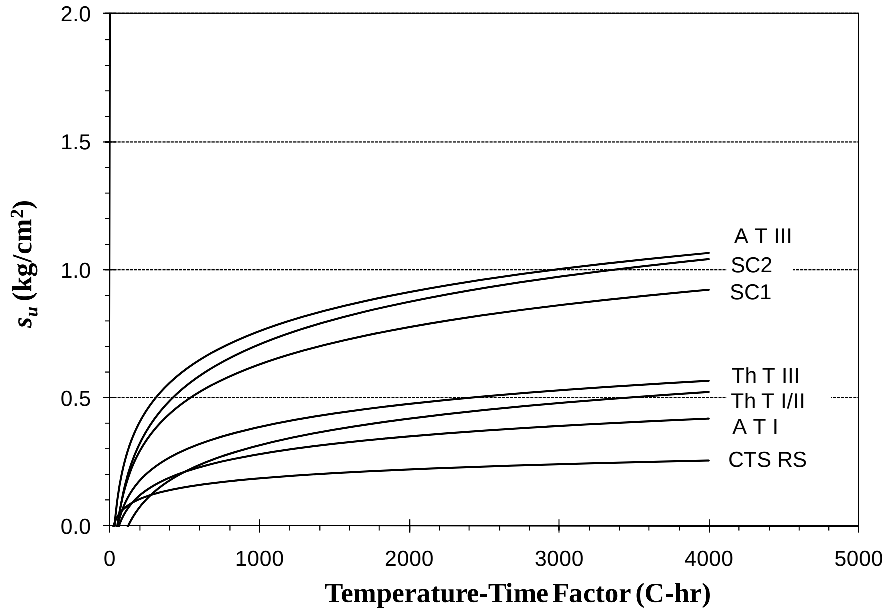


Figure 7.17. Soil 3 Ring Gage Test Results in (5,100) Condition

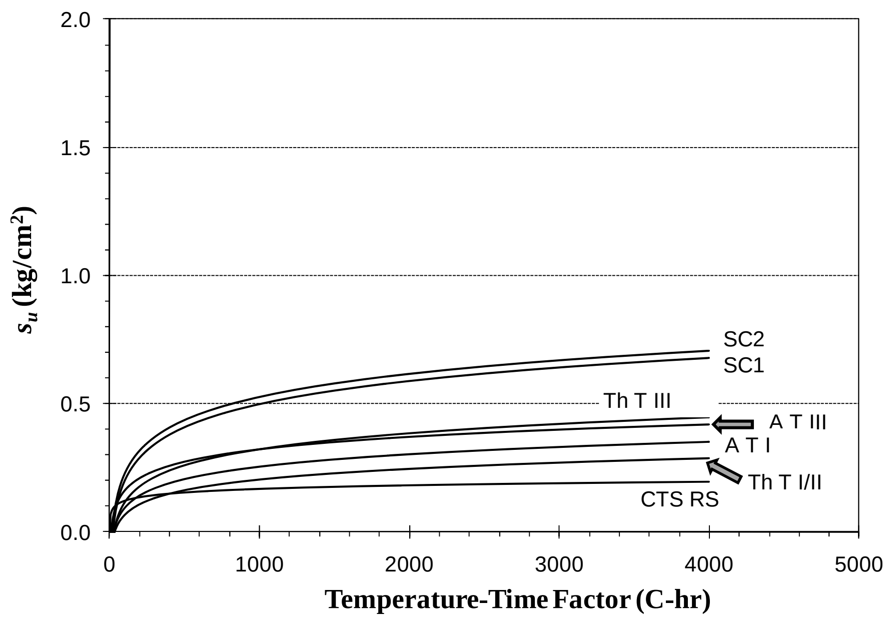


Figure 7.18. Soil 3 Shear Gage Test Results in (5,100) Condition

Five trials were tested multiple times with *SC1* to investigate repeatability; 2 tested *Soil 1*, 1 tested *Soil 2*, and 2 tested *Soil 3*. Results of repeatability testing can be seen in Figures 7.19 through 7.21; the original test plotted in Figures 7.10 through 7.18 are indicated as original (O), while repeat tests are indicated as (R). The raw data from the 5 repeat trials can be seen in Figures A.18 through A.20.

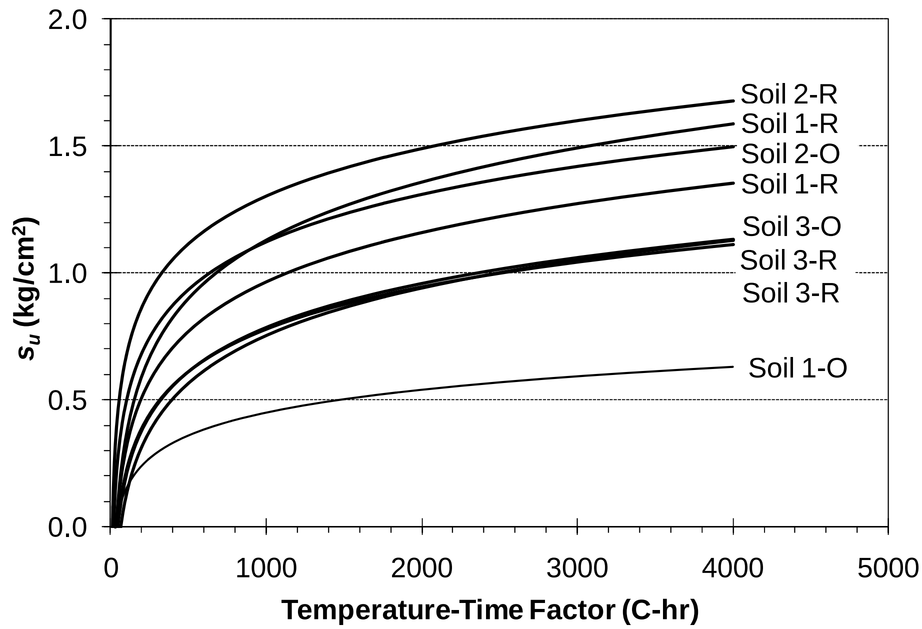


Figure 7.19. Dial Gage Repeatability Test Results in (5,100) Condition

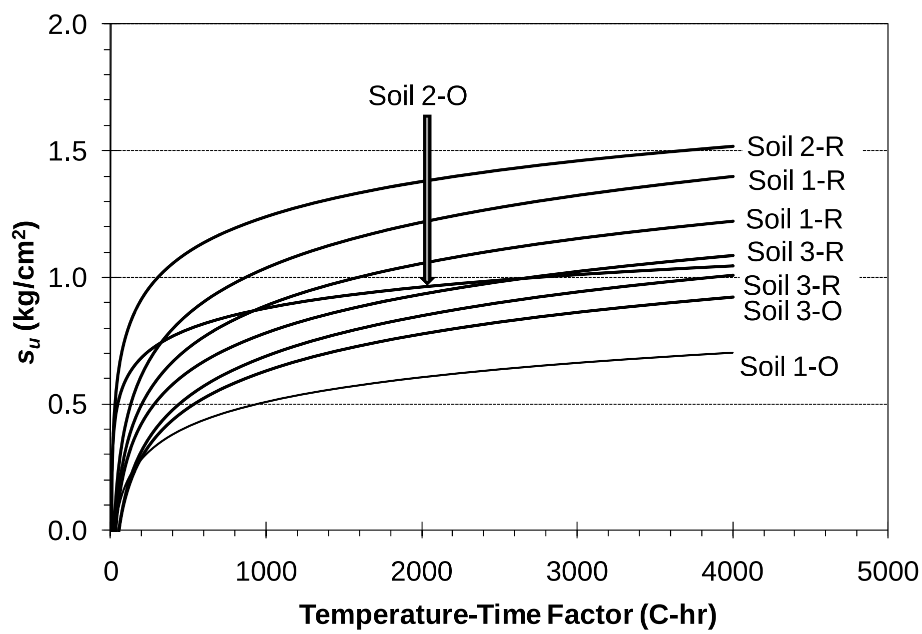


Figure 7.20. Ring Gage Repeatability Test Results in (5,100) Condition

Soil 1 was not repeatable with respect to the original test as measured by all 3 hand held gages. The difference in readings between the original test and the 2 repeat tests was quite large. The *Dial* and *Ring* gages measured fairly repeatable strengths between the 2 repeat trials, while the *Shear* gage measured a moderate difference between the 2 repeat trials. The original trial was conducted largely in the summer of 2008, while the 2 repeat trials were conducted in the spring and summer of 2010. The soil from each trial was taken from a different barrel, though the repeat trials were taken from adjacent barrels. This data alone provides no explanation for the behavior, but indicates that the properties of *Soil 1* between barrels might not be as uniform as originally envisioned. Changing the soil's properties might explain the behaviors observed; routine testing errors, normal variability, and similar have been observed to be much lower than the differences observed in *Soil 1* and between the 3 trials plotted in Figures 7.19 through 7.21.

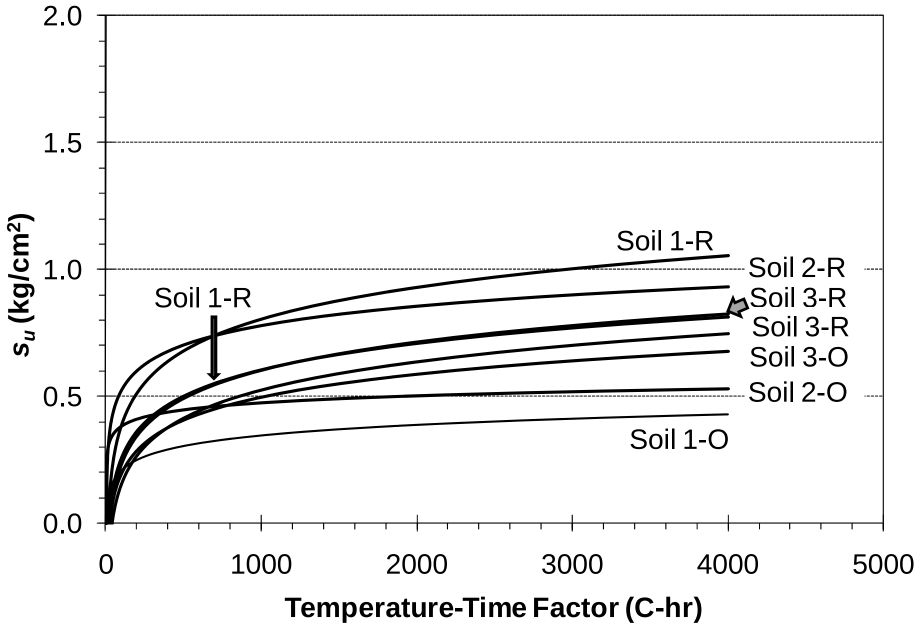


Figure 7.21. Shear Gage Repeatability Test Results in (5,100) Condition

Soil 2 was repeated reasonably well as measured by the *Dial* gage, with a difference at 4,000 C-hr of approximately 11%. The *Ring* gage did not repeat strength as well as the *Dial* gage, with the original test having 68% of the strength of the repeat at 4,000 C-hr. Observing the raw test data in Figures A.15 and A.19, the repeat test had measured values as low as 1.13 kg/cm^2, and the original trial had values as high as 1.15 kg/cm^2, indicating a very slight overlap of results. The *Shear* gage did not provide repeatable results, with the original test strengths being approximately 57% of the repeat strengths at 4,000 C-hr. The *Dial* gage providing very repeatable results would suggest that the lack of repeatability with the *Ring* and especially the *Shear* gages is due to the gages and/or operator-related difficulties between tests. There was no reason to believe that *Soil 2* had substantial variability between barrels of processed soil. The original test was performed in the spring of 2009, while the repeat test was conducted in the spring of 2010. The soil for the trials came from different barrels.

Soil 3 was highly repeatable as measured by the *Dial* gage; the 3 plots are barely distinguishable. This level of repeatability is believed to be higher than typical and would not be expected if the test were repeated dozens of times. Repeatability with the *Ring* and *Shear* gages was also very good as the maximum difference between the 3 trials at 4,000 C-hr was 15% and 16%, respectively. This level of repeatability is likely more reasonable than that measured from the *Dial* gage for these tests. There was no reason to believe that *Soil 3* had substantial variability between barrels of processed soil. The original and repeat tests were conducted in the spring of 2009, the spring of 2010, and the summer of 2010, and the soil from each trial was taken from a different barrel.

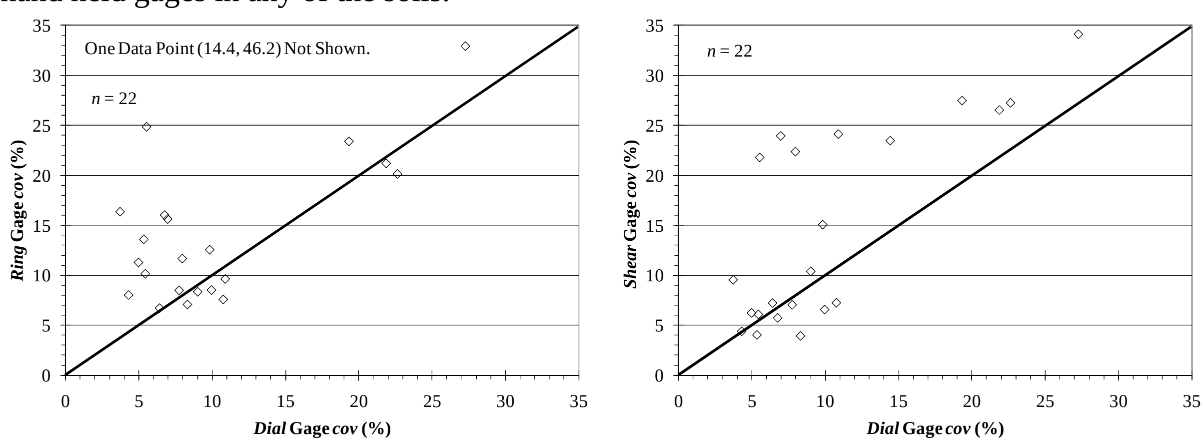
Precision (proximity to each other) was evaluated by considering R^2 values from the logarithmic trend lines for the 23 trials presented in this section and the 3 *Th T III* control trials. Average values were assembled and are provided in Table 7.1. A considerable amount of scatter was present for each gage, indicating that the precision of any 1 measurement would be relatively low. The *Dial* gage was the most precise of the 3 hand held gages. The *Shear* device, in general, was the least precise of the gages as it had the lowest R^2 value in 2 of the 3 soils and for *Soil 1*; it had the same R^2 as the *Ring* gage.

Table 7.1. Average R^2 Values of Hand Held Gages

Soil	*Dial* Gage	*Ring* Gage	*Shear* Gage
1	0.82	0.70	0.70
2	0.49	0.37	0.23
3	0.70	0.69	0.61

7.3.2 Variability Slab Test Results at (5, 100) Condition

Twenty-two variability slabs were tested in the (5, 100) condition, with 2 of them being with the *Th T III* control. Figure 7.22 compares the coefficient of variation (*cov*) of the 3 hand held gages measured using the variability slabs. Raw data is provided in Figure B.3. through B.12. All 3 gages would have been tested on each slab providing a means of direct comparison in absence of material, mixing, or other variability other than variability within the gage itself. The *Dial* gage was less variable than the *Ring* or *Shear* gage. The average *cov* for each of the twenty-two tests was 10.4% for the *Dial* gage, 15.5% for the *Ring* gage, and 14.7% for the *Shear* gage. A plot was generated comparing the *Ring* and *Shear* gages that was not shown for brevity. Data was scattered on either side of the equality line and showed no practical difference between the *cov* measured with these 2 gages. The *Dial* gage was the least variable (i.e. the most precise) at the (5,100) condition with the *Ring* and *Shear* gages being equally variable. There was no correlation between *cov* and s_u for any of the 3 hand held gages in any of the soils.



(a) Dial Gage versus Ring Gage *(b) Dial Gage versus Shear Gage*

Figure 7.22. Coefficient of Variation Comparison of Hand Held Gages at (5,100)

Generally speaking, operators felt more comfortable using the *Dial* gage. The *Ring* gage was said to be more difficult to read, and concerns exist of it achieving peak readings too quickly in the field. The *Ring* gage has a lower peak strength than the *Dial* gage, and both of these devices recorded higher strengths than the *Shear* gage. The *Shear* gage could be inadvertently tilted during testing (especially with the smallest attachment), which could lower the reading. The primary drawback of the *Dial* gage relative to the *Shear* gage would be it recording peak readings and no longer being useful in the field. This did not occur in the (5,100) condition, however.

Seven (5, 100) variability slabs were repeated; 3 replicates were performed with *SC1* and *Soil 1*, 2 replicates were performed with *SC1* and *Soil 3*, and 2 replicates were performed with *SC2* and *Soil 3*. Mean shear strength test results are provided in Figure 7.23 alongside the *cov* of the individual measurements. It is interesting that test results are generally repeatable for all 3 soils, especially as measured by the *Dial* gage. All *Soil 1* testing was performed in the summer of 2010, which indicates the lack of repeatability observed between the summer 2008 and the spring and summer of 2010 trial testing presented in the previous section could be due to variations in the soil between barrels. Barrel numbers were not recorded for variability slab testing.

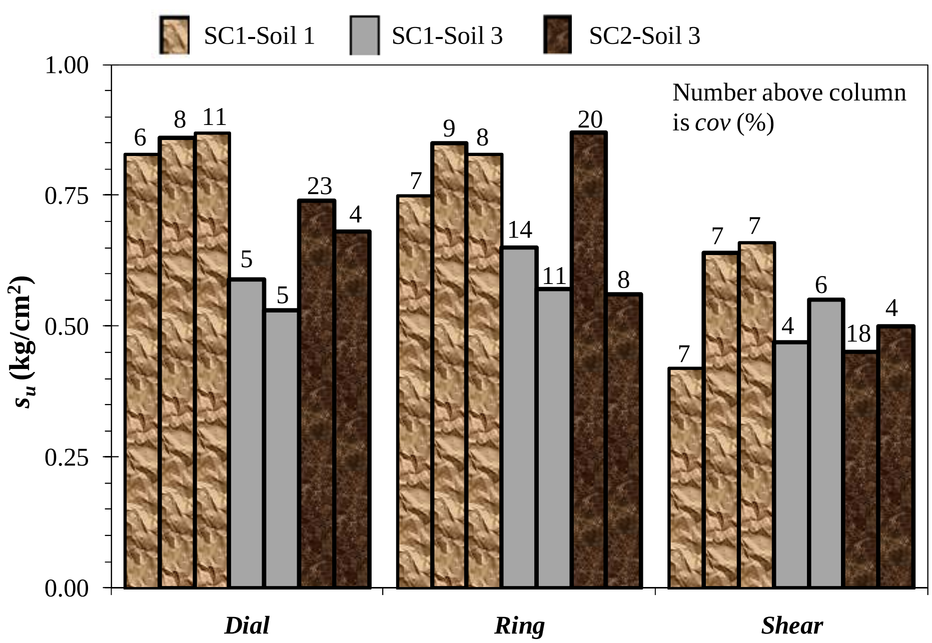


Figure 7.23. Repeatability of Variability Slab Testing at (5, 100)

The 2 *SC1-Soil 3* variability slabs were produced in the summer of 2010, and the *cov* for any given gage was similar. This was also the case for the 3 *SC1-Soil 1* variability slabs. The *SC2-Soil 3* slabs, however, were produced approximately 1 year apart with the lowest variability slab being produced late in the spring of 2010 and the highest variability slab produced early in the summer of 2009. Mixing of the soil slurry is likely the cause of the higher *cov* in the summer of 2009 since it was measured by all 3 gages. It is noteworthy, however, that the data provided in Figure 7.23 is a select subset of Figure 7.22 and that on some occasions a high *cov* was measured with 1 or more of the hand held gages but not all 3 of them. Therefore, it is not advisable to attribute all cases with a high *cov* to mixing. The *cov* for a variability slab is the combination of the uniformity of the slab tested, the operator, and the gage itself. The data collected cannot decouple these behaviors. A conservative approach would be to attribute most to all of the variation in readings to the gage and operator.

7.3.3 Comparison of Trial and Variability Slab Test Results at (5, 100) Condition

Figures 7.24 through 7.26 compare the mean value from each of the twenty-two variability slabs to the value predicted from the trial trendline at the same temperature-time factor. For cases where the trial was repeated, the trial that was performed nearest the variability slab was used. All raw data used to produce these figures has been discussed previously in this chapter.

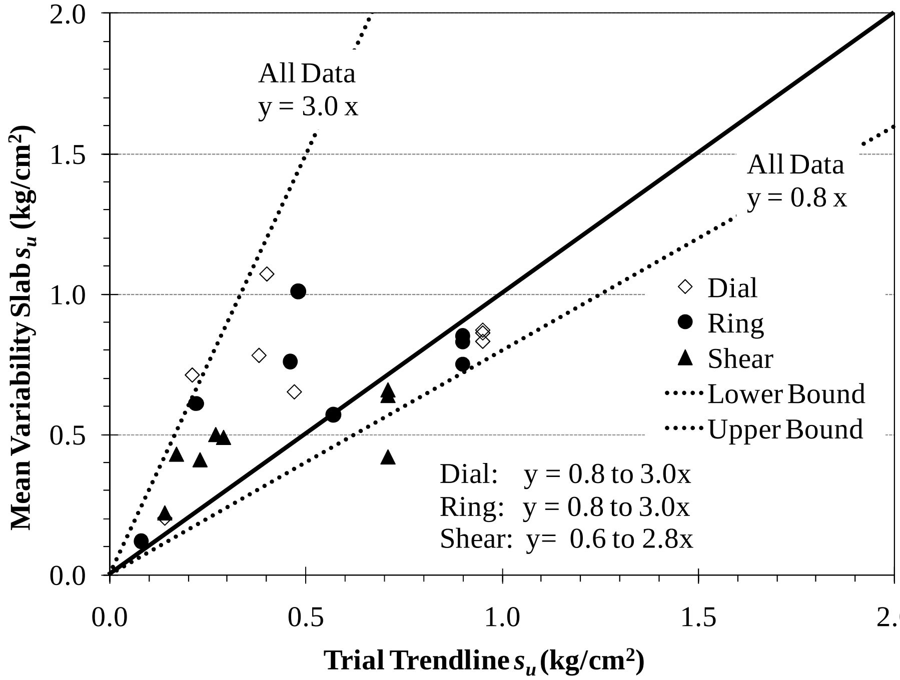


Figure 7.24. Comparison of Trial Trendline and Variability Slab Mean for Soil 1

Figure 7.24 clearly shows problems with *Soil 1*. Variability slab mean values are much higher than trial predicted trendlines in most cases. The trials used to create Figure 7.26 were tested well before the corresponding variability slabs, with the exception of 3 variability slabs tested using *SC1* where a repeat trial was available that was tested along the same time frame. Any data that plotted below the equality line was from this data. The trial data other than the *SC1* repeat was tested in 2008, whereas the variability slabs were tested in late spring and summer of 2010. This data is very convincing and strongly indicates that the properties of *Soil 1* in the barrels tested during the early portion of the experimental program were different than the properties of *Soil 1* in the barrels tested in the later part of the experimental program.

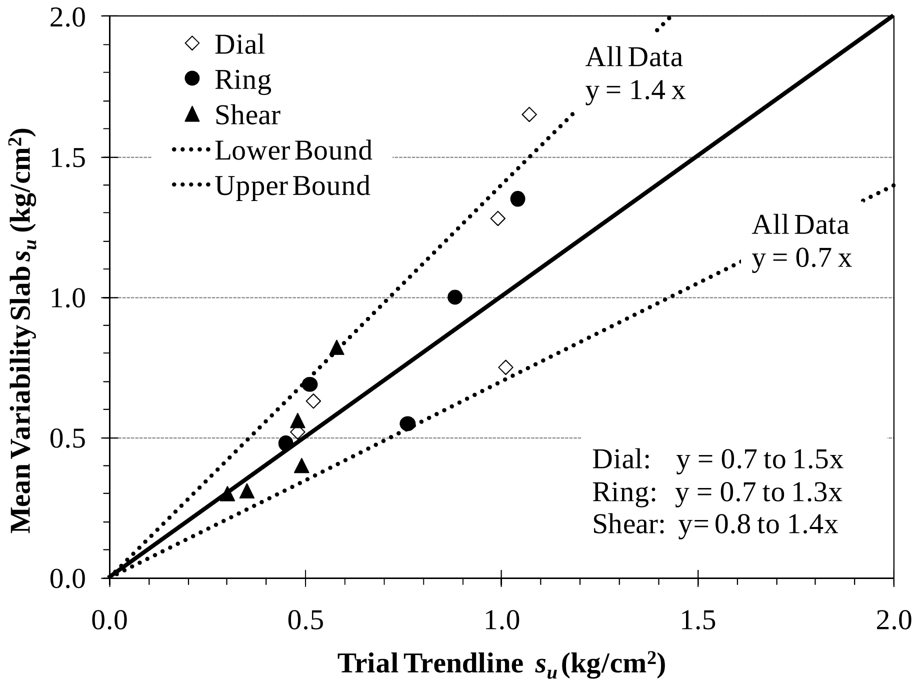


Figure 7.25. Comparison of Trial Trendline and Variability Slab Mean for Soil 2

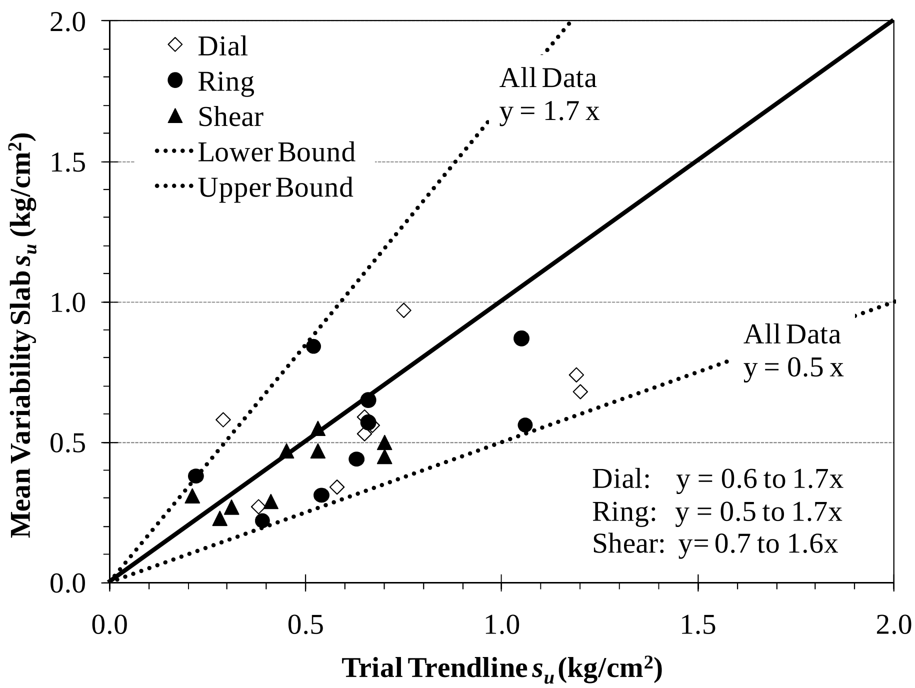


Figure 7.26. Comparison of Trial Trendline and Variability Slab Mean for Soil 3

Unlike *Soil 1*, *Soil 2* and *Soil 3* data was reasonably distributed around the equality line. For *Soil 2*, the outer bound of the data was 70 to 140% of the equality line. The scatter around the trial trendlines suggests that this level of repeatability with respect to the mean value of the variability slabs is reasonable. *Soil 3* was slightly more variable than *Soil 2*, though a noticeable amount of the data plotted at, to very near the equality line. The data provided in Figures 7.26 and 7.27 suggests that the gages have value for use in disaster response and can be used to provide reasonable measurements of strength; but they may not be sensitive enough to detect differences between cement performance within a soil slurry in absence of other data though they can serve as a compliment to other data.

Figure 7.27 expounds on the properties between barrels of *Soil 1* by comparing repeated trials of *Soil 1* and *Soil 3* to variability slabs of the same material at the same temperature time-factor. *SC1* was the cement in all cases. Two of the trials (1 with *Soil 1* and 1 with *Soil 3*) were performed in 2008 and 2009 as discussed previously, or at least 1 year prior to the remainder of the testing shown in Figure 7.27, which was conducted in 2010. As seen, *Soil 3* was repeatable in all cases whereas *Soil 1* testing was substantially different. This data makes a near definitive case that the properties of *Soil 1* between barrels are not uniform with regards to cement stabilization.

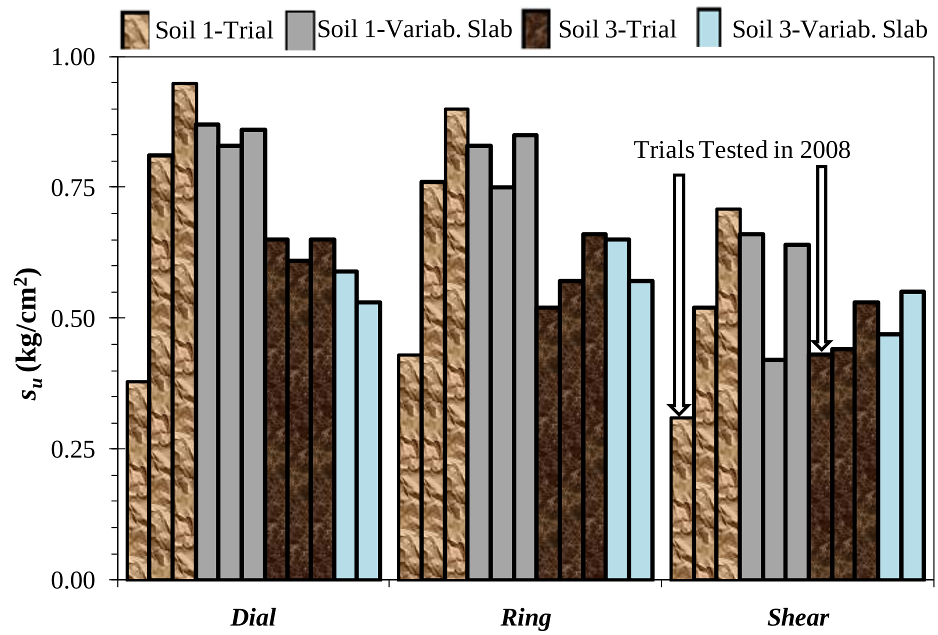


Figure 7.27. Comparison of Repeatability Using Trial and Variability Slab Data

7.4 Test Results at (10, 100) Condition

7.4.1 Trial Test Results at (10, 100) Condition

Four trials were tested at the (10,100) condition for comparison to the *Th T III* control data. All testing incorporated *SC1*. Results of these 4 trials are plotted alongside *Th T III* in Figures 7.28 through 7.30. Raw data used to generate these plots has been provided in Figures A.21 to A.22.

SC1 outperformed *Th T III* in all 3 soils measured by the *Dial* and *Shear* gages, and *SC1* outperformed *Th T III* in *Soil 2* and *Soil 3* with the *Ring* gage while performing just below *Th T III* in *Soil 1*. *Soil 2* was the strongest as measured by each gage, while the relative strengths of *Soil 1* and *Soil 3* varied between the gages. Peak strengths were measured well before 168 hr with the *Dial* and *Ring* gages, while the *Shear* gage did not measure peak strength.

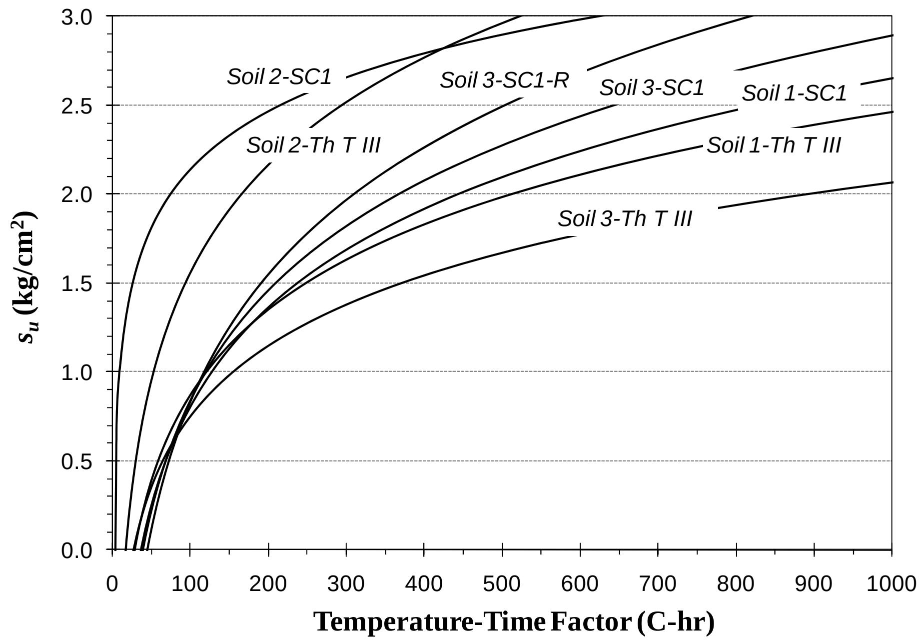


Figure 7.28. Dial Gage Test Results in (10,100) Condition

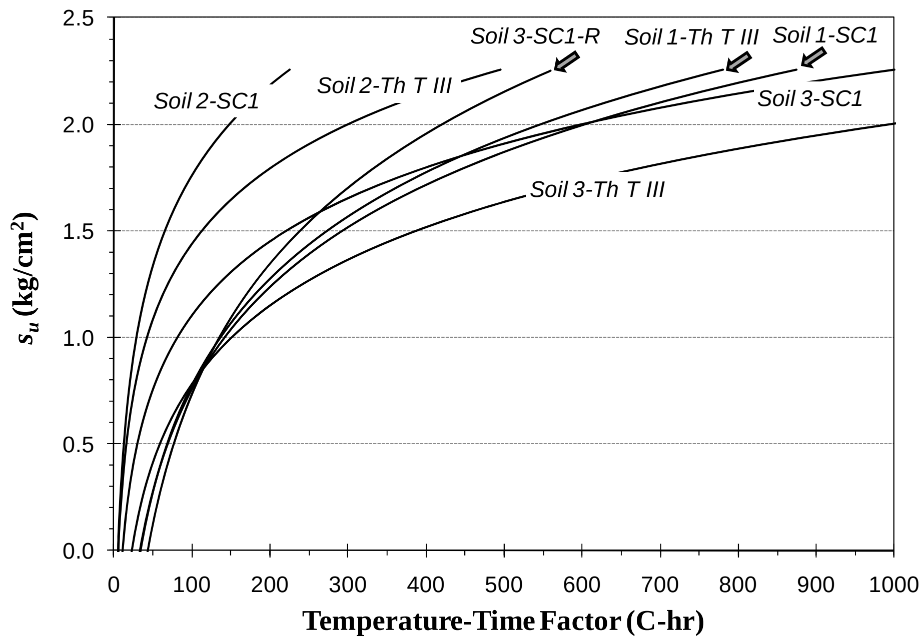


Figure 7.29. Ring Gage Test Results in (10,100) Condition

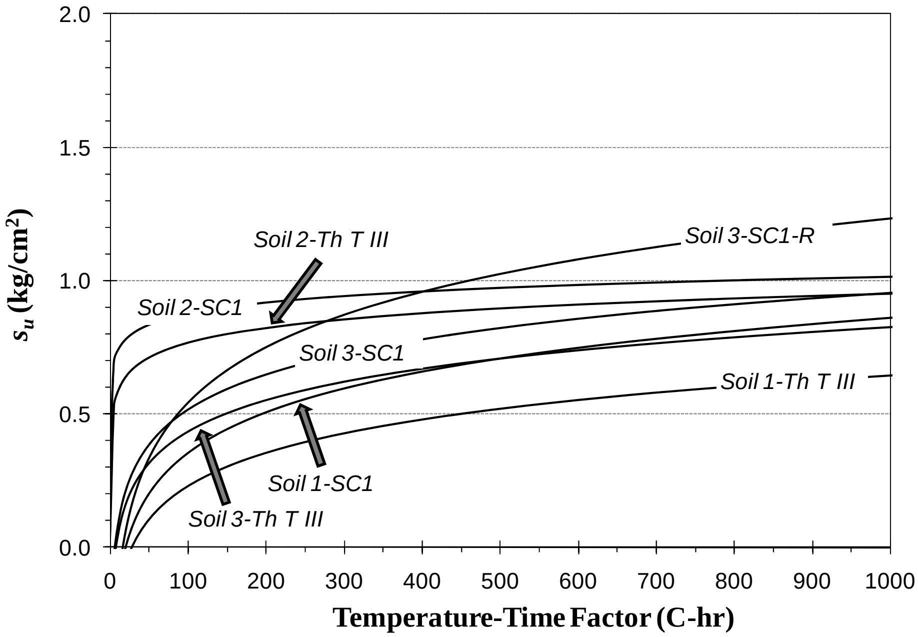


Figure 7.30. Shear Gage Test Results in (10,100) Condition

7.4.2 Variability Slab Test Results at (10, 100) Condition

Five variability slabs were tested in the (10, 100) condition, and all slabs used *SC2*. Results of testing are summarized in Table 7.2. In several instances, peak readings were repeatedly observed, making variability assessments physically meaningless in these instances. Table 7.2 indicates which instances had considerable peak readings. Raw data from these 5 slabs are provided in Figures B.13 through B.15. *Shear* gage data was the least variable in Table 7.2, though *Dial* gage data was only slightly more variable.

Table 7.2. Variability Slab Test Results at (10, 100) with *SC2*

		***Dial* Gage**		***Ring* Gage**		***Shear Gage***	
Soil	***TTF*** **(Ch-r)**	**Mean (kg/cm^2)**	***cov*** **(%)**	**Mean (kg/cm^2)**	***cov*** **(%)**	**Mean (kg/cm^2)**	***cov*** **(%)**
1	532	2.60	9.7	Peak	---	1.16	7.5
2	542	Peak	---	Peak	---	1.01	11.4
3	182	1.55	8.2	1.37	12.8	0.88	6.9
3	532	2.37	9.4	Peak	---	1.05	8.6
3	3855	Peak	---	Peak	---	1.30	8.6

7.4.3 Comparison of Trial and Variability Slab Results at (10, 100) Condition

Insufficient cement was available to test both trials and variability slabs in conjunction at (10, 100) with the same cement. Trials incorporated *SC1*, while variability slabs incorporated *SC2*. As a result, pertinent comparisons between the two types of testing were not performed.

7.5 Test Results at (15, 233) Condition

7.5.1 Trial Test Results at (15, 233) Condition

Four trials were tested at the (15,233) condition for comparison to the *Th T III* control data. All testing incorporated *SC1*. Results of these 4 trials are plotted alongside *Th T III* in Figures 7.31 through 7.33. Raw data used to generate these plots has been provided in Figures A.23 to A.24.

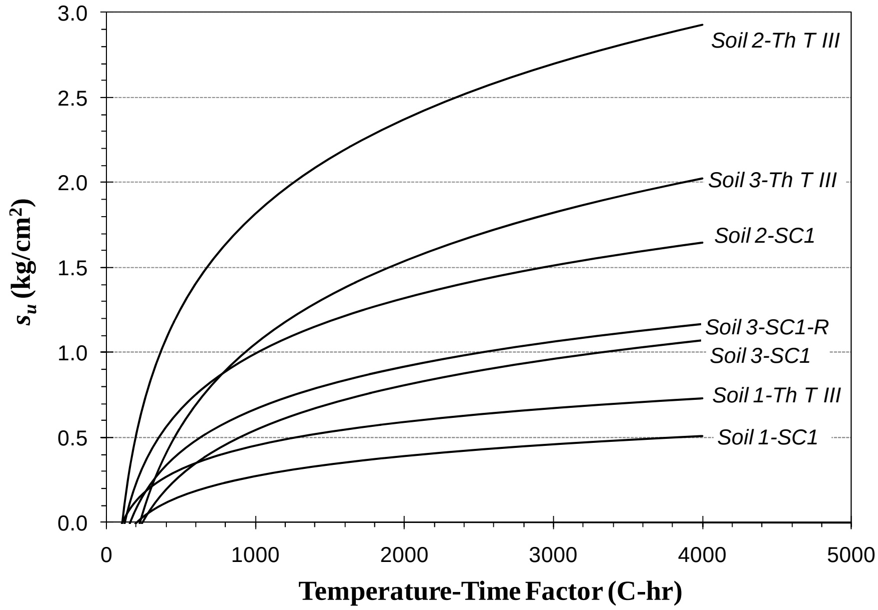


Figure 7.31. Dial Gage Test Results in (15, 233) Condition

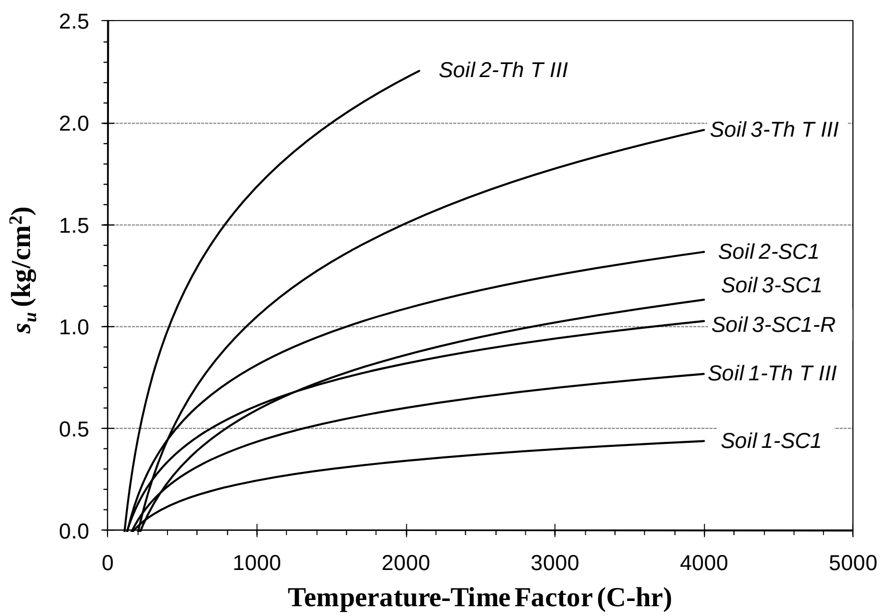


Figure 7.32. Ring Gage Test Results in (15, 233) Condition

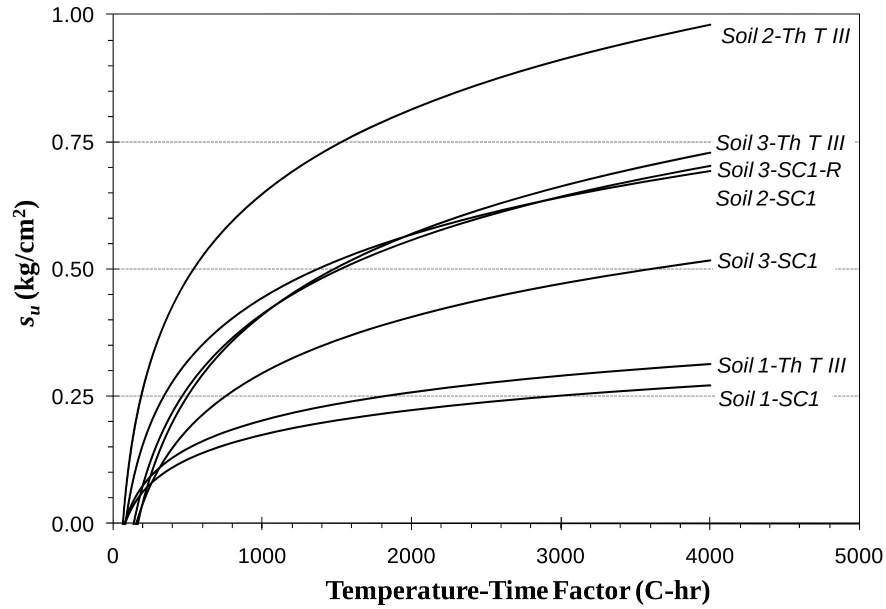
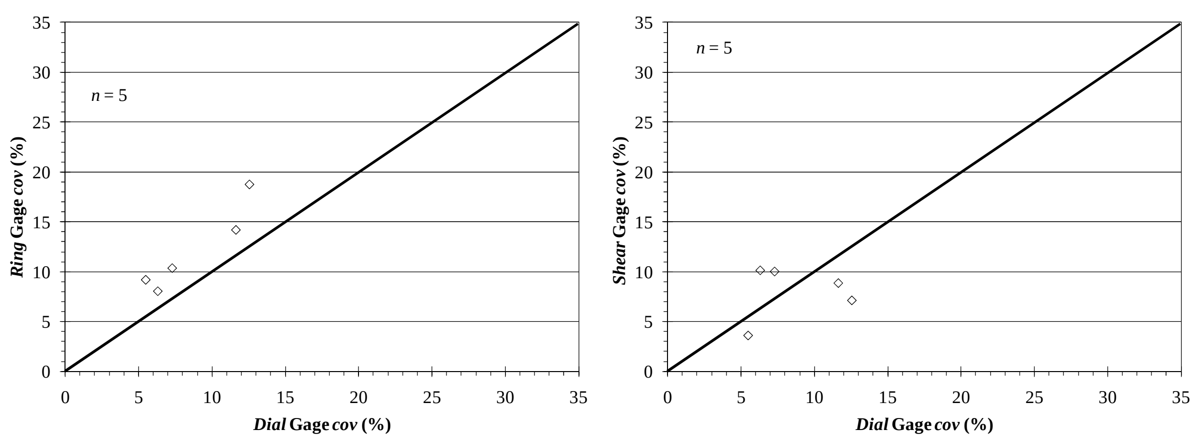


Figure 7.33. Shear Gage Test Results in (15, 233) Condition

Th T III was stronger than *SC1* for all 3 soils and all 3 gages. In *Soil 1*, *Th T III* was slightly to moderately stronger than *SC1*, in *Soil 2* it was considerably stronger, and in *Soil 3* it was slightly to considerably stronger. *Soil 2* produced the strongest blends, followed by *Soil 3*, with *Soil 1* producing the weakest blends. Strength and organic content were correlated with higher organic content equating to higher strength.

7.5.2 Variability Slab Test Results at (15, 233) Condition

Five variability slabs were tested in the (15, 233) condition, and all slabs used *SC2*. Results of testing are summarized in Figure 7.34, and raw data from these 5 slabs are provided in Figures B.16 through B.18. Though not shown, a plot was developed indicating the *Ring* gage was more variable than the *Shear* gage. The *Dial* gage was less variable than the *Ring* gage and had approximately the same variability as the *Shear* gage.



(a) Dial Gage versus Ring Gage *(b) Dial Gage versus Shear Gage*

Figure 7.34. Coefficient of Variation Comparison of Hand Held Gages at (15, 233)

7.5.3 Comparison of Trial and Variability Slab Results at (15, 233) Condition

Insufficient cement was available to test both trials and variability slabs in conjunction at (15, 233) with the same cement. Trials incorporated *SC1*, while variability slabs incorporated *SC2*. As a result, pertinent comparisons between the two types of testing were not performed.

7.6 Summary of Hand Held Gage Testing

Soil 1 was not repeatable when properties were measured from different soil barrels indicating there could be some non-uniformity within the barrels that was not initially envisioned. Fairly strong evidence was presented that soil barrels used during early portions of trial testing were different than soil used later in the testing program to investigate repeatability. *Soil 2* and *Soil 3* were repeatable to an acceptable level when measured with the *Dial* gage.

Dial gage R^2 values indicated it was the most precise at (5, 100). Variability slab testing at (5, 100) indicated that the *Dial* gage was the least variable. The *Dial* gage was only slightly more variable than the *Shear* gage at (10, 100), and had approximately the same variability at (15, 233). Small amounts of data were available at (10, 100) or (15, 233). With exception of peak readings relatively early in (10, 100) soils, *Dial* gage performance appeared acceptable for use in disaster environments. The *Ring* gage also peaked relatively early in (10, 100) soils. A *TTF* of 500 C-hr is a curing level where the gages often peaked. The *Shear* gage did not peak during the testing performed in this chapter.

All 3 hand held gages showed *CTS-RS* to produce the lowest strength in all 3 soils at (5, 100). Of the portland cements, *SC2* performed noticeably worse than the other 5 cements tested in *Soil 1*, while the other cements exhibited somewhat similar behavior. *SC2* performed better than any other cement in *Soil 2*, and the 4 Artesia cements outperformed both Theodore cements as measured by all 3 gages. Overall, *SC1* and *SC2* performed better than the other cements in *Soil 3*. No Theodore cement tested was clearly the strongest cement in (5, 100) testing.

At (5, 100), *Soil 2* blends achieved the highest strength, followed by *Soil 3*, and then *Soil 1*. At 4,000 C-hr, the respective strengths were approximately 1.5, 1.2, and 0.8 kg/cm^2. The higher the organic content of the soil, the higher the measured shear strength.

At (10, 100) *SC1* outperformed *Th T III*, in general, in all soils. *Soil 2* was the strongest with *Soil 1* and *Soil 3* relative strengths varying between gages. At 233% moisture, 15% cement was required to produce slurries with reasonable strength across the range of soils tested. Strength increased with increasing organic content, with the strength of the lowest organic content soil (*Soil 1*) largely dictating the use of 15% cement. At (15, 233), *Th T III* was stronger than *SC1* for all 3 soils and all 3 gages. Higher organic content led to higher strengths at (15, 233).

CHAPTER 8 - UNCONFINED COMPRESSION ANALYSIS TECHNIQUES

8.1 Introduction and Purpose

This chapter presents analysis techniques used in subsequent chapters (though mostly in Chapter 9) alongside the methods used to develop them. The techniques were developed considering analysis goals and data behaviors observed during and after collection. Analysis goals are listed below.

Analysis Goals:

1. Develop shear strength, modulus, and failure strain property trends for high moisture content, fine grained soils stabilized with cementitious materials and in some cases polymer fibers by testing a range of soil types, moisture contents, and cementitious materials in unconfined compression.
2. Develop analysis protocols that could prevent the variability of testing high moisture content soil slurries from leading to incorrect conclusions while still allowing comparison of cementitious materials to control cements, typically produced at the same production facility. This approach used control cement property envelopes to make comparisons to control cements.
3. Determine if specially developed on demand portland cement, *Type I* portland cement, or blended portland cements could outperform high early strength commercially available *Type III* portland cement from the same production facility.

To achieve the analysis goals with the data collected, techniques had to be carefully considered as the experimental program was quite large and some unexpected behaviors occurred with *Soil 1* that forced the analysis in directions that were not expected. In some instances, the approach diverted from predictive response to assessment of trends due to variability in test results. In other instances, the approach was to predict response. The remainder of this chapter provides information on how all pertinent aspects were addressed.

8.2 Cement Considerations

SC1, *A T III*, and *Th T III* were used as control cements. *SC1* was used when investigating fiber effects on stabilized soil properties. Cement type was not significant when evaluating fibers so long as the same cement was used for the entire test regimen and since sufficient *SC1* was available it was selected.

A T III and *Th T III* were used as controls for the majority of the report. *Type III* cements were chosen as controls since they are typically used in applications desiring high early strength. *SC1* and *SC2* were produced at the same facility as *A T III*, so it was used as the control for these cements. *SC5* and *SC6* were produced at the same facility as *Th T III*, so it was used as the control for these cements. A key cement consideration was that during the course of the research the Holcim Artesia plant was mothballed and ceased production for the foreseeable future. This was not expected at the onset of the research. The initial quantity of *SC1* and *SC2* obtained was sufficient for all research activities, however *A T III* was not. A very large sample of *SC1* and *SC2* was obtained since they were specialty products produced only once. Under normal circumstances another sample of *A T III* could have been obtained when the facility was producing cement with key properties (e.g. SO_3 content) that were the same as the original sample. This was unfortunately not the case for this research.

8.3 Soil Considerations

It was initially assumed that a sample taken from any location in 1 soil barrel was reasonably close to another sample taken from any location in any other barrel (i.e. that the soil was uniform). This assumption was based on index test results taken from different soil barrels. Five samples were taken in a stratified random configuration from each soil, and pertinent results are provided in Table 8.1; these values summarize data provided in Chapter 5. No 2 samples were taken from the same barrel.

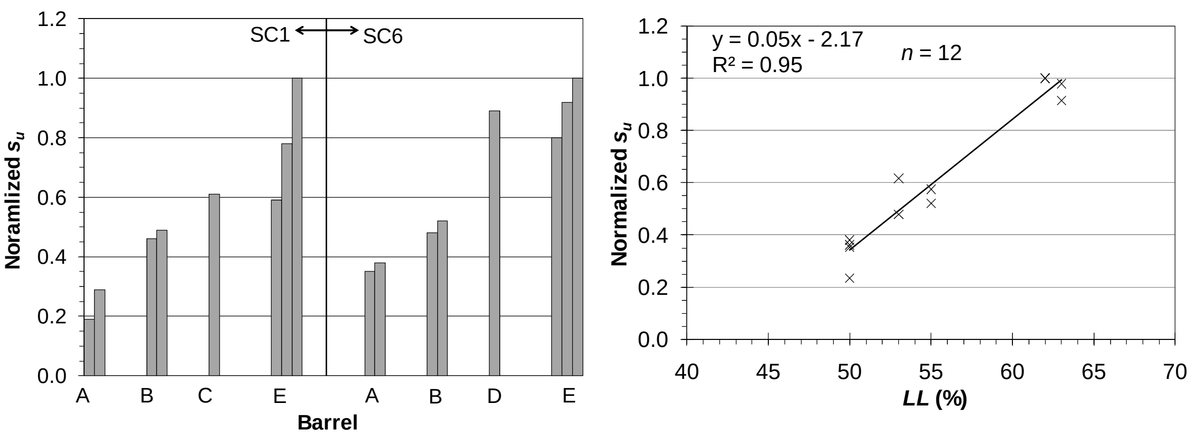
Table 8.1. Soil Index Properties from Five Initial Tests

	Soil 1		***Soil 2***		***Soil 3***	
Property	**Mean**	**Range**	**Mean**	**Range**	**Mean**	**Range**
LL	52	48 to 61	96	65 to 108	81	73 to 94
PI	35	31 to 44	56	37 to 62	49	43 to 63
% Clay	37	30 to 43	61	52 to 66	55	51 to 60
G_s	2.67	2.57 to 2.71	2.45	2.36 to 2.58	2.68	2.62 to 2.72
% Organics	4.2	3.6 to 4.6	24.2	11.4 to 31.3	10.6	10.2 to 11.4
USCS	CL to CH		CH to MH		CH to OH	
AASHTO	A-7-6 w/ GI 27 to 43		A-7-6 to A-8		A-7-5 to A-7-6	

Based on index properties, *Soil 1* was the least variable of the 3 soils. This result was expected since *Soil 1* was used for construction of a levee. *Soil 2* and *Soil 3* were disposal materials so their variability being relatively high was not surprising. Each of the 3 soils tested represent a somewhat distinct range of properties, with *Soil 1* having the most desirable characteristics for routine construction of load bearing objects such as levees or walls, followed by *Soil 3* and then *Soil 2*. For the type of soils of interest to this research, variability was felt to be unavoidable within such a large total sample (several metric tons); as a result, these materials were deemed suitable for testing where any sample taken from a barrel was treated the same as any other sample. This is not the ideal theoretical approach, but performing index tests multiple times within each barrel was not within time and budgetary allotments for the research.

The control cement property envelopes discussed in Section 8.4 were developed at this juncture in the research under the assumption that soil from any barrel was taken as the same. Control cement property envelopes used all available cement. Additional testing beyond control cement property envelopes revealed repeatability problems with *Soil 1* that were not explained by random variability. Laboratory and data reduction procedures were first investigated, and it was determined they were not the cause of the behaviors. Repeat testing of *Soil 1* using hand held gages (presented in Chapter 7) first observed that in some cases there was a considerable difference in measured strengths (even when tested during the same time by the same operators), whereas this did not occur for other soils. By the time the problematic behavior was observed in the test data, no material from any of the barrels used to develop the *Soil 1* control envelopes remained. This discovery resulted in a detailed and, in some cases, retroactive investigation into the behavior of *Soil 1* between barrels that is summarized in the remainder of this section and described in detail by Carruth (2011). *Soil 2* and *Soil 3* were not shown to vary consistently in strength between barrels, so all test data was analyzed as a whole for these soils.

Additional index properties and corresponding shear strengths were measured from known barrels at distinct locations. Figure 8.1 summarizes key test results and shows that *Soil 1* shear strength was a function of the barrel of origin that was strongly correlated to the liquid limit (*LL*). Shear strength was normalized by providing a value of 1.0 to the highest values. Eight barrels of *Soil 1* were used for *UC* strength measurement, and the anticipated properties of each barrel were compared to each other to group together barrels of similar performance. This assessment used the data in Figure 8.1 alongside engineering judgment; the approach is far from ideal but is believed to be the best available under the circumstances.



(a) Soil 1, (5, 100) 24 hr Normalized Results *(b) Liquid Limit to Normalized Strength*

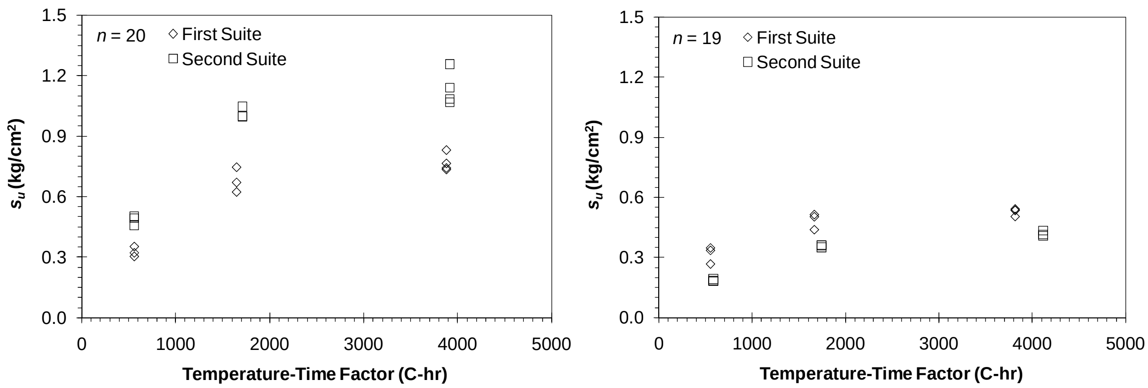
Figure 8.1. Normalized Test Results Showing Soil 1 Variability

Three groups were developed as a result of the investigation and are referred to as *Group 1*, *Group 2*, and *Group 3*. Where pertinent, the *Soil 1* group is identified in the rest of the report. A summary of each group follows.

- *Group 1*: The strongest of the *Soil 1* groups consisting of barrels D, E, F, and G
- *Group 2*: The intermediate strength *Soil 1* group consisting of barrels C and H
- *Group 3*: The weakest of the *Soil 1* groups consisting of barrels A and B

8.4 Development of Control Cement Property Envelopes

Some amount of variability is present in any testing program, and provided reasonable variability estimates are not available in literature, the amount of variability is not known until testing commences. The latter was true in this study. Initially, 1 suite was tested with each cement, and to get an assessment of test variability, a second suite was performed for a select number of combinations. Ideally, 1 suite could have been performed for each soil-condition-cement combination of interest and used for comparison, but test data showed the results were not repeatable enough to make this a reasonable approach. Figure 8.2 provides an example where results were repeatable between suites and another example where they were not. The observation that not all cases were going to be repeatable coupled with the observed soil index property variability shown in Table 8.1 made it apparent that an envelope of strengths from the control cements would be needed to provide any realistic assessment of properties of other cements from the same production facility mixed with the same soil type. The data did, however, indicate that properties should vary within a reasonable range for a given soil-condition-cement combination.



a) Example: Not Repeatable *b) Example: Repeatable*

Figure 8.2. Repeatability of Control Suites

Availability of control cements was considered in determining how many replicate suites could be performed per soil-condition-cement combination. Ideally, 3 replicate suites would have been performed for all combinations, but insufficient *A T III* was available to do so; since it was desired to treat all control cements the same, a compromise was made for some combinations. Table 8.2 provides replicate testing performed for each combination (over 200 tests required per cement type). *Soil 3* was tested 3 times since its index properties were intermediate relative to *Soil 1* and *Soil 2*.

Table 8.2. Control Cement Suites Conducted

	Soil		
Cement	**1**	**2**	**3**
A T III	2	2	3
Th T III	2	2	3
SC1	2	2	3

The concept for the control suites is provided in Figure 8.3. Once all data was collected, the data for each soil-condition-cement combination was plotted, and a logarithmic trend line was fit through the data, shown as the solid line in Figure 8.3. The trend line was multiplied by 2 constants which are shown in the upper right corner of Figure 8.3 to create a data envelope. The data envelope is shown as the 2 dashed lines in Figure 8.3. The envelopes are subjective and do not represent a statistical approach. The constants were adjusted until the envelope effectively captured nearly all of the data while occasionally excluding a small amount of data. For example, the logarithmic trend line equation in Figure 8.3 is y = 0.70 ln(x) - 3.10, and at 1000 C-hr strength it was calculated to be 1.74 kg/cm^2 and represents the trend line. The upper and lower data bounds at 1000 C-hr were 1.25(1.74 kg/cm^2) or 2.18 kg/cm^2 and 0.80(1.74 kg/cm^2) or 1.39 kg/cm^2, respectively.

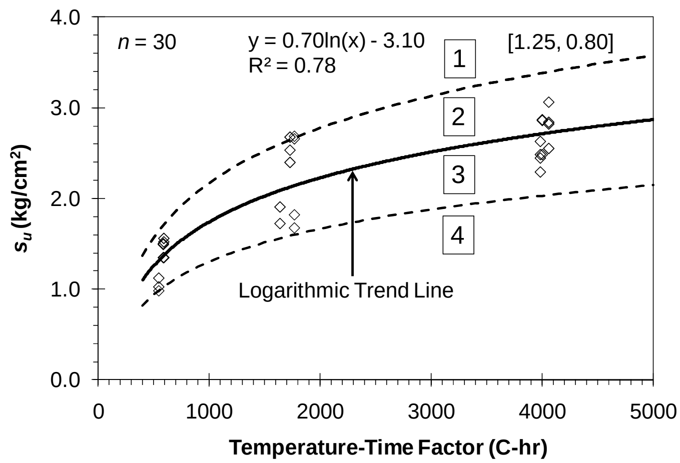


Figure 8.3. Concept of Control Suite Zones

To provide a reasonable method of evaluating cements relative to the controls, 4 zones were developed as seen in Figure 8.3. They were, in general, interpreted as follows.

- Zone 1: Specialty cement considered stronger than control
- Zone 2: Specialty cement could be stronger than control
- Zone 3: Specialty cement could be weaker than control
- Zone 4: Specialty cement considered weaker than control

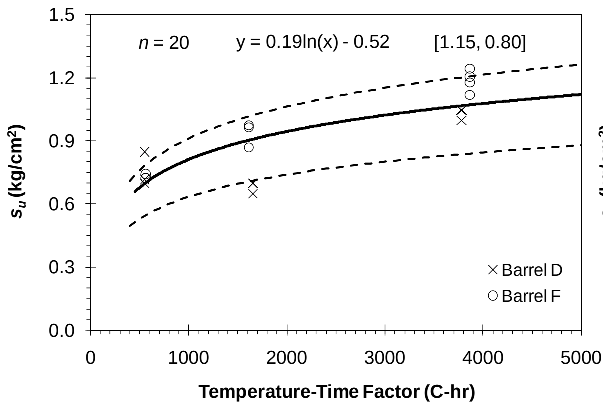
Figures 8.4 through 8.9 plot the envelopes for all control cements. The individual data points and total number of data points are shown in the plots to provide visual indication of how they were developed. The envelopes were used for analysis in absence of the data points for the remainder of the analysis. Pertinent trend line data were also shown on the envelopes, including the 2 constants shown in brackets used to create the envelopes. All *Soil 1* control cements had at least 1 suite which was tested with a barrel from *Group 1*. The individual barrels were shown to provide additional clarity for *Soil 1*. Accordingly, these control envelopes were believed to provide an upper bound that was as strong as possible for the family of soils tested and provided a reasonable strength comparison.

A T III and *Th T III* control cements were compared by Carruth (2011) to assess their relative strength properties. The inherent effect of the raw materials used to produce cement at any given facility can be significant. A summary of the comparison as it pertains to this report is summarized in Table 8.3.

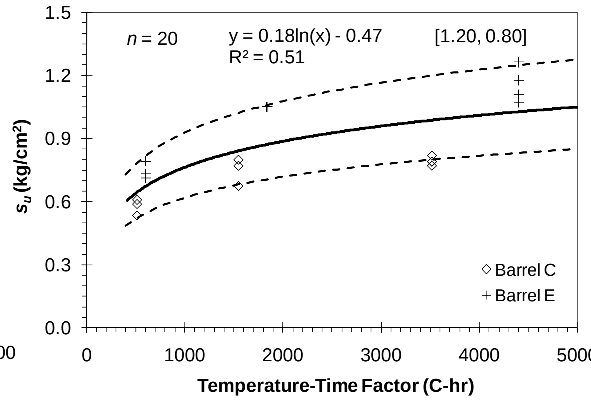
Table 8.3. Summary of Optimum Control Cement

	Condition and Extent		
Soil	**(5, 100)**	**(10, 100)**	**(15, 233)**
1	*A T III (slight)*	*A T III (noticeably)*	*Th T III (noticeably)*
2	*A T III (slight)*	*Th T III (noticeably)*	*Th T III (noticeably)*
3	*Th T III (slight)*	*Th T III (slight)*	*A T III (slight)*

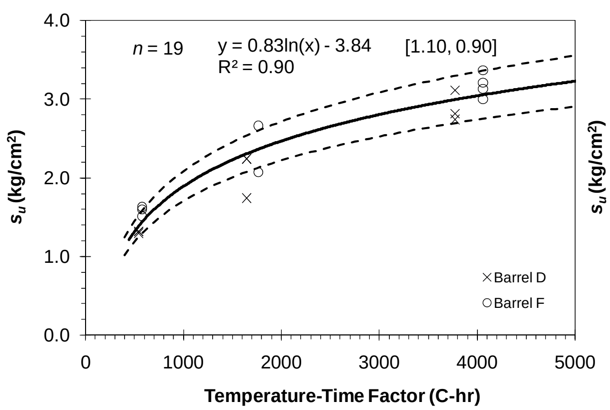
Note: Relative performance was not drastically different in several instances.



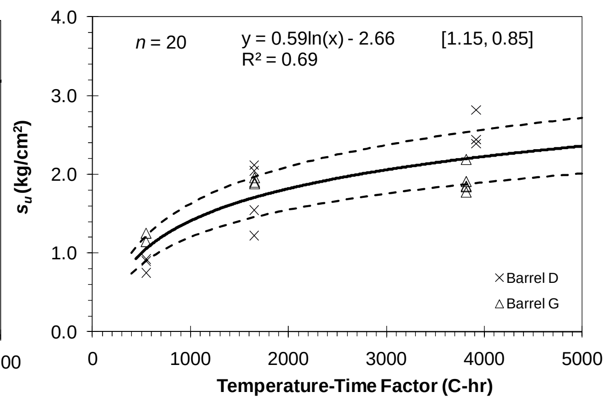
a) (5,100)-A T III



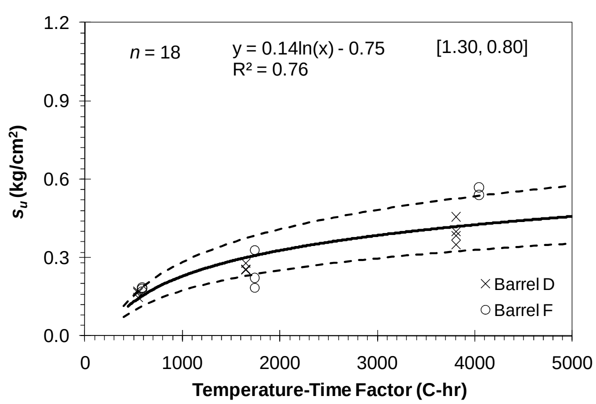
b) (5,100)-Th T III



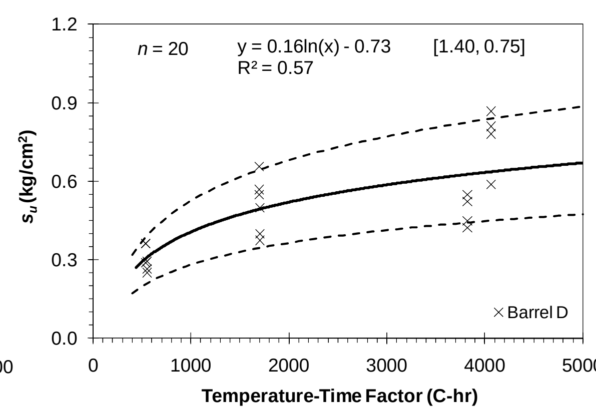
c) (10,100)-A T III



d) (10,100)-Th T III

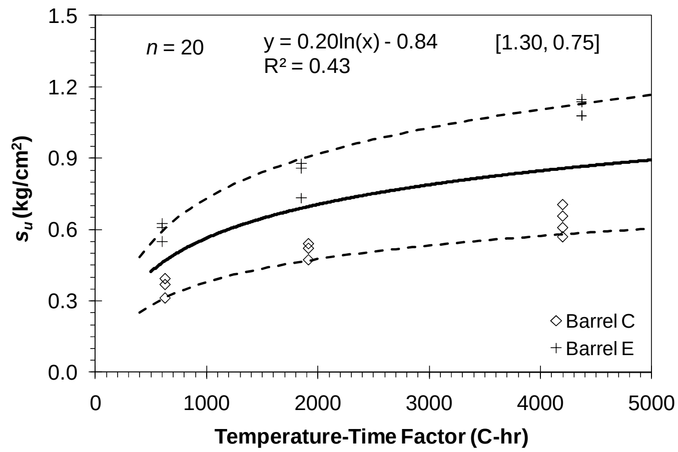


e) (15,233)-A T III

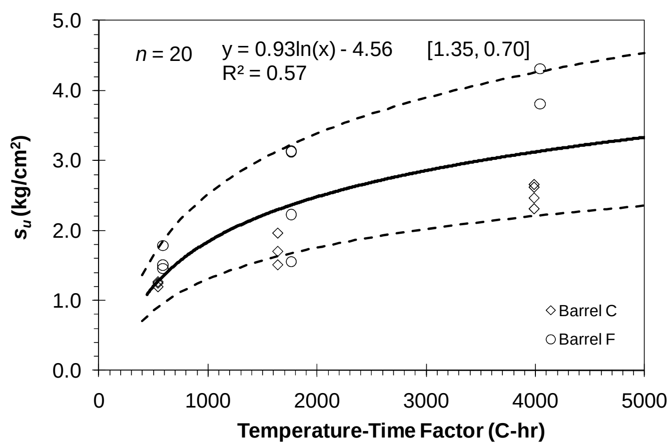


f) (15,233)-Th T III

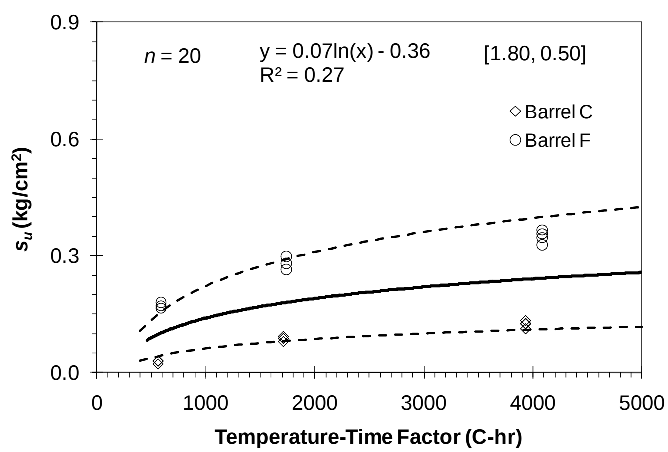
Figure 8.4. Control Cement Plots-Soil 1-A T III and Th T III



a) (5,100)

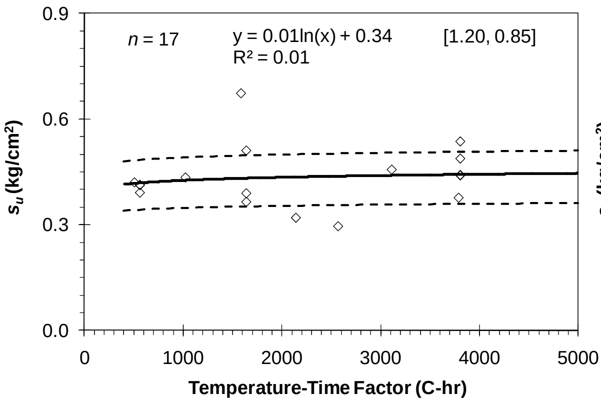


b) (10,100)

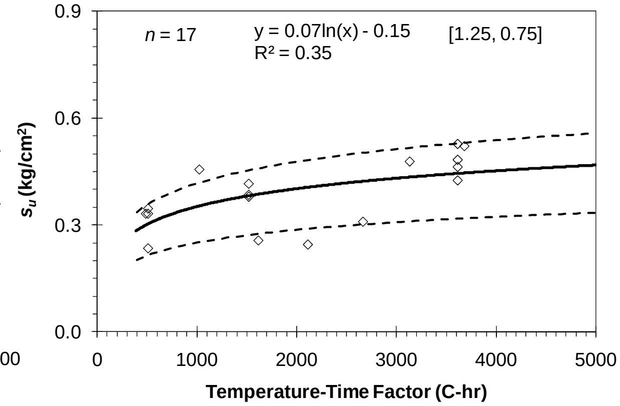


c) (15,233)

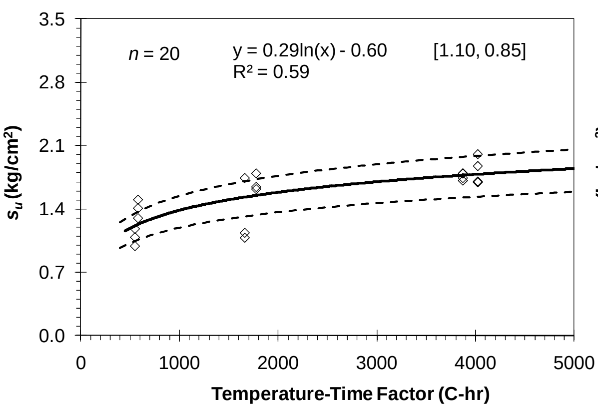
Figure 8.5. Control Cement Plots-Soil 1-SC1



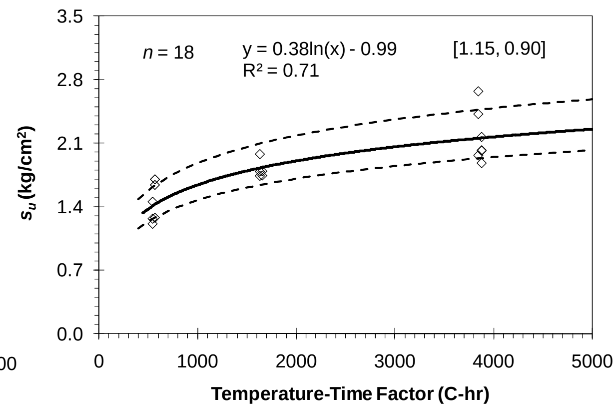
a) A T III-(5,100)



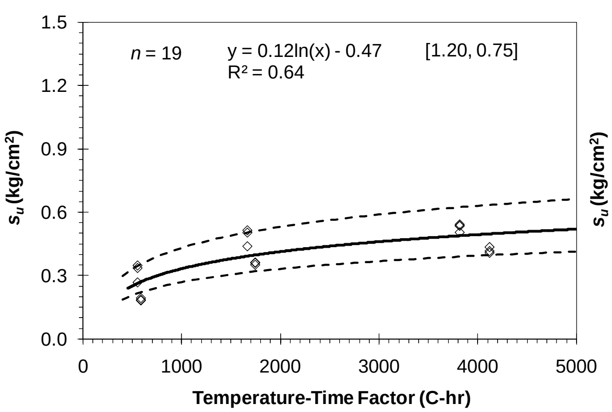
b) Th T III-(5,100)



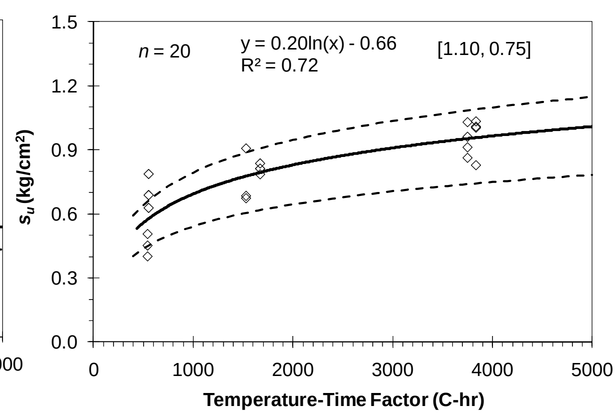
c) A T III-(10,100)



d) Th T III-(10,100)

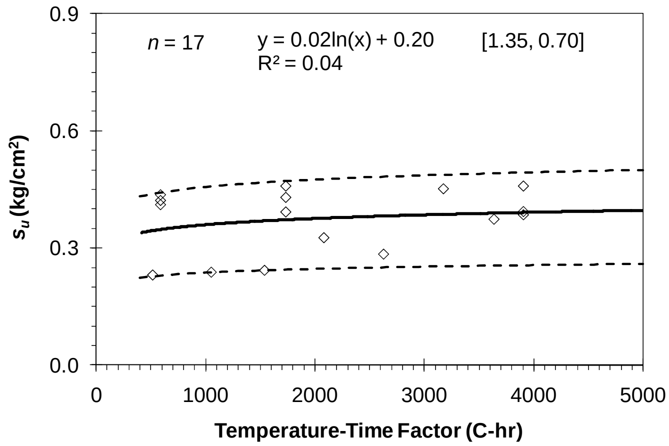


e) A T III-(15,233)

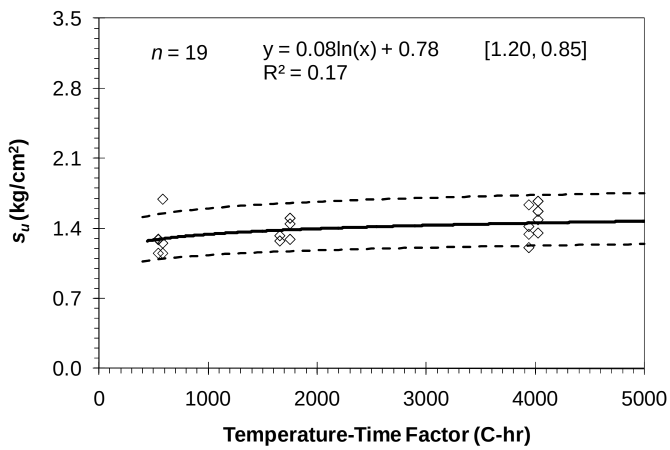


f) Th T III-(15,233)

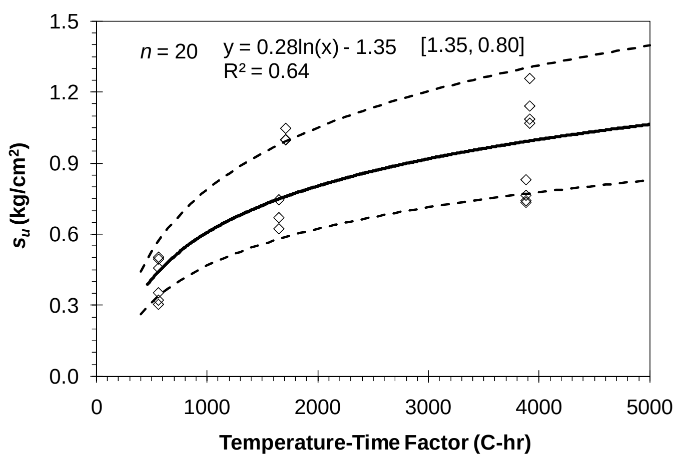
Figure 8.6. Control Cement Plots-Soil 2-A T III and Th T III



a) (5,100)

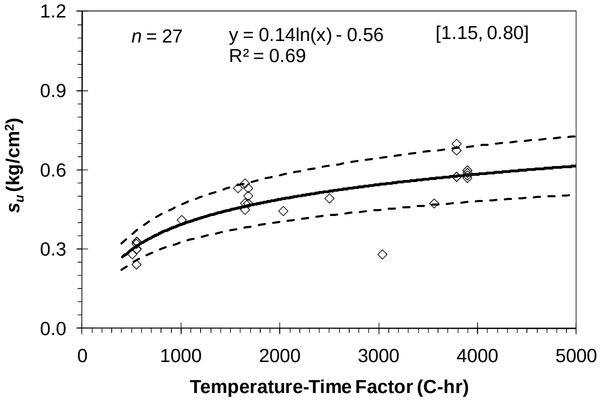


b) (10,100)

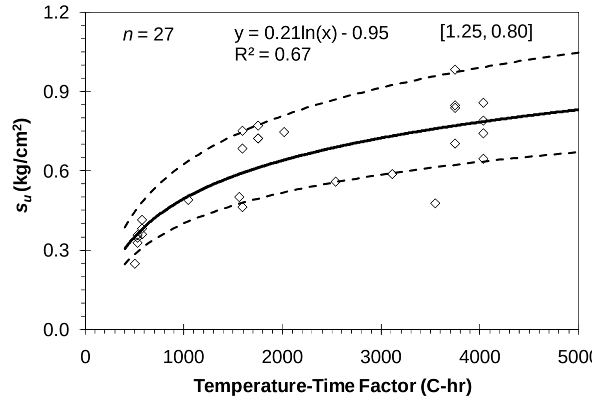


c) (15,233)

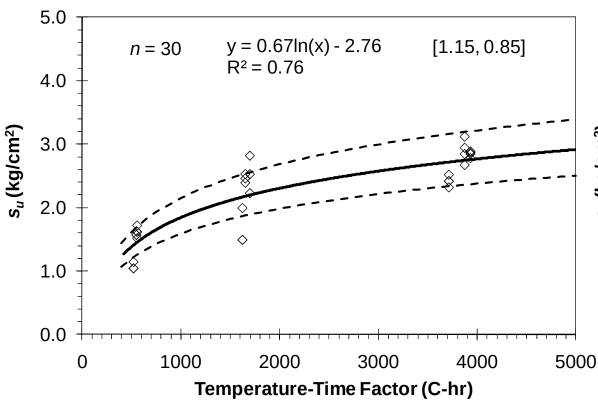
Figure 8.7. Control Cement Plots-Soil 2-SC1



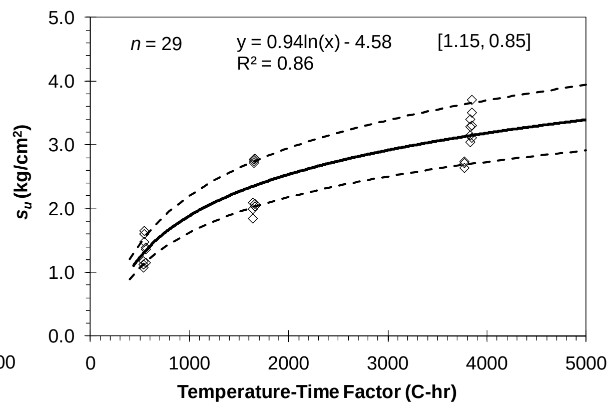
a) A T III-(5,100)



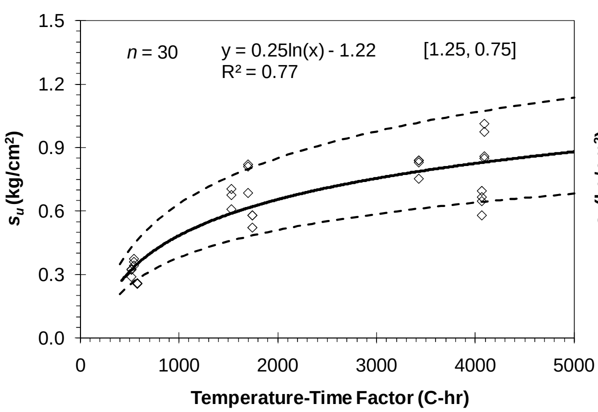
b) Th T III-(5,100)



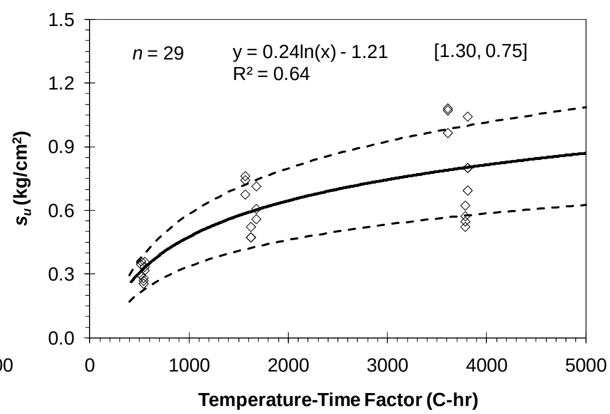
c) A T III-(10,100)



d) Th T III-(10,100)

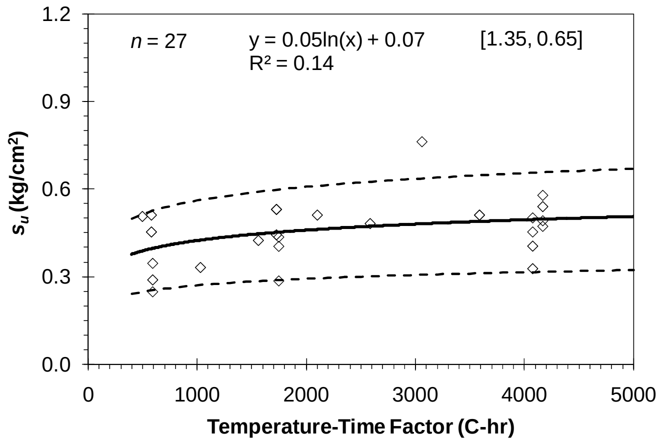


e) A T III-(15,233)

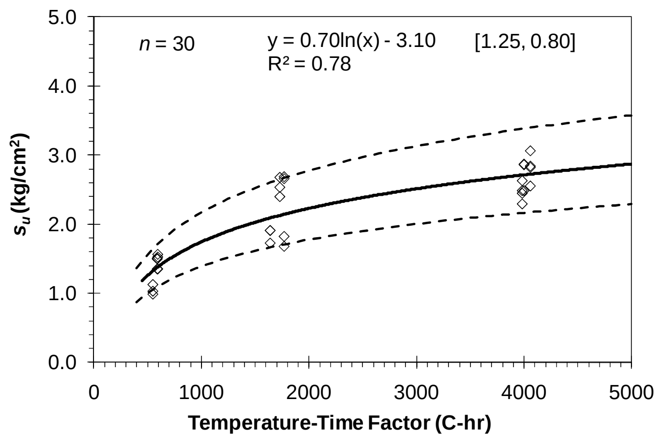


f) Th T III-(15,233)

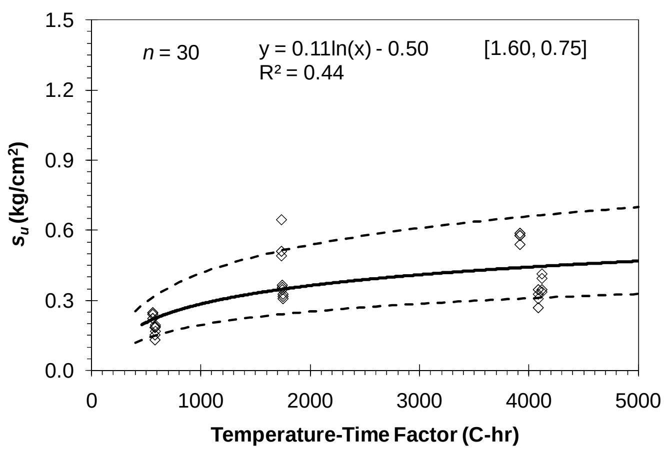
Figure 8.8. Control Cement Plots-Soil 3-A T III and Th T III



a) (5,100)



b) (10,100)



c) (15,233)

Figure 8.9. Control Cement Plots-Soil 3-SC1

CHAPTER 9 – PORTLAND CEMENTS TEST RESULTS

9.1 Portland Cements Test Results Overview

This chapter presents *UC* test results of specimens cured 24 to 168 hours and stabilized with portland cements. Data collected with portland cement stabilized specimens cured less than 24 hours is provided in Chapter 10 as a comparison to calcium sulfoaluminate cements. The majority of the testing focused on specialty grind portland cements, while *Type I* and blended portland cements were tested to a lesser extent. The sets and suites conducted resulted in approximately 1,100 *UC* measured strengths for comparison to control suites. Analysis was performed and presented according to cement production facility.

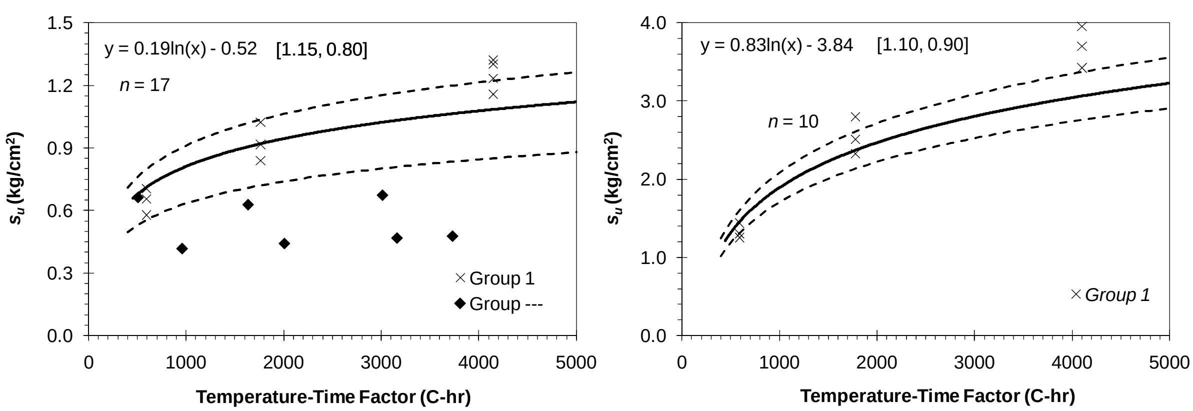
9.2 Shear Strength Analysis of Cements Produced at Holcim Artesia

A T III served as the control cement for the analysis in this section. Other cements produced at Artesia were plotted alongside the control envelopes developed in Chapter 8. The logarithmic trend line equation, R^2 value, and upper/lower bound constants are shown for the control cement, while the number of data points (n) on the figures refers to the number of data points of the cement(s) being compared to the control.

9.2.1 *Soil 1* Shear Strength Analysis of Artesia Cements

9.2.1.1 *Soil 1* Shear Strength Analysis of *Type I* Artesia Cements

Figure 9.1 plots test results of *A T I*. *Group 1* results showed *A T I* to be on par with *A T III* and in some cases to exceed the upper band. One data point at (10, 100) exceeded the upper control band slightly over 20%, and at (10, 100) 4 of the 10 tests were above the upper control band. At (5, 100) 1 of the suites tested was from an unknown *Soil 1* group, and the strength from this suite was well below the control band. The suite using *Group 1* had 2 data points exceed the control band. The data suggests that the *Type I* cement was on par with (perhaps even slightly better than provided one (5, 100) suite is not considered) the *Type III* cement in *Soil 1*, though conclusive statements would require more testing.

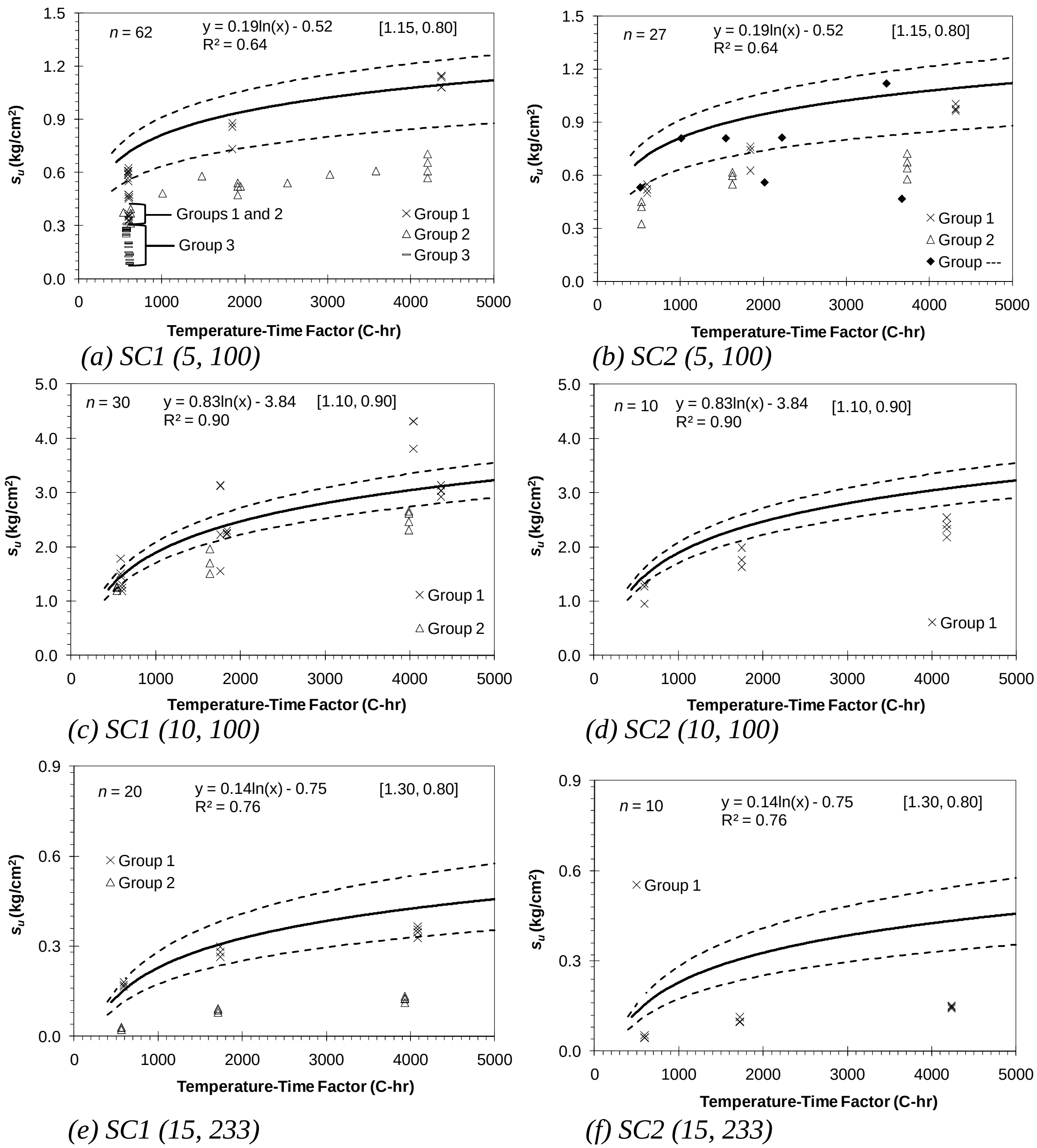


(a) (5, 100) *(b) (10, 100)*

Figure 9.1. Soil 1 Shear Strength Test Results for A T I vs. A T III Control

9.2.1.2 *Soil 1* Shear Strength Analysis of Specialty Grind Artesia Cements

Figure 9.2 plots specialty cement *Soil 1* results. All data fell below the control envelope lower bound with exception of *Group 1* data. *Group 1* data appeared weaker than the control at early intervals but strength improved to rival the control at later intervals. *Groups 2* and *3* data fell below the lower control bound, which is not necessarily informative. *SC1* appeared slightly weaker than the control at (5,100) though the evidence is not conclusive.



(a) SC1 (5, 100) *(b) SC2 (5, 100)*

(c) SC1 (10, 100) *(d) SC2 (10, 100)*

(e) SC1 (15, 233) *(f) SC2 (15, 233)*

Figure 9.2. Results of Artesia Specialty Cements in Soil 1 vs. A T III Control

SC2 was considered weaker than the control at (5,100) below 1000 C-hr, as all data fell below the lower bound of the control envelope. The data above 1000 C-hr mostly fell below the lower control bound. *Group 1* appeared weaker than the control, and only 1 data point fell in Zone 2 of Figure 8.3. *SC2* was thought to be weaker than the control at (5,100) with a moderate but not conclusive amount of evidence.

At (10,100), *SC1* showed similar strength to the control below 1000 C-hr for all groups tested. *Group 2* was weaker than the control at later test times. *Group 1* results above 1000 C-hr were somewhat mixed but did exceed the upper control band in a few instances and was, in one instance, below the lower control bound. Overall, *SC1* was considered to be at to moderately stronger than the control, as data from *Group 1* were near the trend line to above the upper bound. *SC2* was considered weaker than the control at (10,100) as all data fell near or below the lower control band, and all soil tested came from *Group 1*.

Group 1 specimens at (15, 233) stabilized with *SC1* were within the upper and lower control bounds. *Group 2* was significantly weaker than the control, which is not necessarily meaningful. *SC1* appeared to be at to slightly weaker than the control, though the evidence was limited. *SC2* at (15, 233) was weaker than the control at all testing times, and the material came from *Group 1*.

9.2.1.3 *Soil 1* Shear Strength Analysis of Blended Artesia Cements

Figure 9.3 plots test results where *A T III* and *SC2* were blended (5% of each cement) and compared to the *A T III* control at (10, 100) in *Soil 1*. Results are not conclusive. At the intermediate curing interval, test data exceeds the upper control band, while at the later curing interval data is near the bottom control band. More testing would be needed to make definitive statements, but there did not appear to be a decisive advantage gained by replacing 5% of the *A T III* cement with *SC2*.

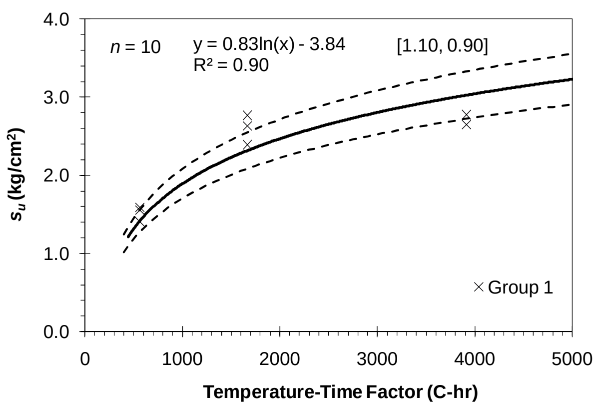
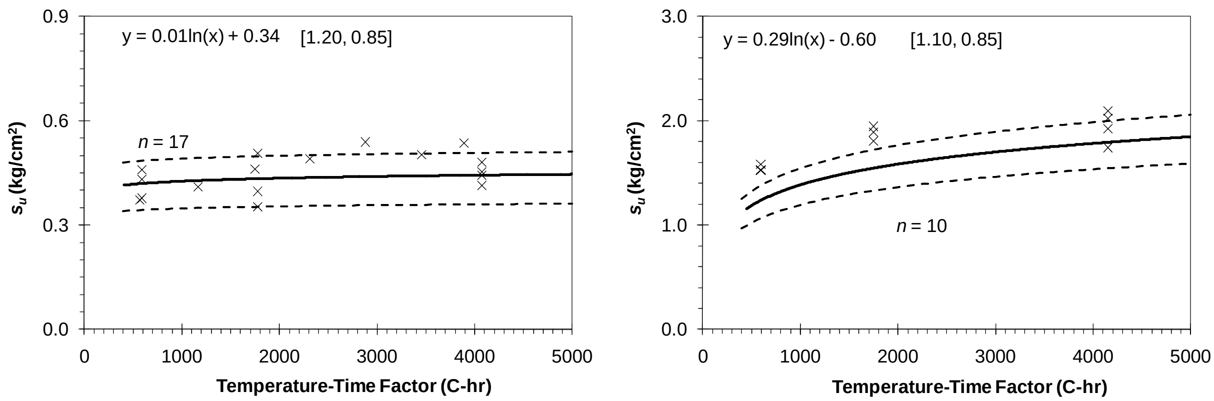


Figure 9.3. Results of Blending 5% A T III and 5% SC2 in Soil 1 with 100% Moisture

9.2.2 *Soil 2* Shear Strength Analysis of Artesia Cements

9.2.2.1 *Soil 2* Shear Strength Analysis of *Type I* Artesia Cements

Figure 9.4 plots results of *A T I* in *Soil 2*, which was on par with *A T III* and in some cases exceeded the upper control band. At (5, 100) 2 data points were above the control band but barely. At (10, 100) 7 of the data points were above the upper control band. The data suggests the *Type I* cement was on par with (perhaps even slightly better than) the *Type III* cement in *Soil 2*, though conclusive statements would require more testing.



(a) (5, 100) *(b) (10, 100)*

Figure 9.4. Soil 2 Shear Strength Test Results for A T I vs. A T III Control

9.2.2.2 *Soil 2* Shear Strength Analysis of Specialty Grind Artesia Cements

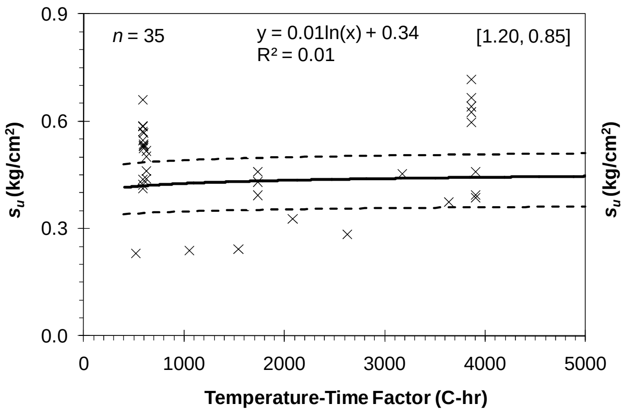
Figure 9.5 plots test results of *Artesia* cements and *Soil 2*. Specimens stabilized with *SC1* at (5,100) had several data points exceeding the upper control band below 1000 C-hr and near 4000 C-hr. Several data points also fell within the control band, while 5 data points fell below the lower bound of the control band. These 5 points came from a single *Protocol 1* suite, which is noteworthy since they are noticeabally lower than the rest of the data that came from other suites and sets. Accordingly, *SC1* was considered at to moderately stronger than the control.

SC2 appeared to be at to stronger than the control at (5,100). Of the 27 *Soil 2* specimens stabilized with *SC2*, 12 had strengths in Zone 1, and the 15 remaining shear strength data points fell within the control envelope. Of those 15, 7 fell above the control trend line (Zone 2).

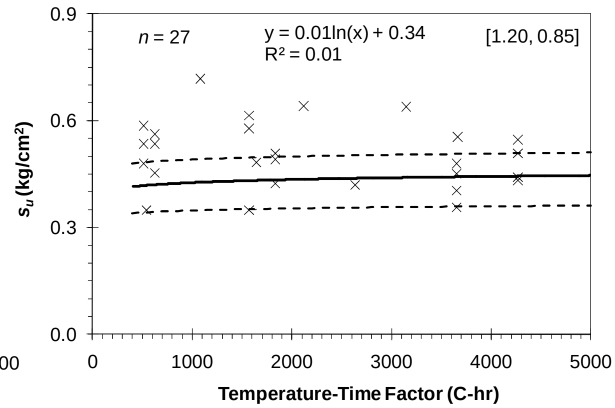
For the (10,100) condition, *SC1* appeared slightly weaker than the control. The majority of the data fell below the trend line, and much of the data fell near or below the lower bound of the control envelope. SC1 stabilized specimens did not gain strength after 1000 C-hr.

SC2 was stronger than the control below 2000 C-hr. All but 1 of the data points fell at or above the upper bound of the control envelope below 2000 C-hr. Above 3000 C-hr, *SC2* results were no different than the control. Most of the data fell near the trend line and within the control envelope, with 1 reading just above and 1 just below the control envelope.

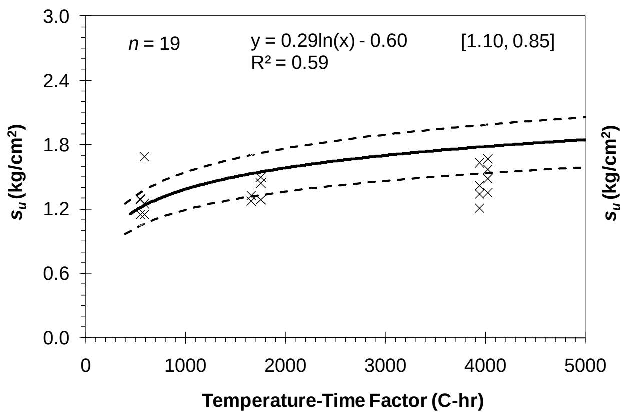
At (15,233), *SC1* was considerably stronger than the control at later test times and moderately stronger than the control at early test times. *SC2* was moderately stronger than the control at (15, 233). *SC2* did not improve strength as much as *SC1* at (15, 233).



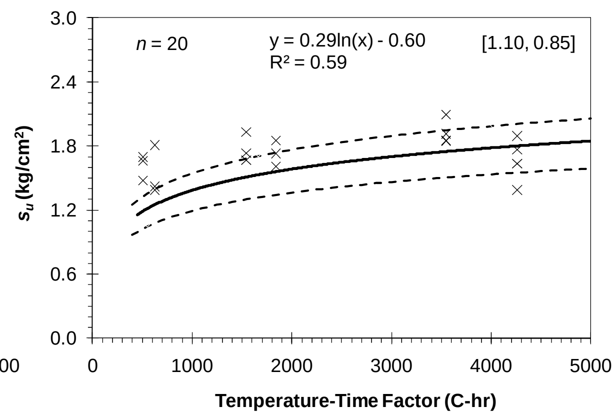
a) (5,100)-SC1



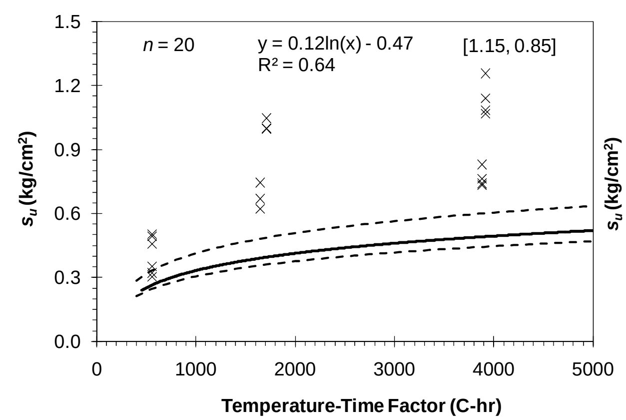
b) (5,100)-SC2



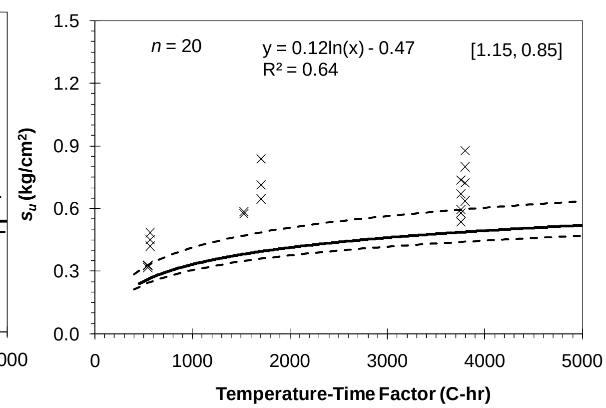
c) (10,100)-SC1



d) (10,100)-SC2



e) (15,233)-SC1



f) (15,233)-SC2

Figure 9.5. Results of Artesia Specialty Cements in Soil 2 vs. A T III Control

9.2.2.3 *Soil 2* Shear Strength Analysis of Blended Artesia Cements

Figure 9.6 plots test results where *A T III* and *SC2* were blended (5% of each cement) and compared to the *A T III* control at (10, 100) in *Soil 2*. The results are not conclusive. The data bounds the control envelope with data slightly above and below the upper and lower control bands. More testing would be needed to make definitive statements, but there did not appear to be a decisive advantage gained by replacing 5% of the *A T III* cement with *SC2*.

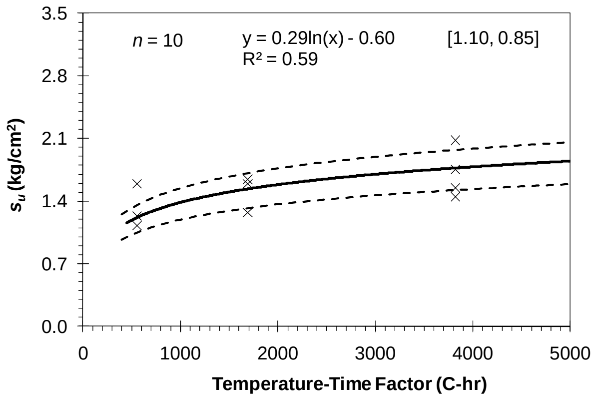
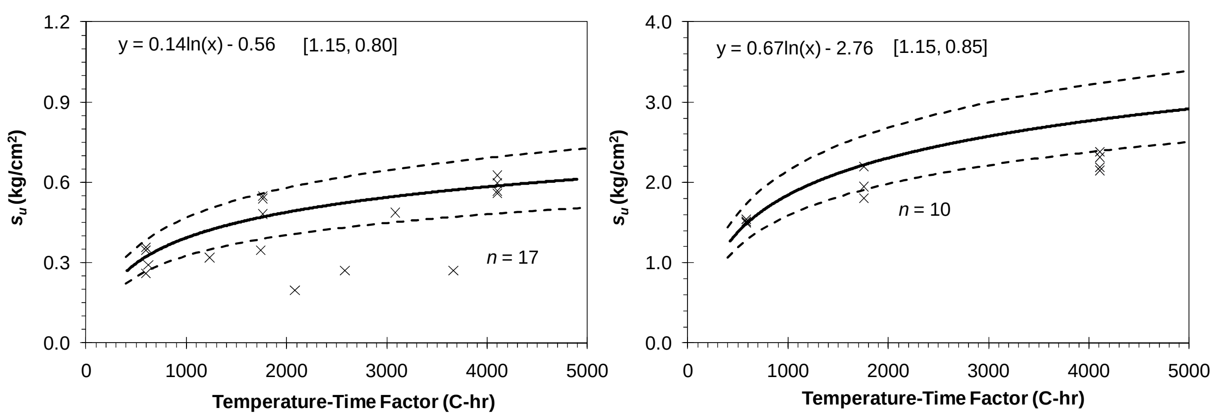


Figure 9.6. Results of Blending 5% A T III and 5% SC2 in Soil 2 with 100% Moisture

9.2.3 *Soil 3* Shear Strength Analysis of Artesia Cements

9.2.3.1 *Soil 3* Shear Strength Analysis of *Type I* Artesia Cements

Figure 9.7 plots results of *A T I* in *Soil 3*, which was on par with *A T III* in some instances and less than *A T III* in other instances. Some of the test data at (5, 100) was well below the control band while other data was within the control band. At (10, 100), shear strength at approximately 4,000 C-hr was slightly below the control band. The data suggests that the *Type I* cement was on par with to less than the *Type III* cement in *Soil 3*, though conclusive statements would require more testing.

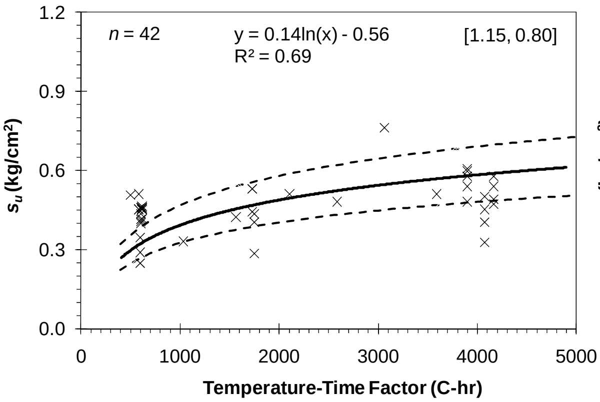


(e) Soil 3 (5, 100) *(f) Soil 3 (10, 100)*

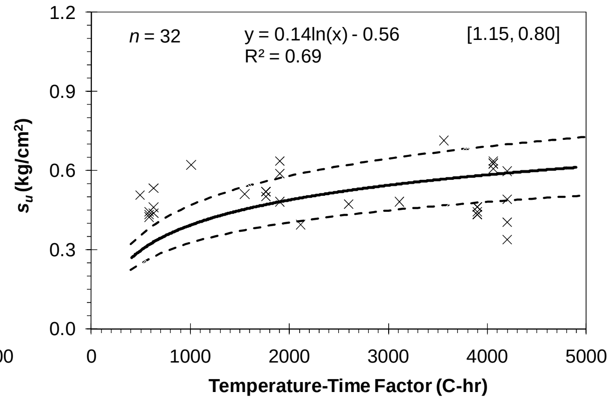
Figure 9.7. Soil 3 Shear Strength Test Results for A T I vs. A T III Control

9.2.3.2 *Soil 3* Shear Strength Analysis of Specialty Grind Artesia Cements

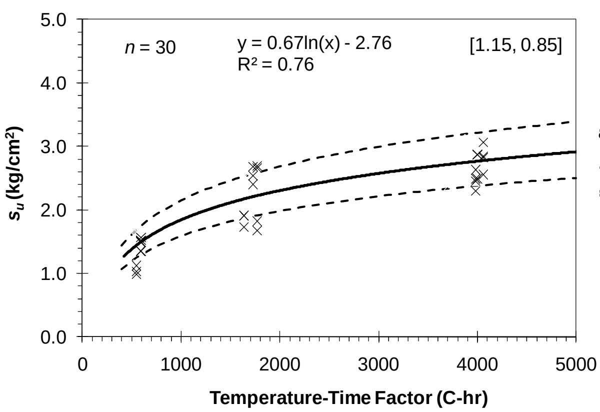
Figure 9.8 plots test results of Artesia cements and *Soil 3*. *SC1* at (5, 100) mostly outperformed the control at approximately 600 C-hr, as the majority of the data fell above the upper bound of the control envelope. *SC1* was mostly weaker than the control above 1000 C-hr, as much of the data fell below the trend line but within the control envelope.



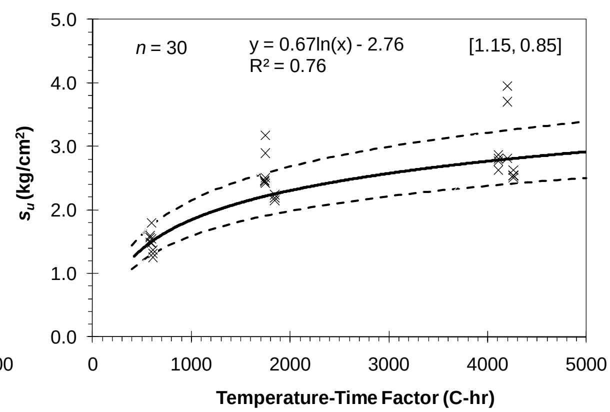
a) (5,100)-SC1



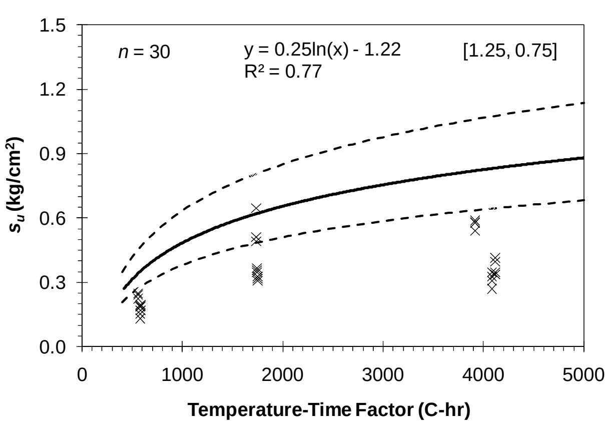
b) (5,100)-SC2



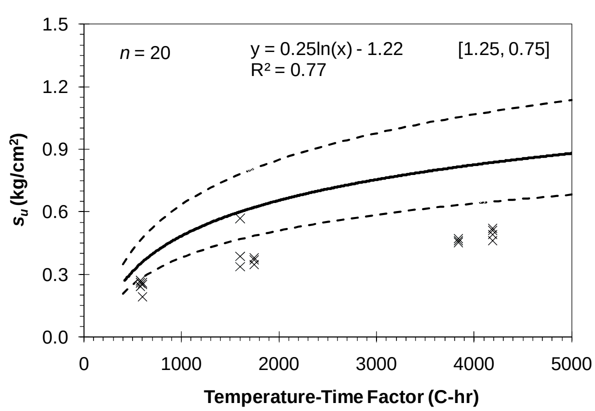
c) (10,100)-SC1



d) (10,100)-SC2



e) (15,233)-SC1



f) (15,233)-SC2

Figure 9.8. Results of Artesia Specialty Cements in Soil 3 vs. A T III Control

SC2 at (5, 100) was stronger than the control until 1000 C-hr, and at to stronger than the control between 1000 to 2000 C-hr. Above 2000 C-hr, *SC2* was considered comparable to slightly weaker than the control. *SC1* and *SC2* appeared to gain strength faster than the control up to approximately 1000 C-hr but also appeared to be slightly weaker by approximately 4000 C-hr.

SC1 at (10, 100) exhibited similar strengths to that of the control at all testing times. *SC2* at (10, 100) displayed strengths that were similar to the control cement in most instances, but moderately higher in a few instances. Most of the data points at all testing times were located either near the trend line or on either side of the trend line, but within the control envelope. At the (15, 233) condition, *SC1* appeared weaker than the control. Nearly all the shear strength data points fell below the lower bound of the control envelope. *SC2* also appeared to be weaker than the control at (15, 233) since all the data but 1 data point fell in Zone 4 or below the lower bound of the control envelope.

9.2.3.3 *Soil 3* Shear Strength Analysis of Blended Artesia Cements

Figure 9.9 plots test results where *A T III* and *SC2* were blended (5% of each cement) and compared to the *A T III* control at (10, 100) in *Soil 3*. Results were inconclusive. Shear strength is at to slightly below the control envelope lower bound at early and intermediate *TTF* values, yet is at to slightly above the control envelope upper bound at later *TTF* values. More testing would be needed to make definitive statements, but there did not appear to be a decisive advantage gained by replacing 5% of the *A T III* cement with *SC2*.

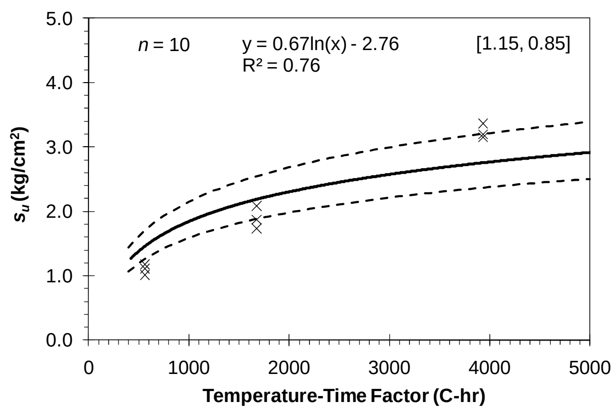


Figure 9.9. Results of Blending 5% A T III and 5% SC2 in Soil 3 with 100% Moisture

9.2.4 Discussion of Artesia Produced Cement Shear Strengths

Limited testing with *Type I* cement suggested there could be potential in investigating its performance in more detail. None of the data suggested *Type I* cement was clearly a better performer than *Type III* cement, though *Type I* cement was competitive with *Type III* cement in many cases. In *Soil 1* and *Soil 2 A T I* was at least on par with *A T III*, and at (10, 100) the data exceeded the upper control band in multiple instances. In *Soil 3*, *A T I* was on par with to weaker than *Type III* cement.

Table 9.1 contains a summary of the *SC1* results obtained from comparisons with the *A T III* control. Only at the (10,100) condition did *SC1* outperform the control for *Soil 1*, and an increase in strength was only observed in some instances. For *Soil 2* at (5,100), *SC1* was moderately stronger than the control on some occasions, while at (10,100) *SC1* was slightly weaker. At (15,233) a clear strength increase was observed. *Soil 3* at (5,100) showed mixed results, as *SC1* was slightly weaker than the control in some cases, while slightly stronger in others. *SC1* strengths were similar to the control at (10,100) and weaker at (15,233).

Table 9.1. Summary of *SC1* Findings Relative to *A T III* Control

Type	*Soil 1*	*Soil 2*	*Soil 3*
(5,100)	At to slightly weaker	At to moderately stronger	Slightly weaker to slightly stronger
(10,100)	At to moderately stronger	Slightly weaker	Same strength
(15,233)	At to slightly weaker	Moderately to considerably stronger	Weaker

Table 9.2 summarizes *SC2* behavior relative to *A T III*. *Soil 1* data was definitively weaker at all conditions. *Soil 2* specimens were considered similar and in some cases stronger than the control at (5,100). At (10,100) and (15,233), *SC2* was similar to moderately stronger than the control. *Soil 3* specimens stabilized with *SC2* exhibited strengths ranging from slightly weaker to slightly stronger than the control at (5,100). *Soil 3* at (10,100) appeared to have the same strength as the control for most instances, but also outperformed the control in a few cases. *Soil 3* at (15,233) was considered weaker than the control.

Table 9.2. Summary of *SC2* Findings Relative to *A T III* Control

Type	*Soil 1*	*Soil 2*	*Soil 3*
(5,100)	Weaker	At to stronger	Slightly weaker to slightly stronger
(10,100)	Weaker	At to moderately stronger	Same strength typically, stronger occasionally
(15,233)	Considerably weaker	At to moderately stronger	Weaker

Soil 2 appeared to benefit most from the use of specialty cements, as strengths were at to stronger than the control in all cases, except for *SC1* at (10,100). The result is somewhat surprising considering the high amount of organics present in *Soil 2*. Organics are believed to inhibit hydration in many instances, so the benefits of increased Blaine Fineness or reduced SO_3 might be expected to be somewhat masked. *SC1* and *SC2* both had reduced SO_3 contents, but *SC2* had an increased Blaine Fineness. The increased fineness appeared to have an effect on *Soil 2*, since *SC2* appeared to slightly outperform *SC1*, particularly at (10,100). *Soil 2* exhibited a very thick consistency at 100% moisture due to its high organic content, so it is possible that a finer cement was needed to react with the free water that was available. Reduced SO_3 appeared to have a considerable effect on *Soil 2* at (15,233), while cement fineness appeared to reduce strength since *SC1* was moderately to considerably stronger than the control and appeared to outperform *SC2*. Plenty of free water was available for reaction with cement for *Soil 2* at 233% moisture content.

Soil 2 strength increased with cement fineness at (10,100) while *Soil 1* and *Soil 3* strengths did not. At (10, 100), *Soil 1* showed similar to moderately higher strengths than the control for *SC1* and lower strengths for *SC2*, indicating a decrease in strength with increased fineness. *Soil 3* specimens showed mixed results at (5,100) and results similar to the control at (10, 100) for both *SC1* and *SC2*, so reduced SO_3 appeared to not have much of an effect on strength for *Soil 3* at these conditions. At (15, 233), reduced SO_3 appeared to produce weaker specimens for both *Soil 1* and *Soil 3* (except for *Soil 1* stabilized with *SC1*) where strengths were similar to weaker than the control. Clare and Farrar (1956) suggested that an optimum SO_3 content should be determined when stabilization of soil is needed; so, it is possible that testing of another range of SO_3 contents could arrive at an SO_3 content which could provide higher strengths for *Soil 1* and *Soil 3* with the Artesia cement source.

Limited blended cement test results were not conclusive in any of the soils. In all cases, the results were essentially the same as the control band. Due to the nature of the materials blended, Blaine Fineness and SO_3 were changed when blending.

9.3 Shear Strength Analysis of Cements Produced at Holcim Theodore

Th T III served as the control cement for the analysis in this section. Other cements produced at Theodore were plotted alongside the control envelopes developed in Chapter 8. The logarithmic trend line equation, R^2 value, and upper/lower bound constants are shown for the control cement, while the number of data points (*n*) on the figures refers to the number of data points of the cement being compared to the control.

9.3.1 *Soil 1* Shear Strength Analysis of Theodore Cements

9.3.1.1 *Soil 1* Shear Strength Analysis of *Type I/II* Theodore Cements

Figure 9.10 plots shear strength test results of *Th T I/II* with respect to the *Th T III* control band in *Soil 1*. Test results were within the lower portion of the control band but never above the trendline. Approximately half the data was below the lower bound of the control band. The limited data indicates no advantage of *Th T I/II* versus the control cement; more testing would be needed to provide definitive statements.

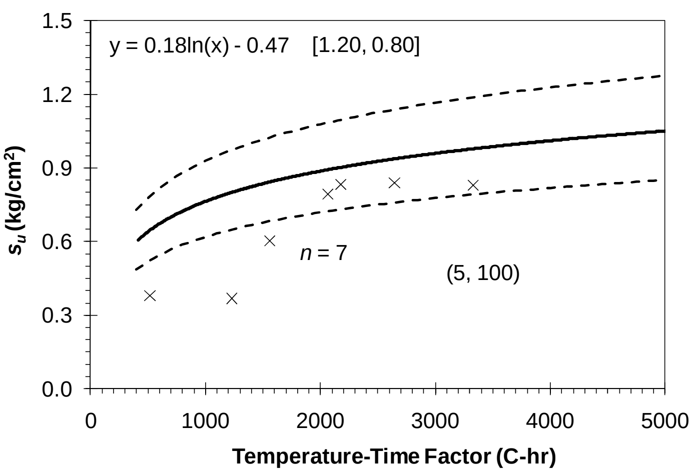


Figure 9.10. Soil 1 Shear Strength Test Results for Th T I/II vs. Th T III Control

9.3.1.2 *Soil 1* Shear Strength Analysis of Specialty Grind Theodore Cements

Figure 9.11 plots test results of *Theodore* cements and *Soil 1*. *SC5* appeared to be slightly stronger than the control below 1000 C-hr, with half of the data above the upper bound and the other half above the trend line. After 1000 C-hr all but 1 data point fell above the upper bound, indicating *SC5* outperformed the control for these test times in *Group 1* soil. *SC6* was similar to *SC5* with *Group 1* soil. *Group 3* soil was considerably weaker than the control. Data below 1000 C-hr from *Group 1* were clustered around the trend line, but the majority fell in Zone 2. Between 1000 and 2000 C-hr, 4 of the 6 data points fell above the upper bound, while the other 2 fell in Zone 2. Above 3000 C-hr, all data fell above the upper bound. Overall, *SC6* was considered moderately stronger than the control.

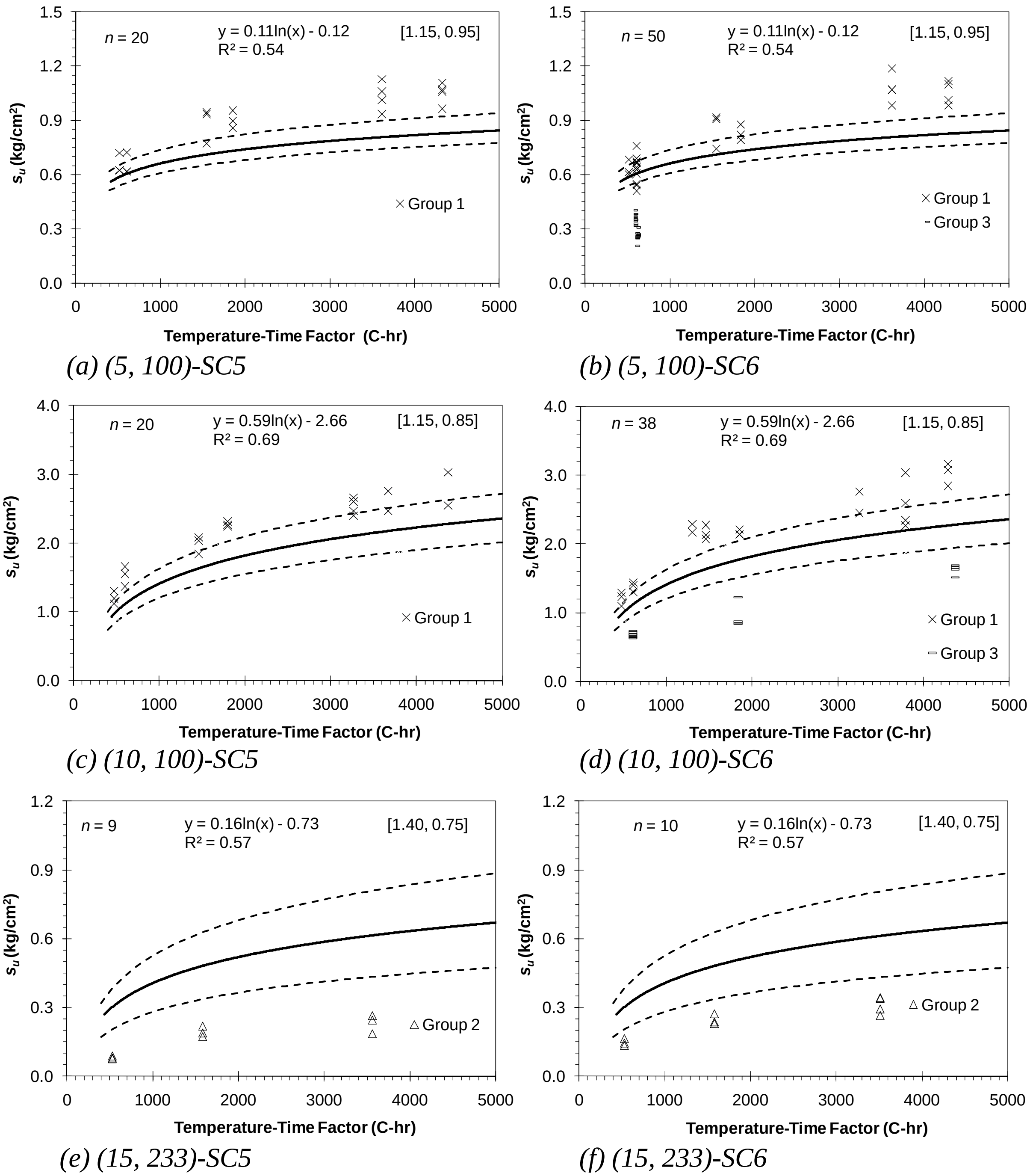


Figure 9.11. Results of Theodore Specialty Cements in Soil 1 vs. Th T III Control

At (10, 100), *SC5* appeared to be slightly to moderately stronger than the control. All *SC5* data below 2000 C-hr except 1 data point fell above the upper bound of the control envelope. Above 3000 C-hr, 5 of the 8 data points fell above the upper bound, while the other 3 fell near the upper bound. Specimens at (10,100) from *Group 1* and stabilized with *SC6* were also stronger than the control. *Soil 1 Group 3* was weaker than the control with *SC6*, but this data is not necessarily meaningful.

At (15, 233), *SC5* and *SC6* were only tested with *Group 2* soil, and all specimens were weaker than the control. Sufficient material was not available from *Group 1* to assess *SC5* and *SC6* further. The (15, 233) results presented should be considered inconclusive.

9.3.1.3 *Soil 1* Shear Strength Analysis of Blended Theodore Cements

Figure 9.12 plots *Theodore* blended cement results in *Soil 1*. A few data points were noticeably different than the rest at a given test condition, which could be due to the way the cements were introduced into the soil. The 2 cements were poured into the slurry at approximately the same rate but were not blended together beforehand. This could have resulted in some of the observed scatter. Overall, however, the data supports an optimum SO_3 content reduced from normal *Th T III* production. The magnitude of the SO_3 reduction cannot be quantified in absolute terms, though the data suggests SO_3 levels bounded by *SC5* and *SC6* (2.2 to 3.5%) produce specimens with higher strength than *Th T III*.

Shear strength of *SC5* and *SC6* was the same in a practical sense (Figure 9.11c and 9.11d). Blending 5% *SC5* and 5% *SC6* resulted in similar shear strength to either *SC5* or *SC6* as shown in Figure 9.13. The *SC5* and *SC6* blend average shear strength below 1000 C-hr was 0.97 kg/cm^2, which was somewhat lower than the next higher strength of 1.21 kg/cm^2. However, when 1 data point (0.62 kg/cm^2) that was noticeabally lower than the other values was removed, the average strength of the blend was 1.15 kg/cm^2, which is very close to the rest of the data. Using *Th T III* reduced strength at (10, 100) as shown in Figure 9.11 and Figure 9.13.

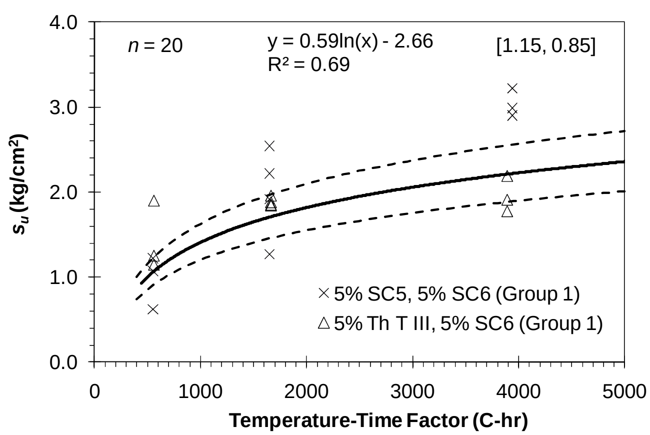


Figure 9.12. Results of Blending Theodore Cements in Soil 1 with 100% Moisture

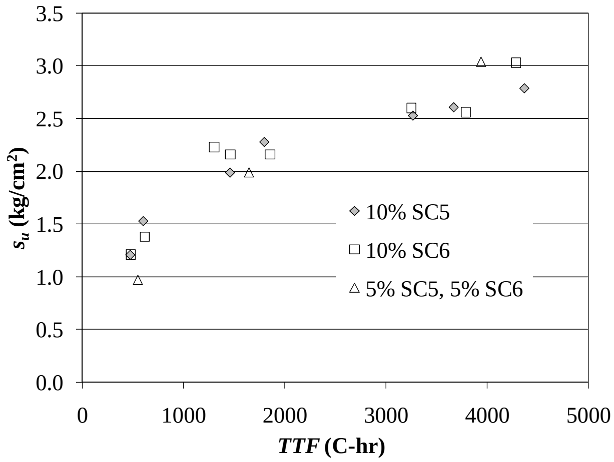


Figure 9.13. Comparison of SC5, SC6, and Blend at (10, 100) in Soil 1

9.3.2 *Soil 2* Shear Strength Analysis of Theodore Cements

9.3.2.1 *Soil 2* Shear Strength Analysis of *Type I/II* Theodore Cements

Figure 9.14 plots shear strength test results of *Th T I/II* with respect to the *Th T III* control band in *Soil 2*. Test results were within the lower portion of the control band and approaching the trendline in several cases. Two data points were below the lower bound of the control band. The limited data indicates no advantage of *Th T I/II* versus the control cement; more testing would be needed to provide definitive statements.

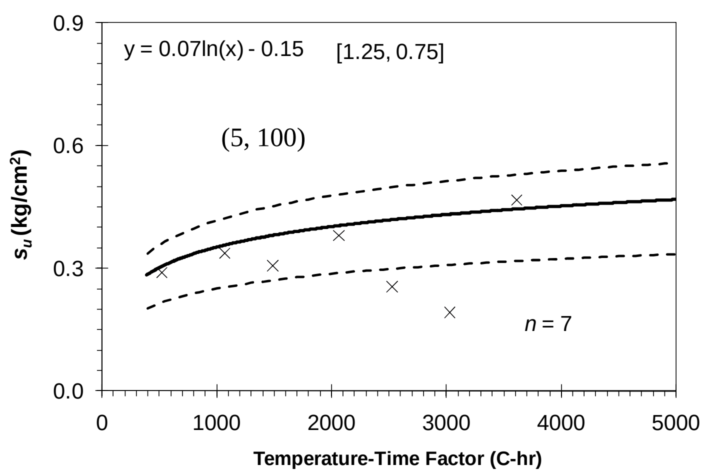
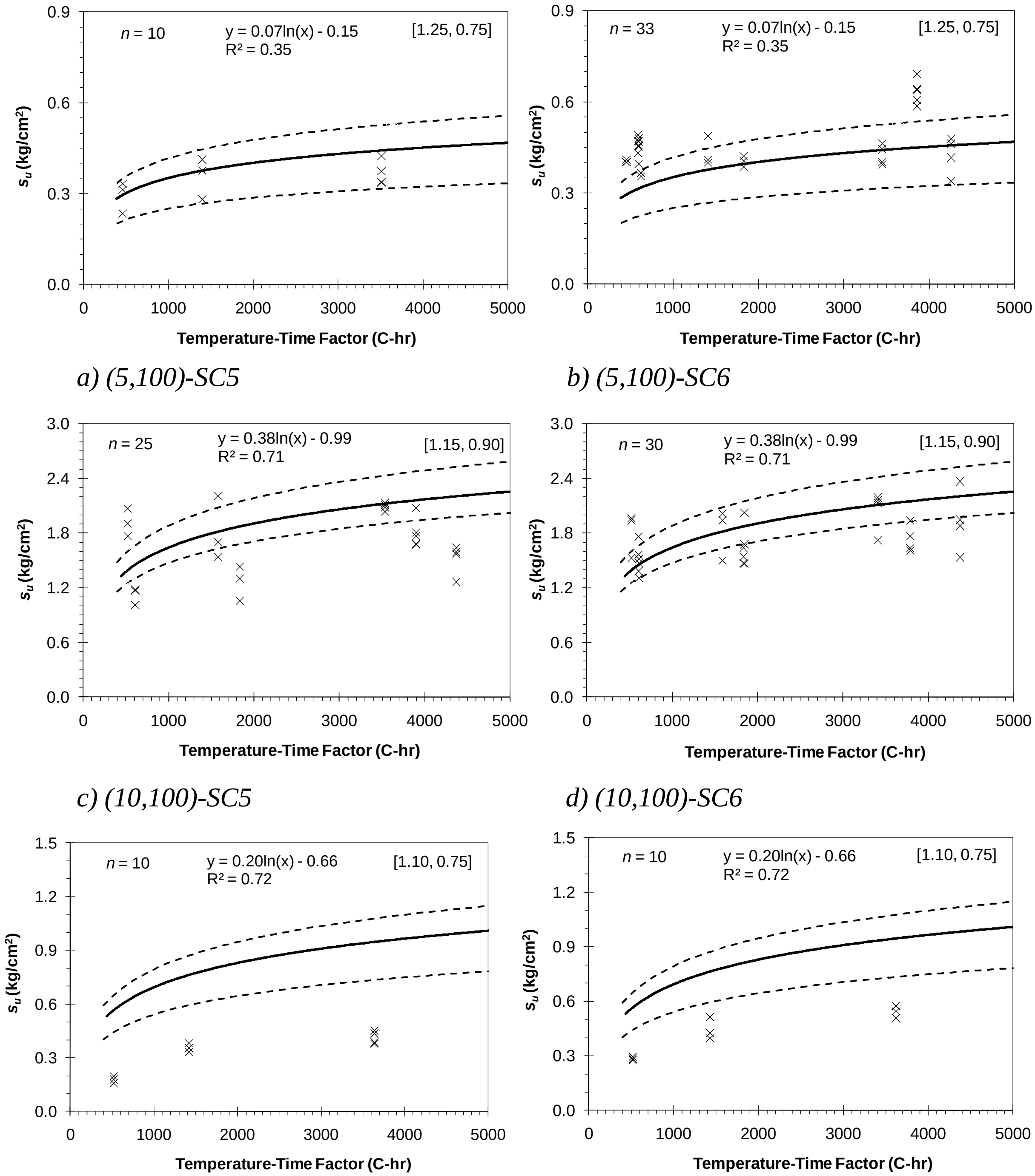


Figure 9.14. Soil 2 Shear Strength Test Results for Th T I/II vs. Th T III Control

9.3.2.2 *Soil 2* Shear Strength Analysis of Specialty Grind Theodore Cements

Figure 9.15 plots test results of *Theodore* cements and *Soil 2*. *SC5* was similar to the control cement at (5,100), as all data fell in Zones 2 and 3. Below 1000 C-hr, *SC6* was stronger than the control at (5,100) as most of the data fell in Zone 1. Above 1000 C-hr, *SC6* exhibited about the same strength as the control in the majority of cases, though 1 set of 5 data points was entirely above the upper control bound (i.e. Zone 1).



a) (5,100)-SC5 *b) (5,100)-SC6*

c) (10,100)-SC5 *d) (10,100)-SC6*

e) (15,233)-SC5 *f) (15,233)-SC6*

Figure 9.15. Results of Theodore Specialty Cements in Soil 2 vs. Th T III Control

At (10,100), *SC5* produced mixed results. Below 1000 C-hr, half of the data fell above the upper bound and half below the lower bound of the control envelope. Between

1000 and 2000 C-hr, the majority of the data was weaker than the lower bound of the control envelope. Above 3000 C-hr, data points ranged from the trendline to well below lower bound of the control envelope. Overall, *SC5* was considered slightly stronger to weaker than the control as the data was mixed and did not provide a clear behavioral trend. *SC6* at (10, 100) appeared to have similar strengths to that of the control below 1000 C-hr, since most of the data fell within the upper and lower control bands. The majority of the data points between 1000 and 2000 C-hr were located near or below the lower bound of the control envelope. Above 3000 C-hr test results were generally weaker than the lower control bound. At (15, 233), *SC5* and *SC6* both produced strengths below the lower bound of the control envelope and were considered weaker than the control in this condition.

9.3.2.3 *Soil 2* Shear Strength Analysis of Blended Theodore Cements

Figure 9.16 plots *Theodore* blended cement results in *Soil 2*. The data was erratic and did not provide any insight into *Soil 2* at (10, 100). Figure 9.15c and 9.15d also behaved somewhat erratic, but not to the extent of Figure 9.16. Investigation into the results provided in Figure 9.16, especially 5% *Th T III* and 5% *SC6*, did not provide any evidence of a procedural or calculation error, so the data was not discarded but is suspect.

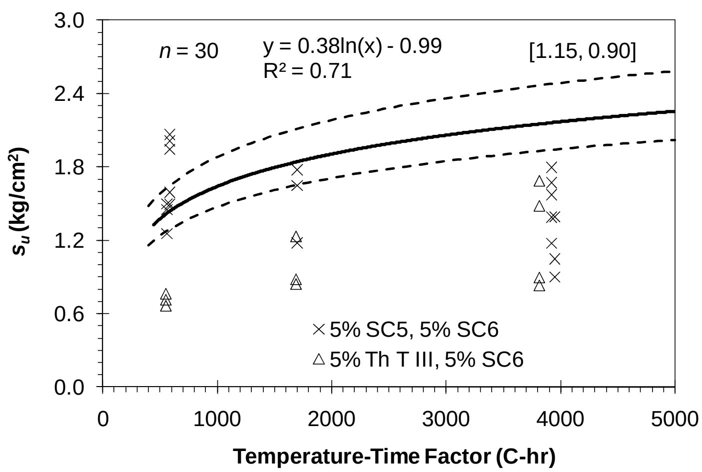


Figure 9.16. Results of Blending Theodore Cements in Soil 2 with 100% Moisture

The range of strengths measured in Figure 9.15c and 9.15d was approximately 1 to 2.3 kg/cm^2. Figure 9.15c and 9.15d data fell above and below the control strength envelope with data generally falling above the envelope at shorter cure times and falling below the envelope at longer cure times. Figure 9.16 data had somewhat of the same pattern if the *Th T III* and *SC6* blend is not considered. In Figure 9.16, however, strength at longer cure times was significantly below the control band in some cases, whereas in Figure 9.15 this was not nearly as common. These low strengths are not fully understood. In Figure 9.16, strength ranged from approximately 0.7 to 2.1 kg/cm^2. Testing of 5% *SC5* and 5% *SC6* was performed with 2 sets and 1 suite. One set produced strengths above the control envelope at early cure times, while the other set produced strengths below the control envelope at later cure times. The suite produced intermediate results.

9.3.3 *Soil 3* Shear Strength Analysis of Theodore Cements

9.3.3.1 *Soil 3* Shear Strength Analysis of *Type I/II* Theodore Cements

Figure 9.17 plots shear strength test results of *Th T I/II* with respect to the *Th T III* control band in *Soil 3*. Test results were below the lower control band in all cases. The limited data indicates no advantage of *Th T I/II* versus the control cement; more testing would be needed to provide definitive statements.

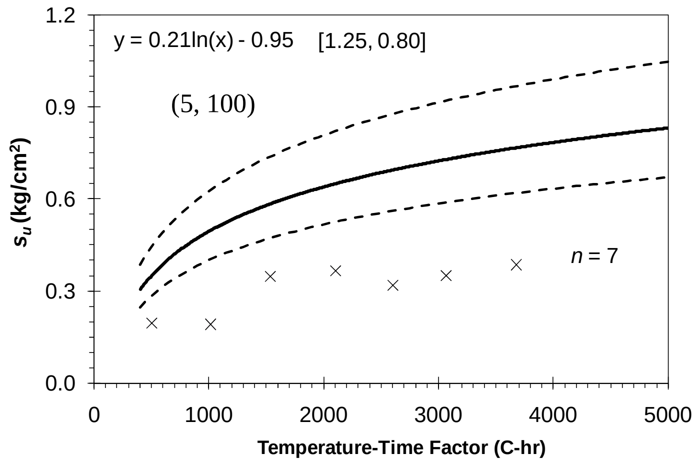


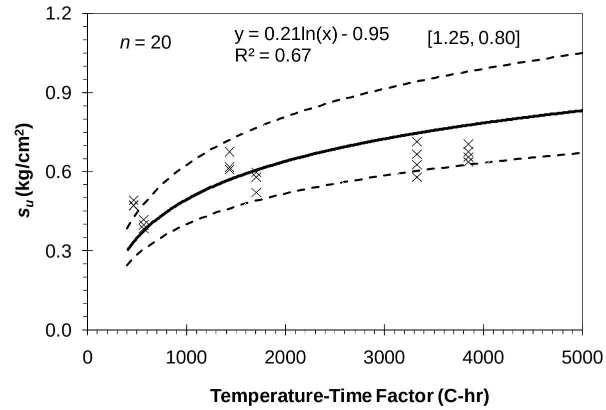
Figure 9.17. Soil 3 Shear Strength Test Results for Th T I/II vs. Th T III Control

9.3.3.2 *Soil 3* Shear Strength Analysis of Specialty Grind Theodore Cements

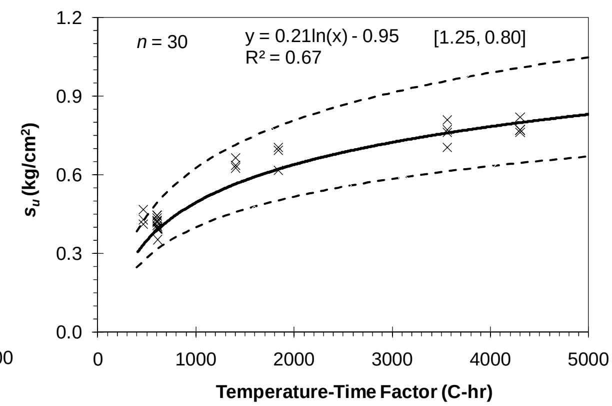
Figure 9.18 plots test results of *Theodore* cements and *Soil 3*. *SC5* and *SC6* performance was deemed the same as the control at (5, 100). Most data was within Zone 2 and Zone 3 and distributed around the trendline.

SC5 at the (10,100) condition was practically the same as the control cement with the possible exception being the data below 1000 C-hr where *SC*5 could have performed better. *SC6* showed somewhat mixed results at (10,100). Below 1000 C-hr, all data points were close to the upper bound of the control envelope. Between 1000 and 2000 C-hr, most of the data fell within the bounds of the control envelope. Above 3000 C-hr, the majority of the data fell below the trend line, and some of that data fell below the lower bound of the control envelope. Consequently, *SC6* was considered on par with the control below 2000 C-hr and slightly below the control above 3000 C-hr.

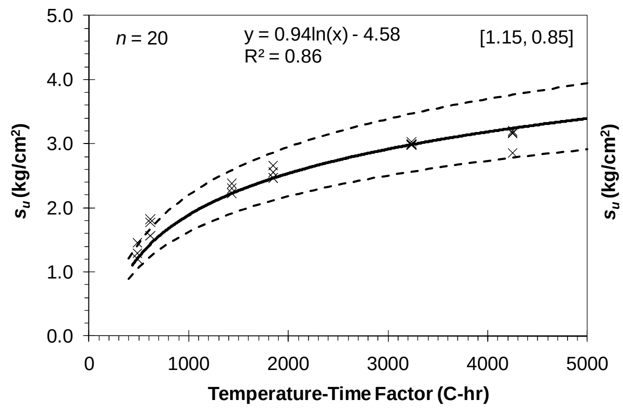
SC5 and *SC6* exhibited similar results at (15,233). Most of the data fell near the lower bound of the control envelope, so both cements were considered slightly weaker than the control. Neither cement appeared capable of producing strengths at longer curing intervals that were as high as the control.



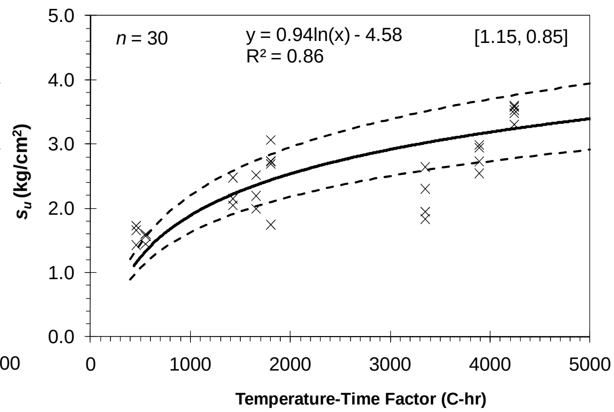
a) (5,100)-SC5



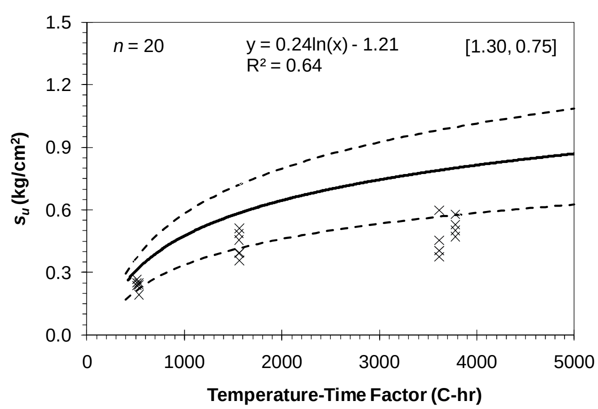
b) (5,100)-SC6



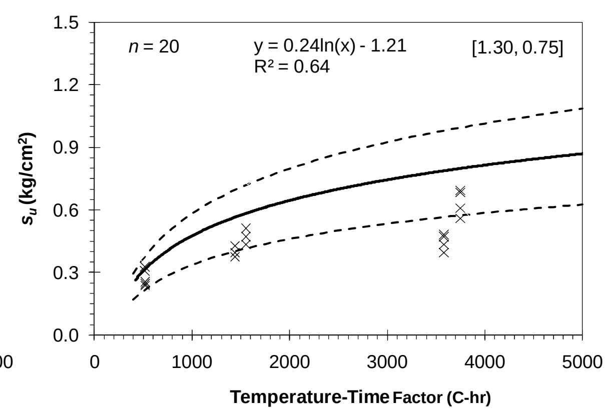
c) (10,100)-SC5



d) (10,100)-SC6



e) (15,233)-SC5



f) (15,233)-SC6

Figure 9.18. Results of Theodore Specialty Cements in Soil 3 vs. Th T III Control

9.3.3.3 *Soil 3* Shear Strength Analysis of Blended Theodore Cements

Figure 9.19 plots *Theodore* blended cement results in *Soil 3*. Blends that contained *Th T III* tended to perform better than those that did not. Above 3,000 C-hr, the blend with 5% *SC5* and 5% *SC6* was noticeabally below the lower bound of the control envelope. Overall, there was not a considerable strength difference between blends, and most of the data fell within Zone 3. The results are moderately surprising when viewed in conjunction with Figure 9.18c and 9.18d; these plots had data in Zone 2 at later test times whereas the blended cement data in Figure 9.19 did not. Figures 9.18 and 9.19 did agree in general terms since strength gain at longer curing times leveled off relative to the control band. The data in Figure 9.19 provides some support to the assessments from Figure 9.18c and 9.18d that any differences between *SC5*, *SC6*, and *Th T III* for *Soil 3* at (10, 100) were minor.

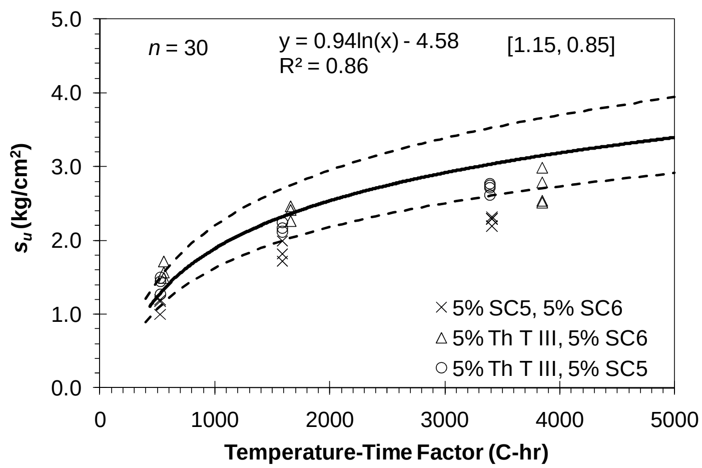


Figure 9.19. Results of Blending Theodore Cements in Soil 3 with 100% Moisture

9.3.4 Discussion of Theodore Produced Cement Shear Strengths

Th T I/II did not indicate any advantage over *Th T III* in any soil based on limited testing. Test data from *Th T I/II* was often below the control envelope lower bound, while a moderate amount of data was in Zone 3. Test data only occasionally achieved Zone 2.

Table 9.3 summarizes the *SC5* analysis compared to the *Th T III* control. For *Soil 1* at (5,100) and (10,100), *SC5* outperformed the control for *Group 1* soils. The results at (15,233) were inconclusive, since no *Group 1* material was available for testing. *Soil 2*, stabilized with *SC5* exhibited similar strength to the control at (5,100); it showed a range of results at (10,100), as some data were slightly stronger than the control while other data were weaker. *SC5*, was weaker than the control at (15,233). *SC5* was unable to outperform the control for *Soil 3*, as the strengths were similar to the control at (5,100) and (10,100) and slightly to moderately weaker at (15,233).

Table 9.4 summarizes the comparisons made between *SC6* and the *Th T III* control. *SC6* for *Soil 1* showed similar results to *SC5*, as the strengths observed were higher than the control for (5,100) and (10,100) *Group 1* soil and inconclusive at (15,233) since insufficient *Group 1* material was available for testing. For *Soil 2, SC6* was at to slightly stronger than the control at (5,100) and at to slightly weaker at (10,100). *SC6* was clearly weaker than the control at (15,233). *Soil 3* results with *SC6* were similar to *SC5*. At (5,100), *SC6* showed about the same strength as the control, and at (10,100) the strengths were at to slightly weaker than the control. At (15,233), *SC6* was slightly weaker than the control.

Table 9.3. Summary of *SC5* Findings Relative to *Th T III* Control

	Soil		
Type	**1**	**2**	**3**
(5,100)	Moderately stronger	Same strength	Same strength
(10,100)	Slightly to moderately stronger	Slightly stronger to weaker	Same strength
(15,233)	Inconclusive	Weaker	Slightly to moderately weaker

Table 9.4. Summary of *SC6* Findings Relative to *Th T III* Control

	Soil		
Type	**1**	**2**	**3**
(5,100)	Moderately stronger	At to slightly stronger	Same strength
(10,100)	Slightly to moderately stronger	At to slightly weaker	At to slightly weaker
(15,233)	Inconclusive	Weaker	Slightly weaker

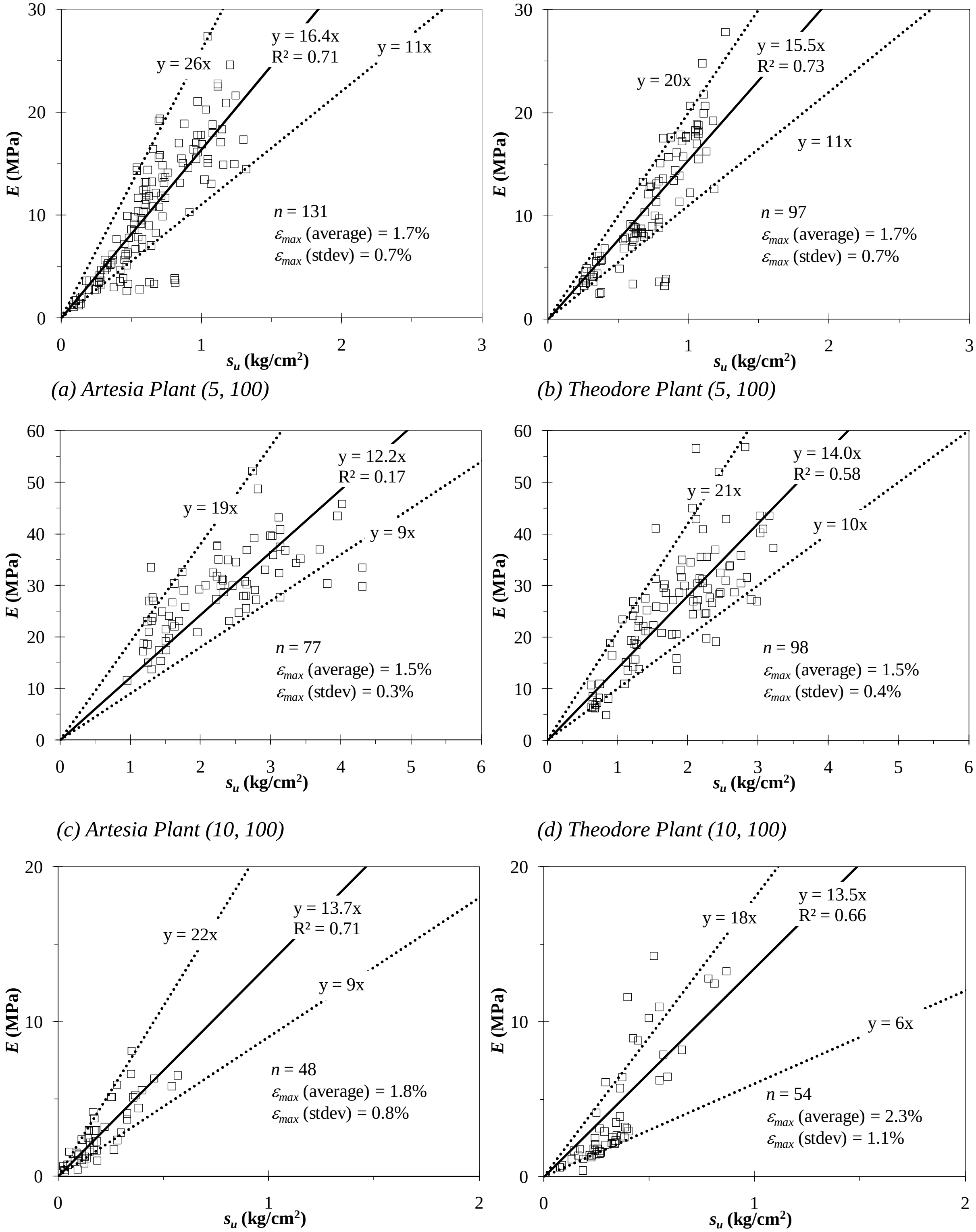
Soil 1 at (5,100) and (10,100) were the only 2 combinations where a considerable strength advantage over the control was observed, and this finding held true for both *SC5* and *SC6*. *SC5* and *SC6* have similar fineness to that of the control, with reduced SO_3 contents. The SO_3 contents of *SC5* and *SC6* were 2.2 and 3.5%, respectively. Since *SC5* and *SC6* were similar for *Soil 1* and both outperformed the control, it is possible that the optimum SO_3 content for *Soil 1* is between the SO_3 contents of *SC5* and *SC6.*

Chew et al (2004) suggests that secondary pozzolanic reactions between hydrated lime and silica and alumina from the soil can add shear strength to soil-cement mixes. Since *Soil 1* showed higher amounts of silica than *Soil 2* and *Soil 3,* perhaps *Soil 1* underwent more pozzolanic reactions at later curing times that allowed strengths to remain above the control. This, coupled by the reduced SO_3 content, could be a justification as to why *Soil 1* was the only soil type that clearly outperformed the control.

In general, *Soil 2* specimens showed similar strengths to that of the control at (5,100) and (10,100) and lower strengths at (15,233) for both cements. *Soil 2* appeared to benefit from increased cement fineness at 100% moisture content (i.e. *SC2*), while *SC5* and *SC6* had fineness values similar to *Type III* cement. The lower fineness could explain why *Soil 2* specimens stabilized with *SC5* and *SC6* did not definitively outperform the control at (5,100) and (10,100). *Soil 2* specimens at (15,233) were weaker than the control. *Soil 3* specimens did not appear to benefit from reduced SO_3 contents, as *SC5* and *SC6* both showed very similar results to that of the control at (5,100) and (10,100). *Soil 3* was slightly weaker than the control at (15,233), again not responding to a change in SO_3 content.

9.4 Elastic Modulus and Ductility Results of Artesia and Theodore Cements

Figures 9.20 through 9.22 plot elastic modulus test results for all portland cements used (includes *Type III* control cements) in conjunction with *Soil 1, Soil 2,* and *Soil 3,* respectively. The number of data points are provided on the individual plots alongside summary failure strain information. The solid line is a trendline fit through the origin while the dashed lines were visually fit to provide an envelope of the data. The slope of the upper and lower portion of the data envelope are shown on the plots.



(a) Artesia Plant (5, 100)

(b) Theodore Plant (5, 100)

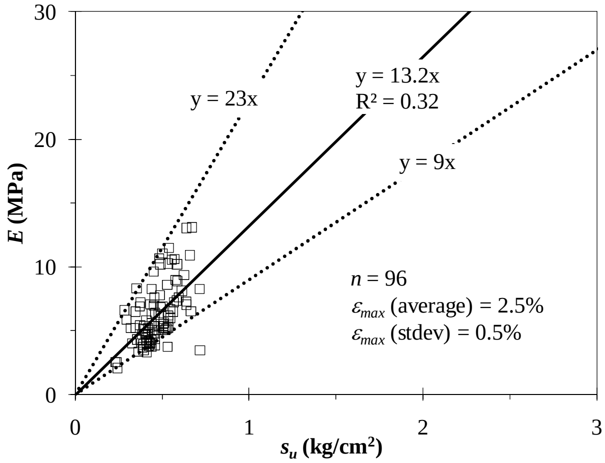
(c) Artesia Plant (10, 100)

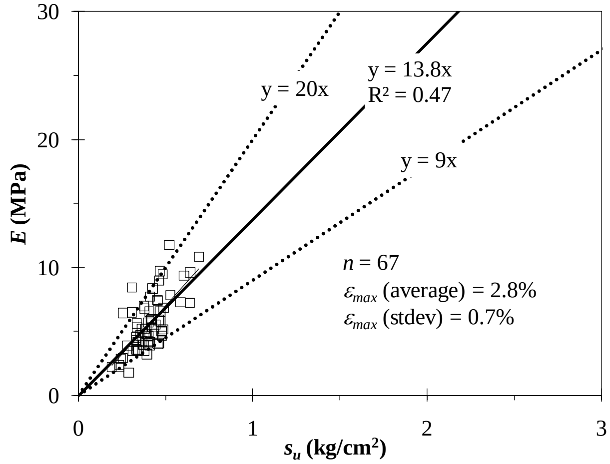
(d) Theodore Plant (10, 100)

(e) Artesia Plant (15, 233)

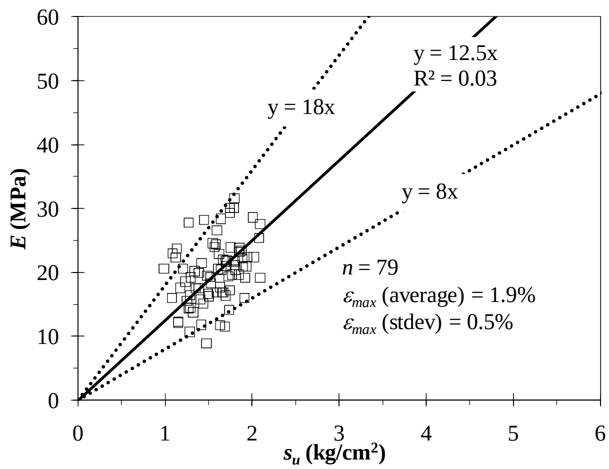
(f) Theodore Plant (15, 233)

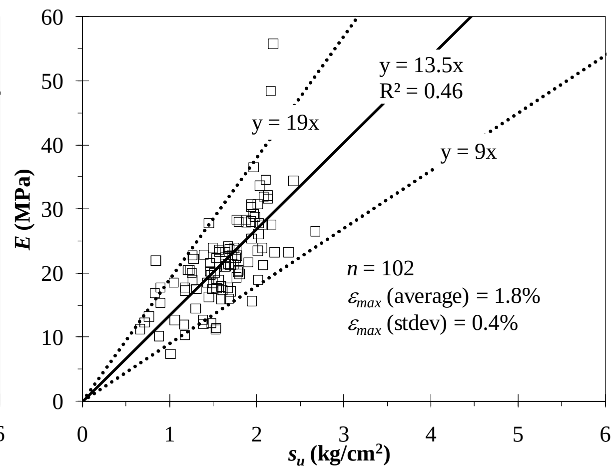
Figure 9.20. Elastic Modulus and Ductility Results of Portland Cement and Soil 1



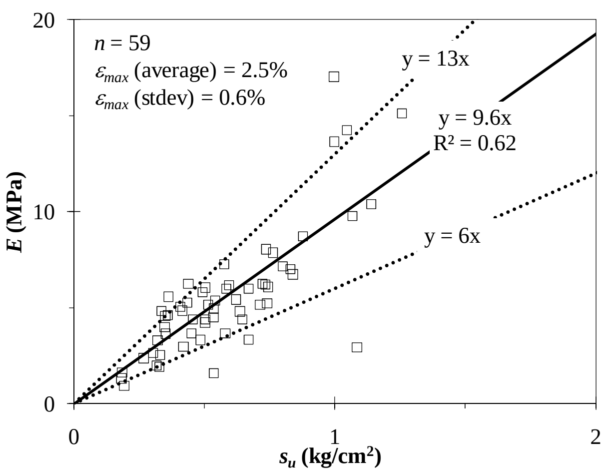


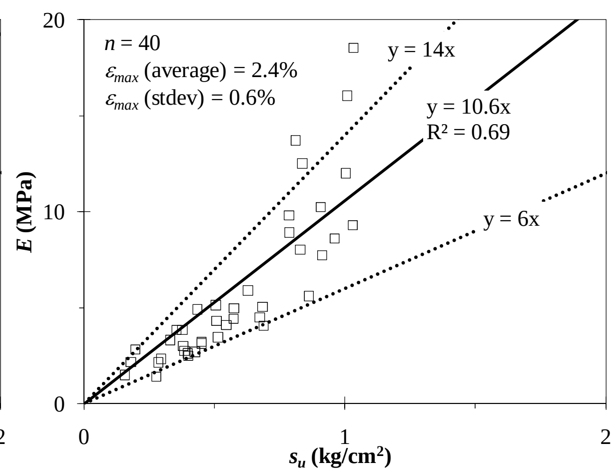
(a) Artesia Plant (5, 100) *(b) Theodore Plant (5, 100)*





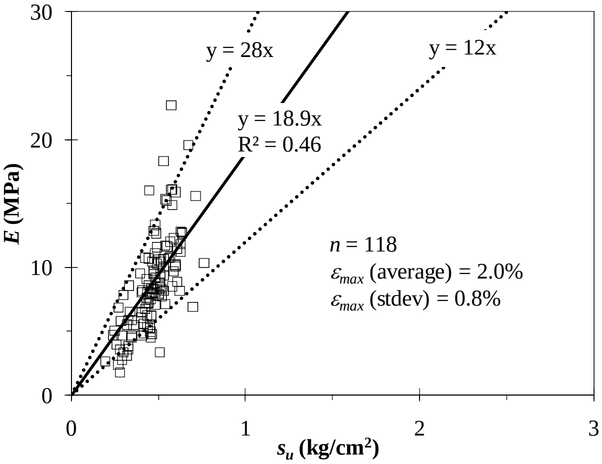
(c) Artesia Plant (10, 100) *(d) Theodore Plant (10, 100)*



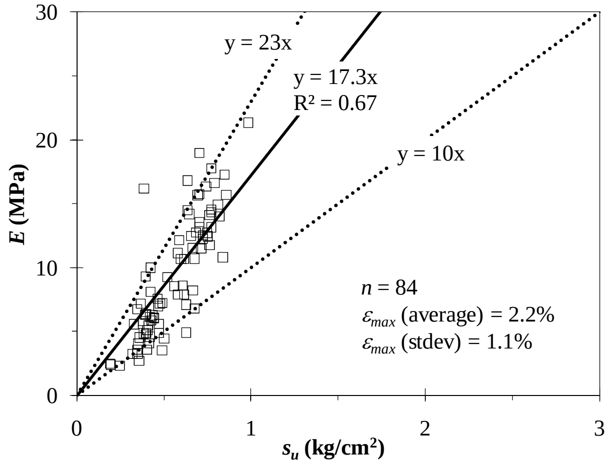


(e) Artesia Plant (15, 233) *(f) Theodore Plant (15, 233)*

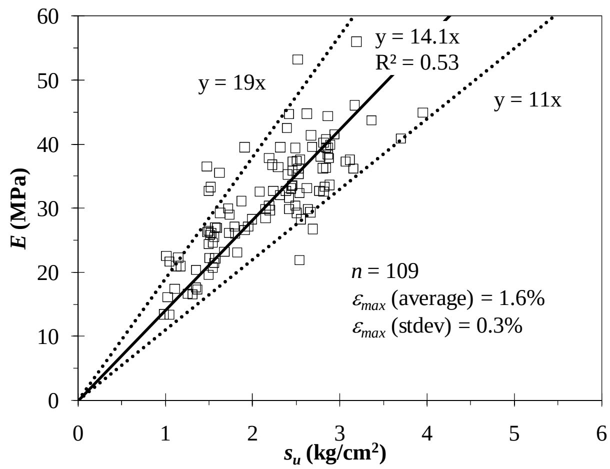
Figure 9.21. Elastic Modulus and Ductility Results of Portland Cement and Soil 2



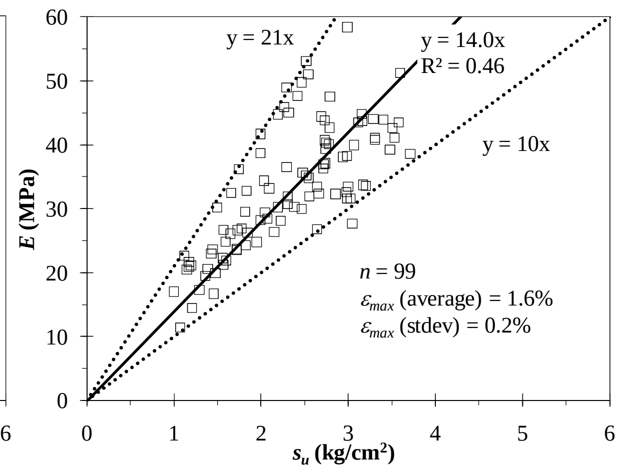
(a) Artesia Plant (5, 100)



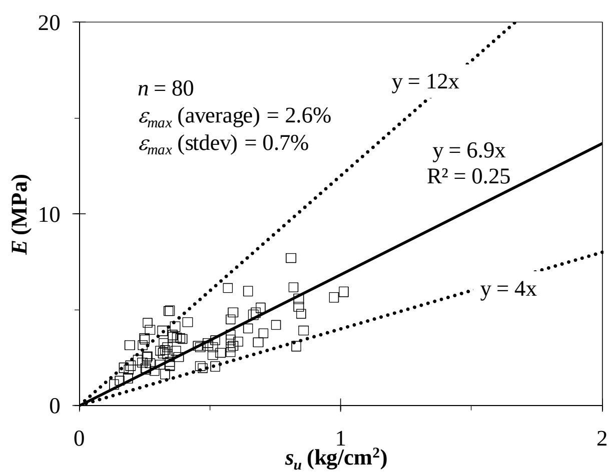
(b) Theodore Plant (5, 100)



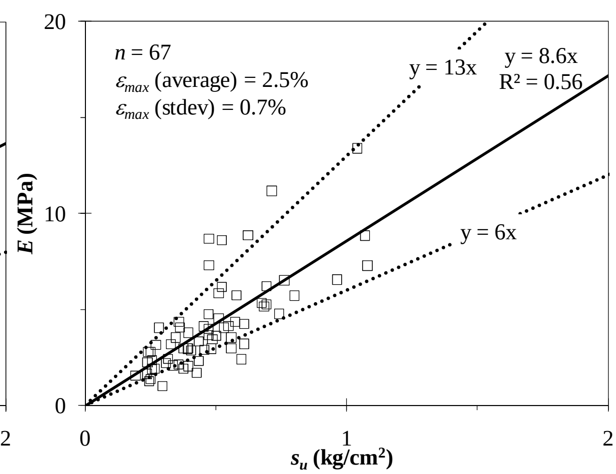
(c) Artesia Plant (10, 100)



(d) Theodore Plant (10, 100)



(e) Artesia Plant (15, 233)



(f) Theodore Plant (15, 233)

Figure 9.22. Elastic Modulus and Ductility Results of Portland Cement and Soil 3

Cements from both plants had a small number of data points (on the order of 5%) that deviated considerably from the envelope for *Soil 1* at (5, 100). Upon further examination all of these data points were collected from 3 *Protocol 1* suites; 2 of the suites were with Artesia produced cement, and 1 of the suites was with Theodore produced cement. All of the data points that were outside the envelope had a low modulus to strength ratio and had higher than typical failure strains. The modulus was calculated with the same protocol in all cases, but for these specimens 2.5% strain or more was used to calculate the modulus.

For all soils, the highest modulus to strength behavior was observed for (5, 100) specimens. Cement source (Artesia or Theodore plant) did not appear to affect strength to modulus behavior. With exception of *Soil 1*, (15, 233) specimens had the lowest strength to modulus behavior, and *Soil 1* behaviors at (15, 233) were on par with (10, 100) but lower than (5, 100). *Soil 3* had a considerable difference in trendline slope as the Artesia cement values reduced from 18.9 at (5, 100) to 14.1 at (10, 100) to 6.9 at (15, 233). A similar trend was observed for Theodore cement and *Soil 3* but the reduction in slope was less than for Artesia. *Soil 1* and *Soil 2* were more consistent in terms of their modulus between conditions, though there were some differences as have already been mentioned. Maximum strains were relatively low for all test conditions, with (10, 100) specimens having the lowest strain in all instances. In general, maximum strains for (5, 100) and (15, 233) conditions were comparable.

9.5 Summary of Portland Cement Test Results

Approximately 1,100 *UC* specimens were tested to evaluate specialty grind portland cements for disaster recovery. The concept is to develop implementable property modifications in the form of a specification for cement plants. The 2 factors of primary investigation were SO_3 content and Blaine Fineness.

Test results indicated that a modified SO_3 content can change shear strength of stabilized soil slurries, but that there is not a specific SO_3 content that works for all applications. On site soil and the cement plant supplying the project should be tested in conjunction with each other at the moisture content in the field and a modified SO_3 content developed. Fortunately, SO_3 reduction did not result in drastic strength loss so the SO_3 levels can be adjusted during the early part of the project. An initial estimate based on the data collected for this project would be to reduce the SO_3 content of the cement by 25% and test strength at 1 day. If the strength exceeds that of the normally produced cement from that plant, reduce the SO_3 content further until strength improvement relative to the highest strength measured to date ceases. This approach would not have caused problematic results for any of the soils tested in this project.

Increased Blaine Fineness was not advantageous except for *Soil 2*, where it appeared to provide strength increase. Increased fineness beyond typical for *Type III* will slow production at the cement plant, so it should only be attempted for highly organic soils (e.g. 25% as with *Soil 2*). Reducing SO_3 content would not reduce cement plant production.

Table 9.5 summarizes specialty grind cements that outperformed the control cement from the same production facility. If the specialty grind product performed the same or only slightly better than the control, it was not listed in Table 9.5. Five of the 9 cases considered benefited from the specialty grind cements. *Soil 3* did not respond to property changes; specialty cements were typically the same strength as the control for *Soil 3*. Portland cement, in general, does not appear to be the best choice for the (15, 233) condition with the possible exception of *Soil 2*. As discussed in Chapter 10, calcium sulfoaluminate cements are a better option for (15, 233) testing needs in a disaster situation for nearly all situations.

Table 9.5. Summary of Specialty Portland Cements Outperforming *Type III* Controls

	Soil		
Type	**1**	**2**	**3**
(5,100)	*SC5*, *SC6*	*SC1*, *SC2*, *SC6*	None
(10,100)	*SC1*, *SC5*, *SC6*	*SC2*	None
(15,233)	None	*SC1*, *SC2*	None

Fairly reliable correlations were developed between shear strength and elastic modulus. These correlations are intended for design purposes as one can select a design shear strength and then estimate the corresponding design modulus. The elastic modulus in units of MPa was 6.9 to 18.9 times the shear strength in units of kg/cm^2. The relationship was dependent on soil type and the amount of water in the blend but did not appear sensitive to the cement source. Maximum strains were less than 3% on average for all specimens.

There was a fair amount of variability in the data collected as a result of several contributing influences. Soil specimens have variability, there are mixing influences, property measurement methods have variability, and so forth. The key to this discussion is that the important trends of interest were still apparent in the presence of this variability.

CHAPTER 10 - CALCIUM SULFOALUMINATE CEMENTS TEST RESULTS

10.1 Overview of Calcium Sulfoaluminate Test Results

Calcium Sulfoaluminate (*CS*) cements were tested that are typically reserved for applications requiring very high early strength. Three *CS* cements were tested in a variety of soil slurry blends at a variety of dosage rates. Approximately 800 *UC* specimens (consisting of ≈660 *CS* and ≈140 portland cements) were tested and are presented in this chapter.

10.2 Preliminary Testing

Figure 10.1 plots early age test results. Six portland cements and *CTS RS* were tested at 2, 4, 8, 16, and 24 hr without replication to assess the general strength gain trend within the first day of curing (defined as early age herein). The data was collected without replication so detailed product comparison is not recommended; but, for the purpose of assessing the general strength gain trend in the first day of curing, one replicate was felt sufficient.

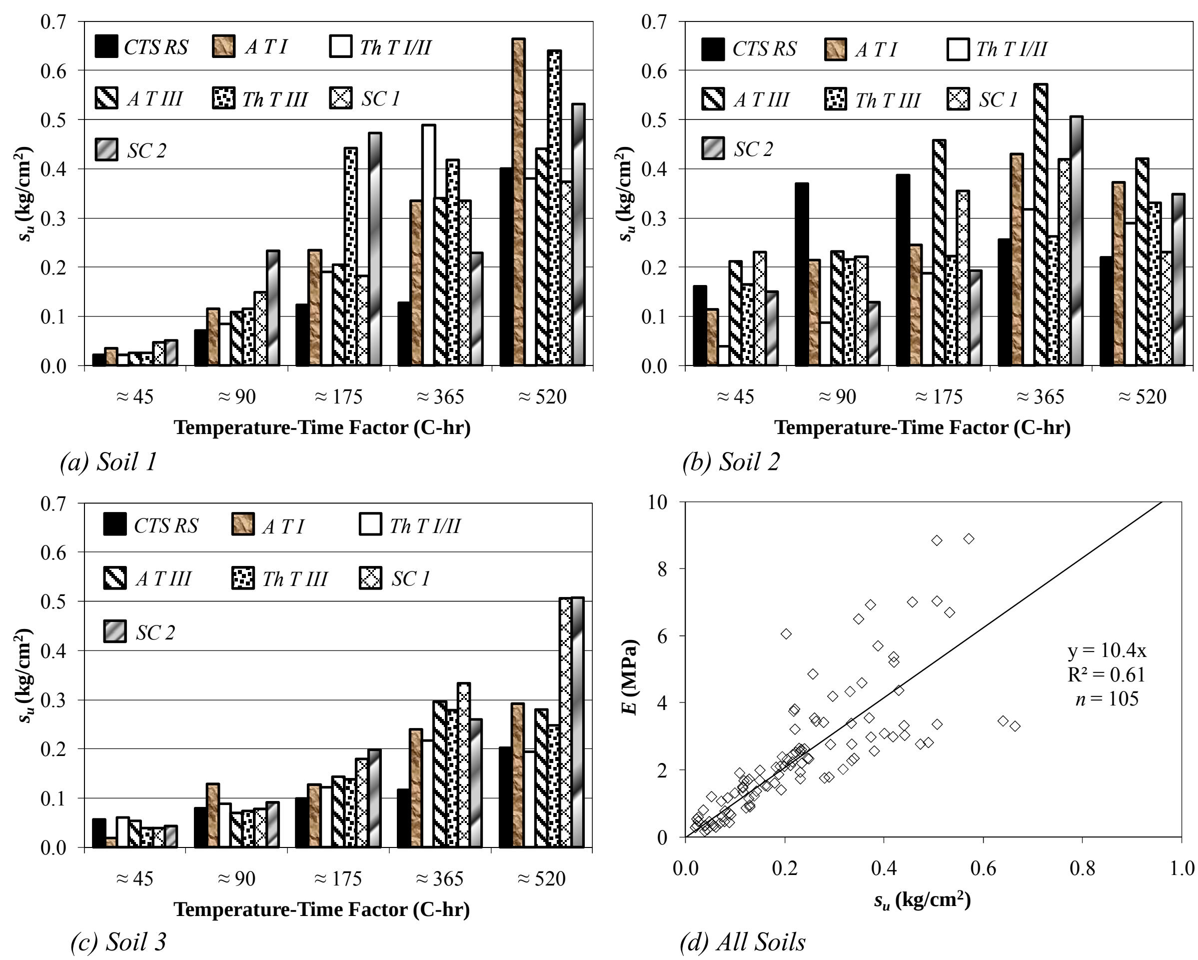


Figure 10.1. Early Strength Properties of Commercially Available Cements (5, 100)

All mixing was according to normal placement (*NP*) when collecting Figure 10.1 data. The data shows that some shear strength is mobilized fairly quickly, and that the rate of strength gain in the first several hours after mixing is not the same between soil types. The relationship between shear strength and elastic modulus has scatter, but does have some correlation for the majority of the data. The average ε_{max} value was 4.7% with 60% of the readings less than 4% and 90% of the readings less than 7%.

One *NP* prepared specimen was tested per day between 1 and 7 days of curing without replication at the (5, 100) condition with *CTS RS* using each soil (21 total specimens). Test results are provided in Table 10.1 alongside corresponding portland cement control data at *TTF* values between 500 and 3,500 C-hr. *CTS RS* did not gain strength between 500 to 3500 C-hr, so a representative strength of all 7 specimens was presented. *CTS RS* was comparable in strength for *Soil 2*, but lower in *Soil 1* and *Soil 3*.

Table 10.1. Preliminary Shear Strength Results at (5, 100) Condition

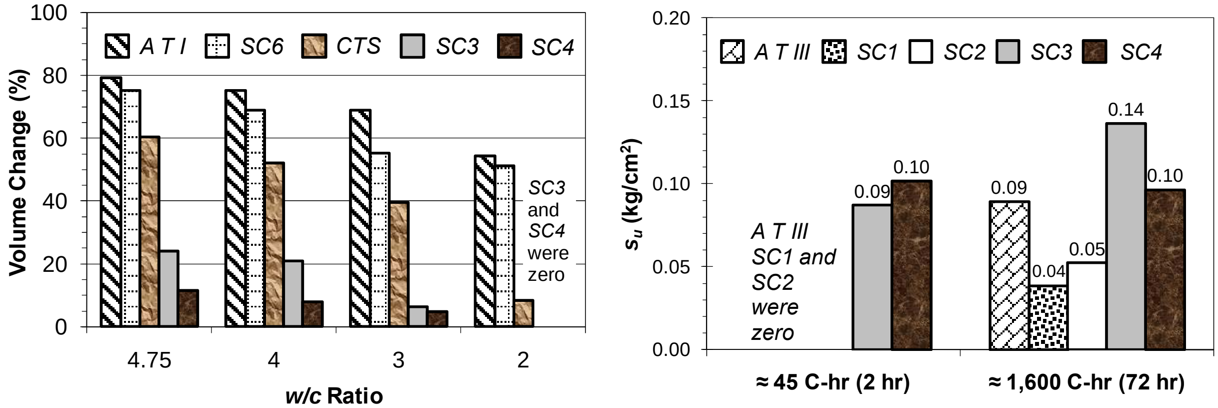
Soil	*CTS RS* (kg/cm^2)	*Th T III* Lower Bound (kg/cm^2)	*Th T III* Trendline (kg/cm^2)
1	0.45	0.52 to 0.80	0.65 to 1.00
2	0.29	0.22 to 0.32	0.29 to 0.42
3	0.28	0.29 to 0.61	0.36 to 0.76

The *CTS RS* product did not gain strength at a typical rate relative to portland cement, if concrete is used as a reference. The rapid set product typically gains strength much faster than portland cement, but in Figure 10.1 *CTS RS* rarely had the highest shear strength for the 3 soils at the 5 early age *TTF* values tested. Table 10.1 showed that the strength within the first 7 days of curing did not exceed that of portland cement. To investigate the relative behavior of *CS* and portland cements, 4 key areas were identified: 1) cement moisture encapsulation; 2) *CS* cement type and its interaction with soil; 3) effects of specimen preparation and in particular mixing time; and 4) cement dosage rate. These issues are addressed in the following sections.

10.3 Moisture Encapsulation

A key characteristic of *CS* cements is they can be designed to encapsulate a large amount of water during hydration, which for soils with excess moisture could be advantageous. Figure 10.2a plots results of an experiment to determine how much water 3 different *CS* cements could encapsulate relative to portland cements. Specimens were made in duplicate where cement and water only were mixed at a given *w/c* ratio and allowed to sit up to 5 minutes before being poured into 75 by 150 mm *UC* molds lined with plastic. The specimens were cured under water for 24 hr; upon removal the free water was allowed to drain, and the height of the solid portion of each specimen was measured to determine volume change, which is an indirect measure of moisture encapsulation.

Figure 10.2a indicates the portland cements (1 from a wet process plant and 1 from a dry process plant) have significantly higher volume change than *SC3* and *SC4*. *CTS RS* performed intermediately with respect to the portland cements and the other two *CS* cements. The *Type I* portland cement has slightly higher volume change than the *Type III* portland cement. The volume change of *SC3* and *SC4* cements reached zero at a *w/c* of 2, and *SC4* consistently had less volume change than *SC3*.



(a) Cement-Water Blends *(b) Non-Plastic Soil 5*

Figure 10.2. Moisture Encapsulation of Calcium Sulfoaluminate Cements

Figure 10.2b plots results of shear strength testing of *Soil 5*, which was non-plastic with 89% fines. *Soil 5* had a low affinity for water, and at the obtained moisture content of 100.5%, there was ample free water. Specimens were prepared in triplicate at a *w/c* ratio of 10 corresponding to a dosage rate of 5%. After curing for 2 hr, only *SC3* and *SC4* had measurable shear strengths indicating that the *CS* cements were able to encapsulate considerable amounts of water and allow quick development of shear strength. At 2 hr curing, the portland cements were slightly more viscous than pudding. After curing for 72 hr, the portland cements had developed some shear strength, but less than the *CS* cements had developed in 2 hr. Average ε_{max} was 1.9%, the average slurry density was 1.5 g/cm^3, and the relationship between E with units of MPa to s_u in units of kg/cm^2 was $E \approx 16(s_u)$.

10.4 Effects of Soil and *CS* Dosage Rate on Strength Gain

Table 10.2 plots average strength of *NP* protocol test results for *Soils 1* to *3* at 3 cement dosage rates and moisture contents. The values shown represent 3 or more individual test specimens. The data indicated by *Type III* was collected using the *A T III* and *Th T III* control cements. The control envelopes of the 2 cements were used to calculate a range of shear strengths at 550, 1750, and 4000 C-hr (average temperature of ≈23 to 24 C for curing duration) and the highest and lowest values were included in the table. As an example, at 24 hr for *Soil 1* (5, 100), *TTF* of 550 C-hr was entered into the trendline equation for *A T III*, which for this condition is 0.19 ln(550)-0.52 or 0.679 kg/cm^2 (Figure 8.4a). Multiplying this value by the upper and lower control band adjustments of 0.80 and 1.15 also shown in Figure 8.4a results in the range of expected values for *A T III* in these conditions of 0.54 to 0.78 kg/cm^2. This same approach was applied to *Th T III* and produced a range of 0.53 to 0.80 kg/cm^2. The lowest number was 0.53 kg/cm^2, and the highest number was 0.80 kg/cm^2 so they were entered into Table 10.2 as shown.

In *Soil 1*, *SC3*, *SC4*, and *CTS RS* were considerably weaker than *Type III* cement at (5, 100) and (10, 100) conditions. At (15, 233), however, all 3 cements were considerably stronger than *Type III* cement. Table 10.3 shows the extent *CS* cements outperformed *Type III* at (15, 233) in *Soil 1*. *Type III* cement was weakest at (15, 233), while the 3 *CS* cements were strongest at (15, 233). The result is significant as it indicates there are a range of conditions where portland cement is most suitable for disaster response and a distinct set of other conditions where *CS* cements are most suitable for disaster response.

Table 10.2. Results of Soil and Cement Dosage Effects on Strength With *NP* Protocol

Cement	Soil	Condition	24 hr s_u (kg/cm^2)	72 hr s_u (kg/cm^2)	168 hr s_u (kg/cm^2)
Type III	*1*	*(5, 100)*	0.53 to 0.80	0.70 to 1.05	0.82 to 1.23
		(10, 100)	0.90 to 1.54	1.48 to 2.59	1.89 to 3.35
		(15, 233)	0.11 to 0.39	0.24 to 0.65	0.33 to 0.84
Type III	*2*	*(5, 100)*	0.22 to 0.48	0.28 to 0.50	0.35 to 0.51
		(10, 100)	1.05 to 1.62	1.33 to 2.12	1.53 to 2.49
		(15, 233)	0.22 to 0.66	0.32 to 0.92	0.40 to 1.10
Type III	*3*	*(5, 100)*	0.26 to 0.47	0.39 to 0.77	0.48 to 0.99
		(10, 100)	1.15 to 1.69	1.91 to 2.81	2.38 to 3.70
		(15, 233)	0.23 to 0.45	0.44 to 0.81	0.59 to 1.07
SC3	*1*	*(5, 100)*	0.07	0.09	0.14
		(10, 100)	0.25[a]	0.43	0.54
		(15, 233)	1.39	2.52	2.89
SC3	*2*	*(5, 100)*	0.19	0.20	0.22
		(10, 100)	0.42	0.37	0.39
		(15, 233)	0.58	0.61	0.62
SC3	*3*	*(5, 100)*	0.02	0.03	0.03
		(10, 100)	0.32	0.34	0.46
		(15, 233)	0.60	0.68	1.35
SC4	*1*	*(5, 100)*	0.06	0.07	0.09
		(10, 100)	0.63	0.72	0.95
		(15, 233)	0.84	1.22	1.51
SC4	*2*	*(5, 100)*	0.16	0.17	0.16
		(10, 100)	0.42	0.39	0.39
		(15, 233)	0.35	0.37	0.48
SC4	*3*	*(5, 100)*	0.03	0.04	0.04
		(10, 100)	0.23	0.36	0.40
		(15, 233)	0.38	0.46	0.85
CTS RS	*1*	*(5, 100)*	0.17	---	---
		(10, 100)	0.58	---	---
		(15, 233)	0.72	1.05	0.87
CTS RS	*2*	*(5, 100)*	---	---	---
		(10, 100)	---	---	---
		(15, 233)	---	---	---
CTS RS	*3*	*(5, 100)*	0.10	---	---
		(10, 100)	0.73	---	---
		(15, 233)	0.85	1.85	1.57

a: Soil 1 with SC3 at (10, 100) was highly variable using the NP protocol. Nine cylinders were produced to provide insight into variability and produced strengths of 0.15, 0.22, 0.24, 0.28, 0.34, 0.38, 0.51, 0.73, and 0.88 kg/cm^2. The 0.25 kg/cm^2 value shown was the average of 0.22, 0.24, 0.28 kg/cm^2 and were specimens made at the same time as the 72 and 168 hr strength results for this combination.

Table 10.3. *Soil 1 CS* to Portland Cement Strength Ratios at (15, 233)

Cement	Ratio at 24 hr	Ratio at 72 hr	Ratio at 168 hr
SC3	3.6 to 12.6	3.9 to 10.5	3.4 to 8.8
SC4	2.2 to 7.6	1.9 to 5.1	1.8 to 4.6
CTS RS	1.8 to 6.5	1.6 to 4.4	1.0 to 2.6

In *Soil 2*, *SC3* and *SC4* were weaker than portland cement at (5, 100) and (10, 100), while *CTS RS* was not tested due to lack of *Soil 2*. At (15, 233), *SC3* and *SC4* were comparable with *Type III* portland cement as their average values fell within the range of values from the 2 portland cements used as a control in all instances. There does not appear to be any advantage to *CS* cements in highly organic soils such as *Soil 2*.

In *Soil 3*, *SC3*, *SC4*, and *CTS RS* were considerably weaker than portland cement at (5, 100) and (10, 100). At (15, 233), portland cement strengths were comparable with *SC3* and *SC4*, but were lower than *CTS RS*. This is interesting as *CTS RS* performed worse in *Soil 1* than *SC3* or *SC4* but better in *Soil 3*.

Results of testing specimens prepared according to the *NP* protocol indicated that behaviors in lower organic soils were of more interest with *CS* cements. Behaviors where the cement and moisture content are relatively high appear to be of particular concern. The effect of specimen preparation (i.e. *NP* vs. *RP* protocols) are investigated in the next section.

10.5 Effects of Specimen Preparation

Properties of normally placed (*NP*) specimens (in mold within 35 minutes of cement addition) were compared to rapidly placed (*RP*) specimens (in mold within 20 minutes of cement addition) in this section. Data is separated by soil type. In some instances, insufficient soil was available to conduct testing with all cements, dosage rates, and moisture contents.

10.5.1 *Soil 1* Specimen Preparation Results

Figure 10.3 compares the *NP* and *RP* methods with *CTS RS* for *Soil 1* at 3 moisture content and dosage rate combinations using the average of triplicate tests. Results were not consistent between moisture content and dosage rate combinations. At (5, 100), there was no practical strength difference. At (10, 100), *RP* specimens appeared noticeably stronger than *NP* specimens, whereas at (15, 233) *NP* specimens appeared somewhat stronger than *RP* specimens. The (10, 100) test results were repeated and did not show considerable differences; the original values for *NP* and *RP* were 0.58 and 0.97 kg/cm^2, respectively, whereas the repeat test results were 0.68 and 1.02 kg/cm^2, respectively. No discernable mixing method pattern was observed with *CTS RS* at a 24 hr cure time.

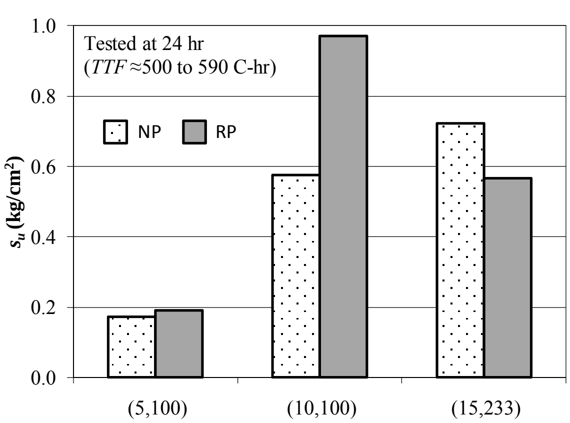
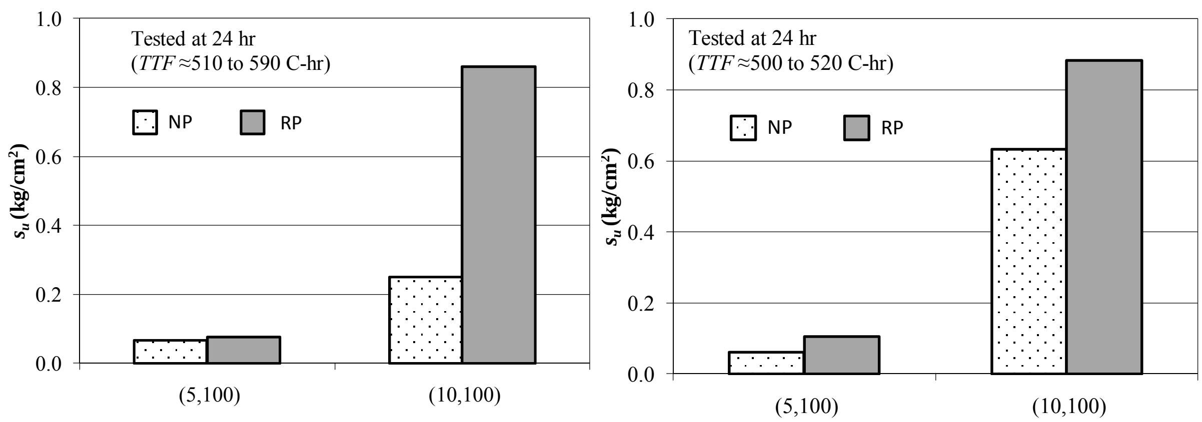


Figure 10.3. CTS RS Mixing Method Results With Soil 1

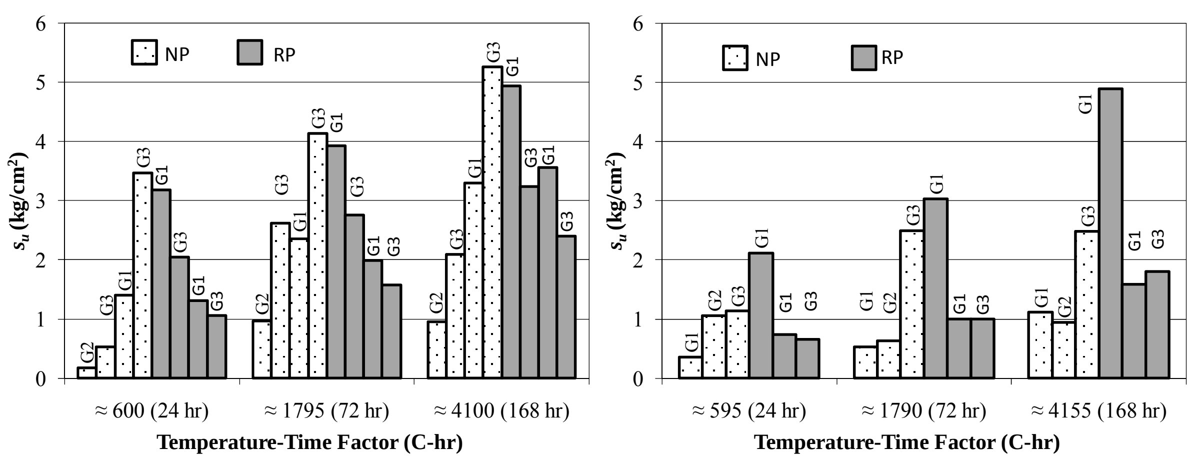
Figure 10.4 plots *SC3* and *SC4* results at (5, 100) and (10, 100) employing both preparation protocols after curing at room temperature for 24 hr using the average of triplicate tests. One outlier was removed from the *RP* data of *SC3* at (10, 100). Shear strength was higher with the *RP* protocol in all instances. At (5, 100) the difference between preparation methods was of no practical significance. At (10, 100) the difference in preparation methods was apparent but not dramatic for *SC4* and significant for *SC3*. For *SC3*, rapid preparation increased strength by a factor of 3.4 when 0.25 kg/cm^2 was used as shown in Figure 10.4 and Table 10.2; if the average of all values indicated in the Table 10.2 notes is used, the increase factor reduces to 2.1, which is still a considerable difference.



(a) SC3 *(b) SC4*

Figure 10.4. SC3 and SC4 Mixing Method Results With Soil 1 at (5,100) and (10, 100)

Figure 10.5 plots *SC3* and *SC4* results with *Soil 1* at (15, 233) and both preparation protocols tested at 3 curing intervals while reporting the average of 3 or 4 replicate tests. The *Soil 1* group is indicated above each test result (e.g. G1 refers to *Group 1*). Figure 10.5 indicates *RP* specimens are stronger than *NP* specimens in an overall sense, but not necessarily for any 1 suite; Figure 10.5 provides results of 14 suites. The data does not provide any evidence that the *Soil 1* group had a predictable effect on *CS* cements as it did on portland cements. Strength behaviors were intermingled within groups.



(a) SC3 *(b) SC4*

Figure 10.5. SC3 and SC4 Mixing Method Results with Soil 1 at (15, 233)

There is considerable scatter in the data, especially with the *NP* protocol, indicating that the material is sensitive to time and handling in the first few minutes after cement is added. The *RP* protocol does not appear to be nearly as sensitive and did not see any results where shear strength was considerably lower than the rest of the data. The *NP* protocol produced very low relative strength in some cases, probably when mixing times were extended to the upper limits of those allowed by the protocol.

Table 10.4 combines the 3 or 4 suites of each preparation method and cement type and provides average strengths representing 9 to 12 specimens. The *RP* protocol produced specimens 1.02 to 1.83 times stronger on average than the *NP* protocol. Specimen preparation method results with *Soil 1* provide indication that the properties presented in Table 10.2 and 10.3 can be further improved in a disaster environment for applications where the soil can be mixed quickly and placed in its final location for curing.

Table 10.4. Combined *SC3* and *SC4* Mixing Method Results with *Soil 1* at (15, 233)

Cement	Protocol	24 hr s_u (kg/cm^2)	72 hr s_u (kg/cm^2)	168 hr s_u (kg/cm^2)
SC3	*NP*	1.39	2.51	2.89
	RP	1.90	2.56	3.53
SC4	*NP*	0.84	1.22	1.51
	RP	1.17	1.68	2.76

10.5.2 *Soil 2* Specimen Preparation Results

Soil 2 was available in limited quantities to perform testing of *CS* cements. Figure 10.6 plots the only specimen preparation method comparison performed with *Soil 2*. The *RP* protocol produced higher strengths in all cases, with considerably higher strengths with *SC4*. When compared to *Type III* portland cement properties, the *RP* strengths are on the upper end of the Table 10.2 range and exceed it slightly in some cases. The most any data point exceeds the Table 10.2 range is 14%, which occurs for *SC3* at 24 hr curing. The same condition, though, is 67% of the upper end portland cement value at 168 hr curing. The precision of the data collected provides no evidence that *SC3* or *SC4* would outperform portland cement even when prepared according to the *RP* protocol, though their performance is comparable to portland cement in these conditions.

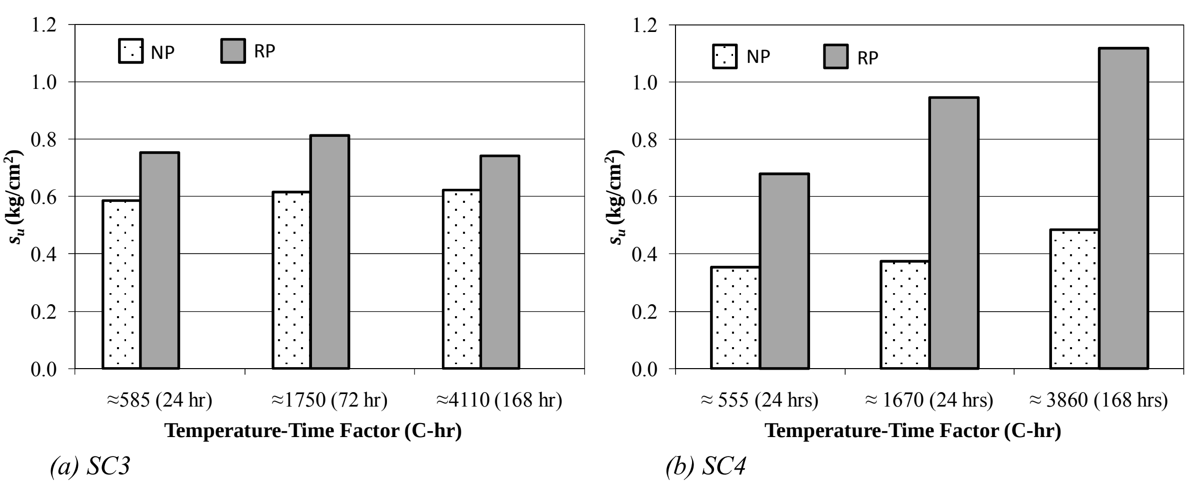


Figure 10.6. Soil 2 Mixing Effect (15, 233)

10.5.3 *Soil 3* Specimen Preparation Results

Figure 10.7 compares the *NP* and *RP* methods with *CTS RS* for *Soil 3* at 3 moisture content and dosage rate combinations at 24 hr curing. Results show the *RP* protocol decreased in strength relative to the *NP* protocol; this occurred 1 time in *Soil 1* with *CTS RS* as well. With either mixing protocol, strength was comparable to below portland cement for the same test conditions.

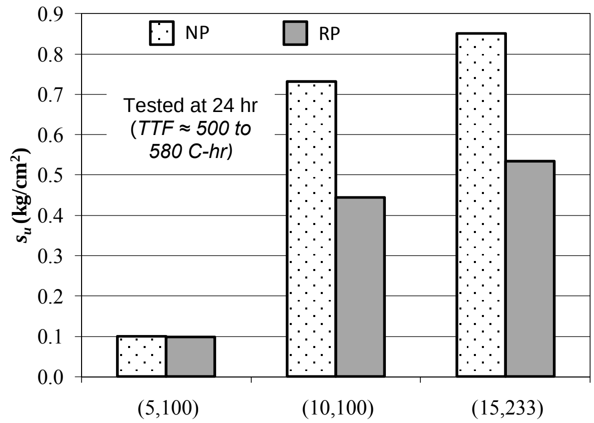
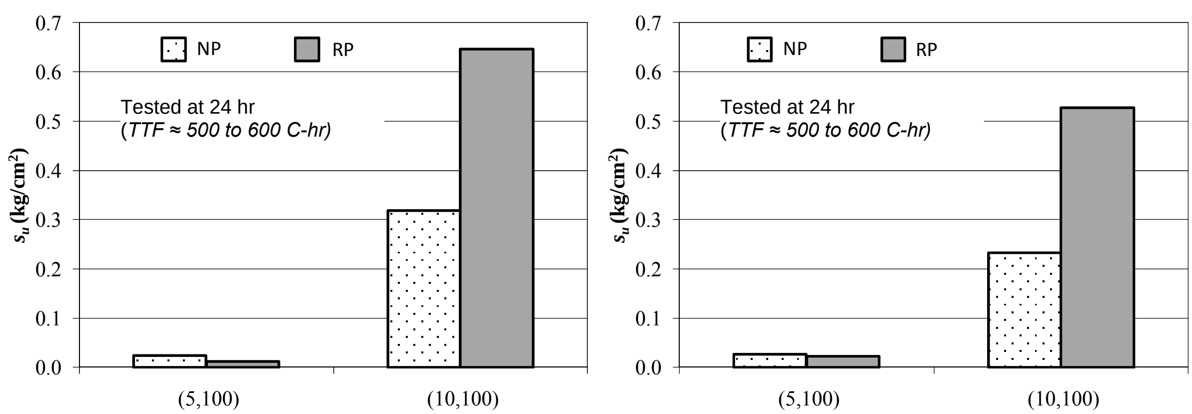


Figure 10.7. CTS RS Mixing Method Results With Soil 3

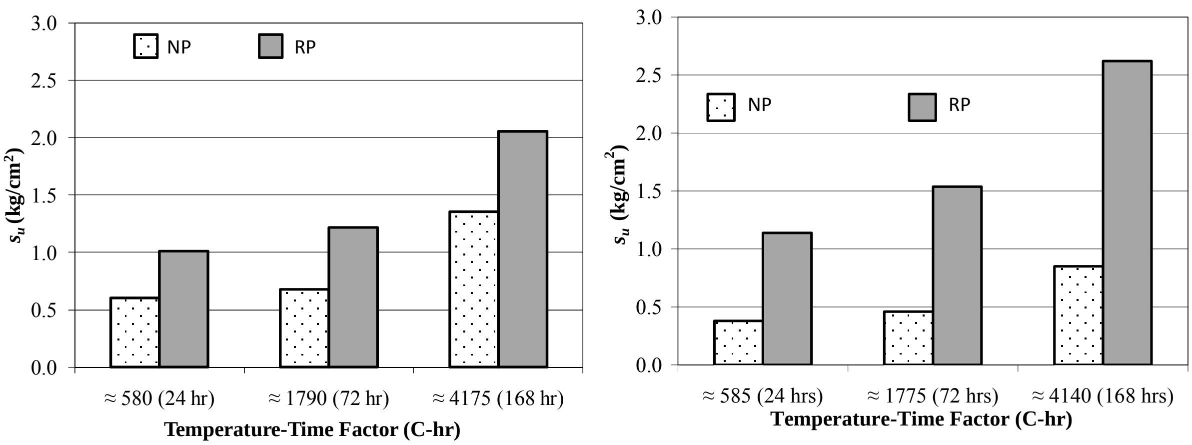
Figure 10.8 plots *SC3* and *SC4* results at (5, 100) and (10, 100) employing both preparation protocols and tested at 24 hr. There was no practical difference at (5, 100) as strength was near 0 for both *NP* and *RP* protocols. At (10, 100) *RP* specimens were on the order of twice as strong as *NP* specimens. *RP* specimens, though, were still considerably weaker than *Type III* portland cement for the same conditions. Additional *RP* testing at 72 and 168 hr with *SC3* and *SC4* at (10, 100) produced strengths of 0.69 and 0.85 kg/cm^2 for *SC3* and 0.55 and 0.73 kg/cm^2 for *SC4*. The later age strengths for *RP* specimens remained considerably lower than *Type III* portland cement.



(a) SC3 *(b) SC4*

Figure 10.8. SC3 and SC4 Mixing Method Results With Soil 3 at (5, 100) and (10, 100)

Figure 10.9 plots *SC3* and *SC4* results at (15, 233) employing both preparation methods. The *RP* specimen preparation method produced stronger specimens for both cements at all curing times. With *SC3*, *RP* to *NP* ratios were 1.7, 1.8, and 1.5 at 24, 72, and 168 hr, respectively. With *SC4*, *RP* to *NP* ratios were 3.0, 3.4, and 3.1 at 24, 72, and 168 hr, respectively. When prepared according to the *RP* protocol, *Soil 3* at (15, 233) was comparable to the lower end of the portland cement strengths for *Soil 3* at (10, 100) at 24 and 168 hr and moderately lower at 72 hr. The amount of strength increase for *Soil 3* when using the *RP* protocol at (15, 233) made it considerably stronger than *Type III* portland cement at (15, 233). *Soil 3* (15, 233) *RP* specimens incorporating *SC3* were still noticeably weaker than (15, 233) *Soil 1* specimens, whereas *SC4* specimens were only slightly different. Interestingly, *SC3* was the better performer for *RP* specimens in *Soil 1*, while *SC4* was the better performer for *RP* specimens in *Soil 3*. *Soil 3* has a higher *LL* than *Soil 1*, and *SC4* experienced less volume change for a given *w/c* ratio in Figure 10.2; so, from the perspective of moisture encapsulation improving strength gain, the relative performance of the cements should have been reversed for these conditions.



(a) SC3 *(b) SC4*

Figure 10.9. Soil 3 Mixing Effect (15, 233)

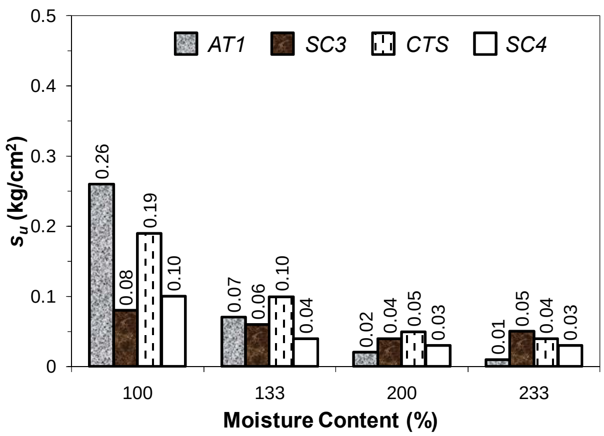
10.6 *Soil 1* Strength Comparison of Portland and Calcium Sulfoaluminate Cements

An expanded experiment was performed using *Soil 1-Group 3* using only the *RP* specimen preparation method at 24 hr curing. *Soil 1* was selected over *Soil 3* since it produced higher strengths with the *NP* protocol and had comparable to higher strengths with the *RP* protocol. *Soil 2* did not perform at a level warranting further consideration with *CS* cements. Four moisture contents, 4 cements, and 4 dosage rates were tested in triplicate after 24 hr underwater curing at room temperature in a full-factorial experiment. Test results are provided in Figure 10.10.

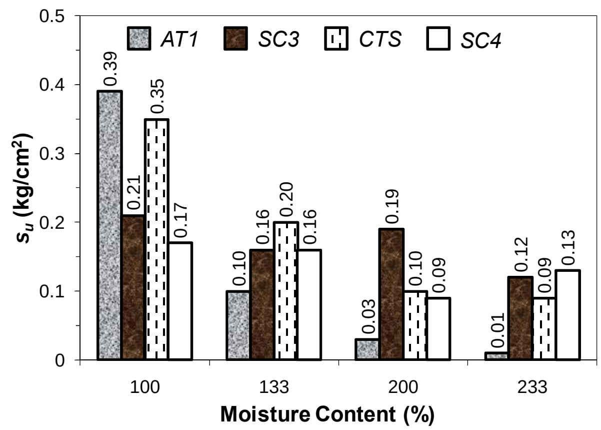
At 5 and 6.7% dosage rates, strengths were low with *CS* cements; portland cement was stronger at 100% moisture which was the only combination where meaningful strength was obtained. At 5% cement and 100% moisture, *Type III* cement produced shear strengths of 0.53 to 0.80 kg/cm^2 (Table 10.2) after 24 hr curing, which is considerably higher than any of the *CS* cements in this condition. At 6.7% cement and moisture exceeding 100% *CS* cements performed noticeably better than portland cements, but the magnitude of strength produced would not be of use for most applications.

At 10% cement dosage rate and 100% moisture, *Type I* portland cement was approximately 2/3rd the strength of *CS* cement while *Type III* cement (Table 10.2) was at to 1.5 times stronger than *CS* cement. *SC3* maintained strength better than other products in Figure 10.10c. No *CS* cement at 10% dosage exceeded 1 kg/cm^2; so, considering cost and rapid handling requirements there does not appear to be any advantage to using *CS* cement at or below 10% dosage except for very rare cases where strength gain is needed at over 100% moisture and 10% dosage needs to be used due to cement availability (or equivalent).

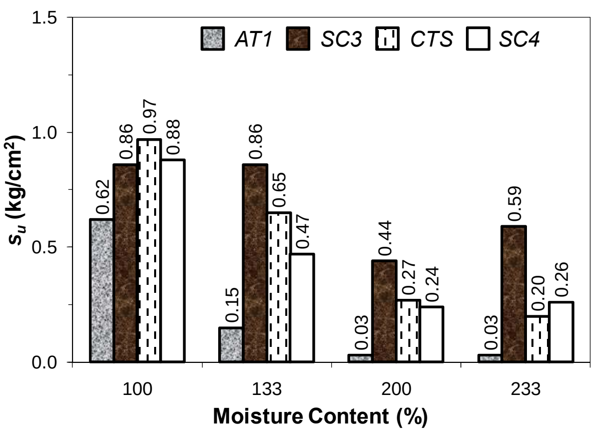
At 20% cement dosage rate *CS* cement was noticeably stronger than portland cement in all cases. The extent *CS* cement strength exceeded portland cement strength increased with moisture content. Portland cement strength steadily decreased with moisture content increase, while *CS* strength either decreased at a slower rate (*CTS RS* and *SC4*) or stayed relatively consistent (*SC3*). *SC3* was the best performing cement as it had the highest shear strength and was able to maintain a reasonably consistent strength over a wide range of moisture contents. *SC4* was more stable than *CTS RS* as the moisture content increased. Moisture contents of 133% or higher and high cement dosage (15 to 20%) in conjunction with a relatively low organic soil with moderate liquid limit (e.g. 45 to 65) appears to be a good application for *CS* cements.



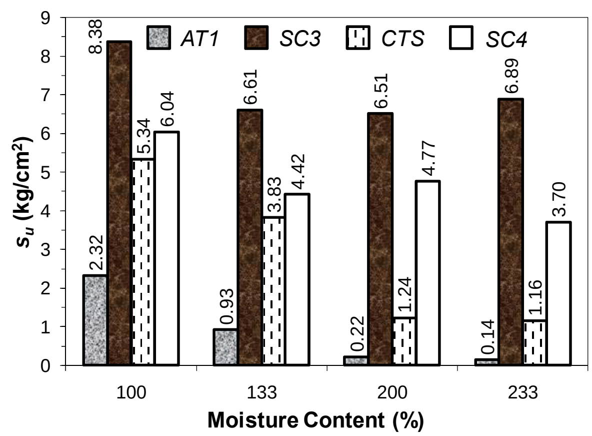
(a) 5% Dosage Rate



(b) 6.7% Dosage Rate



(c) 10% Dosage Rate



(d) 20% Dosage Rate

Figure 10.10. Results of Expanded Soil 1 Experiment After 24 Hr Curing

Figure 10.11 plots test results with only *SC3* cement as it was the best performer in Figure 10.10. *Soil 1* was tested at 24 hr curing at 5 cement dosage rates and 5 moisture contents. The majority of the Figure 10.11 data was taken from Figure 10.10; testing at 15% cement with multiple moisture contents and testing at 167% moisture and multiple cement contents were performed specifically for Figure 10.11.

Figure 10.11 agrees with the findings of Figure 10.10 that cement contents of 15 to 20% appear to be the optimal range for *SC3*. The mechanism leading to very low strength at 5% cement and very high strength at 20% with respect to portland cement could be due to interaction with the soil. At lower dosages the soil may be inhibiting reactive behaviors, but as the dosage increases the cement properties are likely governing behavior. The ability to maintain considerable strength in the presence of large moisture content increases is likely due in part to the ability to encapsulate moisture (Figure 10.2a). It is also likely that cement chemistry affects the behavior as well as *SC3* did not encapsulate as much moisture as *SC4* but was stronger in *Soil 1* yet weaker in *Soil 3*. *Soil 1* has a lower liquid limit than *Soil 3* indicating it does not take on as much water prior to changing state. *SC3* had the highest Blaine fineness of any cement evaluated.

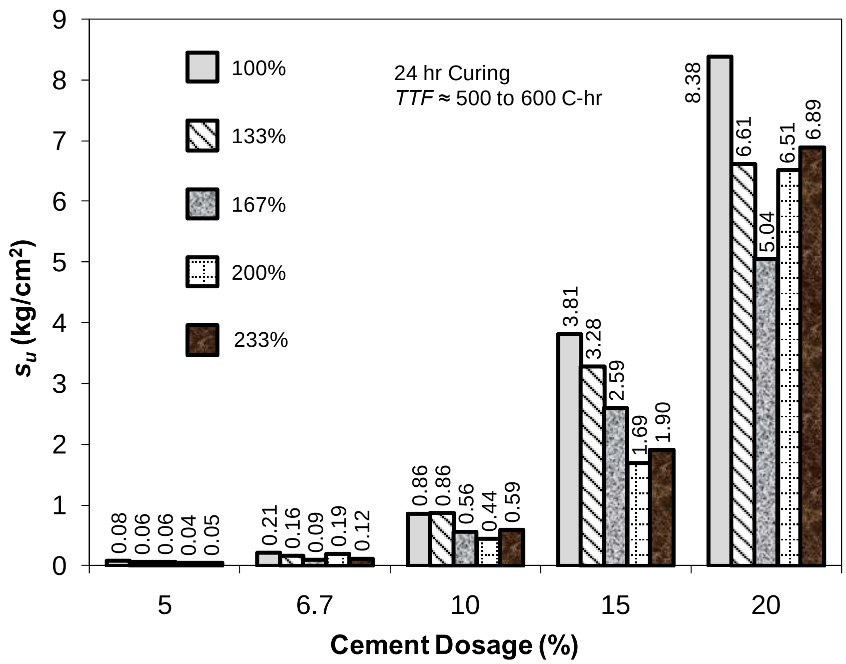


Figure 10.11. SC3 Strength Results Over a Range of Conditions with Soil 1

10.7 Elastic Modulus and Ductility Results of Calcium Sulfoaluminate Cements

Figure 10.12 plots elastic modulus to shear strength correlations for the expanded *Soil 1* experiment shown in Figure 10.10. Portland cement had a slightly higher slope than the *CS* cements. There was not a substantial enough difference in the slopes of all 4 cements to allow relative quantification between *CS* and portland cements with only 48 data points per cement.

Maximum strain values were similar between portland and calcium cements in Figure 10.12. Figure 9.20 plots *Soil 1* behaviors with portland cement where significantly more data was available with respect to Figure 10.12a. Portland cement maximum strain values in Figure 9.20 were 1.5 to 2.3%, and all values in Figure 10.12 fell into this range.

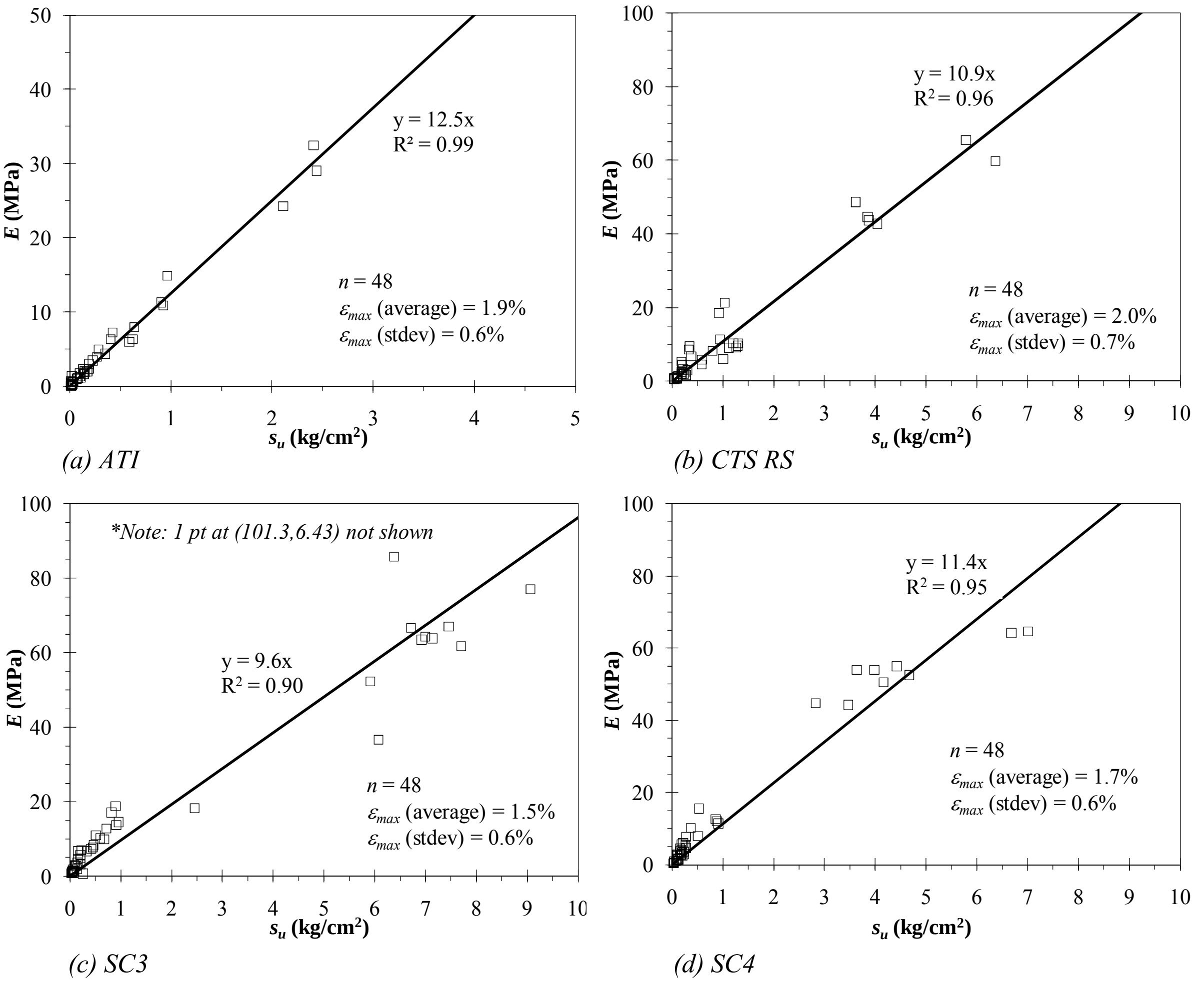


Figure 10.12. Elastic Modulus Results of Expanded Soil 1 Experiment

To further investigate modulus behavior, *Soils 1* to *3* stabilized with *SC3* or *SC4* at (15, 233) were compared to slopes of portland cement at (15, 233) shown in Figures 9.20 to 9.22. *SC3* and *SC4* results are plotted in Figure 10.13; *SC3* and *SC4* data already plotted in Figure 10.12 was not re-used. A (15, 233) blend was investigated since more data was available and dosages of 10% or lower with *CS* cements do not appear valuable. *SC3* and *SC4* data was combined after observing no noticeable differences in Figure 10.12.

Soil 1 behaved in a similar manner to Figure 10.12 and Figure 9.20e and 9.20f. The maximum strain of 1.5% was slightly lower than Figure 9.20e and 9.20f, which is not surprising with a rapid set cement used in Figure 10.13. A modulus to shear strength slope of 12.2 was slightly below the portland cement plots of Figure 9.20e and 9.20f but well within the band of data. Overall, no meaningful difference in modulus to shear strength behavior was observed for *CS* cements relative to portland cements in *Soil 1*.

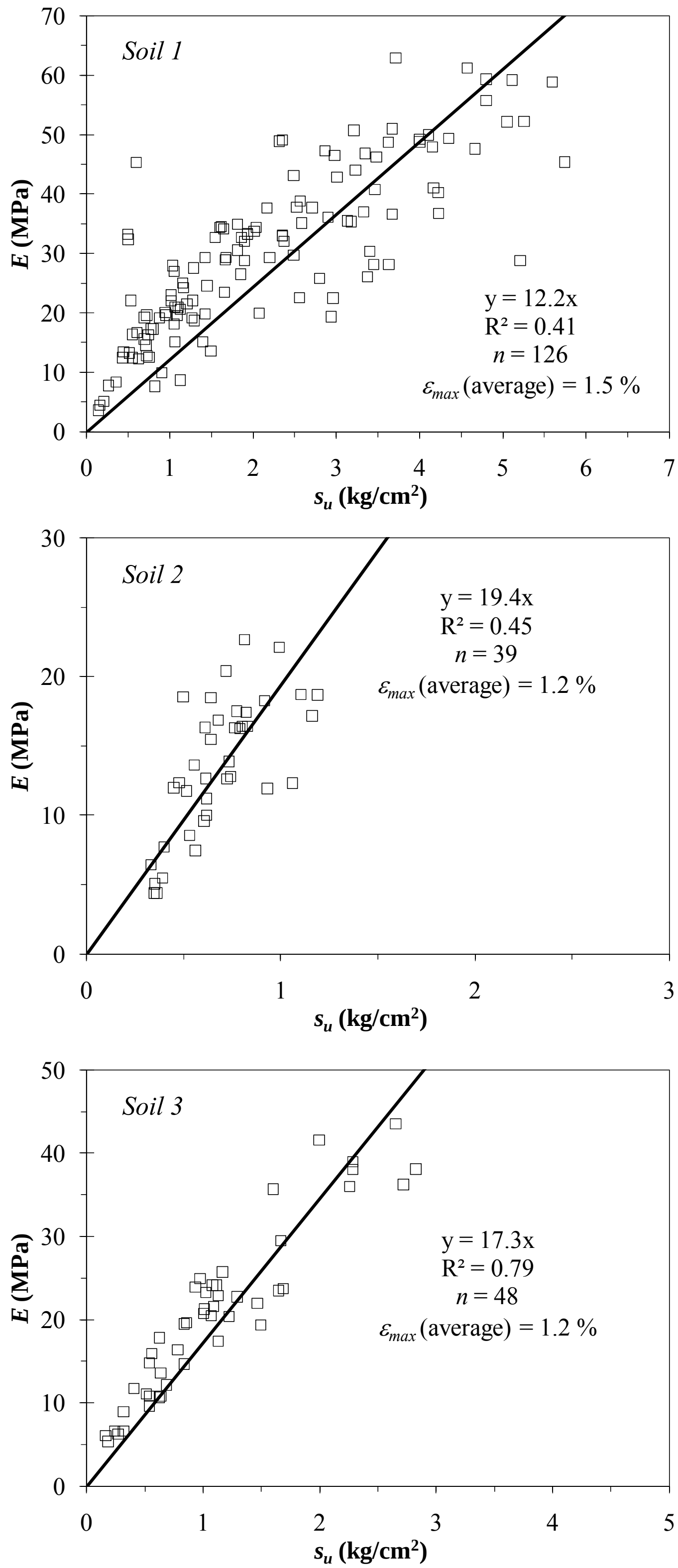


Figure 10.13. Elastic Modulus to Shear Strength Correlation of CS Cements at (15, 233)

Soil 2 behaved very differently in terms of modulus between *CS* and portland cement. The slope of modulus to strength was approximately 10 (Figure 9.21e and 9.21f) with portland cement, while the slope was approximately 20 with *CS* cement (Figure 10.13). The maximum strain was approximately halved when *CS* cement was used instead of portland cement (1.2% versus approximately 2.4%). The data indicates the materials became stiffer but not stronger. In *Soil 2*, *CS* cement was not able to exceed the strength of portland cement by any meaningful amount at (15, 233).

Soil 3 behaved very differently in terms of modulus between *CS* and portland cement. Portland cement (Figure 9.22e and 9.22f) had a modulus to shear strength slope of 7.8 when Artesia and Theodore cements were averaged, which is less than half the slope for *CS* cement under the same conditions (Figure 10.13). The maximum strain with *CS* cement was also less than half of portland cement (1.2% versus approximately 2.6%). The data indicates the materials became stiffer and stronger. In *Soil 3*, *CS* cement was able to exceed the strength of portland cement by a meaningful amount at (15, 233).

10.8 Summary of Calcium Sulfoaluminate Cement Test Results

CS cements were able to encapsulate more water than portland cements as evidenced by volume change measurements. *SC4* changed volume the least followed by *SC3*; both changed volume quite a bit less than *CTS RS*, which was an intermediate performer as portland cement easily changed volume the most. Preparation time affected strength; the *NP* protocol had all specimens in the mold within 35 minutes whereas the *RP* protocol was within 20 minutes. *CS* cement blends were affected by handling time within the first few minutes after cement addition. Table 10.5 summarizes *NP* versus *RP* behaviors.

Table 10.5. Summary of Preparation Method Test Results

Cement	*Soil 1*	*Soil 2*	*Soil 3*
CTS RS	No pattern	---	*NP* ≈1.4 times > *RP*
SC3	*RP* ≈1.2 times > *NP*	*RP* ≈1.3 times > *NP*	*RP* ≈1.7 times > *NP*
SC4	*RP* ≈1.5 times > *NP*	*RP* ≈2.3 times > *NP*	*RP* ≈3.2 times > *NP*

A distinct set of conditions existed where portland cement was the appropriate choice, and another distinct set of conditions existed where *CS* cement was the appropriate choice. With exception of very rare and isolated instances, there is no advantage to using *CS* cement below 10% dosage and portland cement should be used. Moisture contents of 133% or higher and high cement dosage (15 to 20%) in conjunction with a relatively low organic soil with moderate liquid limit (e.g. 45 to 65) appears to be a good application for *CS* cements. *Soil 1* would be a good candidate for *CS* cements under high dosage and moisture conditions. Interestingly, the elastic modulus to shear strength relationship was similar between *CS* and portland cement in *Soil 1*, whereas in *Soil 2* and *Soil 3* the slope was approximately twice as steep for *CS* cement than for portland cement.

The 3 *CS* cements tested performed differently depending on soil type indicating some interaction with the differing cement chemistries. A few differing *CS* products should be tested with soil from the disaster area to select the best performer; this can be performed on site in an expedient manner. *SC3* performed best with *Soil 1*, while *CTS RS* and *SC4* performed best with *Soil 3*. Use of *CS* cements with highly organic soils (e.g. *Soil 2*) is not recommended based on the data collected and presented in this chapter.

CHAPTER 11 – BLENDED CEMENTS TEST RESULTS

11.1 Overview of Blended Cements

Blending cements was performed for 2 purposes. The first was to further investigate the effect of SO_3 content on shear strength. SO_3 effects were investigated by adding Plaster of Paris to adjust the total SO_3. The second purpose of blending cements was to investigate properties of blending ground-granulated blast furnace slag (*GGBFS*) and portland cement since these blends are being used successfully in some more conventional applications.

11.2 Blending Portland Cement and Plaster of Paris

A key characteristic of the specialty portland cements was reduced sulfates. calorimetry testing discussed in Chapter 14 provids evidence that *SC2* had a sulfate content that had been reduced too much. Plaster of Paris (*PoP*) was added to increase the sulfate content. *PoP* would not provide the same form of sulfates but did provide a relatively efficient method to investigate SO_3 behavior. An SO_3 increase of 2% was desired based on calorimetry testing described in Chapter 14, which required *PoP* at 0.28% of slurry weight. One specialty cement from each plant (*SC2* and *SC5*) was tested to produce additional information.

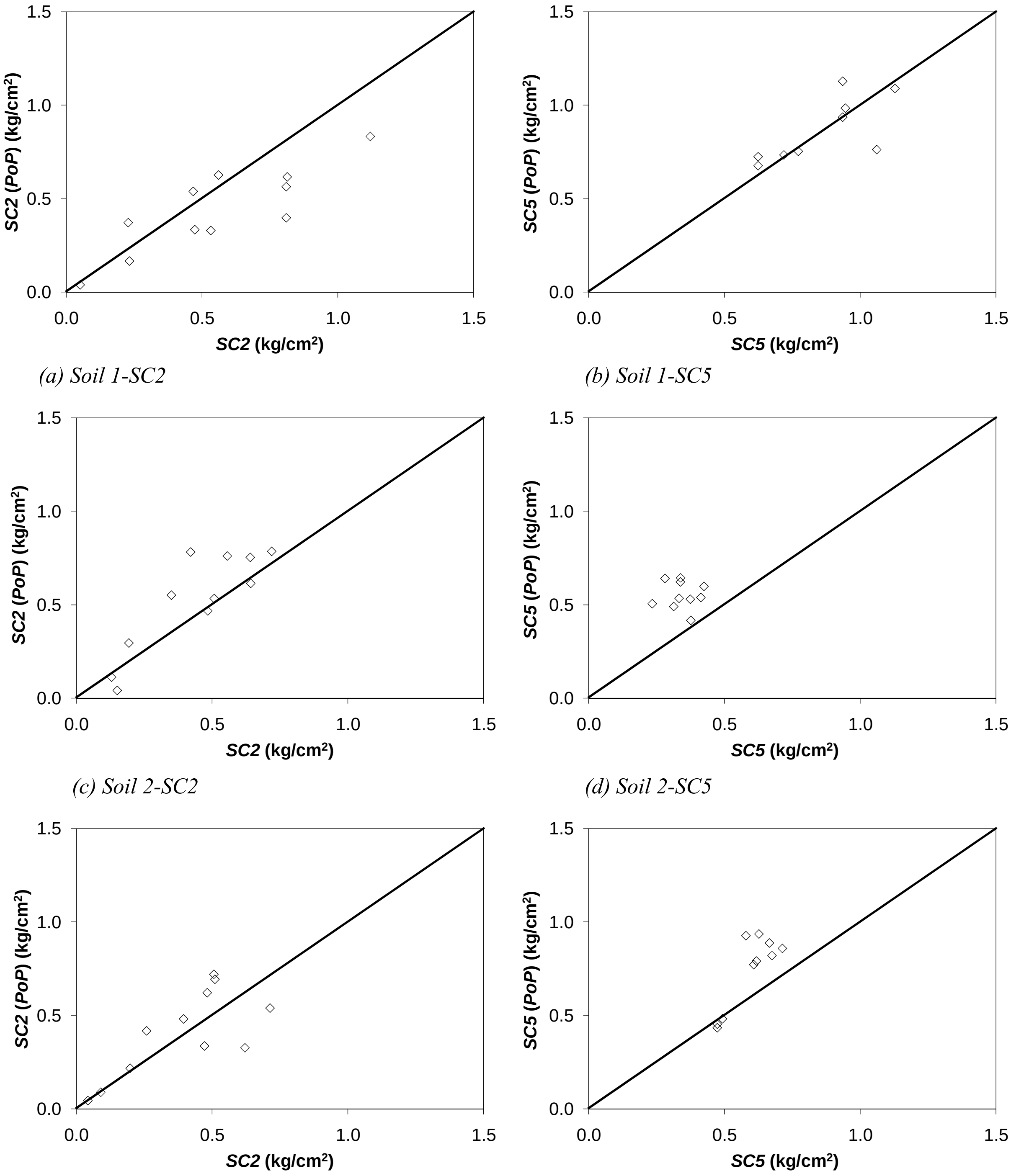
11.2.1 *UC* Results of Blending Portland Cement and Plaster of Paris

Figure 11.1 plots results of the 6 suites conducted where portland cement was blended with Plaster of Paris (*PoP*) and compares them to control suites where only portland cement was used. Overall, there was no clear pattern in terms of relative *PoP* effect as data fell on both sides of the equality line. Test results were relatively close to the line of equality in the majority of instances indicating any effects of the *PoP* were not large. Some of the scatter around the equality line could be explained by test variability observed in the rest of this report.

SC2 may have been slightly stronger without the *PoP* in *Soil 1* as the majority of the data fell below the equality line. The reverse was observed in *Soil 2* as the majority of the data fell above the equality line with *SC2*. Results from *SC2* were mixed in *Soil 3* as data was fairly evenly distributed on either side of the equality line. There were no substantial differences in shear strength in any of the soils based on the addition of *PoP* that were consistent over several measurements. With *SC2* there was a very slight trend of positive *PoP* effects dissipating as the organic content decreased.

Soil 1 results where *SC5* was used were close to the equality line in most instances indicating no measurable effect of *PoP*. *Soil 2* with *SC5* indicated *PoP* increased shear strength as all data fell above the equality line. What is interesting about this case (Figure 11.1d) is that all data was grouped together even though a range of curing was used to develop the data. It appears that peak strength was developed early with this case and remained essentially constant with additional curing. It is possible that under these conditions some of the apparent effect of *PoP* could be due to random variability in the materials. More suites with and without *PoP* would have to be conducted to make more definitive statements. *Soil 3* with *SC5* appeared to be somewhat improved by addition of

PoP at higher strengths, while at lower strengths the values were very near the equality line. With *SC5*, any positive effects of *PoP* appeared to dissipate as the organic content decreased.



(a) Soil 1-SC2

(b) Soil 1-SC5

(c) Soil 2-SC2

(d) Soil 2-SC5

(e) Soil 3-SC2

(f) Soil 3-SC5

Figure 11.1. UC Test Results of Blending Portland Cement and PoP

Interpretation of the data presented in Figure 11.1 should consider that there was 0.28% more additive in the specimens with *PoP* as both types of specimens had 5% portland cement. One set was tested per soil type where 5% *PoP* was the only additive and no strength was produced in those mixes indicating *PoP* alone does not produce shear strength in soil slurries. One additional set was tested per soil type where 4.72% *SC5* was blended with 0.28% *PoP* resulting in a total additive percentage of 5%. These 3 sets were weaker than corresponding test results where 5% *SC5* was used with 0.28% *PoP*, which was expected. The differences observed in Figure 11.1 were on the order of the differences in strength due to removal of 0.28% of the portland cement. The amount of portland cement removed that would be truly comparable to addition of 0.28% *PoP* is unknown, though it would be less than 0.28%. The key point is that no conclusive statements could be made from *UC* testing incorporating *PoP*.

11.2.2 Trial Results of Blending Portland Cement and Plaster of Paris

Figures 11.2 through 11.10 compare blends incorporating *SC2* to those incorporating *SC2* and *PoP*. The data was collected with the 3 hand held gages. Raw data used to generate these plots can be found in Figures A.25 to A.27.

In *Soil 1*, the effect of the *PoP* varied between gages. The *Dial* gage showed the *PoP* to have little (if any) affect on shear strength, the *Ring* gage showed a considerable benefit from the *PoP*, and the *Shear* gage showed a moderate strength reduction from the *PoP*. In *Soil 2*, all 3 gages showed the *PoP* to produce minor increases in shear strength; these apparent increases could be due to material and test variability, though, and not the *PoP*. In *Soil 3*, all 3 gages showed the *PoP* to cause moderate reduction in shear strength. As with *Soil 2*, the changes in shear strength were not large enough to make specific statements as the differences could be largely due to material and test variability. The strength reduction in *Soil 3* was more than the increase in strength in *Soil 2*. *Soil 1* was the only condition where noticeable changes were observed, and the effect of *PoP* was different between gages.

One variability slab was tested incorporating *PoP*; Figure B.19 contains the raw data. The mean value using the *Dial* gage was 1.53 kg/cm^2, whereas the trend line from the trial resulted in 1.13 kg/cm^2 or 74% of the variability slab for the same conditions. *Bottom* readings from the trial resulted in a *Dial* value of 1.42 kg/cm^2, which aligns reasonably well with the variability slab. The mean value using the *Ring* gage was 1.45 kg/cm^2, whereas the trend line from the trial resulted in 1.09 kg/cm^2 or 75% of the variability slab for the same conditions. *Bottom* readings from the trial resulted in a *Ring* value of 1.22 kg/cm^2, which aligns moderately well with the variability slab. The mean value using the *Shear* gage was 0.55 kg/cm^2, whereas the trend line from the trial resulted in 0.57 kg/cm^2 or 104% of the variability slab for the same conditions. *Bottom* readings from the trial resulted in a *Shear* value of 0.78 kg/cm^2, which is considerably higher than the variability slab. Interestingly, the *Dial* and *Ring* gages produced different trends than the *Shear* gage both in magnitude of readings and in behavior between the trial and variability slab.

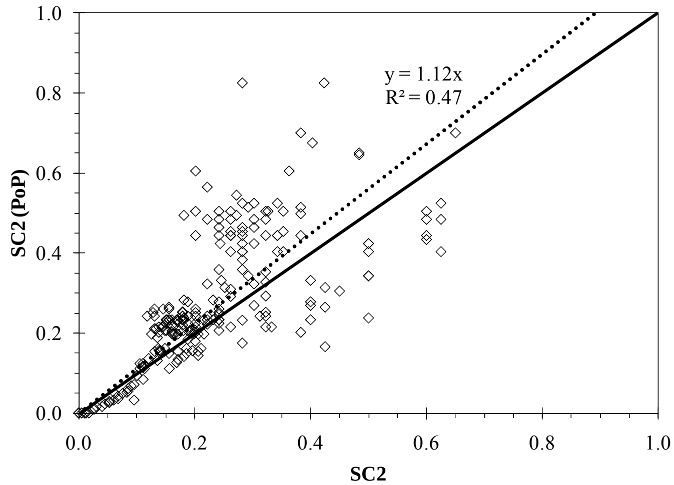


Figure 11.2. Plaster of Paris Effect-Dial Gage-Soil 1

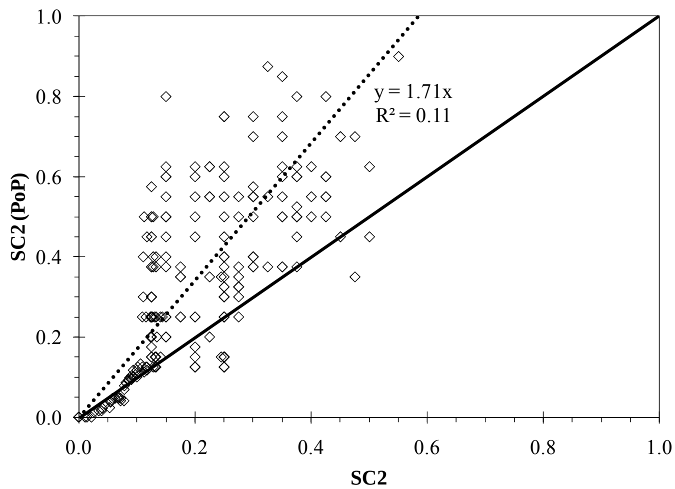


Figure 11.3. Plaster of Paris Effect-Ring Gage-Soil 1

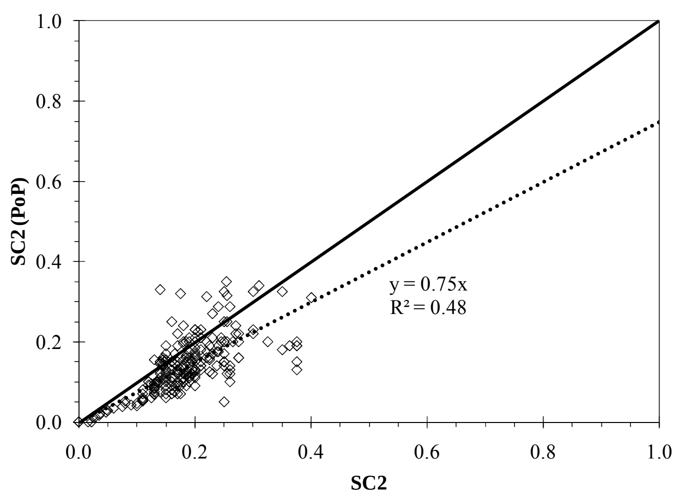


Figure 11.4. Plaster of Paris Effect-Shear Gage-Soil 1

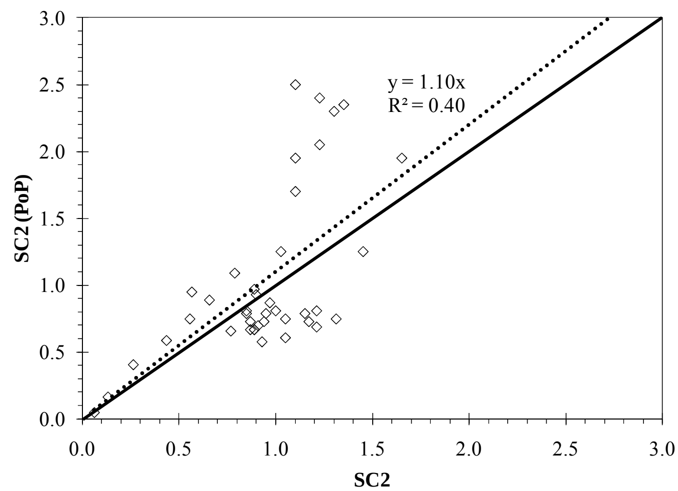


Figure 11.5. Plaster of Paris Effect-Dial Gage-Soil 2

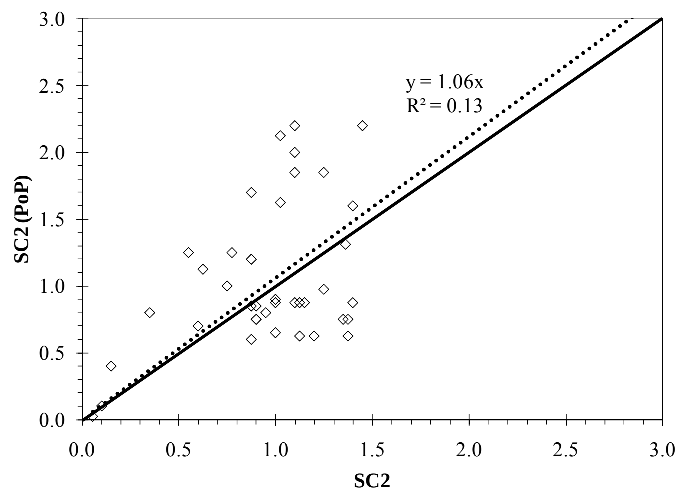


Figure 11.6. Plaster of Paris Effect-Ring Gage-Soil 2

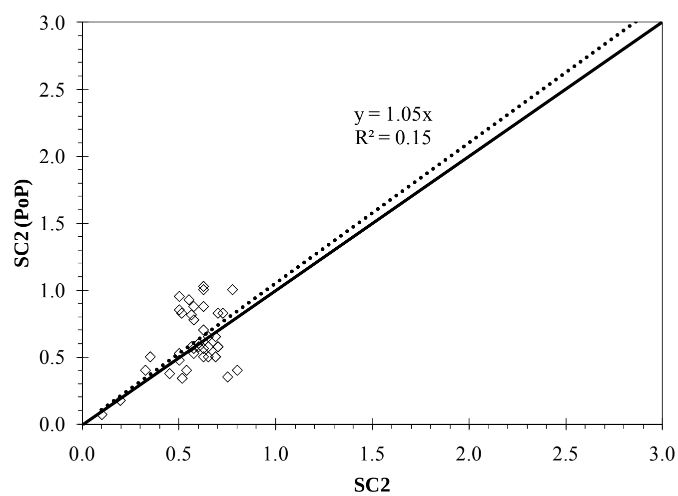


Figure 11.7. Plaster of Paris Effect-Shear Gage-Soil 2

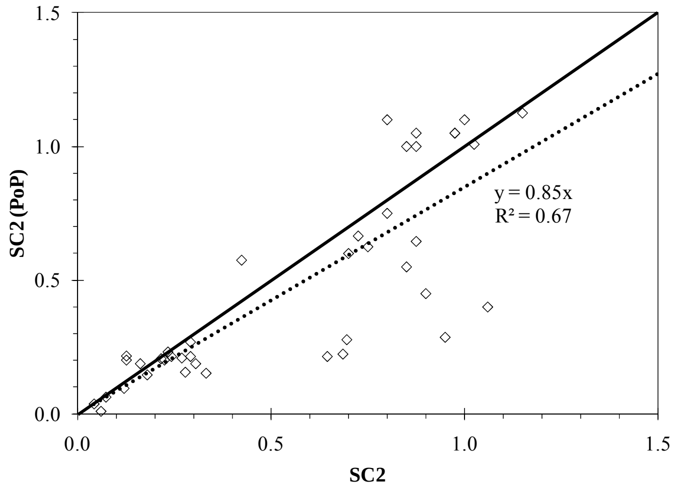


Figure 11.8. Plaster of Paris Effect-Dial Gage-Soil 3

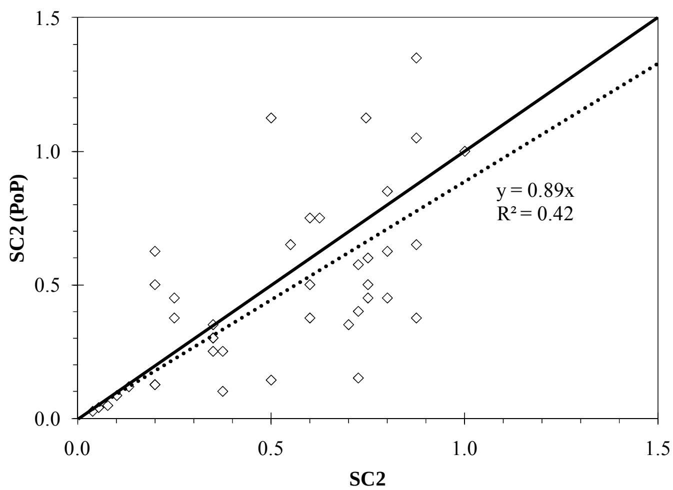


Figure 11.9. Plaster of Paris Effect-Ring Gage-Soil 3

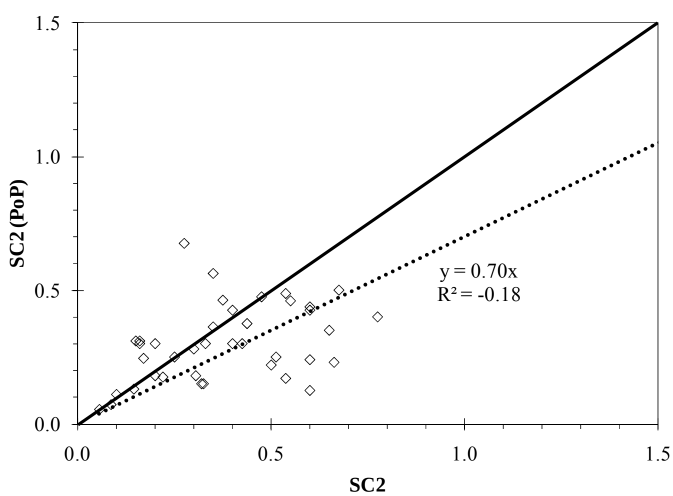


Figure 11.10. Plaster of Paris Effect-Shear Gage-Soil 3

11.3 Blending Portland Cement and *GGBFS*

Blending *GGBFS* and portland cement is becoming fairly common in soil stabilization projects. To investigate performance within the first 7 days with high moisture content soil slurries (all testing was performed at 100% moisture), 17 suites, 3 trials, 1 variability slab, and 1 field sample were tested using *GGBFS*. The majority of the testing was performed with *Soil 1* and *Soil 3,* with moderate testing performed with *Soil 2* and *Soil 4*. Results are presented by soil type in the remainder of this section, with the majority of the unconfined compression testing performed in triplicate.

11.3.1 Results of Blending Portland Cement and *GGBFS* in *Soil 1*

One suite and one trial (Figure A.25) were initially conducted with 3.75% *GGBFS* and 1.25% *A T I* to assess strength gain potential at a modest dosage rate. All hand held gages used in the trial showed no considerable strength gain below a *TTF* of 2,300 C-hr. After 2,300 C-hr, shear strengths were measured between 0.15 to 0.50 kg/cm^2 with considerable scatter. The shear strengths measured with the *UC* suite agreed with the trial in general terms, as shear strength did not exceed 0.2 kg/cm^2 until 2,100 C-hr and peaked at 0.50 kg/cm^2 at 3,600 C-hr. Strength gain in the first few days of curing at room temperature was slower than what occurred when only portland cement was used, and the strength achieved at 7 days curing at room temperature did not exceed what could be provided by portland cement at the same total dosage rate (5%) as the blend.

To further investigate blending *GGBFS* and portland cement with a 5% total dosage rate, *SC6* was used as 100, 75, and 50% of the total cementitious material. Test results are provided in Figure 11.11 in terms of average values. Increasing the amount of the 5% cementitious blend that was portland cement increased strength at the early and intermediate *TTF* values, while at the later *TTF* range of values, strength was practically independent of *SC6* (i.e. portland cement) content. At *TTF* values over 3,500 C-hr blends with *GGBFS* were the same as those with only portland cement. In *Soil 1*, there was no advantage in using *GGBFS* when the total cementitious dosage rate was 5%.

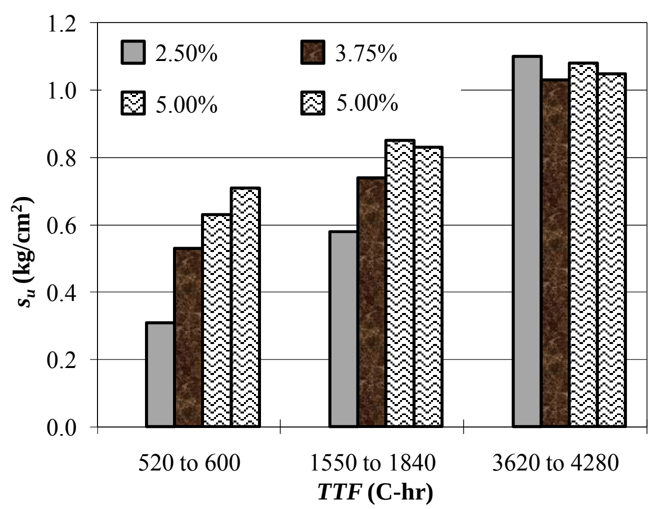


Figure 11.11. Soil 1 Shear Strength Results by SC6 Content: Total Dosage Rate 5%

To determine the effect of *GGBFS* at higher dosages, the total cementitious content was increased to 10% and a similar type of investigation conducted as with the 5% dosage. Test results are provided in Figure 11.12 in terms of average values. At low *TTF* values, the more *SC6* the higher the strength. However, upon curing to the intermediate *TTF* level (1,460 to 1,870 C-hr), the blend with 2.5% *SC6* and 7.5% *GGBFS* was at to slightly stronger than 10% *SC6*. When cured to the highest *TTF* level considered in this research (3,250 to 4,380 C-hr), the blend with 2.5% *SC6* significantly outperformed 10% *SC6*. When the total dosage was 5%, it took 7 days of room temperature curing for *GGBFS* blends to be comparable to only portland cement; whereas, when the total dosage was 10%, it only took 3 days of room temperature curing. Blends with 5% or 7.5% *SC6* performed in an intermediate fashion indicating that if *GGBFS* is to be used with this type of soil, approximately 7.5% of the 10% total blend should be *GGBFS*.

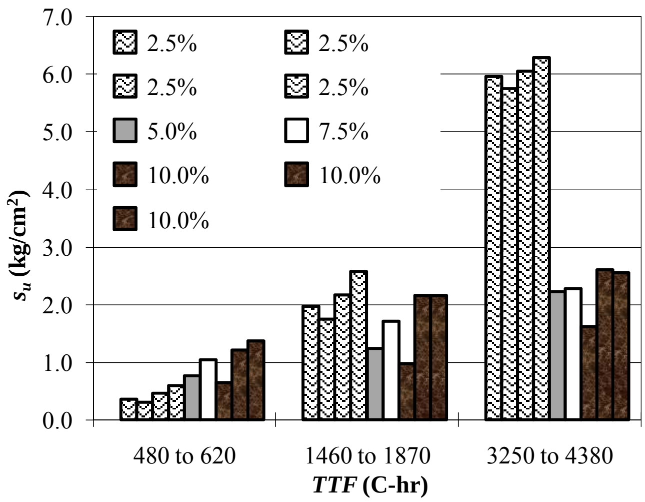
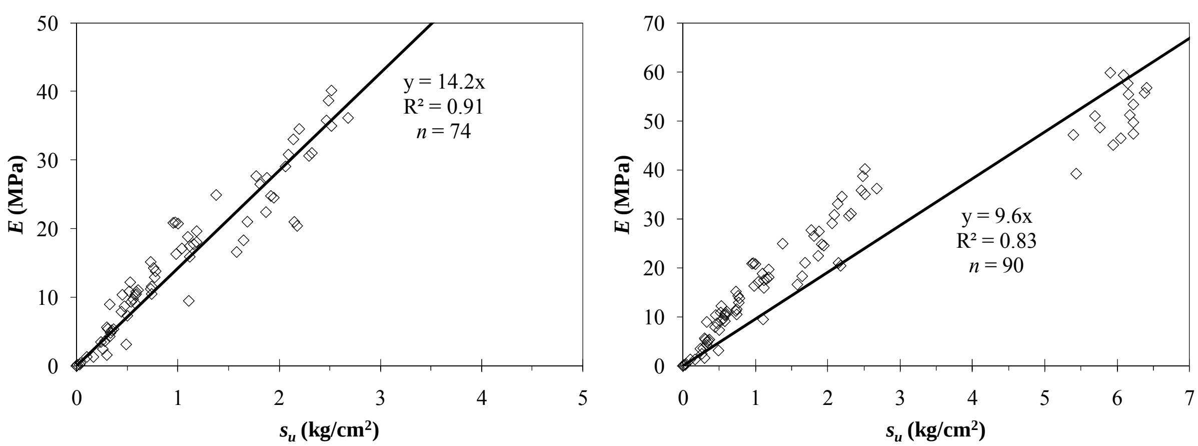


Figure 11.12. Soil 1 Shear Strength Results by SC6 Content: Total Dosage Rate 10%

Figure 11.13 plots the relationship between shear strength and elastic modulus with *Soil 1*. Plotting all data resulted in 2 distinct groups; specimens with shear strengths below 3 kg/cm^2 had a different shear strength to modulus relationship than specimens with shear



(a) High Strength Data Removed *(b) All Data*

Figure 11.13. Correlation of Elastic Modulus and Shear Strength in Soil 1 with GGBFS

strength above 5 kg/cm^2. The slope of the modulus to strength plot was higher when only data below 3 kg/cm^2 was used indicating modulus increases at a lower rate above 5 kg/cm^2 than does strength. Maximum strain was 1.8% on average with a standard deviation of 0.9%. Low strength specimens had higher maximum strains than those with high shear strength.

11.3.2 Results of Blending Portland Cement and *GGBFS* in *Soil 2*

One trial and one suite were conducted with 3.75% *GGBFS* and 1.25% *A T I*. Results were considerably lower than when 5% portland cement was used in the same blend. Most trial shear strength readings were 0.1 to 0.2 kg/cm^2, with an occasional reading exceeding 0.2 kg/cm^2 as seen in Figure A.26. Results from the suite resulted in similar values as the shear strength ranged from 0.07 to 0.18 kg/cm^2 with an average value of 0.13 kg/cm^2.

11.3.3 Results of Blending Portland Cement and *GGBFS* in *Soil 3*

One suite, one trial (Figure A.27), and one variability slab (Figure B.19) were initially conducted with 3.75% *GGBFS* and 1.25% *A T I* to assess strength gain potential at a modest dosage rate. All hand held gages used in the trial and variability slab showed no meaningful strength gain throughout the 7 day curing period. Strength was always less than 0.1 kg/cm^2. *UC* results from the suite supported test results from the hand held gages as the shear strength was always below 0.1 kg/cm^2 and in some instances the specimens were too weak to test. Using 5% portland cement produced specimens that were considerably stronger than those produced with the blend at the same total dosage rate.

To further investigate blending *GGBFS* and portland cement with a 5% total dosage rate, *SC6* was used as 100, 75, and 50% of the total cementitious material. Test results are provided in Figure 11.14 in terms of average values. Increasing the amount of the 5% cementitious blend that was portland cement increased strength at all *TTF* levels. This result differs with that observed in *Soil 1* in the sense that at above 3,500 C-hr the blends with *GGBFS* performed the same as all portland cement.

Figure 11.15 investigates the effect of 10% total cementitious material with *Soil 3* in terms of the *GGBFS* and *SC6* proportions. Increasing the amount of the 10% cementitious blend that was portland cement (i.e. *SC6*) increased strength at all *TTF* levels. This result agrees with the trend observed in *Soil 3* with a 5% dosage rate, but disagrees with the behavior of *Soil 1* with 10% dosage and also disagrees with the relative performance of *Soil 1* between 5 and 10% dosage. The data suggests *GGBFS* was not effective in stabilizing *Soil 3*, which has a higher *LL* and organic content than *Soil 1*. As with *Soil 1*, there was no advantage in using *GGBFS* at an intermediate level.

Figure 11.16 plots the relationship between shear strength and elastic modulus with *Soil 3*. The correlation is fairly strong and is essentially the same as the low strength data from *Soil 1* (Figure 11.13a) as the slopes are 14.2 for *Soil 1* and 14.0 for *Soil 3*. The maximum strain was 4.0% on average with a standard deviation of 2.5%. The average maximum strain of 4.0% is much higher than *Soil 1*, but one possible reason is that 70% of the specimens tested had shear strengths below 0.5%; lower strength specimens have higher maximum strains in other parts of the report. Strains for specimens with shear strength in excess of 0.5 kg/cm^2 were on the order of those observed for *Soil 1*.

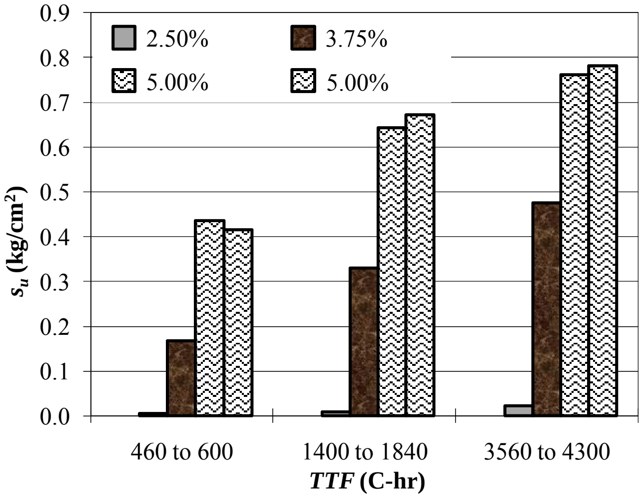


Figure 11.14. Soil 3 Shear Strength Results by SC6 Content: Total Dosage Rate 5%

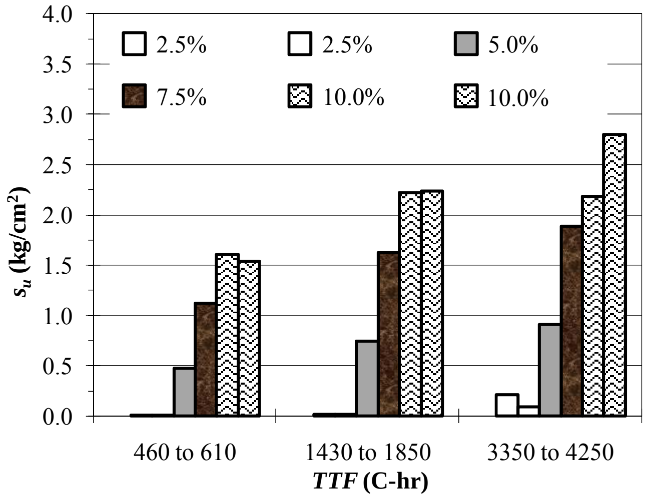


Figure 11.15. Soil 3 Shear Strength Results by SC6 Content: Total Dosage Rate 10%

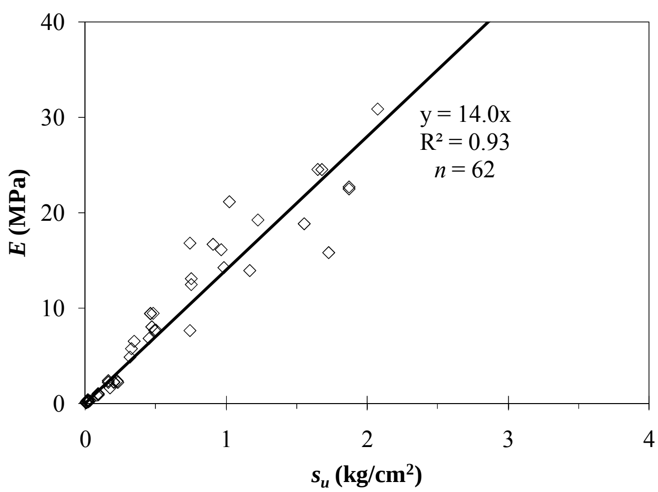


Figure 11.16. Correlation of Elastic Modulus and Shear Strength in Soil 3 with GGBFS

11.3.4 Test Results From the *Inner Harbor Navigation Channel* (*Soil 4*)

The *Inner Harbor Navigation Channel (IHNC) Reach III* project in New Orleans, LA was sampled on May 27, 2009. A random near surface soil sample from the slope (*Soil 4*) was taken at a single location along with a sample of the cementitious material from the cement shuttle (*SB-HB*). The project dosage rate of 75% *GGBFS* and 25% *Type I* portland cement was on the order of 250 kg/m^3.

Boring logs showed a wide variety of soils within the treated area (on the order of 1 to 8 m deep) ranging from coarse grey sand to organic clays. Three day laboratory bench scale shear strengths ranged from 1.94 to 5.11 kg/cm^2, and 7 day shear strengths ranged from 2.92 to 7.75 kg/cm^2. Laboratory dosage rates were 225 to 275 kg/m^3. These properties cannot be directly compared to the properties presented in this section since the soil sample was at the surface, but they do provide some level of reference.

Figure 11.17 provides average *UC* test results; testing was performed in triplicate. Average maximum strain (ε_{max}) values were on the order of 1.7%, and the stabilized slurry (γ_T) had an approximate density of 1.5 to 1.6 g/cm^3 depending on cementitious content, with an average value of 1.54 g/cm^3. Dosage rates used translated to *w/c* ratios of 9.5 and 2.7 and dosages by percent wet slurry of 5 and 17 for *D* values of 75 and 250, respectively. *SC1* performed noticeably better than *SB-HB* at the low dosage rate at 72 hr room temperature curing (*TTF* ≈ 1,600 C-hr), and had the same strength at the low dosage rate at 168 hr room temperature curing (*TTF* ≈ 3,700 C-hr). At the high dosage rate the reverse was true as *SC1* only had 65% of the strength of *SB-HB* at 72 hr and 52% of *SB-HB* at 168 hr. Shear strength (kg/cm^2) and elastic modulus (MPa) were well correlated at s_u = 10.1E (R^2 = 0.96).

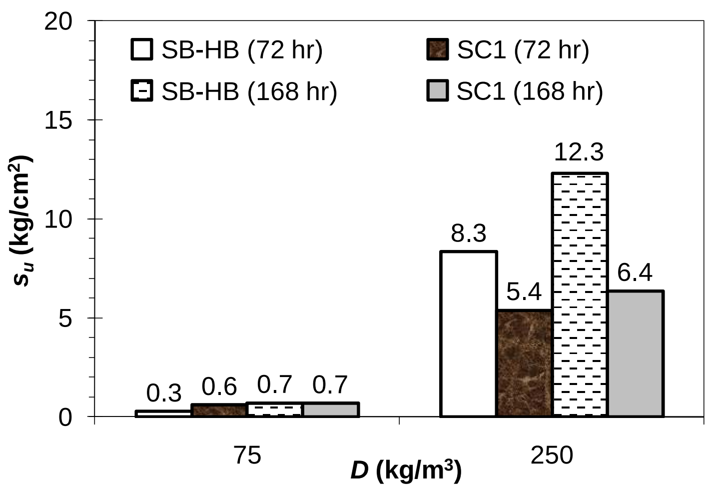


Figure 11.17. Soil 4 Test Results

11.4 Summary of Blended Cement Test Results

Using Plaster of Paris to adjust the SO_3 content of cementitiously stabilized clay slurries did not provide conclusive data in an overall sense. Positive effects were observed in some instances and negative effects in others, with few discernable patterns that could be attributed to SO_3 content. With *SC2*, there was a very slight trend of positive Plaster of Paris effects dissipating as soil organic content decreased.

At a total dosage rate of 5%, there was no advantage to using a blend of *GGBFS* by comparison to using all portland cement. In *Soil 1* and *Soil 4*, the blend of 3.75% *GGBFS* and 1.25% portland cement produced the same strength after 7 days of room temperature curing, but at earlier test times portland cement strengths exceeded that of the blend. *Soil 2* and *Soil 3* were not improved with *GGBFS* during the curing period investigated.

At higher total dosage rates (10% for *Soil 1* and *Soil 3*; 17% for *Soil 4*), blends including *GGBFS* were beneficial in some instances after 3 days of room temperature curing, but never before. After 7 days of room temperature curing, blends with 75% *GGBFS* and 25% portland cement were significantly stronger than only portland cement for *Soil 1* and *Soil 4*, but significantly weaker for *Soil 3*. Interestingly, *Soil 1* and *Soil 4* have similar *LL* (54, 55) and organic contents (5.7, 6.3) while *Soil 3* has a higher *LL* (77) and organic content (10.4). If a *GGBFS* blend is to be used, 75% *GGBFS* and 25% portland cement is the only blend recommended based on the testing performed in this chapter as other proportions provided no advantage for any test time, soil type, or dosage rate.

In *Soil 1*, modulus to shear strength behaviors were different for high shear strength specimens (i.e. over 5 kg/cm^2) than for lower shear strength specimens (i.e. below 3 kg/cm^2). *Soil 3* had a similar relationship between shear strength and modulus as *Soil 1* for shear strengths below 3 kg/cm^2 (slopes of 14.0 and 14.2, respectively). The shear strength to modulus relationship of *Soil 4* was similar to that of *Soil 1* (slopes of 10.1 and 9.6, respectively) when all data was considered; both soils had shear strength values in excess of 5 kg/cm^2.

CHAPTER 12 – INVESTIGATION OF BRACKISH AND SALT WATER EFFECTS

12.1 Brackish and Salt Water Test Results Overview

Sixteen *Protocol 2* suites were conducted with brackish water (3 at (10, 100), 4 at (10, 233), nine at (15, 233)), and nine protocol 2 suites were conducted with salt water at (15, 233). Two sets were conducted with brackish water (15, 233), and 3 sets were conducted with salt water at (15, 233). One trial each was conducted at (15, 233) in brackish and salt water. Fresh water testing was used as a control. The remainder of this chapter analyzes water effects on shear strength and elastic modulus.

12.2 Brackish Water Test Results at (10, 100)

Figure 12.1 compares the fresh water control bands of *A T III* to testing with brackish water. Maximum strain values were 1.7% on average with a standard deviation of 0.3%. *Soil 1* shear strengths with brackish water were below the control band, while *Soil 2* and *Soil 3* were, in general, within the control band. A reasonable correlation was obtained between shear strength and elastic modulus when all soils were incorporated.

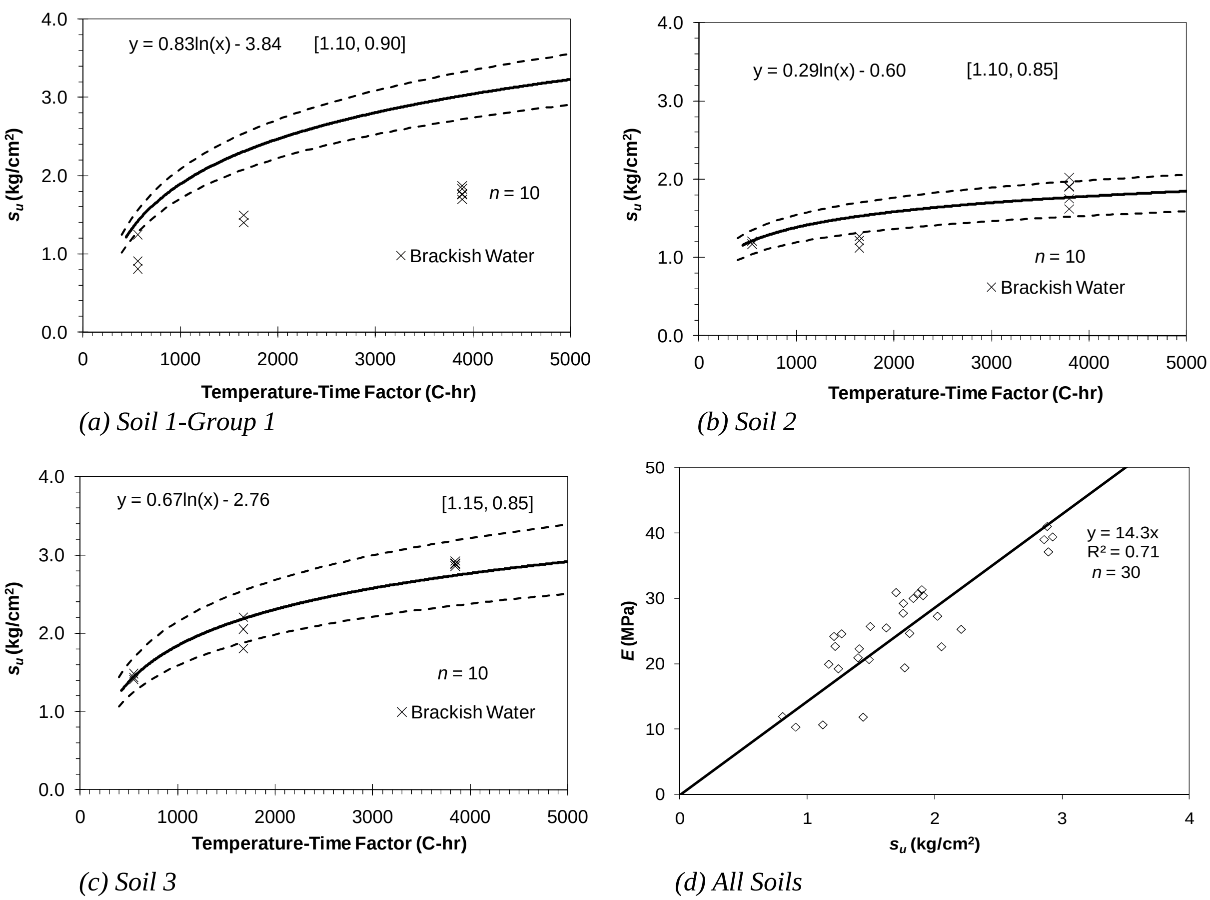


Figure 12.1. Effect of Brackish Water at (10, 100) with A T III

12.3 Brackish Water Test Results at (10, 233)

Table 12.1 provides average test results for all (10, 233) testing incorporating brackish water. A fresh water equivalent was not performed, though testing incorporating *Th T III* at (10, 233) was performed. Test results provided no readily apparent evidence that brackish or salt water negatively affected shear strength.

Table 12.1. Brackish Water Test Results at (10, 233)

Soil	**1**	**1**	**2**	**3**
Cement	***SC1***	***A T III***	***A T III***	***A T III***
***TTF* (C-hr)**	**s_u (kg/cm^2)**	**s_u (kg/cm^2)**	**s_u (kg/cm^2)**	**s_u (kg/cm^2)**
550	0.03	0.02	0.13	0.06
1665	0.06	0.05	0.17	0.12
3800	0.08	0.06	0.18	0.17

12.4 Brackish and Salt Water Test Results at (15, 233)

12.4.1 *UC* Brackish and Salt Water Test Results at (15, 233)

Figure 12.2 indicates that brackish and salt water are not a deterrent to strength development for *Soil 1* at (15, 233). For both cements salt water produces higher strength than brackish water, which has a higher strength than fresh water. All material was sampled at the same time to reduce variability observed with *Soil 1* in other parts of the report.

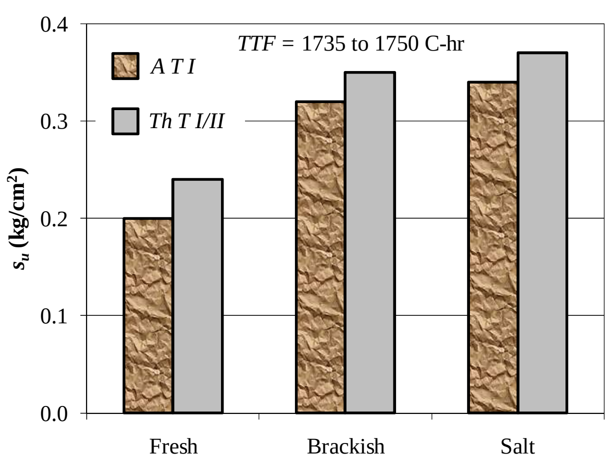
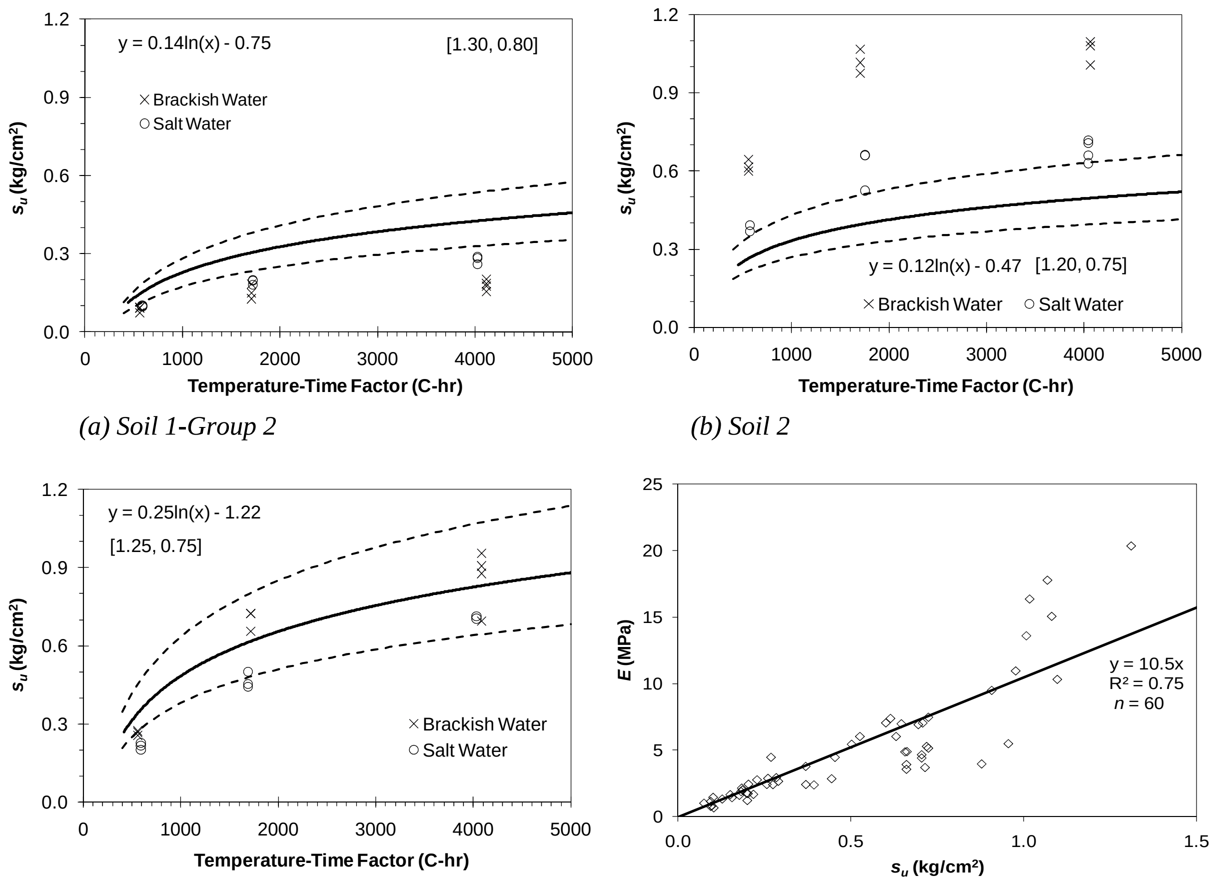


Figure 12.2. Water Effects in Soil 1-Group 1 (15, 233)

Figure 12.3 plots brackish and salt water test results of *A T III* relative to fresh water control bands. Maximum strains were 2.3% on average with a standard deviation of 0.5%. *Soil 1* shear strengths were below the control for brackish and salt water. *Soil 2* shear strengths were moderately above the control with salt water and substantially above the control with brackish water. *Soil 3* shear strengths were within the control band with brackish water and at to slightly below the control with salt water. A reasonable correlation was obtained between shear strength and elastic modulus when all soils were incorporated.

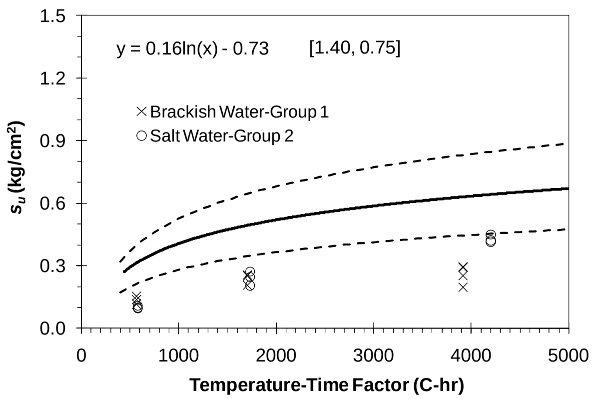


(a) Soil 1-Group 2 *(b) Soil 2*

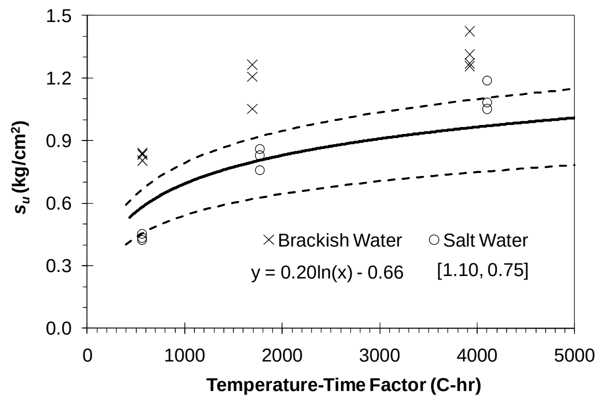
(c) Soil 3 *(d) All Soils*

Figure 12.3. Water Effects at (15, 233) with A T III

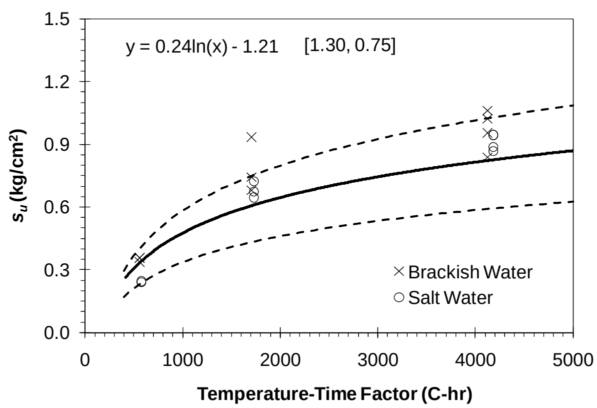
Figure 12.4 plots brackish and salt water test results of *Th T III* relative to fresh water control bands. Maximum strains were 2.2% on average with a standard deviation of 0.8%. *Soil 1* shear strengths were below the control with brackish and salt water. *Soil 2* shear strengths were within the control with salt water and noticeably above the control with brackish water. *Soil 3* shear strengths were within the control band and almost entirely above the trend line. A reasonable correlation was obtained between shear strength and elastic modulus when all soils were incorporated.



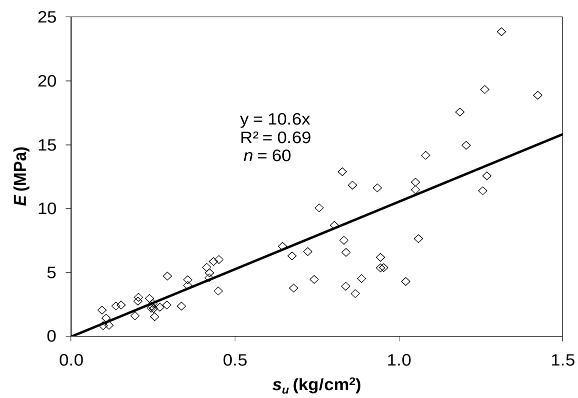
(a) Soil 1-Groups 1 and 2



(b) Soil 2



(c) Soil 3



(d) All Soils

Figure 12.4. Water Effects at (15, 233) with Th T III

Figure 12.5 plots brackish and salt water test results of *SC1* relative to fresh water control bands. Maximum strains were 2.1% on average with a standard deviation of 0.6%. *Soil 1* shear strengths were within the control band, though the control band was fairly weak. *Soil 2* shear strength was near the upper bound of the control band with brackish water, yet was moderately below the lower bound of the control band with salt water. A set was also conducted with *Soil 2* and salt water to verify the result, and the results are plotted along with the suite in Figure 12.5b. *Soil 3* shear strengths aligned with the trend line below 1000 C-hr and were at to slightly greater than the upper bound of the control band above 1000 C-hr. A reasonable correlation was obtained between shear strength and elastic modulus when all soils were incorporated.

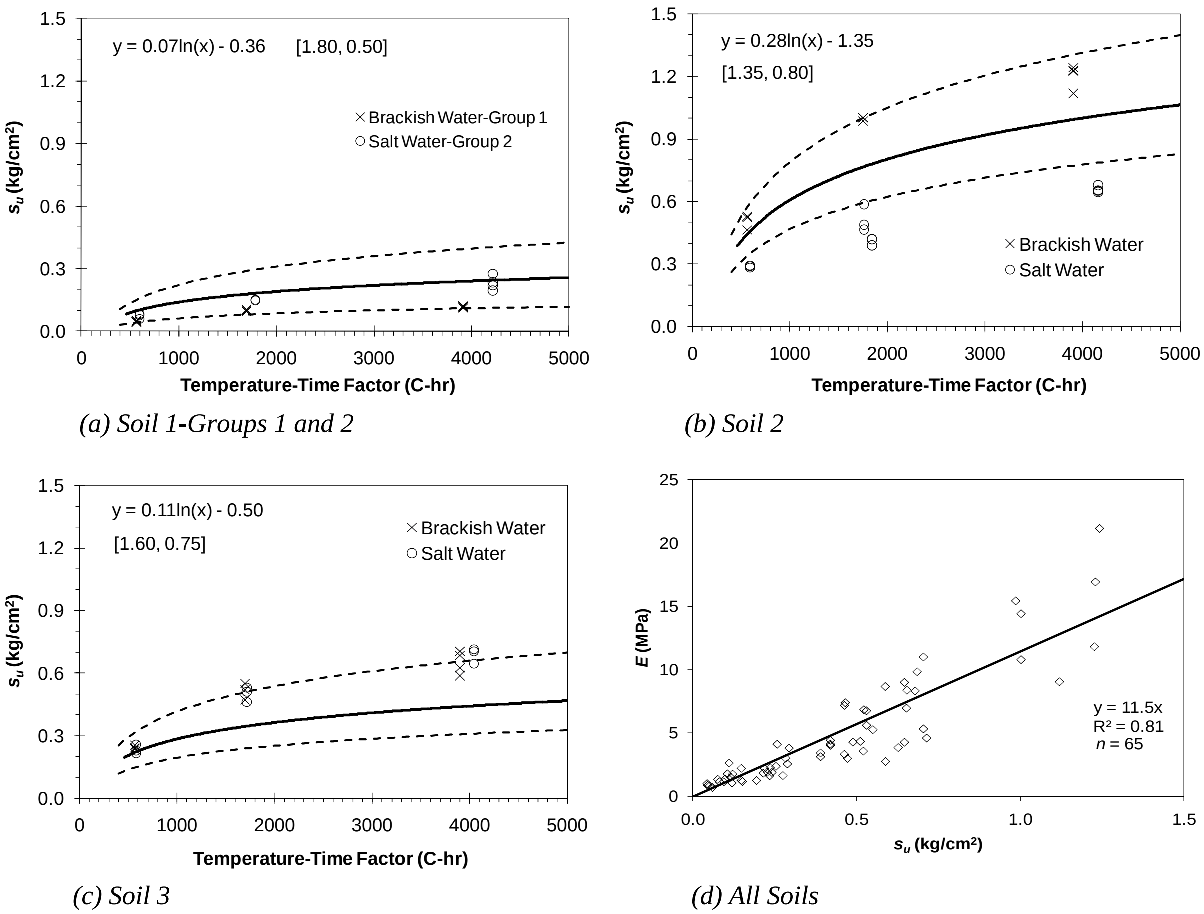
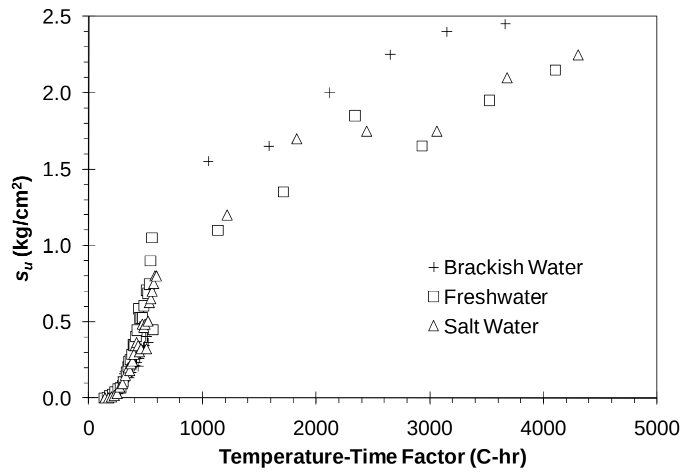


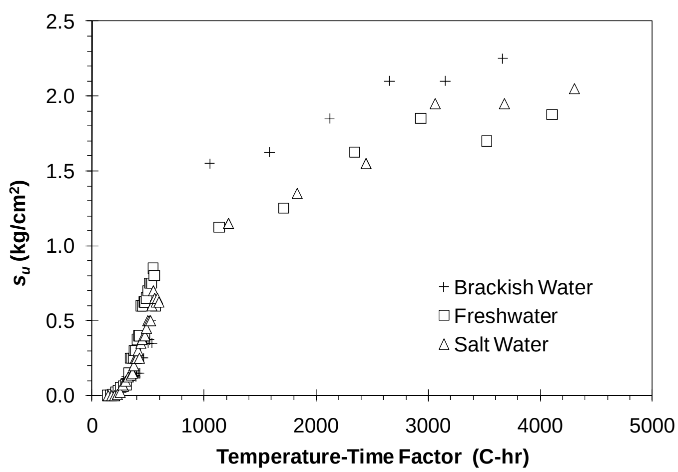
Figure 12.5. Water Effects at (15, 233) with SC1

12.4.2 *Slab* Brackish and Salt Water Test Results at (15, 233)

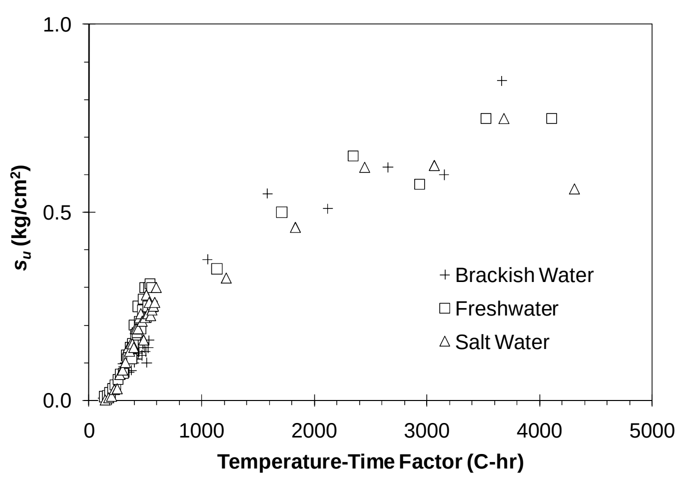
Figure 12.6 plots hand held gage results from the trials comparing shear strengths with different water types. Raw data is provided in Figure A.28 for the brackish and salt water trials. There is no evidence that the brackish water or the salt water negatively affected shear strength. The plots are similar in shape for all hand held gages. There is some evidence that brackish water increased shear strength above 1000 C-hr, especially from the *Dial* and *Ring* gages.



(a) Dial Gage



(b) Ring Gage



(c) Shear Gage

Figure 12.6. Water Effect Slab Test Results for Soil 3 at (15, 233) with Th T III

12.5 Summary of Brackish and Salt Water Test Results

The type of water appeared to have some effect on strength gain, and the effect was not consistent with soil type. *Soil 1* appeared to be weakened by either brackish or salt water, though in 1 instance brackish and salt water increased strength in *Soil 1*. *Soil 2* appeared to be strengthened by brackish water, and the strength increases were considerable in some instances. Effects of salt water on *Soil 2* were mixed as shear strengths ranged from moderately higher to moderately lower than the freshwater control. *Soil 3* with brackish water was, in general, above the freshwater control trend line (i.e. Zone 2), indicating its shear strength could be higher than the control. Effects of salt water on *Soil 3* were mixed as shear strengths ranged from slightly higher to slightly lower than the freshwater control.

The effect of water type did indicate a trend with respect to soil organic content. *Soil 2* had the highest organic content followed by *Soil 3* and then *Soil 1*. Strength improvement with brackish water had the same relative trend with *Soil 2* gaining strength, *Soil 3* possibly gaining strength, and *Soil 1* appearing to lose strength. Strength effects due to salt water were more variable as *Soil 2* ranged from moderately higher to moderately lower, *Soil 3* ranged from slightly higher to slightly lower, and *Soil 1* appeared to be lower.

Elastic modulus was correlated reasonably well to shear strength. At (15, 233) cement type did not appear to have a noticeable effect on the relationship as the slopes between cements varied within a small range (10.5 to 11.5) with R^2 values between 0.69 and 0.81. Maximum strain values were typical of fresh water testing as (10, 100) testing had an average value of 1.7%, and (15, 233) testing had average values of 2.1 to 2.3% depending on cement type.

Use of high moisture content cement stabilized slurries in areas with brackish or salt water does not appear prohibitive based on the testing conducted. Salt water slurries appear to be more problematic than brackish water slurries. Shear strength was reduced in some instances, but not to a point where the slurries would not be useful in disaster recovery. It is worth noting that local sea water was used to dissolve the foaming agent in 1 of the cementitiously stabilized projects studied by Tanaka et al. (2009) with no reported problems. Minimal on site testing of the slurry blended with on site soil and water sources will be needed to verify properties, but this can be performed quickly and easily during the mobilization stage of recovery efforts.

CHAPTER 13 - INVESTIGATION OF FIBER REINFORCED SPECIMENS

13.1 Overview of Fiber Reinforced Specimens

The effect of adding fibers along with cementitious material for soil stabilization is explored in this chapter. Two fiber types were tested at the (5,100) condition with trial and *UC Protocol 1* testing, with each fiber evaluated in terms of shear strength in order to determine the better performing fiber. The remainder of the investigation focused on the better performing fiber. Shear strength, elastic modulus, and ductility of specimens reinforced with the better performing fiber were evaluated at all conditions and soil types using *UC Protocol 2*, and the results were compared to non-fiber reinforced specimens. Shear strength of the better performing fiber was also evaluated with trial testing on a limited basis. One-hundred and fifty-six fiber reinforced *UC* specimens were tested alongside 6 trials (632 readings per hand held gage), and data is provided in Figures A.29 to A. 31.

13.2 Comparison of *F20* and *F70* Fibers

13.2.1 Comparison of *F20* and *F70* Fibers via Trial Testing

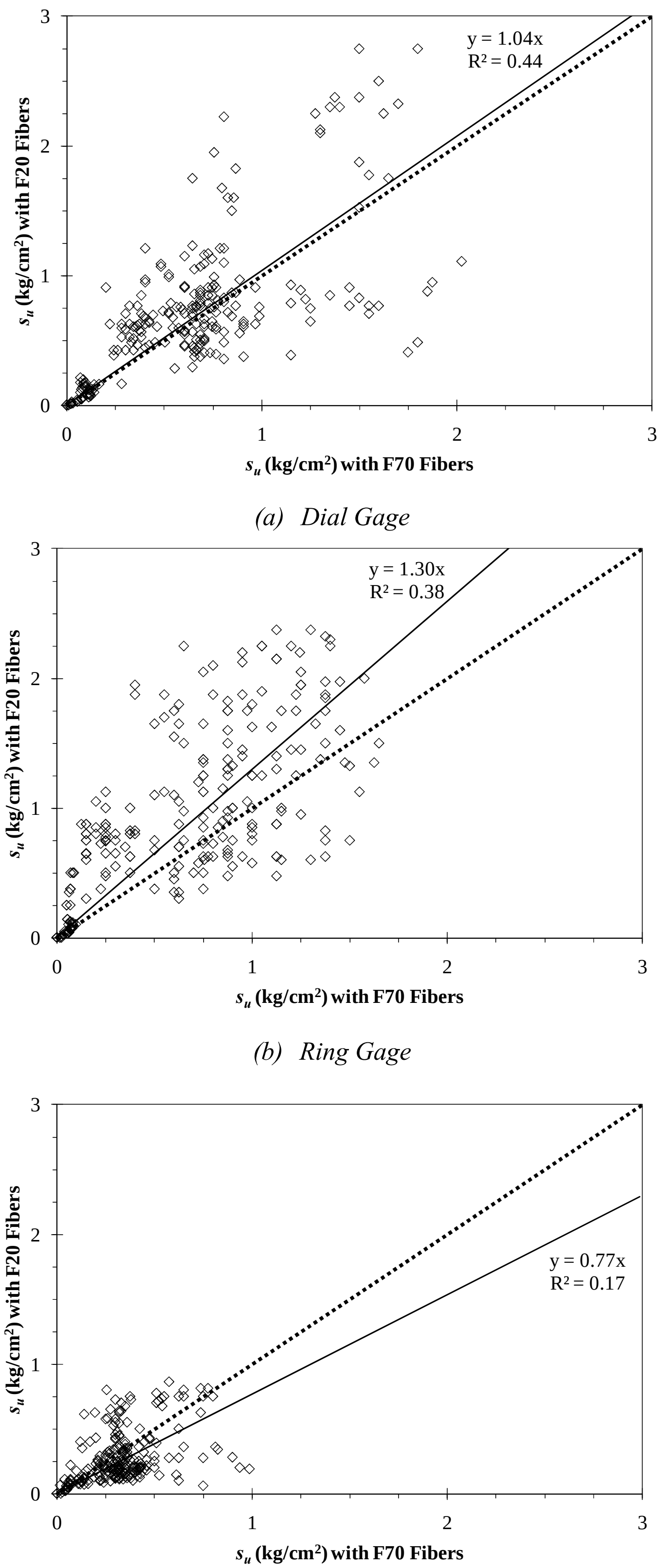
Figures 13.1 to 13.3 compare the shear strength of the *F70* fibers to the *F20* fibers for all 3 soils; all data was tested in conjunction with *SC1* in the (5, 100) condition. *Soil 1* did not provide conclusive results as the *Dial* gage indicated the strength of the 2 fibers to be practically equal, the *Ring* gage predicted the *F20* fibers to be somewhat better than *F70*, and the *Shear* gage predicted the *F20* fibers to be somewhat worse than the *F70* fibers.

Soil 2 test data did not show any difference between *F20* and *F70* fibers. *Soil 3* test data indicated *F20* fibers provided higher shear strength than *F70*, in particular the *Dial* and *Ring* test results. The *Shear* gage indicated *F20* fibers provided higher strength, but not enough higher to be of practical interest. Overall, *F20* fibers performed better than *F70* fibers, but not by a considerable margin. *F70* fibers performed reasonably well.

13.2.2 Comparison of *F20* and *F70* Fibers via *UC* Testing

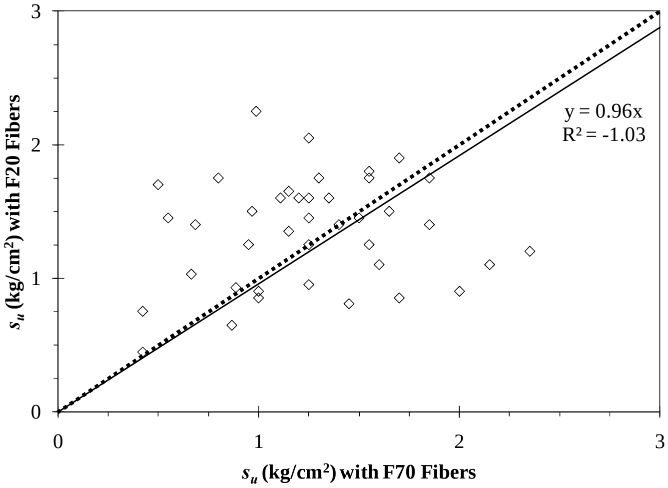
Figure 13.4 plots *F20* versus *F70* shear strength for *Protocol 1* at the (5, 100) condition. Both fibers showed similar results with *F20* holding a very slight advantage over *F70*. The *Soil 1* group(s) used to generate the data in Figure 13.4a were unknown, but since the testing for both fibers was performed around the same time period, the data are believed to have all come from the same group.

F20 was chosen as the better performing fiber and was used for the rest of the fiber analysis. The data from trial testing was considered when choosing the *F20* fiber. Some literature presented previously suggests that shorter fibers are better suited for clay soils, and the results obtained do not disagree with literature. Since the *F20* fiber did not drastically outperform the *F70* fiber for the limited testing performed, it is unclear whether shorter fibers work better for the soils tested, but nothing suggests they are worse.

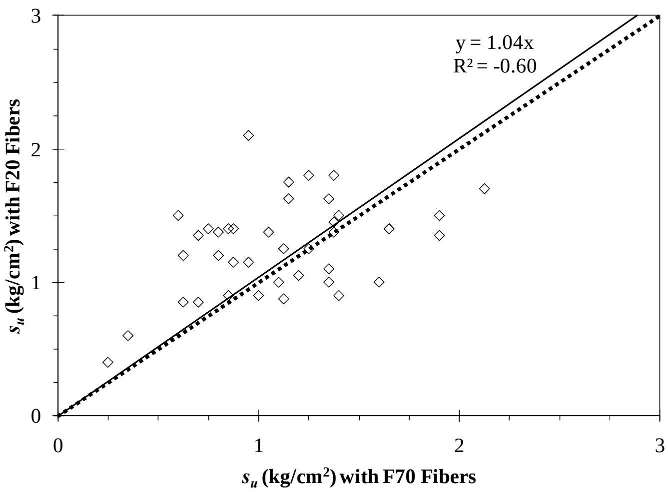


(c) Shear Gage

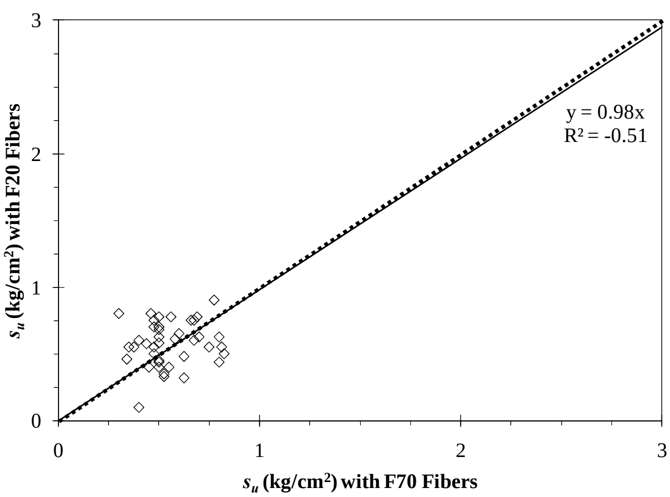
Figure 13.1. Comparison of F20 and F70 in Soil 1



(a) Dial Gage

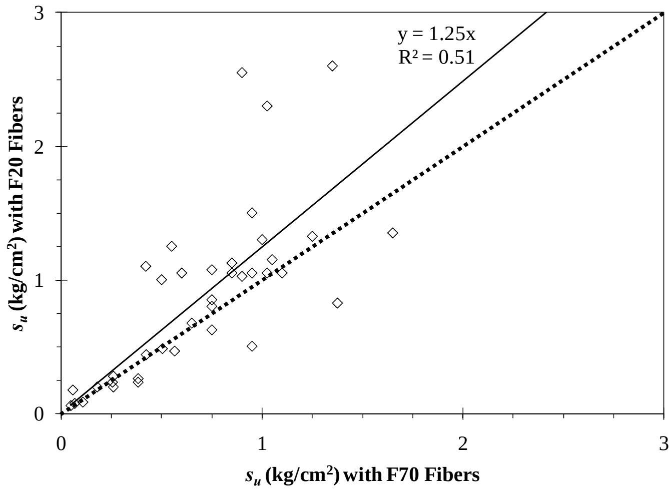


(b) Ring Gage

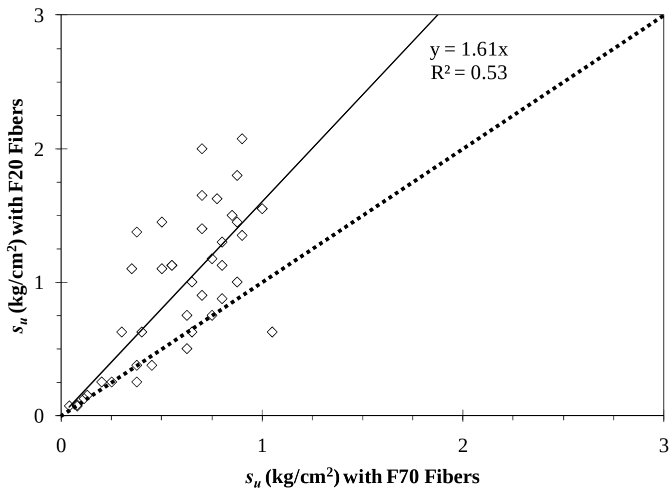


(c) Shear Gage

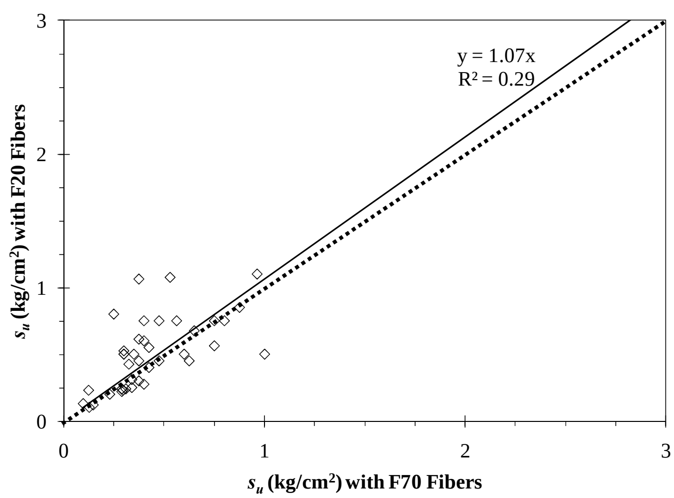
Figure 13.2. Comparison of F20 and F70 in Soil 2



(a) Dial Gage

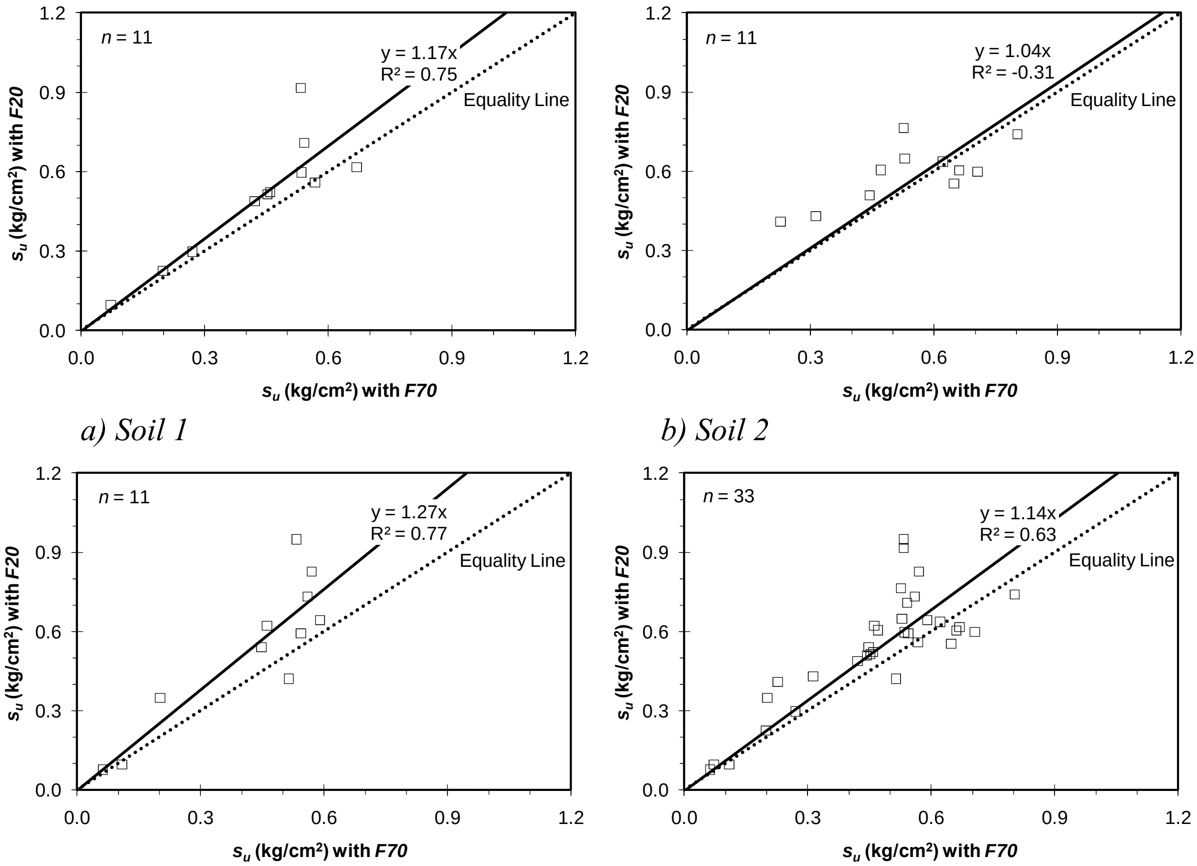


(b) Ring Gage



(c) Shear Gage

Figure 13.3. Comparison of F20 and F70 in Soil 3



a) Soil 1 *b) Soil 2*

c) Soil 3 *d) All*

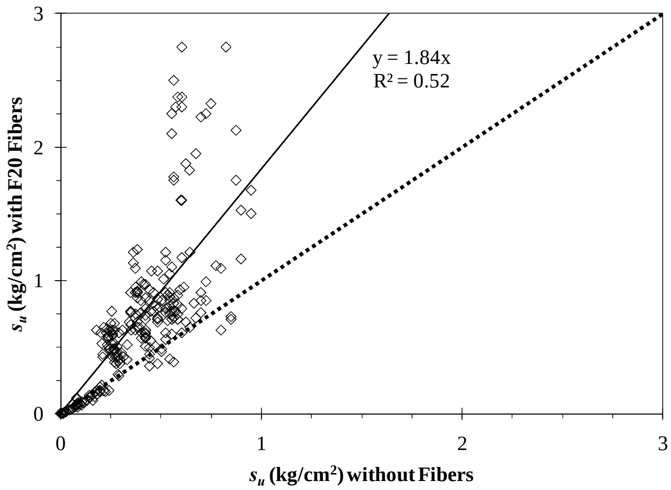
Figure 13.4. UC Strength Comparison of F20 and F70 Fibers

13.3 Effect of Fibers on Shear Strength

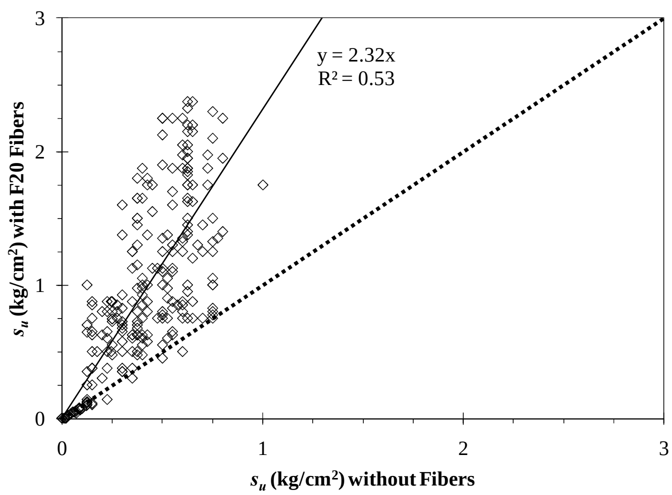
13.3.1 Effect of Fibers on Shear Strength via Trial Testing

Figures 13.5 to 13.7 compare shear strengths of each soil stabilized with *SC1* to those stabilized with *SC1* and *F20*; all comparisons were made in the (5, 100) condition. The gages predicted fibers to increase shear strength, though the extent varied considerably from gage to gage. The *Shear* gage predicted the smallest improvement of 1.00 to 1.30 depending on soil type, the *Dial* gage predicted an intermediate improvement relative to the other gages of 1.38 to 1.84, and the *Shear* gage predicted the most improvement at 1.65 to 2.32.

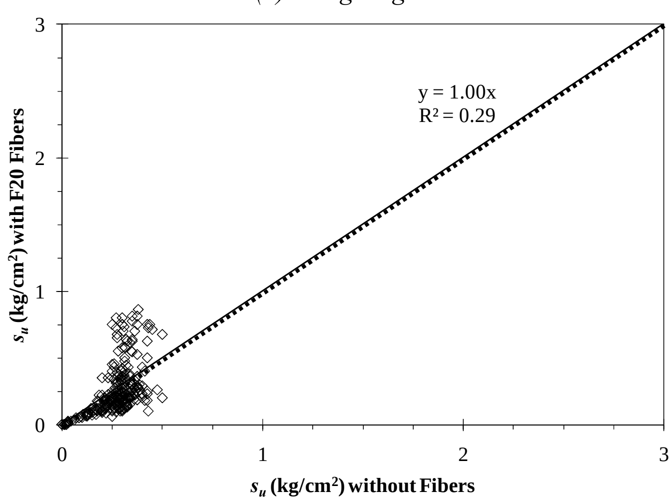
One variability slab was tested with fibers (Figure B.20) that served as a repeatability reference. At the same *TTF*, the *Dial* gage mean and *cov* from the variability slab were 1.38 kg/cm^2 and 36%, respectively. The trial trendline predicted a shear strength of 1.61 kg/cm^2, or 17% higher than the variability slab mean. The *Ring* gage mean and *cov* from the variability slab were 1.07 kg/cm^2 and 46%, respectively. The trial trendline predicted a shear strength of 1.59 kg/cm^2, or 49% higher than the variability slab mean. The *Shear* gage mean and *cov* from the variability slab were 0.41 kg/cm^2 and 37%, respectively. The trial trendline predicted a shear strength of 0.70 kg/cm^2, or 70% higher than the variability slab mean.



(a) Dial Gage

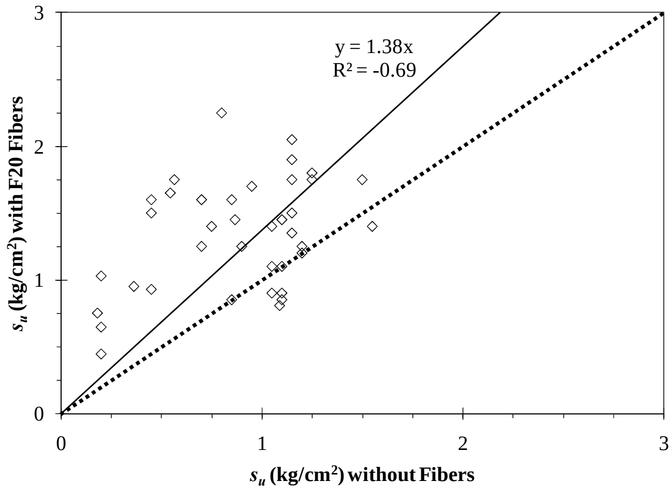


(b) Ring Gage

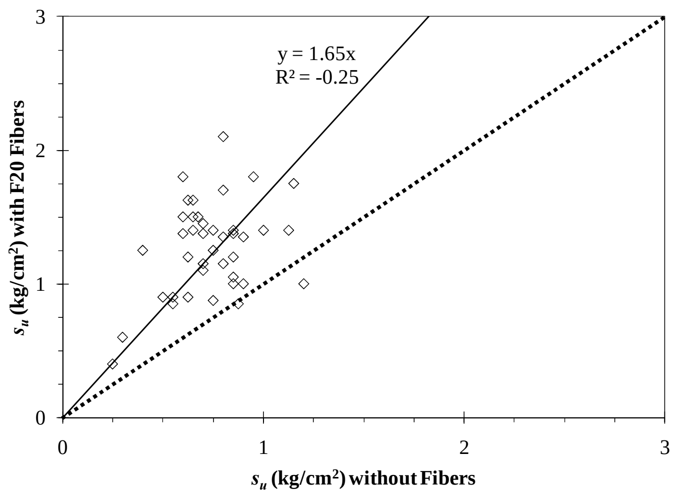


(c) Shear Gage

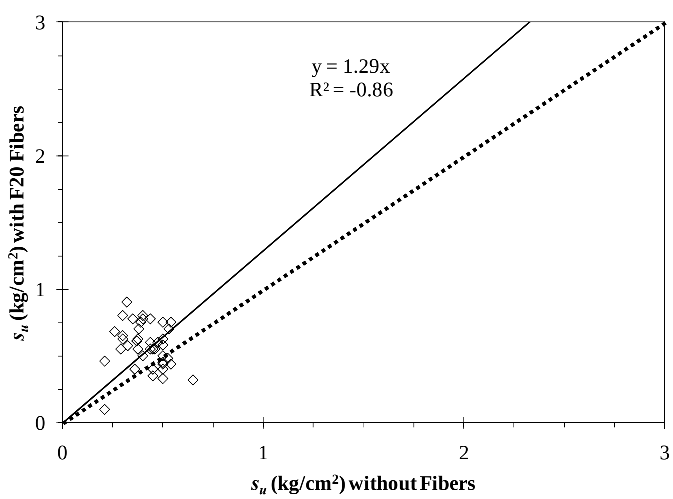
Figure 13.5. Comparison of F20 and Control in Soil 1



(a) Dial Gage

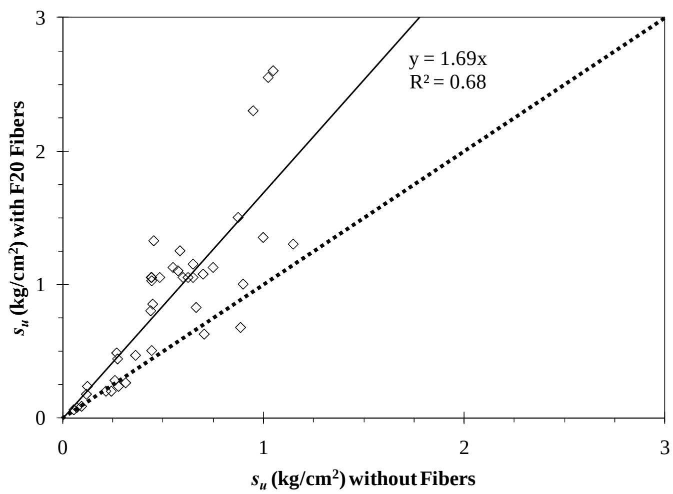


(b) Ring Gage

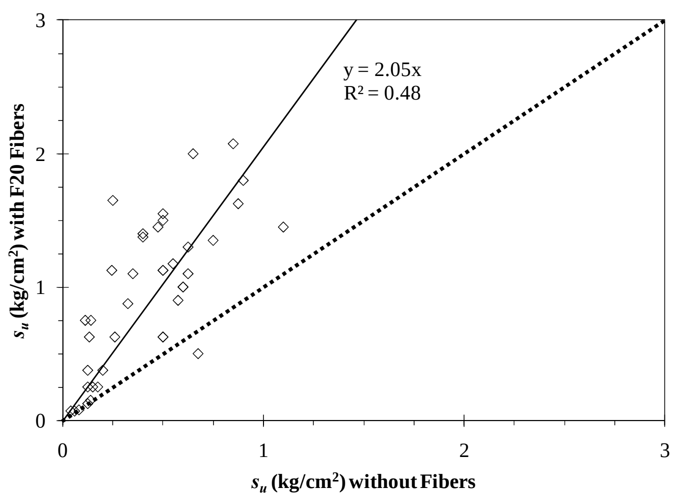


(c) Shear Gage

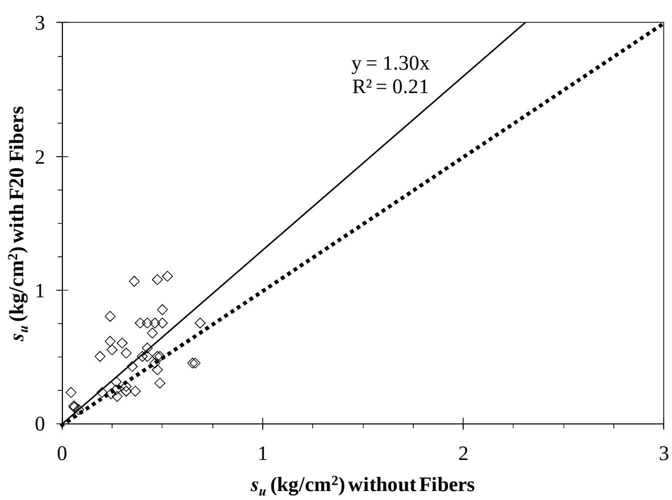
Figure 13.6. Comparison of F20 and Control in Soil 2



(a) Dial Gage



(b) Ring Gage



(c) Shear Gage

Figure 13.7. Comparison of F20 and Control in Soil 3

13.3.2 Effect of Fibers on Shear Strength via *UC* Testing

The *SC1* control suites presented previously were used to compare specimens without fibers to specimens with *SC1* and fibers tested according to *Protocol 2*, as shown in parts a, c, and e of Figures 13.8 to 13.10. All *Soil 1* fiber reinforced specimens were *Group 2*. As discussed in Chapter 8, all *Soil 1* control suites were found to provide the strongest upper control bound possible for the soils tested. As a result, the comparison between *Soil 1* fiber reinforced and non-fiber reinforced data was conservative in that the control data had what was believed to be stronger *Soil 1*; so, if fibers were shown to be beneficial the results were based on an unfavorable case for the fibers.

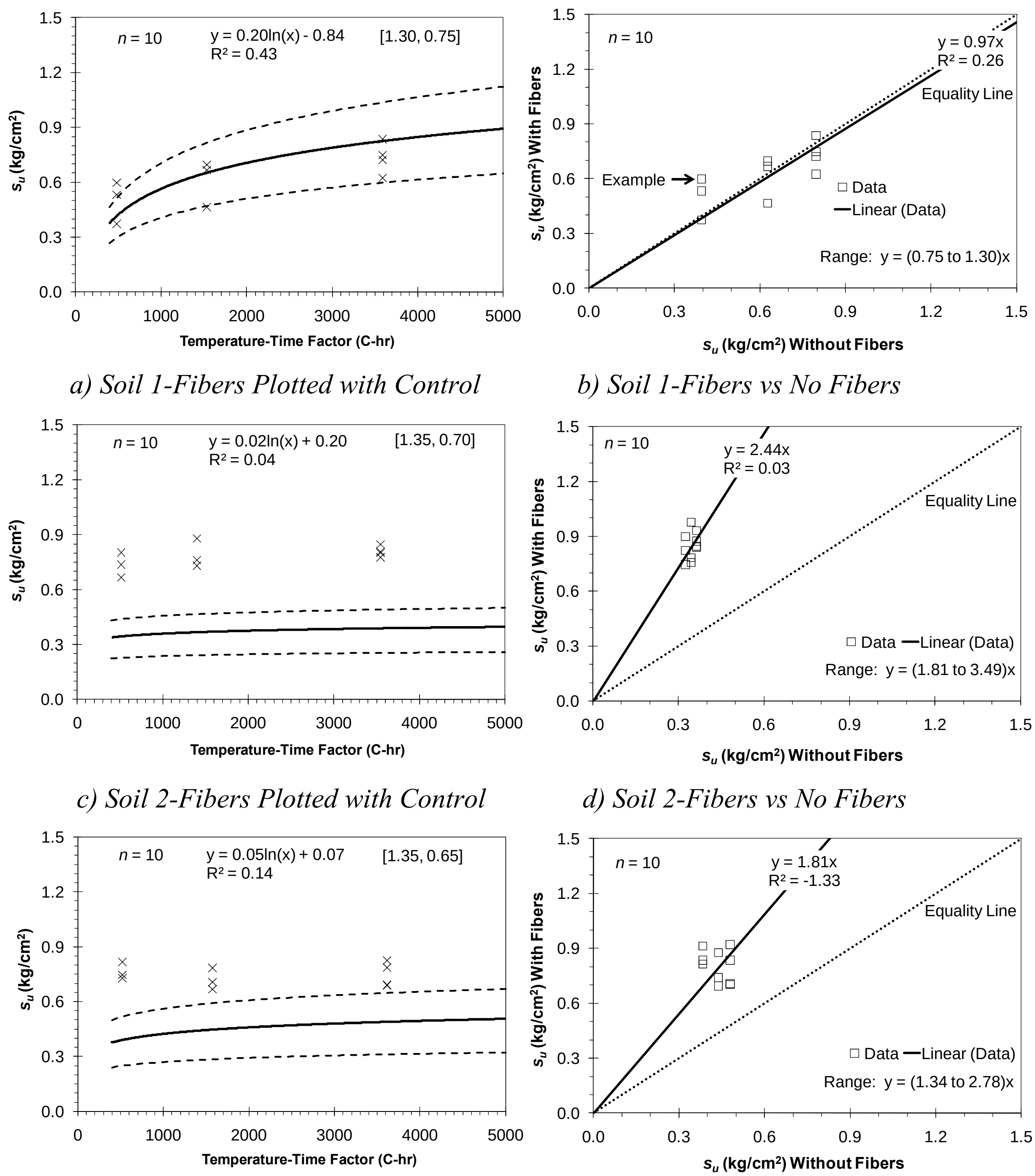
The plots in parts b, d, and f of Figures 13.8 to 13.10 were generated as follows. Equations of the control suite logarithmic trend lines were used to calculate the shear strength of specimens without fibers at the same *TTF* as the shear strength of corresponding specimens with fibers, and the 2 values were plotted together as 1 data point. A data point has been highlighted in Figure 13.8b as an example calculation. The *TTF* was 481 C-hr, which resulted in a no fiber shear strength of 0.40 kg/cm^2; the corresponding measured shear strength with fibers was 0.67 kg/cm^2. The result was the data point (0.32, 0.67).

A zero-intercept linear trend line was also fit through the data and is shown as the solid line. The shear strength of fiber reinforced specimens was plotted against the upper and lower control bounds from parts a, c, and e of Figures 13.8, 13.9, and 13.10. Zero- intercept linear trend lines were also fit through these 2 sets of data, and the equations of the lines are listed as the range in parts b, d, and f in Figures 13.8, 13.9, and 13.10.

At (5,100), mixed strength results were observed for fiber reinforced specimens. *Soil 2* showed considerable improvement, with fiber reinforced specimens showing a 144% higher shear strength than the control trend line and an 81% higher shear strength than the upper bound of the control envelope. *Soil 3* also showed a noticeable increase with 92% and 34% higher strengths than the control and upper control bound, respectively. *Soil 1* was the only soil type at the (5,100) condition where some of the fiber reinforced specimens exhibited shear strengths that fell within the control envelope. Some of the data fell below the control trend line, but still within the control envelope. A 3% decrease was demonstrated as a result of fibers being added compared to the control, and a 25% lower shear strength was exhibited compared to the upper control bound. Note that what was believed to be weaker *Soil 1* was present in the fiber mixtures which should be considered when viewing the results.

Specimens at the (10,100) condition exhibited varying responses due to the addition of fibers. *Soil 1* fiber reinforced specimens showed a 5% decrease in shear strength compared to the control trend line, and a 30% decrease when compared to the upper control bound. *Soil 2* specimens responded differently, exhibiting a 97% increase in shear strength compared to the control and a 64% higher shear strength compared to the control upper bound. *Soil 3* specimens showed a very slight improvement in shear strength due to fibers as shown in Figure 13.9e; the specimens with fibers were 11% stronger than the control trend line, but 11% weaker than the upper control bound.

Fiber reinforced specimens at the (15,233) condition also showed differing responses according to soil type. *Soil 1* fiber reinforced specimens all exhibited shear strengths that fell within the control envelope and slightly above the trend line. The fibers improved shear strength 27% compared to the control, but decreased shear strength 30% compared to the control upper bound. The increase in shear strength for *Soil 2* specimens was not as pronounced as the (5,100) and (10,100) *Soil 2* specimens, but all shear strength data fell well above the upper bound of the control envelope showing a 64% increase in strength overall and a 22% advantage over the control upper bound. For *Soil 3*, the shear strength data points for specimens with fibers fell very close to the upper bound of the control, indicating a moderate improvement in terms of shear strength of 60% over the control.

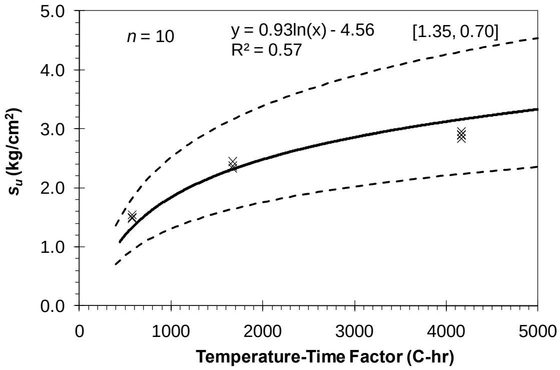


a) Soil 1-Fibers Plotted with Control *b) Soil 1-Fibers vs No Fibers*

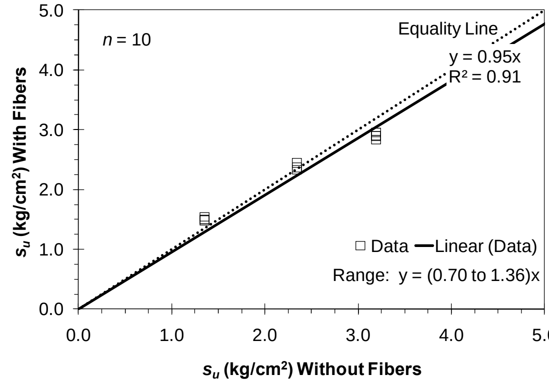
c) Soil 2-Fibers Plotted with Control *d) Soil 2-Fibers vs No Fibers*

e) Soil 3-Fibers Plotted with Control *f) Soil 3-Fibers vs No Fibers*

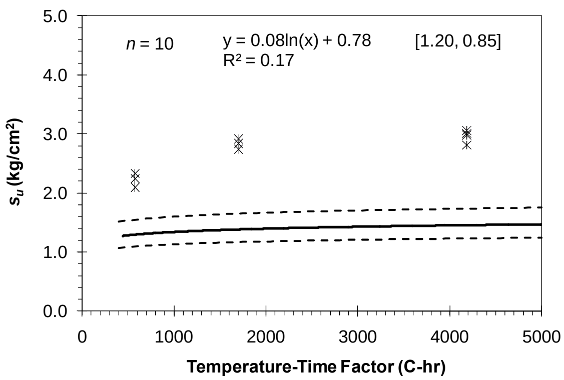
Figure 13.8. UC Fiber Shear Strength Plots at (5,100)



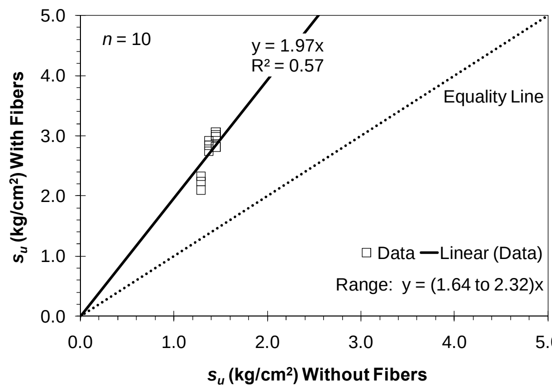
a) Soil 1-Fibers Plotted with Control



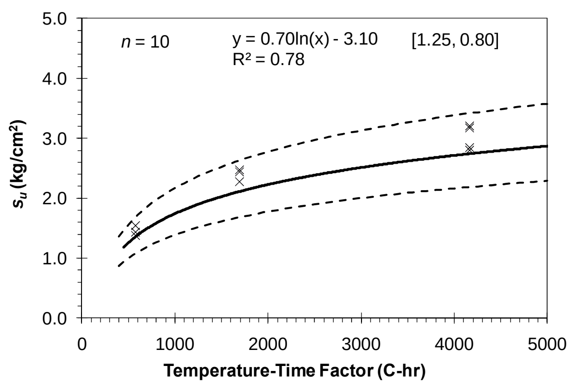
b) Soil 1-Fibers vs No Fibers



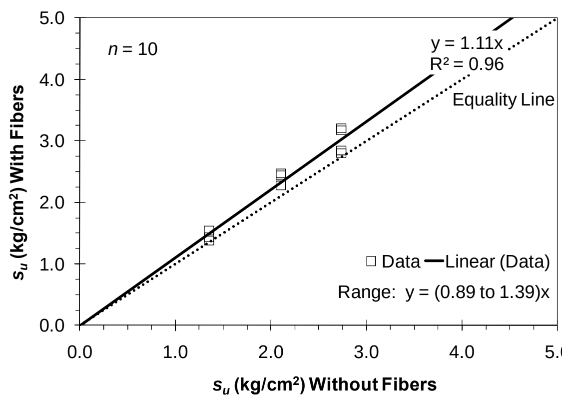
c) Soil 2-Fibers Plotted with Control



d) Soil 2-Fibers vs No Fibers



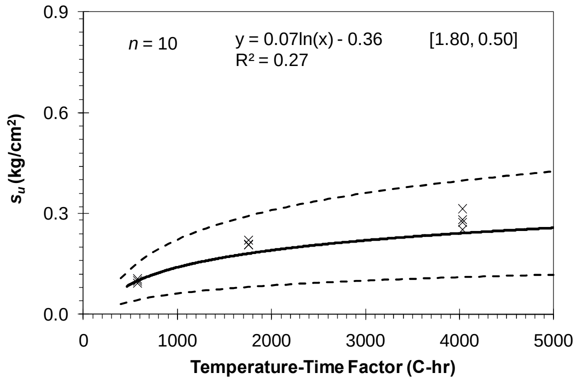
e) Soil 3-Fibers Plotted with Control



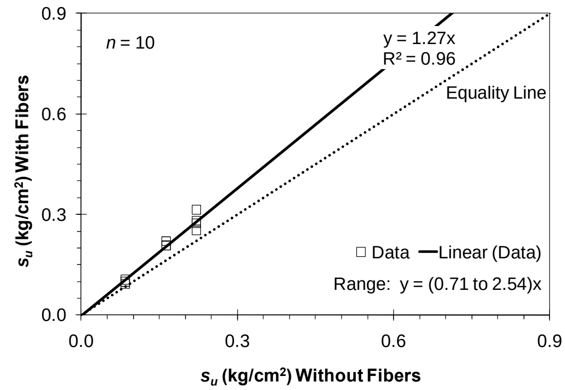
f) Soil 3-Fibers vs No Fibers

Figure 13.9. UC Fiber Shear Strength Plots at (10,100)

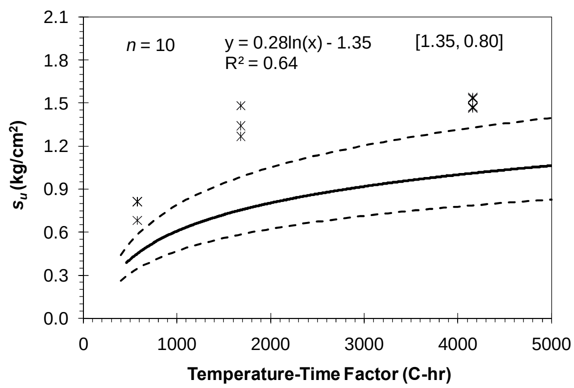
Overall, the addition of fibers increased shear strength predicted by trend lines in all but 2 soil-condition combinations, and in those conditions the effects were essentially non-existent. Both of these conditions occurred in *Soil 1* with what was believed to be different soil strengths between fiber and no fiber specimens. This conclusion disagrees with literature that is largely related to other applications, which reported little or no effects on strength due to fiber addition as a secondary stabilizer. The degree of shear strength increase varied, but a trend was noticed when the overall fiber to no fiber ratio ($F\text{-}NF_{ratio}$) was plotted against organic content in Figure 13.11. The fiber to no fiber ratios were taken from the linear trend line equations developed in Figures 13.8 to 13.10. The higher shear strength increases for soils with higher organic contents could be the result of less free water. Organics tend to absorb free water that could prevent the fibers from binding with clay particles.



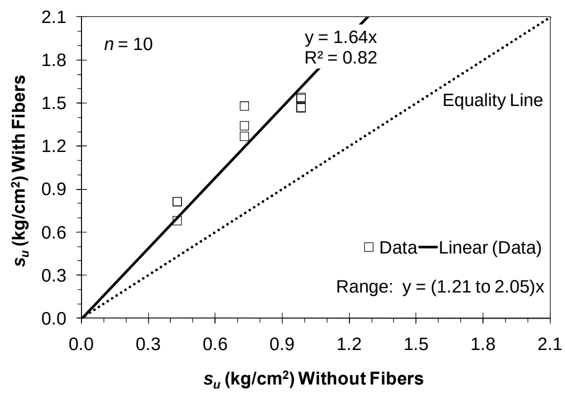
a) *Soil 1*-Fibers Plotted with Control



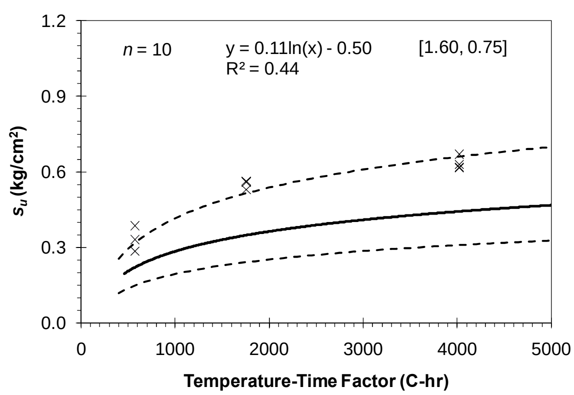
b) *Soil 1*-Fibers vs No Fibers



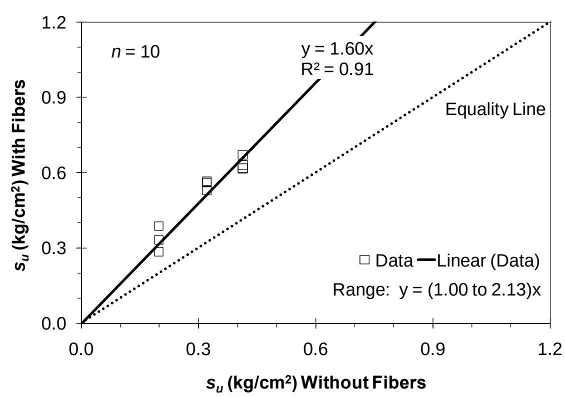
c) *Soil 2*-Fibers Plotted with Control



d) *Soil 2*-Fibers vs No Fibers



e) *Soil 3*-Fibers Plotted with Control



f) *Soil 3*-Fibers vs No Fibers

Figure 13.10. UC Fiber Shear Strength Plots at (15,233)

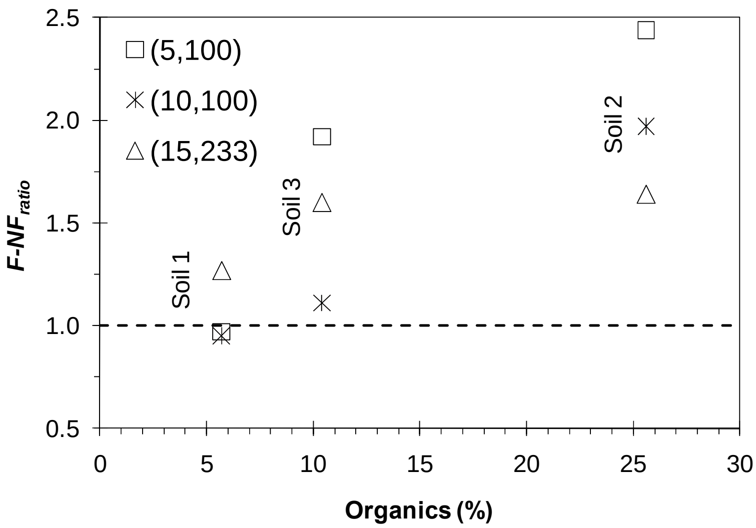


Figure 13.11. Plot of Fiber to No Fiber Ratio vs Organic Content

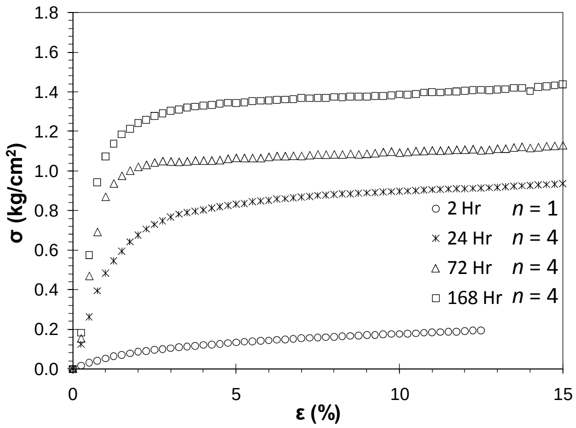
13.4 Effect of Fibers on Ductility

The maximum strain (ε_{max}) of each specimen was used along with Figures 13.12 and 13.13 to evaluate fiber and non-fiber reinforced specimen ductility. Non-fiber reinforced specimens did not carry load after ε_{max} was reached (complete failure occurred). Table 13.1 contains all ε_{max} values for fiber reinforced and non-fiber reinforced specimens.

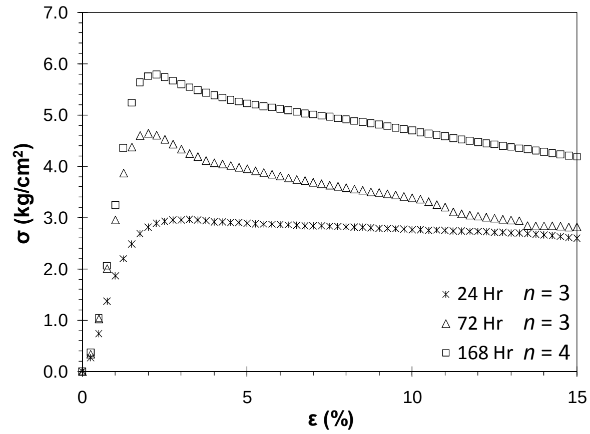
For the (5,100) condition, the shapes of the stress-strain curves were similar for each soil type. The strength increased with strain up to the yield point, and then the specimens were able to maintain strength or show only a slight strength reduction after the yield point was reached. As shown in Figure 13.12c, the normal stress increased at a slower rate for *Soil 2* specimens than *Soil 1* and *Soil 3* specimens, indicating a lower elastic modulus. Also, there was a noticeable increase in ε_{max} values for all soils at all testing times due to fiber addition.

Stress-strain behavior of specimens at the (10,100) condition was quite different compared to (5,100) specimens. The higher cementitious content increased strength but also decreased ductility compared to the (5,100) specimens. Despite the addition of fibers, the specimens experienced a considerable amount of post-peak reduction in strength, although not a complete failure as with non-fiber reinforced specimens. All 3 soils exhibited very similar behavior in terms of σ_{ult} and elastic modulus. At 10% cementitious content, it appears that the higher strength provided by the additional cement could have diminished the effects of differing soil properties. In terms of ε_{max}, fiber reinforced specimens had higher ε_{max} values than non-fiber reinforced specimens in all cases at (10,100), although the increase was small in some cases. Also, the increase in ε_{max} was not as drastic as the (5,100) condition.

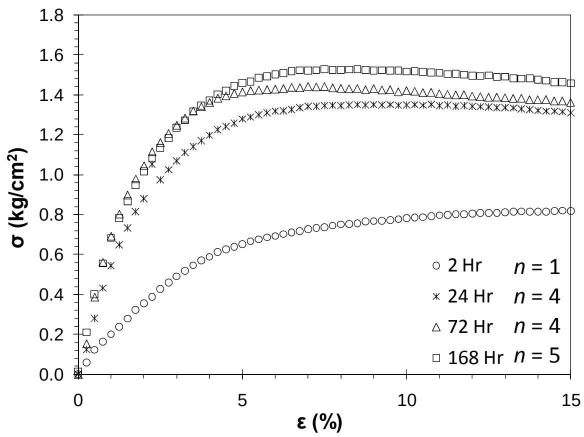
The general shapes of fiber reinforced specimens at the (15,233) condition were similar for each soil type at each testing time. All specimens continued to maintain strength or show a slight increase in strength after the yield point was reached. However, σ_{ult} varied considerably by soil type, which was not the case for (5,100) and (10,100) specimens. A considerable increase in ε_{max} was noticed for all fiber-reinforced specimens compared to non-fiber reinforced specimens, and there was less variability in the data than the (5,100) and (10,100) conditions.



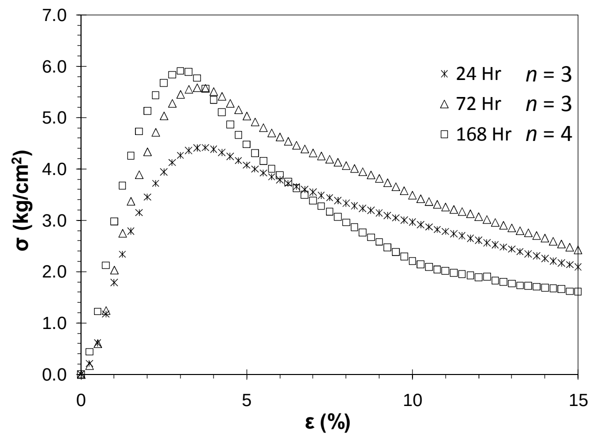
(a) Soil 1-(5,100)



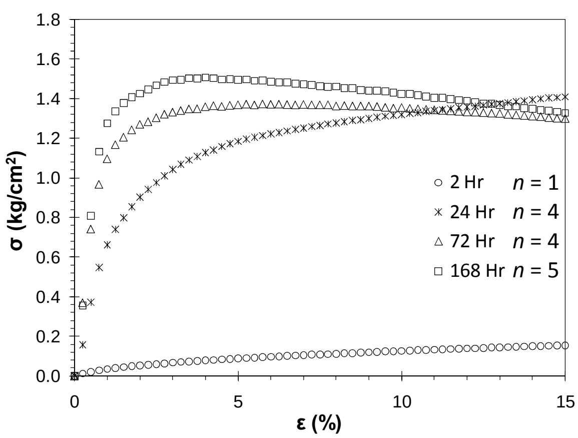
(b) Soil 1-(10,100)



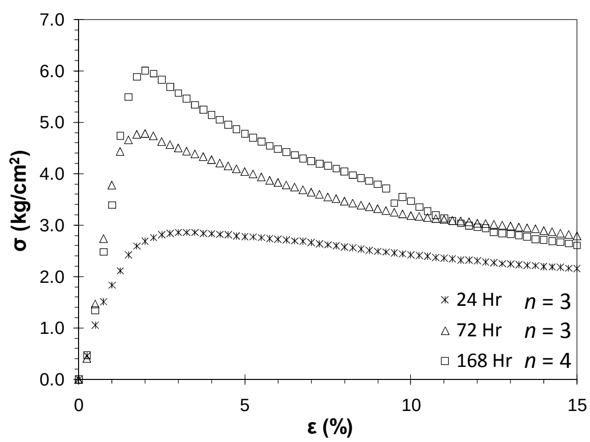
(c) Soil 2-(5,100)



(d) Soil 2-(10,100)

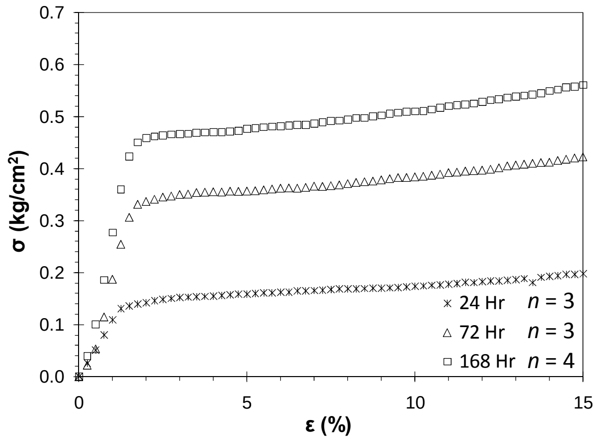


(e) Soil 3-(5,100)

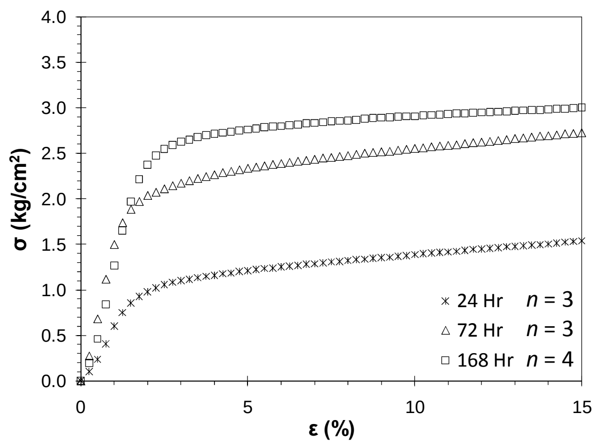


(f) Soil 3-(10,100)

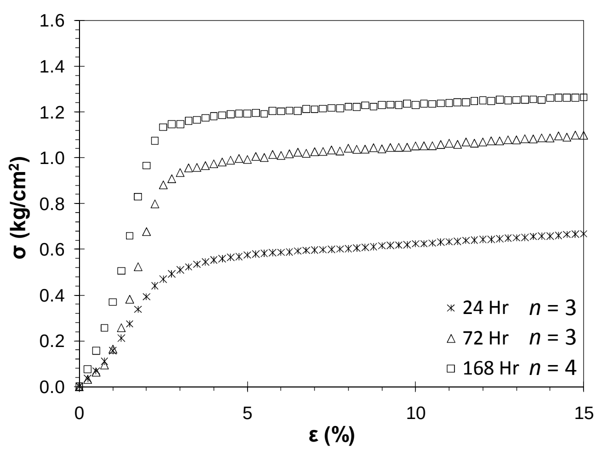
Figure 13.12. Stress-Strain Plots for (5,100) and (10,100)-F20 Specimens



(a) Soil 1



(b) Soil 2



(c) Soil 3

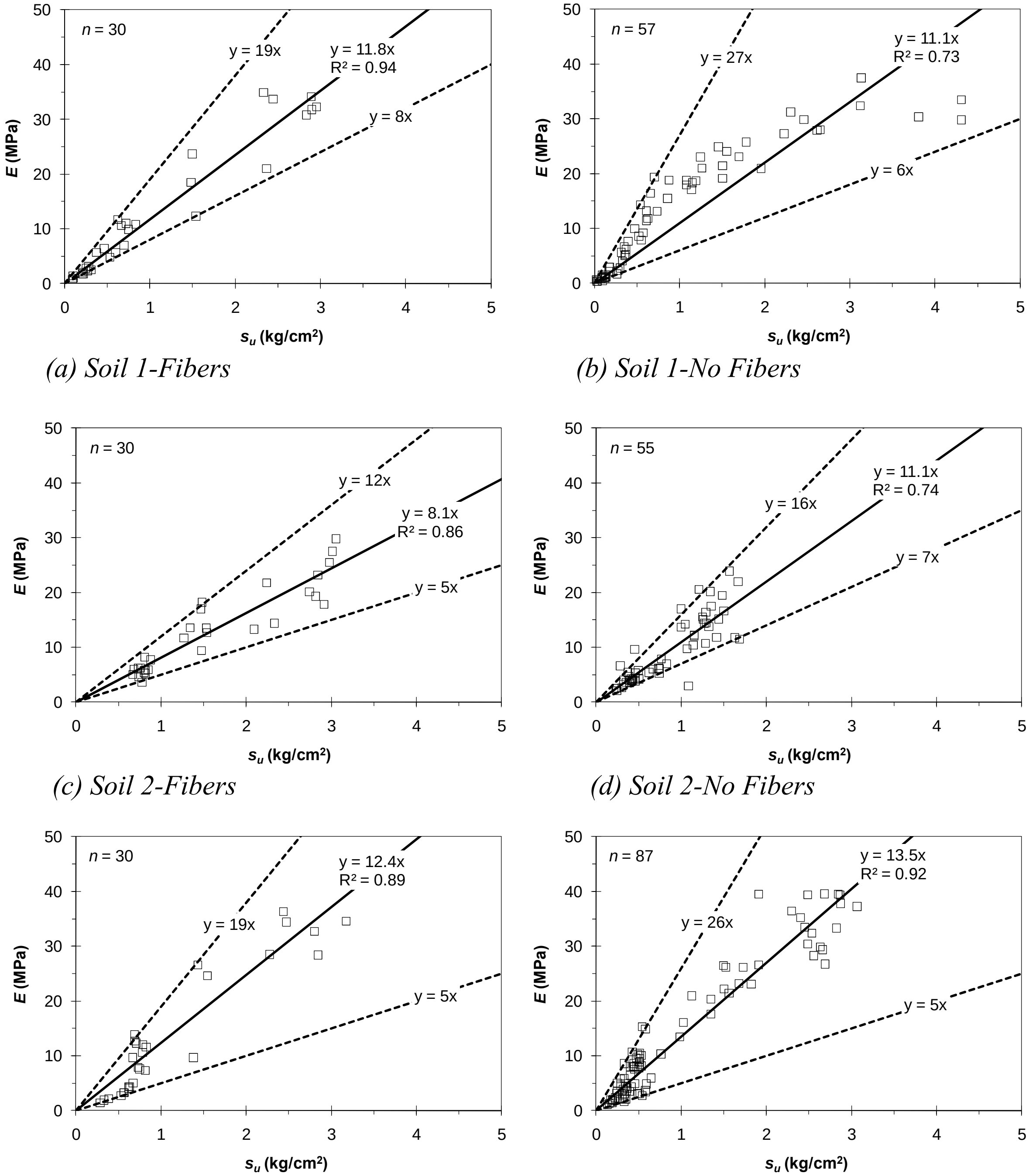
Figure 13.13. Stress-Strain Plots for (15,233)-F20 Specimens

Table 13.1. Maximum Strain Test Results

			ε_{max} (%)	
Condition	**Soil**	**Time (hr)**	**No Fiber**	**Fiber**
(5,100)	1	2	3.0	---
		24	1.8	15.0
		72	1.4	14.9
		168	1.3	15.0
	2	2	4.5	15.0
		24	3.3	13.2
		72	3.0	12.3
		168	2.8	14.4
	3	2	15.0	15.0
		24	2.3	15.0
		72	1.8	14.3
		168	1.5	9.4
(10,100)	1	24	1.6	10.3
		72	1.6	6.3
		168	1.8	5.6
	2	24	2.4	3.5
		72	2.3	3.8
		168	2.1	3.2
	3	24	1.9	6.8
		72	1.5	2.2
		168	1.7	2.1
(15,233)	1	24	2.3	15.0
		72	2.5	15.0
		168	2.6	15.0
	2	24	2.1	15.0
		72	2.0	15.0
		168	2.3	15.0
	3	24	2.0	15.0
		72	2.6	14.8
		168	3.0	14.9

13.5 Effect of Fibers on Elastic Modulus

Figure 13.14 plots elastic modulus versus peak shear strength for fiber and non-fiber reinforced specimens. Figure 13.15a combines Figures 13.14a, 13.14c, and 13.14e, while Figure 13.15b combines Figures 13.14b, 13.14d, and 13.14f. A zero-intercept linear trend line is displayed alongside linear boundaries that portray the envelope of data alongside pertinent statistical results. More data was available for non-fiber reinforced specimens since these tests were repeated to create the *SC1* control suites; only data from the control suites are presented for this analysis. A good trend between shear strength and modulus was exhibited overall and by soil type, and the data could be used to estimate elastic modulus for design purposes. Table 13.2 summarizes the information displayed in Figures 13.14 and 13.15 and also shows data obtained for all combinations, which were not plotted.

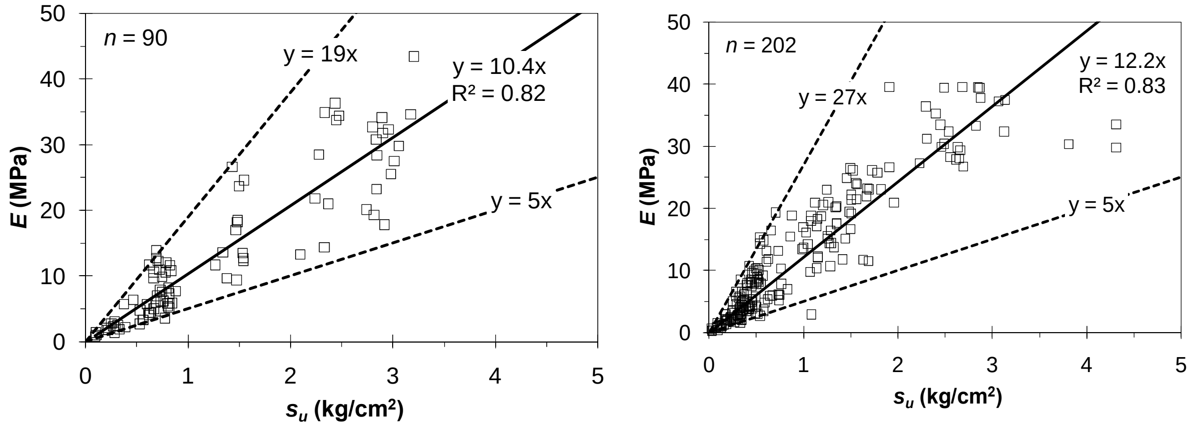


(a) Soil 1-Fibers *(b) Soil 1-No Fibers*

(c) Soil 2-Fibers *(d) Soil 2-No Fibers*

(e) Soil 3-Fibers *(f) Soil 3-No Fibers*

Figure 13.14. Elastic Modulus vs Shear Strength Plots



(a) Fiber Reinforced *(b) Non-Fiber Reinforced*

Figure 13.15. Overall Elastic Modulus vs Shear Strength Plots

Table 13.2. Summary of Fiber Modulus Results

					Slope		
Fibers	**Soil**	**Condition**	***n***	**R^2**	**Trend line**	**Lower**	**Upper**
Yes	All	All	90	0.82	10.4	5	19
		(5,100)	30	-0.07	11.3	5	19
		(10,100)	30	0.25	10.5	6	16
		(15,233)	30	0.88	8.9	5	12
	1	All	30	0.94	11.8	8	19
		(5,100)	10	0.49	13.4	8	19
		(10,100)	10	0.61	11.7	8	15
		(15,233)	10	0.80	9.4	8	12
	2	All	30	0.86	8.1	5	12
		(5,100)	10	0.22	7.5	5	10
		(10,100)	10	0.50	7.9	6	10
		(15,233)	10	0.67	9.3	6	12
	3	All	30	0.89	12.4	5	19
		(5,100)	10	-0.32	14.1	8	19
		(10,100)	10	0.59	12.5	8	16
		(15,233)	10	0.86	6.4	5	8
No	All	All	202	0.83	12.2	5	27
		(5,100)	63	0.72	17.7	8	27
		(10,100)	69	0.43	12.0	7	18
		(15,233)	70	0.73	9.8	5	17
	1	All	60	0.73	11.1	6	27
		(5,100)	20	0.68	18.4	13	27
		(10,100)	20	-1.07	10.4	7	18
		(15,233)	20	0.78	12.7	6	17
	2	All	55	0.74	11.1	7	16
		(5,100)	16	0.20	11.8	8	15
		(10,100)	19	-0.01	11.3	7	16
		(15,233)	20	0.56	10.1	7	14
	3	All	87	0.92	13.5	5	26
		(5,100)	27	0.44	18.6	11	26
		(10,100)	30	0.62	13.5	10	18
		(15,233)	30	0.38	8.1	5	14

For the same shear strength, *Soil 1* elastic modulus slope with fibers increased slightly relative to no fibers (11.8 versus 11.1). For the same shear strength, the elastic modulus of *Soil 2* and *Soil 3* decreased. For *Soil 2* the elastic modulus slope decreased moderately from 11.1 to 8.1, and for *Soil 3* the elastic modulus slope decreased slightly from 13.5 to 12.4.

If one were to consider that shear strength was affected by the fibers, one approach would be to enter Figure 13.14 with different x-coordinates and compare the resulting fiber and no fiber moduli. Figures 13.8 through 13.10 depict shear strength relationships of fiber reinforced and non-fiber reinforced specimens. The following are averages of the slopes for each soil: *Soil 1* - 0.97, 0.95, and 1.27 results in 1.06; *Soil 2* - 2.44, 1.97, and 1.64 results in 2.02; *Soil 3* - 1.81, 1.11, and 1.60 results in 1.51. Utilizing no fiber shear strength as 1 kg/cm^2 as an example, the estimated shear strengths with fibers would be 1.06, 2.02, and 1.51 kg/cm^2, respectively. Entering each Figure 13.14 plot with the aforementioned data results in the following elastic modulus estimates: *Soil 1* - 11.1 MPa without fibers and 12.5 MPa with fibers; *Soil 2* - 11.1 MPa without fibers and 16.4 MPa with fibers; *Soil 3* - 13.5 MPa without fibers and 18.7 MPa with fibers. Modulus was improved in all soils as a result of fiber addition for the same shear strength.

13.6 Summary of Fiber Reinforcement

The *F2*0 fiber outperformed the *F70* fiber with respect to shear strength at the (5,100) condition. Accordingly, the *F2*0 fiber was used to evaluate the effects of fiber inclusion at all conditions. The addition of fibers increased shear strength for all but 2 conditions where there were no considerable differences, and the level of increase was shown to be affected by soil organic content. Ductility was improved considerably as a result of fibers being added. The stress-strain behavior appeared to be mostly influenced by moisture condition. Correlations were developed by soil type so that design elastic modulus values could be calculated from shear strength. The correlations were used to assess the effect of fiber addition on modulus, and fibers were observed to increase modulus values for the same shear strength.

CHAPTER 14 - SEMI-ADIABATIC CALORIMETRY TESTING

14.1 Calorimetry Test Results Overview

Calorimetry is the science of measuring heat evolution associated with chemical reactions (e.g. portland cement hydration). While isothermal or near-adiabatic calorimetry methods actually quantify evolved heat, simple records of temperature changes (thermal profiles) of hydrating mixtures often referred to as semi-adiabatic calorimetry (*SAC*) can be similarly used for most applications as an *indication* of evolved heat, in sample configurations and hydrating environments more representative of field applications. *SAC* was conducted on cement paste mixtures and on cementitiously stabilized soil slurries. *SAC* has been used on cementitious mixtures such as paste and mortar by numerous investigators, while testing cementitiously stabilized soil slurries in this manner has not been performed and reported by many, if any, others outside of the authors of this report. The purpose of testing cement paste was to investigate behaviors that could help explain shear strength data presented in Chapter 9, in particular issues associated with SO_3 content. The purpose of testing cementitiously stabilized soils slurries was to determine their applicability to compare cements and how useful they might be on site as a quality control tool. Each *SAC* investigation was referred to in this report as a series. Three series were conducted (2 on cement paste, 1 on stabilized soil slurries), and they are described individually in the following sections.

14.2 Paste Mixture *SAC Series 1*

SC2 was produced with high Blaine fineness and a very low SO_3 content to intentionally shorten initial set time. When slurry mixture tests indicated unexpected low strength development in some cases (See Chapter 9), the question of whether calcium sulfate content of this sample was adequate for normal hydration at even mild temperatures was raised and whether a minimum effective SO_3 level for strength would be somewhat higher. *SAC* was used to investigate this question. Six paste mixtures (Table 14.1) were prepared and tested at 23 C. Neat paste mixtures were compared to mixtures with added SO_3 from *PoP*.

Table 14.1. ***SAC Series 1*** **Mix Proportions**

Mixture	*SC2* (g)	*PoP* (g)	Water (g)	Total SO_3 (%)
1	500	0.00	225.0	1.6
2	500	7.14	228.2	2.1
3	500	14.28	231.4	2.6
4	500	21.42	234.6	3.1
5	500	28.56	237.9	3.6
6	500	35.70	241.1	4.1

Notes: w/cm = 0.45 and SO_3 *=1.6% in SC2.*

Figure 14.1 plots *SAC* test results for *Series 1* with start times synchronized in absolute terms (i.e. no adjustment for the slight laboratory ambient temperature variations during the testing period). *Mixture 1* clearly indicates abnormal hydration that is associated with insufficient calcium sulfate in solution for control of aluminate hydration. This is characterized by the rapid associated exotherm immediately after mixing (Figure 14.1 insert). This aluminate hydration results in rapid stiffening or flash set with little associated strength gain, and also causes disruption of normal calcium silicate hydration as evidenced by the reduced and delayed silicate main peak exotherm that begins around 5 hr and peaks around 8 hr. The data indicates that the very short (10 min) initial Vicat time for this cement is the result of abnormal aluminate hydration, not silicate hydration.

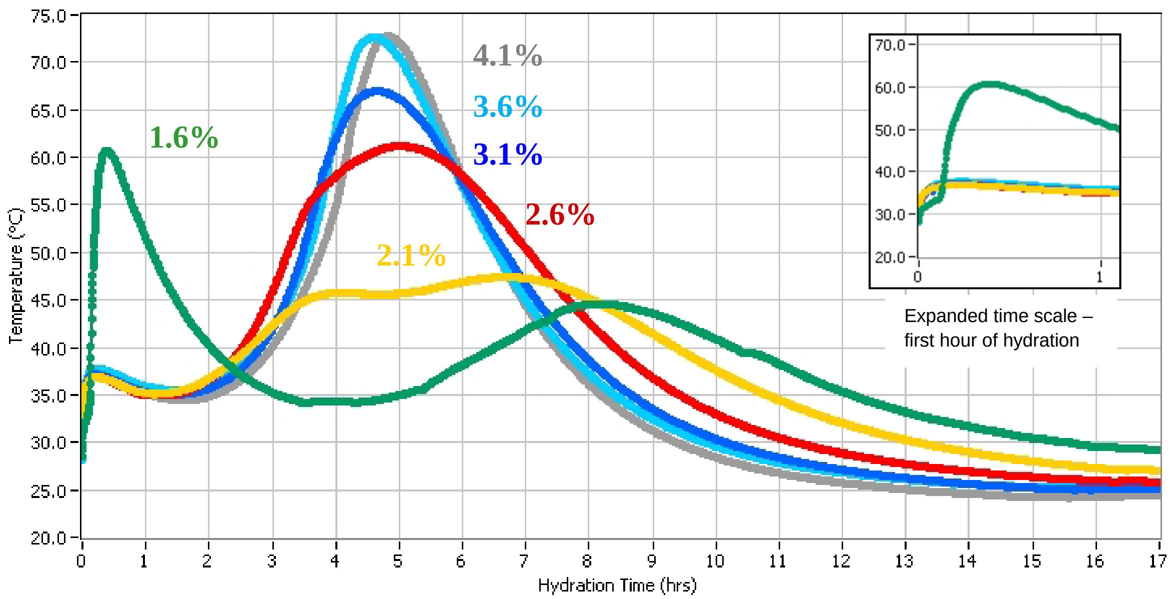


Figure 14.1. Paste Temperature (T_{samp}) vs. Time

Performance of successive mixtures was improved by calcium sulfate addition, preventing the uncontrolled early aluminate hydration. Thermal profile shapes were somewhat influenced by the mixture sulfate balance until mixture SO_3 levels approached "optimum" for early strength production (likely around 3.6% or so based on thermal profiles – this is consistent with established production SO_3 optimum for this source considering results obtained using *PoP* additions would be expected to differ slightly).

SO_3 content is optimized for commercial cements using various methods including *ASTM C563*, which generally result in SO_3 levels that produce near-optimum strength performance at a selected age and also provide some excess SO_3 in reserve for the higher "sulfate demand" conditions common to concrete and other mixtures with chemical and mineral admixtures at temperatures higher than laboratory ambient. This is practical since beyond the absolute minimum SO_3 needed for control of early aluminate hydration, SO_3 content is not an especially sensitive parameter to cement performance. Since such sulfate demand influences would not be present in slurry mixtures used for disaster recovery construction, reducing SO_3 in a special cement for the application could be useful in minimizing set time, as long as minimum SO_3 requirements for aluminate control are met. Based on these results, it appears that *SC2* was actually produced below this minimum; a safer target SO_3 for this material would appear to be in the 2.1% to 2.6% range. Since SO_3

optimums and minimum requirements for aluminate control are specific to the cement source and affected by many materials and production variables, it would be recommended that this be evaluated for any source of a special cement.

It should be noted that this type of evaluation of minimum SO_3 requirement was not performed for *SC1*. While the SO_3 of this material is only slightly higher than that of *SC2*, it is not likely too near this "minimum" since it is significantly coarser (lower Blaine). SO_3 optimums and minimum thresholds for aluminate control increase as fineness increases, other factors being equal.

SC2 performed better in *Soil 2* than the *A T III* control (Table 9.5). The cause is unknown but is speculated to be related to the fineness increase more than SO_3 reduction. *SC1* performed better than *A T III* for (5, 100) and (15, 233). The properties of *Soil 2* are speculated to have affected behaviors, but no *SAC* data could be collected to support this position.

14.3 Paste Mixture *SAC Series 2*

Chapter 9 reports performance trends that seem to differ for different soils according to cementitious SO_3 levels. Initially this was a perceived concern, so it was investigated with 1 soil and a few cements; the investigation occurred prior to the completion of Chapter 9. To investigate whether organic or mineral content of soils used in testing might chemically interact with cementitious stabilization materials to an extent that should be considered in special cement chemistry definition, the effects of selected soil slurry preparations were evaluated using *SAC* paste mixtures, and compared with similar paste mixtures made using laboratory mix water alone. *SAC Series 2* is an example of this work.

Neat paste mixtures with *w/c* of 0.55 (selected for ideal paste consistency) were tested at 23 C for comparison with similar slurry mixtures also with *w/c* of 0.55. Slurry with a concentration of 450 g/L of *Soil 1* was conditioned for 48 hr prior to use. Batch quantities for neat paste mixtures were 700 g cement and 385 g water, while slurry mixture batch quantities were 700 g cement, 385 g water, and 173 g soil solids. Cements included in testing were *A T III*, *Th T III*, and *SC1*. Figure 14.2 shows the thermal profile comparisons.

Note that profile relationships in the Figure 14.2 comparisons are similar for *Soil 1* slurry mixtures as compared with water mixtures, and the only clearly apparent influence of the slurry material is a slight acceleration of hydration likely due to some chlorides content in the soil. Based on this comparison, chemistry influences of *Soil 1* do not appear to be the direct result of sulfate balance effects alone. It may or may not be possible to predict optimum special cement properties based on the knowledge of specific soil characteristics in the disaster area, and recommendations for in-project testing for development of special cement properties as described in Section 9.5 are appropriate.

Additional testing using *SAC* in conjunction with several soils and several cements could prove useful. The data could determine if there is a correlation among soil properties, cement properties, and strength gain. The investigation was beyond the scope of this project. Testing of *Soil 2* using *SAC* would have also proved useful, but insufficient material was available.

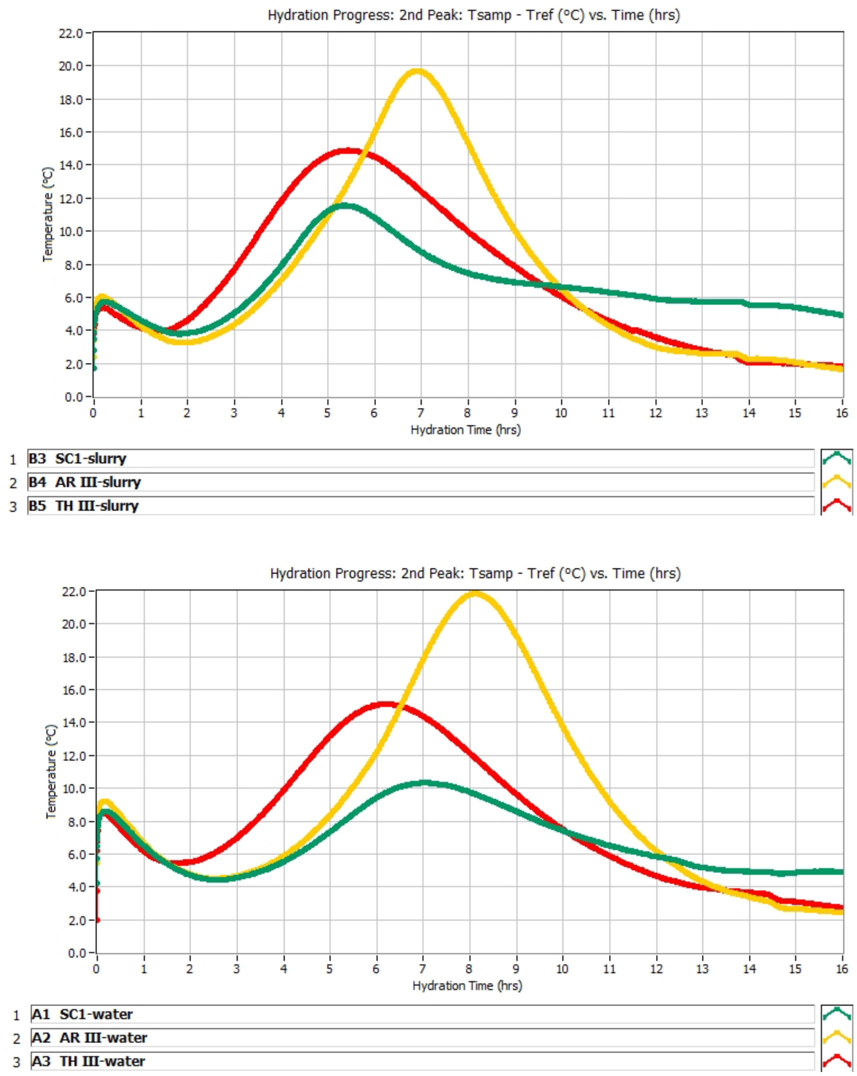


Figure 14.2. Paste Temperature (T_{samp} – T_{ref}) vs. Time, Water (top) vs. Slurry (bottom) Mixtures at Lab Ambient, w/cm = 0.55

14.4 Slurry Mixture SAC *Series 3*

Slurry mixture testing within *Series 3* was performed with 3 sub-series (A, B, C). Series A evaluated equipment types, Series B evaluated curing temperatures, and Series C evaluated different mixture proportions. Each series is described individually in the remainder of this section

14.4.1 Slurry Mixture Series A -- Equipment Evaluation

In Series A, 4 mixtures were tested (Figure 14.3). Temperature from all devices was compiled and synchronized for direct comparisons. All specimens were tested at laboratory temperatures (i.e. no preconditioned materials). Figure 14.3 thermal profiles represent the heat generated from cement hydration. Tests were conducted using default sensor calibration and without using an inert reference specimen to track the effects of changing ambient temperatures.

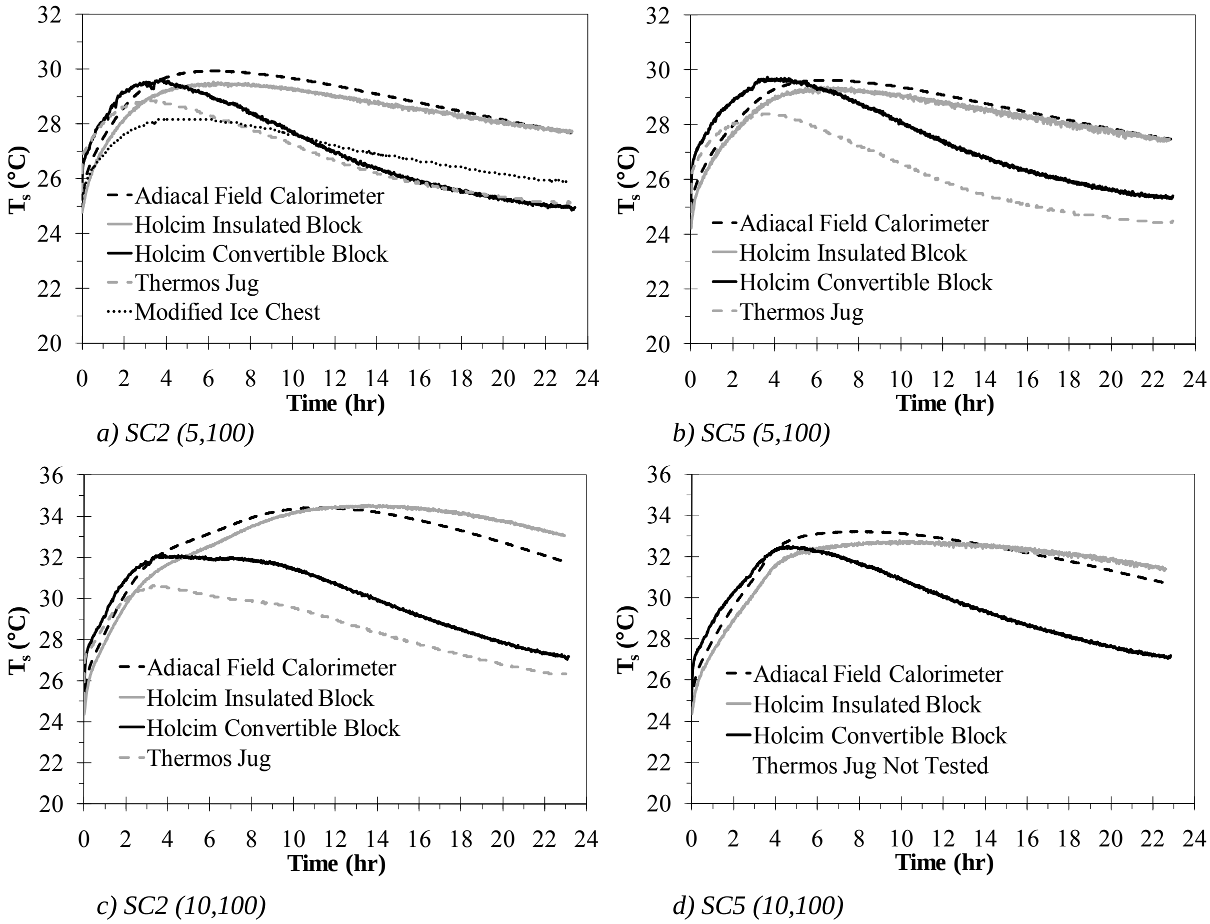


Figure 14.3. Series A -- Soil 1 100% Moisture

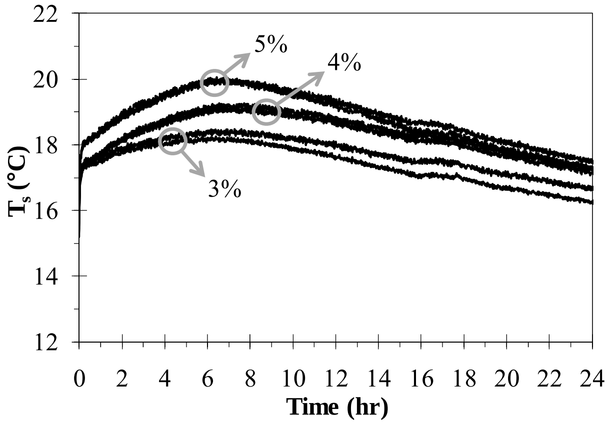
Overall, the calorimetry devices demonstrated adequate signal to noise ratios to produce useful data. The cement dosage rate had a noticeable effect on the thermal profiles. The 10% mixtures recorded peak temperatures 3 to 4 degrees higher than 5% mixtures. Other than the shift due to cement dosage, the relationships of thermal profile peaks, shapes, and areas among the 4 mixtures were consistent and similar for all devices. The peaks, shapes, and areas appeared to vary from 1 device to the other due only to differences in sample mass and device insulation.

The *Holcim Convertible Block* was selected as the most appropriate device as it provides an ideal specimen size for soil cement *UC* testing. An insulated lid for the *Holcim Convertible Block* was later found to reduce ambient temperature effects, and the use of an inert reference specimen likewise improved data.

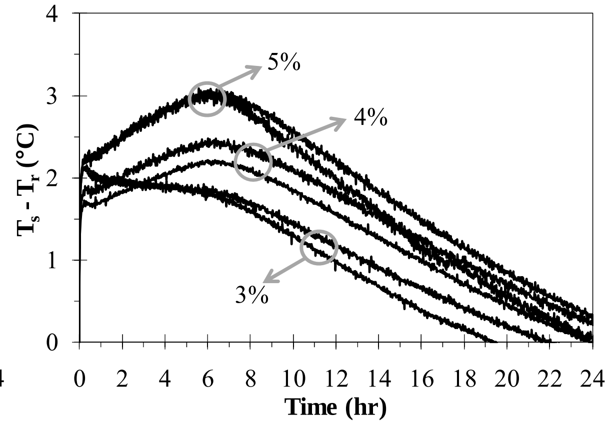
14.4.2 Slurry Mixture Series B -- Varying Curing Temperatures

In Series B, the *Holcim Convertible Block* with insulated lid was used. *SC5* was used at 3 dosage rates (3, 4, and 5%), in 3 separate experiments at different curing temperatures

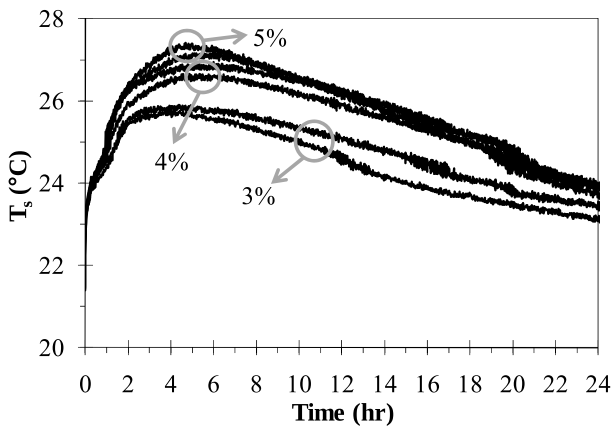
(15, 21, and 30 °C). Figure 14.4 shows thermal profiles recorded during testing. Sudden offsets in the data are due to curing chamber heating and cooling cycles, and most of the signal noise can be attributed to the serial port connection between the data logger and computer.



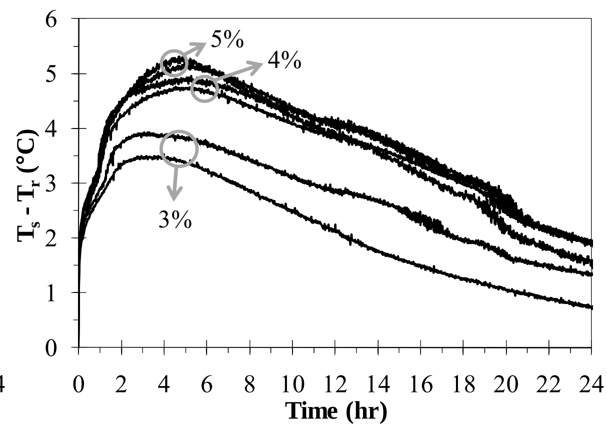
a) Initial Material Temp. 15°C



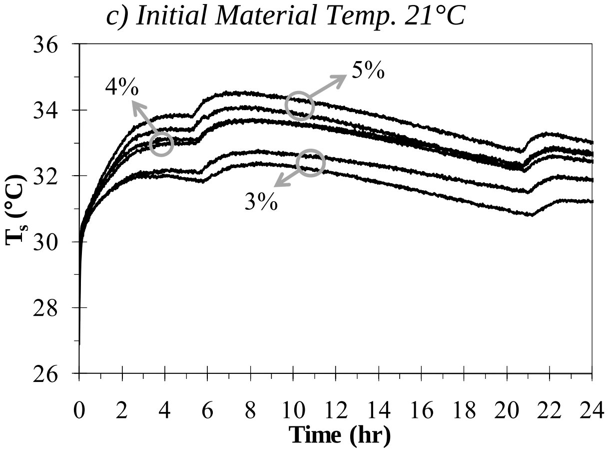
b) Initial Material Temp. 15°C



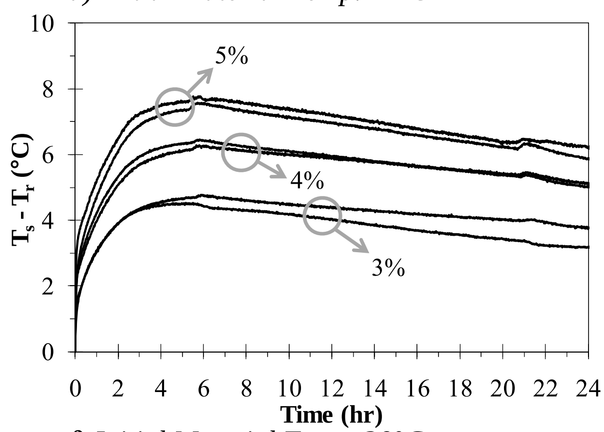
c) Initial Material Temp. 21°C



d) Initial Material Temp. 21°C



e) Initial Material Temp. 30°C



f) Initial Material Temp. 30°C

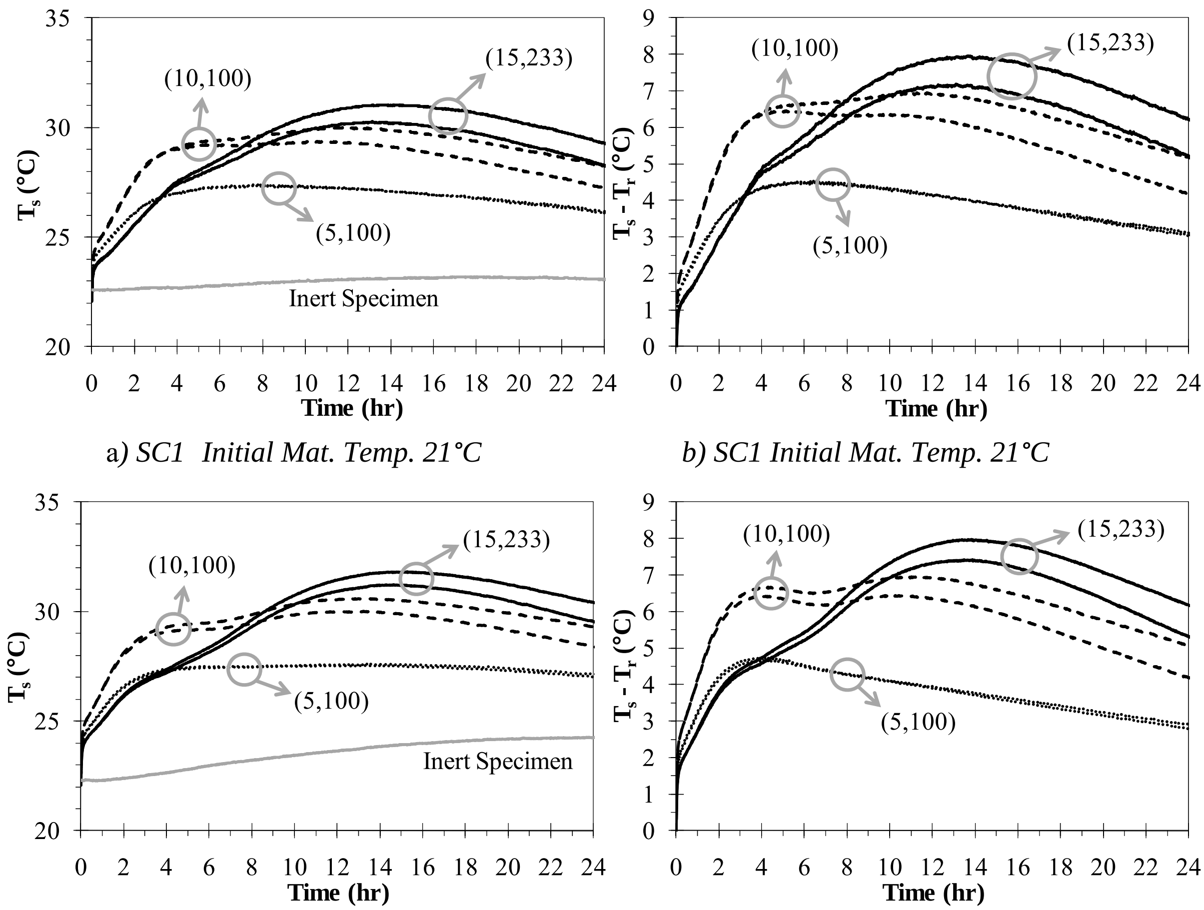
Figure 14.4. Series B-- Soil 1 100% Moisture, SC5

Overall, the *Holcim Convertible Block* produced useful data for all dosage rates at all curing temperatures. The use of an inert reference specimen improved the data quality and provided a more realistic representation of specimen heat generation. The use of an insulated lid for the *Holcim Convertible Block* helped improve the data, and a pre-test calibration of thermocouple sensors also improved data quality.

Peak temperature changes of 2 to 3 °C were observed during the cold temperature experiment, and peak changes of 3 to 5 and 4 to 7 °C were observed for the room temperature and hot temperature experiments, respectively. Figures 14.4b, 14.4d, and 14.4f show temperature change due to cement hydration over time. There were some discrepancies with initial mixture temperatures. Materials and *SAC* equipment were conditioned prior to specimen preparation, but temperatures at the start of data collection were not the same as initial temperatures. Discrepancies are very likely due to the heating/cooling of mixing materials when taken from the curing chamber for mixing. To alleviate these discrepancies a constant correction factor was used to adjust the reference specimen throughout the testing period as it was not removed from the curing chamber. The correction factor was different for every specimen and taken as the first recorded temperature difference between T_s and T_r since they are equal for conditioned materials. This approach is more realistic to compare specimen thermal profiles, though better temperature control during specimen preparation is needed in future efforts. Figure 14.4 plots adjusted data. Adjustment was only needed for 15 and 30 ^{o}C data (≈1.8 and 2.9 ^{o}C, respectively) as 21 ^{o}C was near zero (≈0.1 ^{o}C).

14.4.3 Slurry Mixture Series C -- Varying Mixture Proportions

In Series C, the *Holcim Convertible Block* with insulated lid was used, and 6 mixtures were tested at 21°C. Figure 14.5 shows thermal profiles recorded during testing. Again, it is clear that cement dosage rate has a noticeable effect on the profiles. The (15,233) mixtures (i.e. 15% cement and 233% moisture) have the highest peak and change in temperature followed by (10,100) and (5,100) mixtures, respectfully. Although *SC1* and *SC2* mixes achieve approximately the same magnitude for each mixture, the shapes of the thermal profiles are slightly different likely due to slightly different chemistry and physical properties of the 2 cements.



a) *SC1 Initial Mat. Temp. 21°C*

b) SC1 Initial Mat. Temp. 21°C

c) SC2 Initial Mat. Temp. 21°C

d) SC2 Initial Mat. Temp. 21°C

Figure 14.5. Results of Series C -- Soil 1

14.5 Summary of *SAC* Testing

Testing cement paste indicated *SC2* was likely produced with too little SO_3. A more reasonable SO_3 content would likely have been on the order of 2.1 to 2.6%. Testing cement paste with a moderate amount of *Soil 1* indicated portland cement and *Soil 1* interaction was not a major issue. Testing more cements and soils would be required to make more definitive statements.

SAC showed merit for cement stabilized soil slurries. The *Holcim Convertible Block* offers the most advantages with respect to equipment configuration and is well suited for disaster recovery quality control operations. The equipment is economical, portable, requires minimal operator skill, and provides meaningful data with an adequate signal to noise ratio. The most useful disaster recovery application is likely evaluating in place mixing effectiveness of cementitiously stabilized soil. The use of *SAC* on cement stabilized soils is still in its infancy and additional investigations to improve data quality (e.g. equipment improvements to reduce signal noise and improved preparation protocols at lowered and elevated temperatures) are needed. However, the process as presented in this report is suitable for implementation during disaster recovery.

CHAPTER 15 – HAND HELD GAGE AND *UC* TEST RESULTS COMPARISON

15.1 Overview of Comparison

This chapter takes data presented previously and compares shear strength measured with hand held gages to that of *UC* testing. *UC* measured shear strength was taken as the correct value since it is a well accepted method of testing soil specimens. Accuracy of the gages is the primary variable under consideration in this chapter as precision was addressed in Chapter 7;however, the *Dial* gage was observed to be the most precise.

15.2 *Combined* Test Results

The *Ring* gage provided higher strengths than *Shear* or *UC* methods, which aligned reasonably well (Figure 15.1). Dashed lines from *Surface* measurements align reasonably well with *Bottom* measurements. *UC* specimens had a 1:1 rather than 2:1 aspect ratio and were sawn and extruded prior to testing. Maximum *UC* strains were 6.6 to 8.6%, which is higher than observed for the 2:1 aspect ratio testing discussed previously. The same suite test condition resulted in a shear strength and maximum strain of 0.83 kg/cm^2 and 5%, respectively. Figure 15.1 data should not be interpreted as more than an initial estimate; specimens could have been damaged during extrusion or sawing.

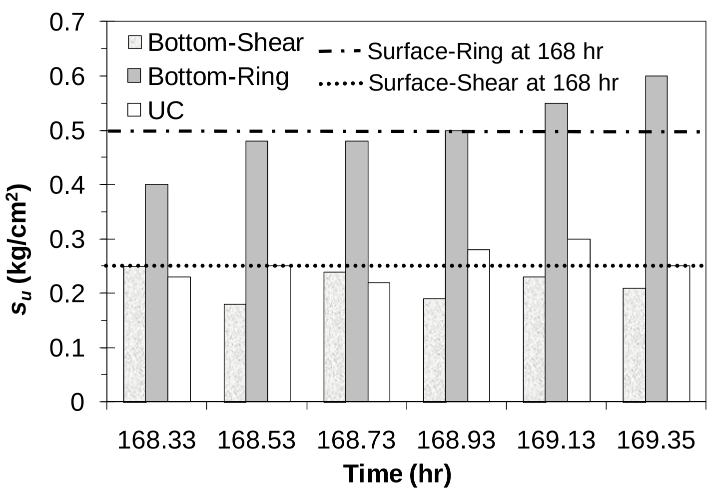


Figure 15.1. Results of Combined Testing: Soil 1 Th T I/II (5, 100) (TTF ≈ 3,700 C-hr)

15.3 Hand Held Gage and *UC* Measured Strength (5, 100) Comparison w/out Fibers

The *w/cm* ratio was 10 for all testing discussed in this section. The accuracy (proximity to true value) of the hand held gages was evaluated in terms of a gage to *UC* strength ratio defined as the shear strength of the gage in question divided by the *UC* measured shear strength for the same cement blend at the same temperature-time factor. *UC* measured values were taken as correct. The first investigation used trendlines from trials measured by the hand held gages and *UC* suite trendlines. The second investigation used mean values measured from variability slab testing and *UC* suite trendlines.

15.3.1 Comparison Using Trendlines at (5, 100)

In general, data collected from the spring of 2008 through the summer of 2009 was used for the hand held gage to *UC* strength comparison presented in this section. *Soil 1* was not as uniform between all barrels as originally believed, and the analysis in this section addresses the issue to the extent possible by using soil from similar test durations. *Soil 1* shear strength was shown to vary up to approximately a factor of 3 depending on the barrel where the soil was sampled (Figure 8.1). This section's comparison made use of twenty-7 trials having a total of 2,844 readings per gage. Each of the 3 soils was tested with 9 cementitious blends; 7 of the blends were described in Section 7.3.1 in Figures 7.10 through 7.18 and 2 of the blends are plotted in Figures A.25 to A.27. *UC* data from corresponding *Protocol 1* suites (297 data points) were used as the control.

Figures 15.2 and 15.3 plot *Soil 1* test results. Seven of the 9 plots had *UC* measured strength as the highest of the 4 measurements, while the other 2 plots had *Ring* measured strengths as the highest. *Shear* measured strengths were the lowest measurement for 8 of the 9 plots, with *Ring* measured strength the lowest for 1 of the 9 plots. The *Dial* gage was never the highest or the lowest measurement.

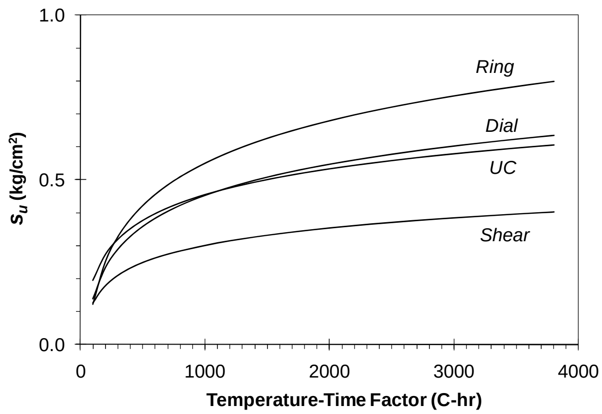
Figures 15.4 and 15.5 plot test results for *Soil 2* measured by all 4 methods. All 9 plots had *Dial* measured strength as the highest. Seven of the 9 plots had the measured strength order beginning with the highest as *Dial*, *Ring*, *Shear*, and *UC*. *UC* strengths were the lowest measured with exception of *SC1*, where *UC* and *Shear* readings were near identical and significantly lower than *Ring* and *Dial* readings.

Figures 15.6 and 15.7 plot test results for *Soil 3* measured by all 4 methods. Seven of the 9 plots had *Dial* measured strengths as the highest, while 2 of the plots had either *Shear* or *UC* readings as the highest (note these 2 plots were considerably weaker than the other 7 plots). Six of the 9 plots had *Dial* and *Ring* measured strengths exceeding *UC* and *Shear* measured strengths.

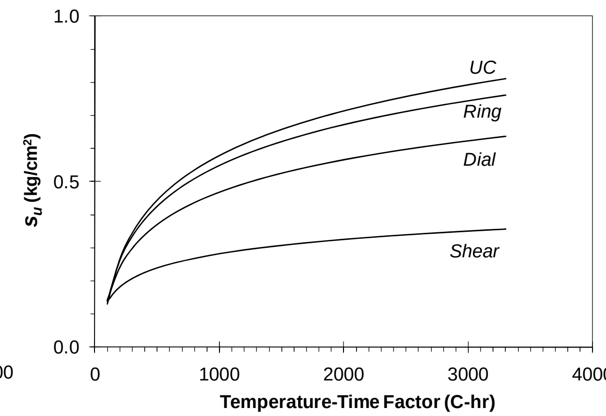
Figures 15.8 through 15.10 were developed using all test data collected by plotting data into bins according to their gage to *UC* ratio. Shear strength was calculated at *TTF* values of 100, 500, 1500, and 3500 C-hr using the logarithmic trend lines for each hand held gage and for *UC* testing. The ratio of hand held gage shear strength to *UC* determined shear strength was then determined for all twenty-seven test cases at each of the 4 *TTF* values. Thirty-six data points were available per gage and per figure. As an example, 1 data point used to create Figure 15.8 would be produced by calculating shear strength at 3500 C-hr using the *UC* and *Ring* trendline equations shown in Figure 15.2a (0.60 kg/cm^2 and 0.78 kg/cm^2, respectively) and taking the ratio (1.3).

For *Soil 1* (Figure 15.8), 90% of the penetrometer (*Dial* and *Ring*) test results were either in the 0.25 to 0.75 bin or the 0.75 to 1.25 bin relative to *UC* test results. Both gages had approximately 50% of their readings in the 0.75 to 1.25 bin. The *Shear* gage test results fell into the 0.25 to 0.75 bin 86% of the time and in the 0.75 to 1.25 bin 14% of the time. Shear strength over prediction was not common in *Soil 1*; strengths were under predicted in the majority of instances but not in all instances.

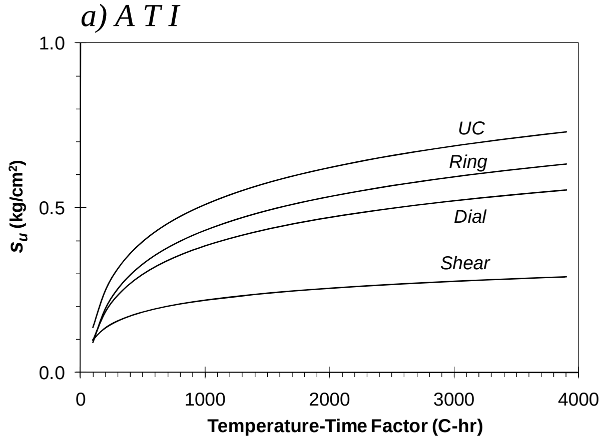
For *Soil 2* (Figure 15.9), none of the *Dial* and *Ring* test results were in the 0.75 to 1.25 bin with the majority of results in either the 1.75 to 2.25 or the 2.25 to 2.75 bins. Conversely, the majority of the *Shear* test results (56%) were in the 0.75 to 1.25 bin while 25% were in the 1.25 to 1.75 bin. Shear strength over prediction was common in *Soil 2* for both *Dial* and *Ring* readings. *Shear* gage measurements tended to be at or greater than *UC* readings. Behaviors in *Soil 2* were opposite to those in *Soil 1*.



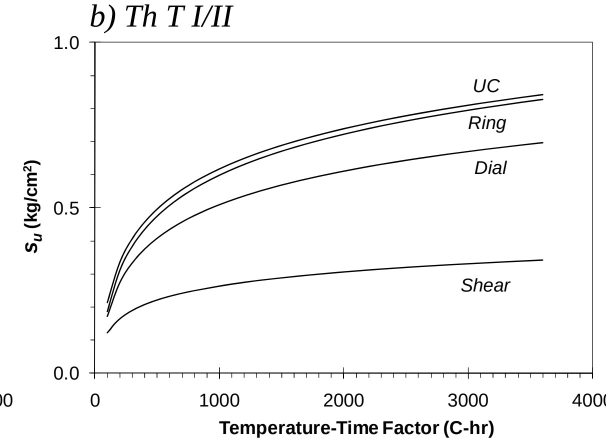
a) A T I



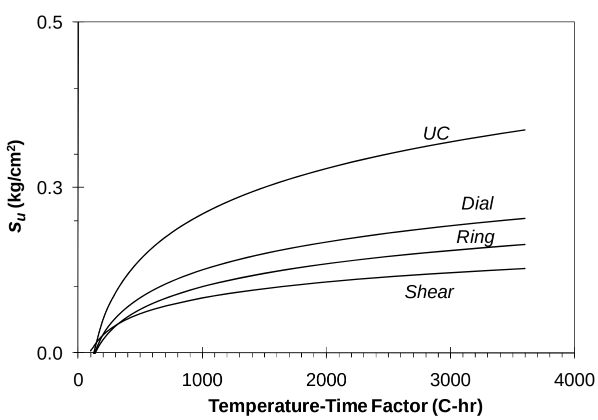
b) Th T I/II



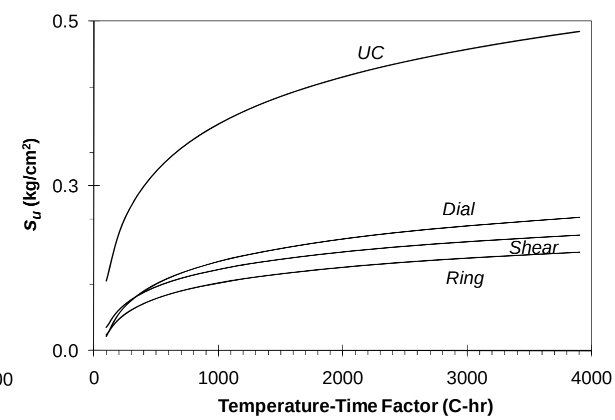
c) A T III



d) Th T III

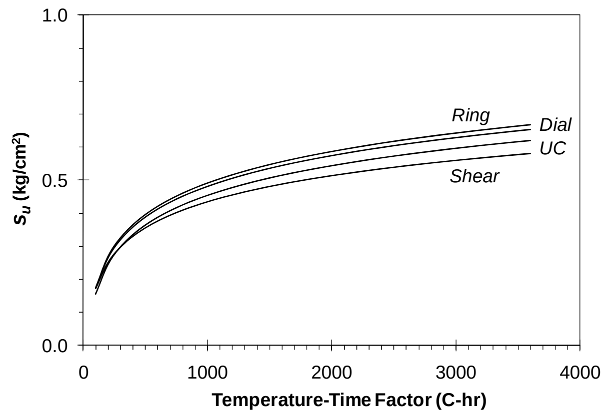


e) A T I (GGBFS)

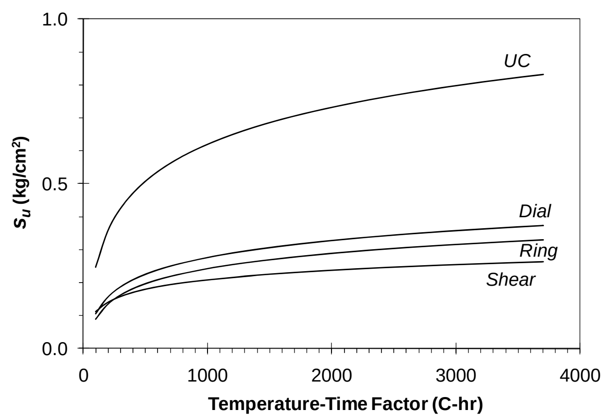


f) CTS RS

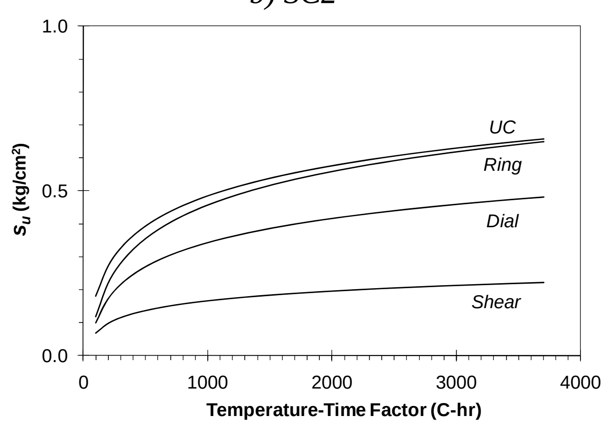
Figure 15.2. Shear Strength Test Results, (5, 100), Soil 1 (1 of 2)



a) SC1

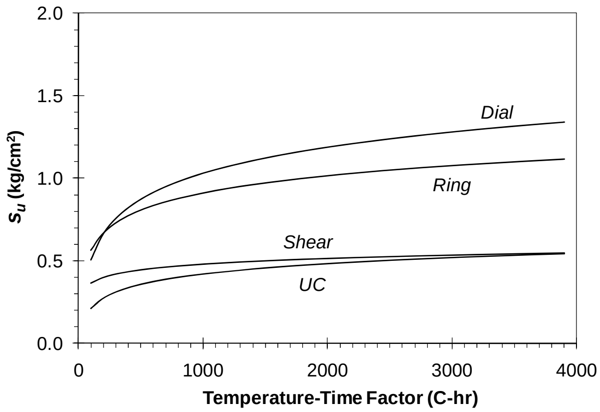


b) SC2

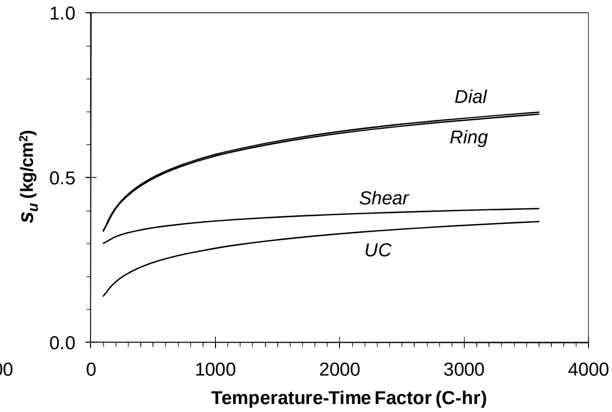


c) SC2 (PoP)

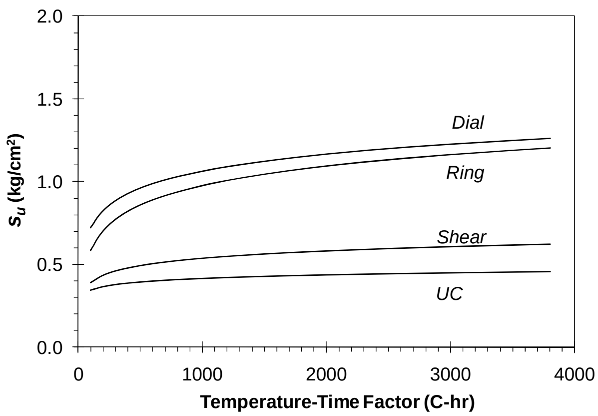
Figure 15.3. Shear Strength Test Results, (5,100), Soil 1 (2 of 2)



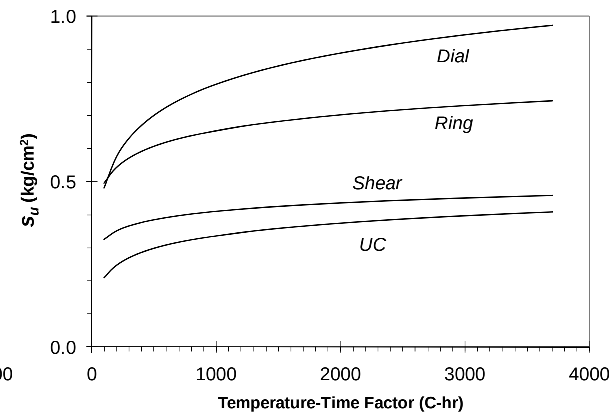
a) A T I



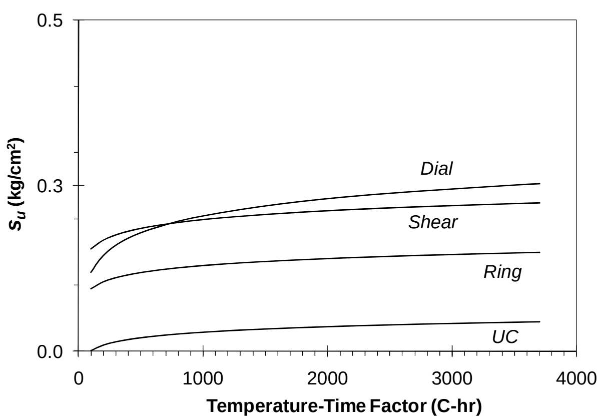
b) Th T I/II



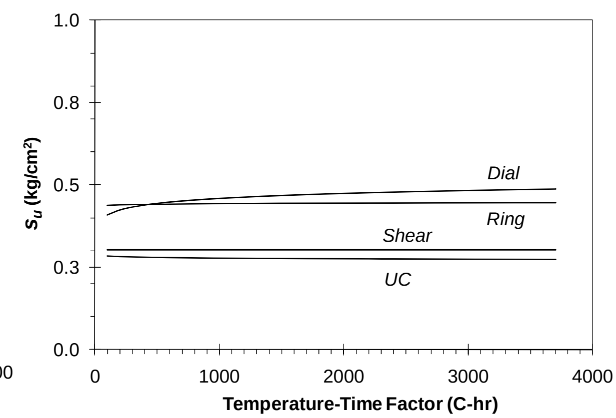
c) A T III



d) Th T III

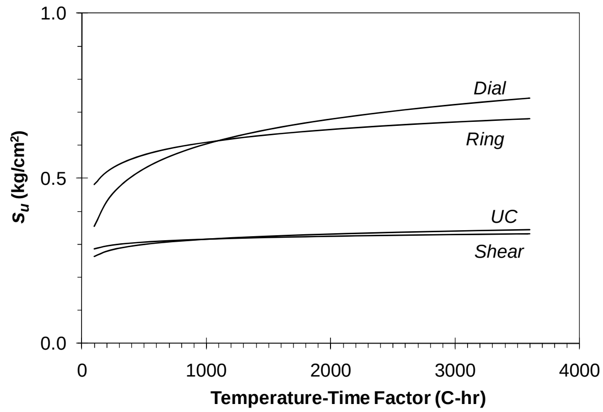


e) A T I (GGBFS)

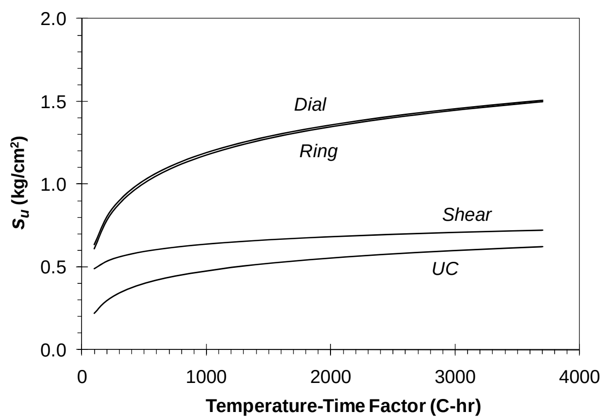


f) CTS RS

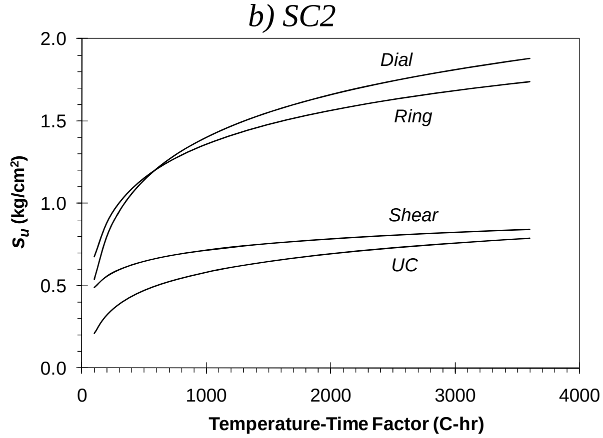
Figure 15.4. Shear Strength Test Results, (5,100), Soil 2 (1 of 2)



a) SC1

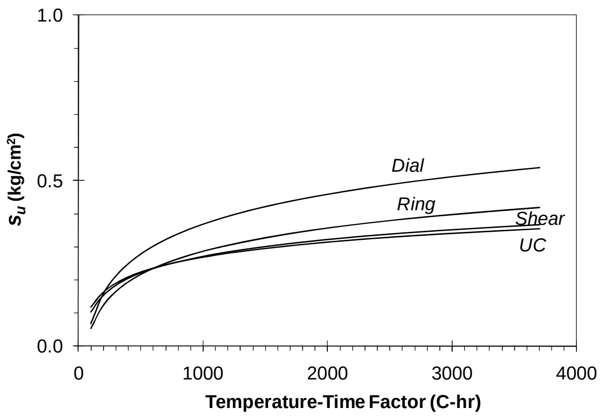


b) SC2

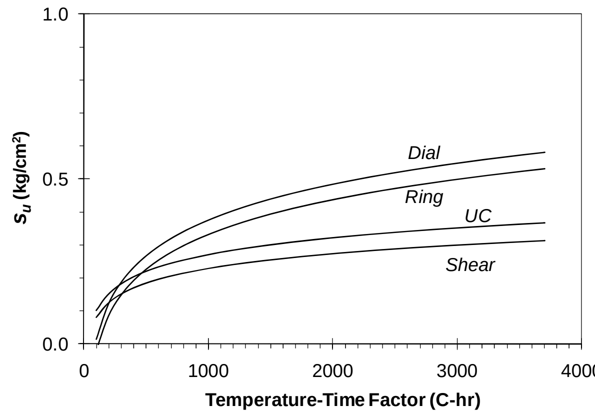


c) SC2 (PoP)

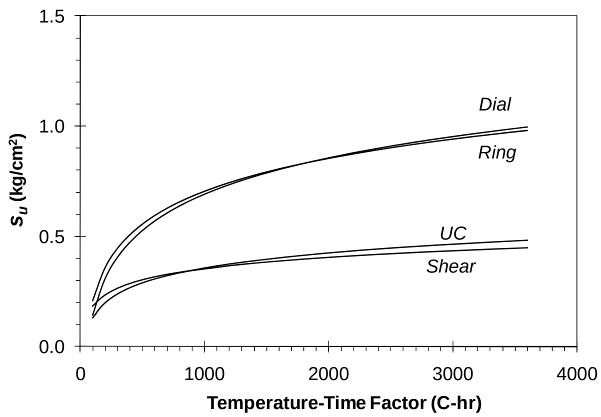
Figure 15.5. Shear Strength Test Results, (5, 100), Soil 2 (2 of 2)



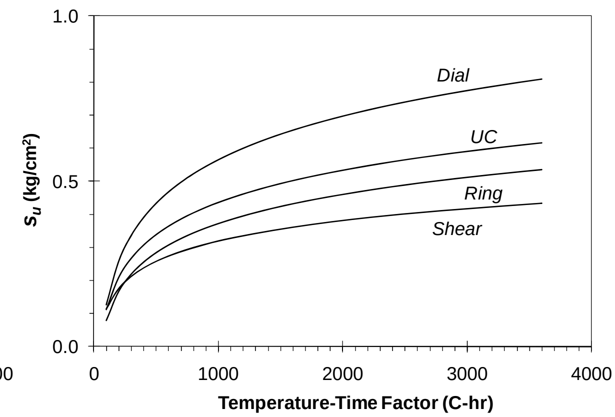
a) A T I



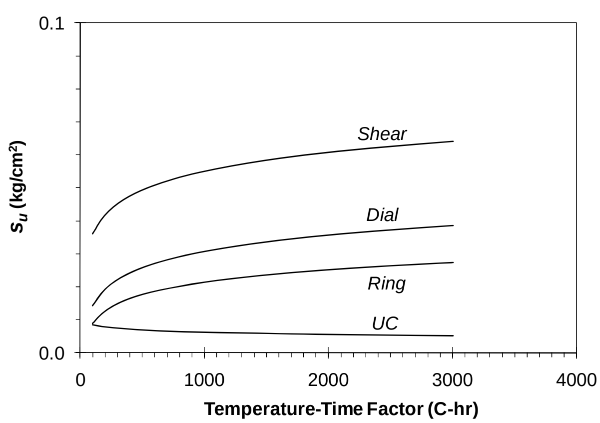
b) Th T I/II



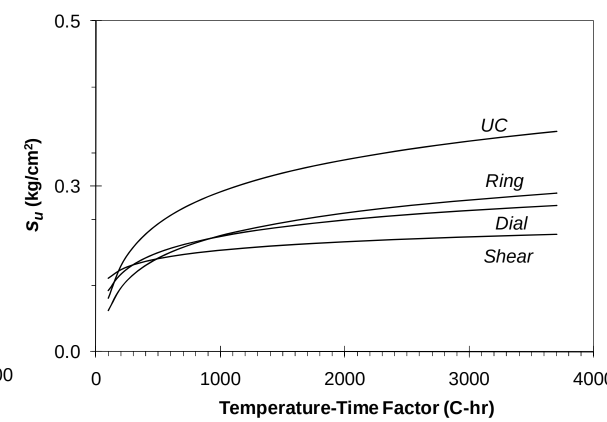
c) A T III



d) Th T III

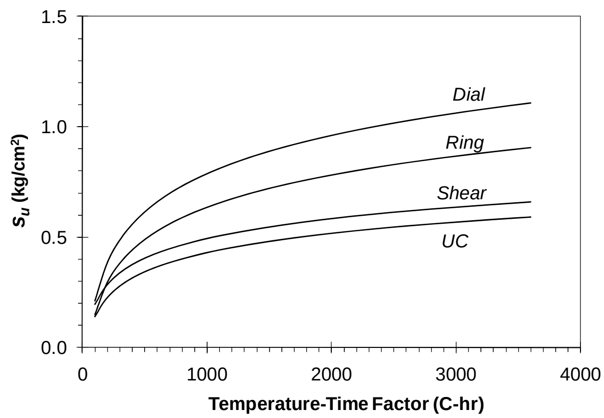


e) A T I (GGBFS)

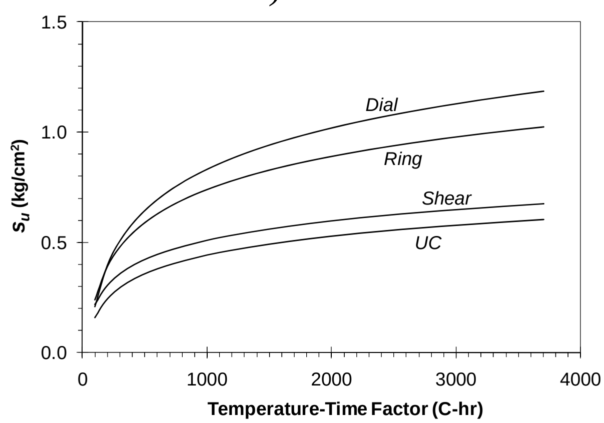


f) CTS RS

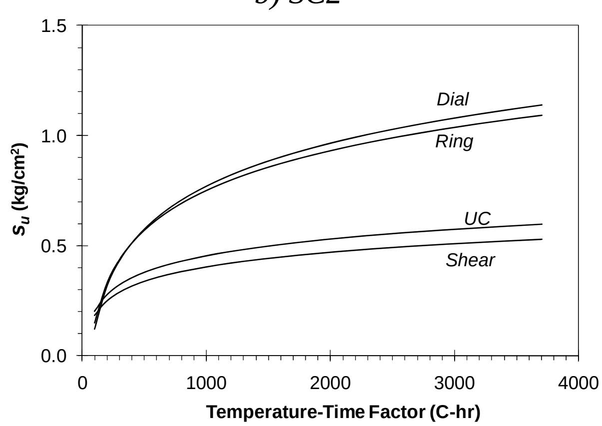
Figure 15.6. Shear Strength Test Results, (5,100), Soil 3 (1 of 2)



a) SC1



b) SC2



c) *SC2 (PoP)*

Figure 15.7. Shear Strength Test Results, (5, 100), Soil 3 (2 of 2)

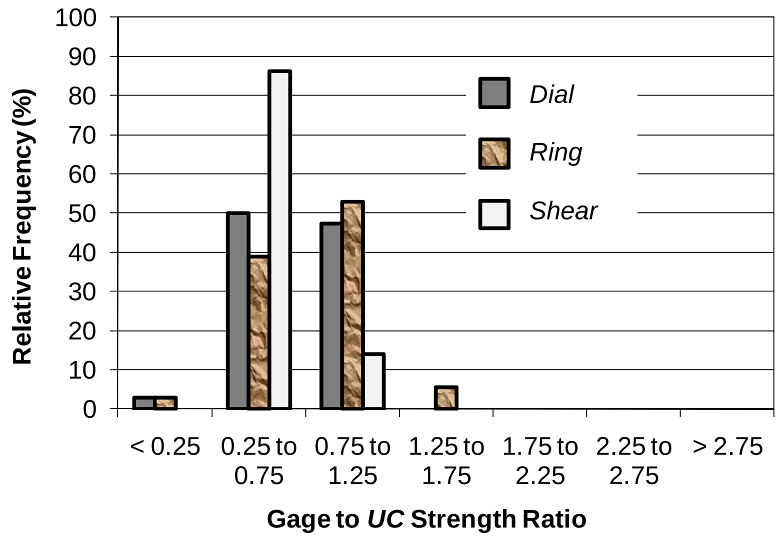


Figure 15.8. Gage Comparisons for Soil 1 (5, 100) Using Trial Data

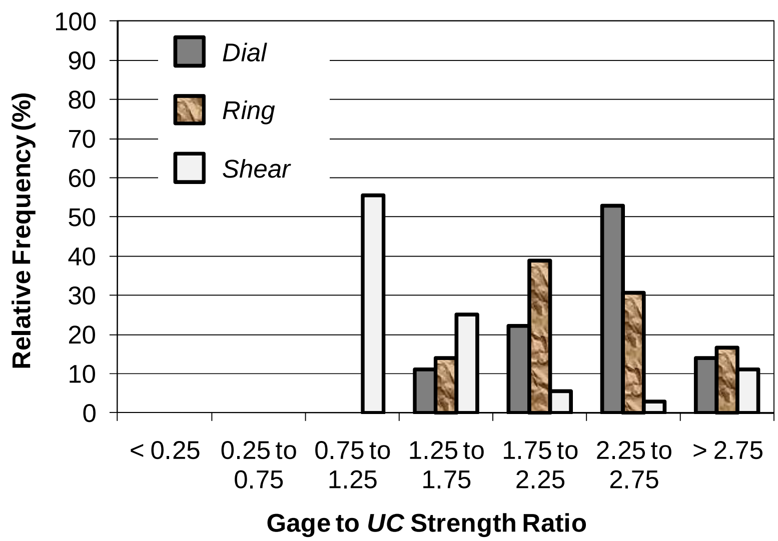


Figure 15.9. Gage Comparisons for Soil 2 (5, 100) Using Trial Data

For *Soil 3* (Figure 15.10), *Dial* and *Ring* test results were dispersed within multiple bins with no more than 33% of the *Dial* results nor more than 31% of the *Ring* results residing in any bin. *Shear* results were more concentrated with 64% of the readings occurring in the 0.75 to 1.25 bin. Shear strength was both over predicted and under predicted in *Soil 3*. The behavior in *Soil 3* was a mixture of the behaviors in *Soil 1* and *Soil 2*; organic content of *Soil 3* was between the values of the other soils.

On rare occasion was a gage to *UC* ratio calculated below 0.25, and with exception of 1 cementitious material, gage to *UC* ratios were rarely above 2.75. Figure 15.11 plots average gage to *UC* ratios excluding values below 0.25 and above 2.75 as a function of soil organic content. A clear trend was observed; as the organic content increased so did the gage to *UC* ratio. *Dial* and *Ring* readings were essentially linear, while the *Shear* gage ratio tapered off at higher organic contents. The data indicates that organic content of the soil should be taken into account when considering the use of hand held gages to measure shear strength.

Figure 15.10 data could be used as a calibration factor (multiply s_u by factor) for similar materials and stabilization material contents. Values of 1.25, 0.50, and 0.75 provide reasonable mean estimates for *Soils 1*, *2*, and *3*, respectively when using the penetrometers. Values of 2, 0.75, and 1 provide reasonable estimates for *Soils 1*, *2*, and *3*, respectively when using the miniature vane shear device. Care should be exercised when using a single factor, and the distribution of the data shown in Figures 15.8 through 15.10 should be considered. The factors, however, do provide some guidance. The scatter observed makes is difficult to assign a factor to any condition if a high level of confidence is required of the prediction. The data suggests that an on site calibration using *UC* data may be the most effective manner in which to use the hand held gages. On site *UC* testing could be performed relatively easily.

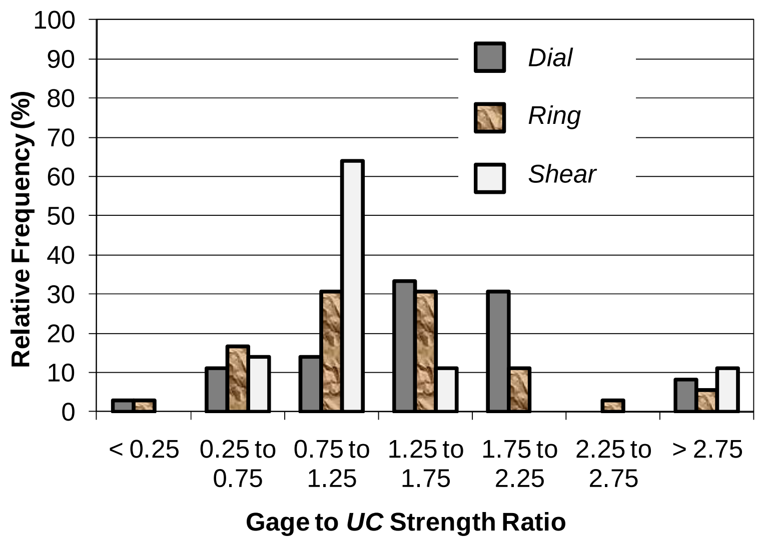


Figure 15.10. Gage Comparisons for Soil 3 (5, 100) Using Trial Data

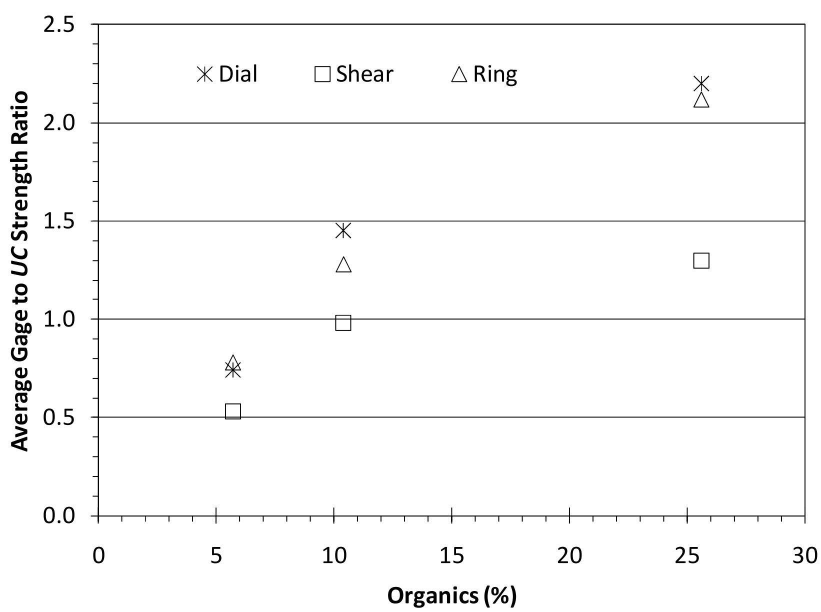


Figure 15.11. Gage Comparisons vs. Organics

Accuracy was observed to be a function of the organic content in the soil with measured shear strength increasing relative to *UC* data as a function of organic content for all 3 hand held gages. The *Shear* gage, in general, predicted the lowest strengths of the 3 gages and was the least affected by soil organic content. The *Shear* gage was the most accurate, while the accuracy of the *Dial* and *Ring* gages was similar. Calibration using *UC* measured values is recommended on the material under investigation to avoid errors.

15.3.2 Comparison Using Variability Slab Means at (5, 100)

Figures 15.12 to 15.14 compare variability slab means to calculated values from *UC* trendlines (*Protocol 1*) at the same *TTF* factor (same as Section 15.3.1 approach). The *Soil 1* data should be questioned in the sense that it was taken from a variety of barrels over an extended period of time. Barrel numbers were not recorded for variability slabs, though it is known that variability slab testing occurred after *Protocol 1 UC* testing. The gage to *UC* ratios of Figure 15.12 are, in general, higher than those in Figure 15.8 and are more dispersed; both behaviors could be due to soil taken from different barrels. Additional investigation of the behavior with *UC* test data from later in the testing program was not performed for this report.

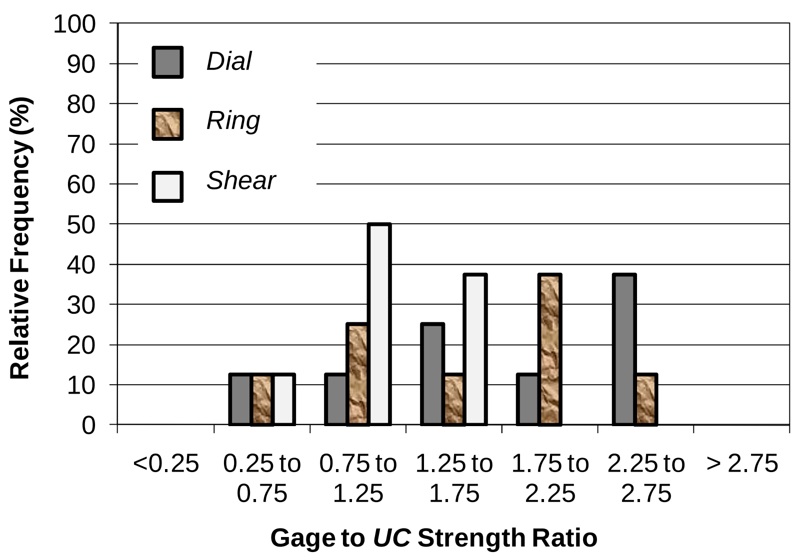


Figure 15.12. Gage Comparisons for Soil 1 (5, 100) Using Variability Slab Means

Figure 15.13 is similar to Figure 15.9, where the *Shear* gage was the only device that predicted strength data in the 0.75 to 1.25 bin for *Soil 2*. All remaining data was above a gage to *UC* ratio of 1.25 indicating considerable strength over prediction with the hand held gages in highly organic soils. Figure 15.14 is similar to Figure 15.10 in the sense that the 0.75 to 1.25 and 1.25 to 1.75 bins are populated with a fair amount of data from all 3 gages. The *Shear* readings are evenly distributed between these 2 bins in Figure 15.14, whereas in Figure 15.10 the 0.75 to 1.25 bin had significantly more readings. Generally speaking, *Soil 2* and *Soil 3* performed in a comparable manner with trendline calculated values or with measured variability slab mean values.

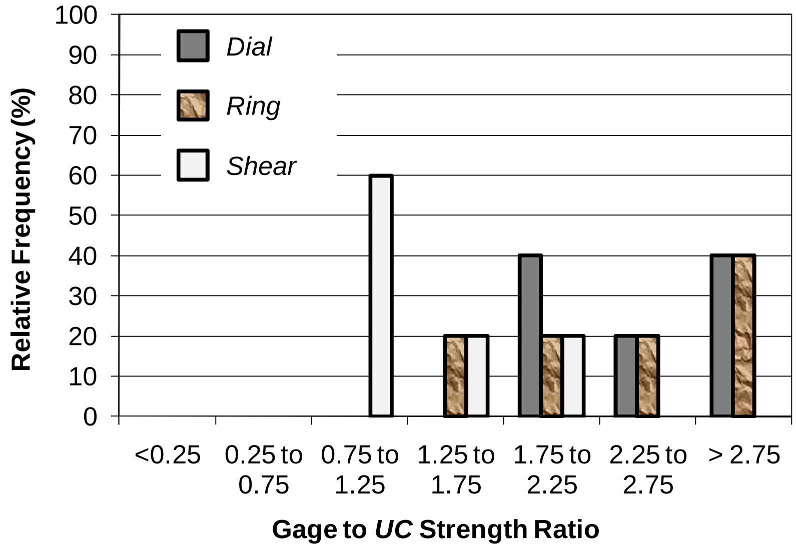


Figure 15.13. Gage Comparisons for Soil 2 (5, 100) Using Variability Slab Means

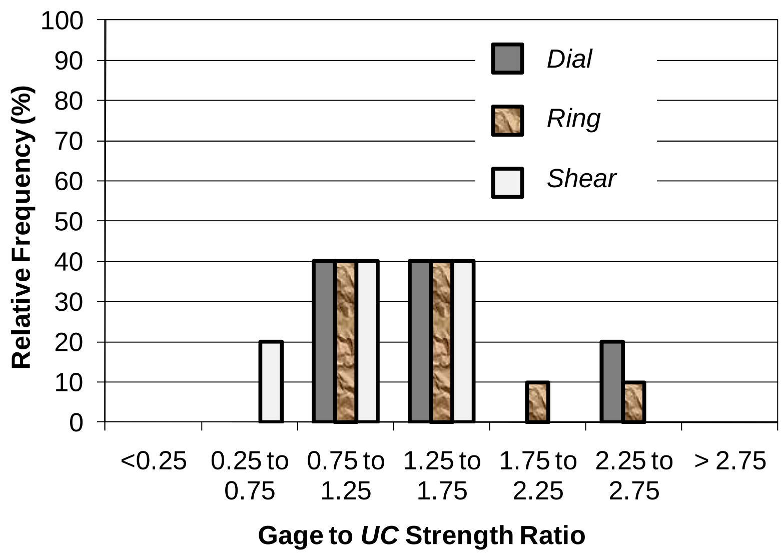
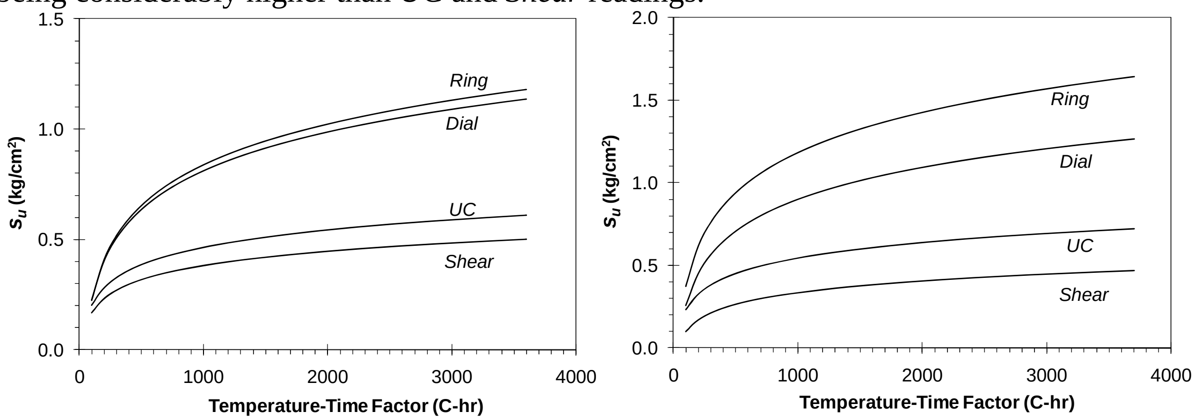


Figure 15.14. Gage Comparisons for Soil 3 (5, 100) Using Variability Slab Means

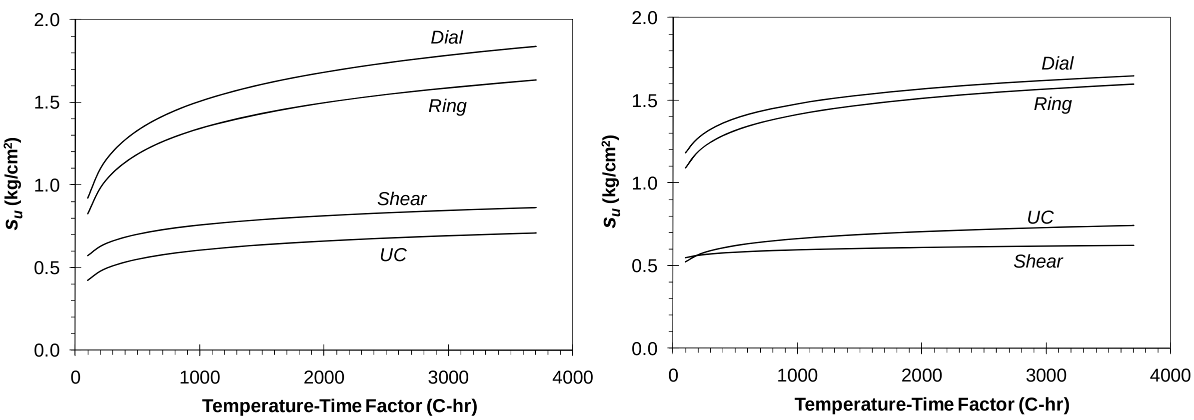
15.4 Hand Held Gage and *UC* Measured Strength (5, 100) Comparison With Fibers

Figures 15.15 through 15.17 plot test results for *Soils 1* through *3* stabilized with portland cement and fibers (632 readings per hand held device). *Protocol 1 UC* data was used as the control in all plots. For *Soil 1* the order of strength from highest to lowest was *Ring*, *Dial*, *UC*, and *Shear*, with *Ring* and *Dial* readings being considerably higher than *UC* and *Shear*. For *Soil 2*, *Dial* measured strengths were always highest followed by *Ring* measured strengths. *UC* and *Shear* measured strengths were considerably lower than *Dial* and *Ring* measured strengths as with *Soil 1*. Unlike *Soil 1*, *Shear* measured strength exceeded *UC* measured strength in 1 of the 2 cases (Figure 15.16a). For *Soil 3*, the order of strength from highest to lowest was *Dial*, *Ring*, *Shear*, and *UC* with *Ring* and *Dial* readings being considerably higher than *UC* and *Shear* readings.



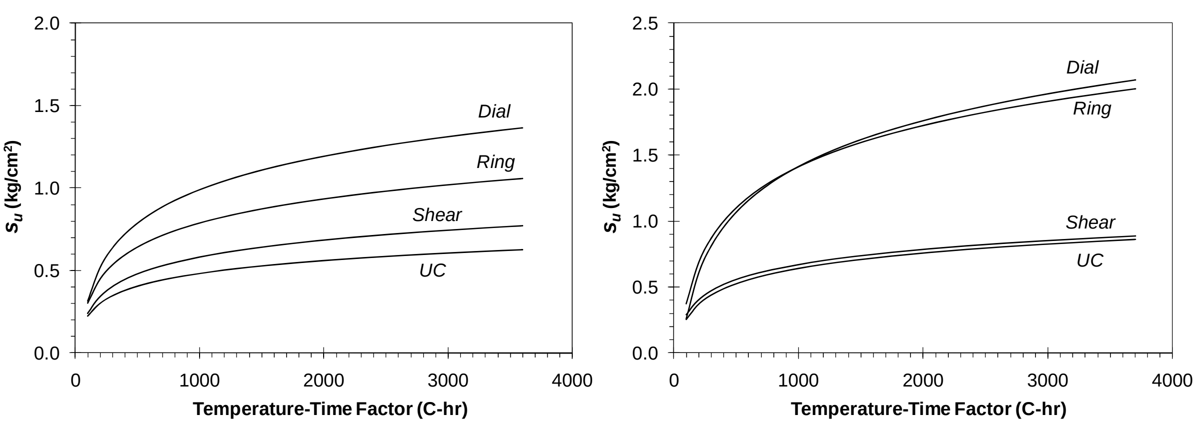
a) SC1 (F70) *b) SC1 (F20)*

Figure 15.15. Shear Strength Test Results, (5,100), Soil 1 with Fibers



a) SC1 (F70) *b) SC1 (F20)*

Figure 15.16. Shear Strength Test Results, (5, 100), Soil 2 with Fibers



a) SC1 (F70) *b) SC1 (F20)*

Figure 15.17. Shear Strength Test Results, (5, 100), Soil 3 with Fibers

Figures 15.18 to 15.20 were developed by organizing data into bins according to their gage to *UC* ratio in the same manner as Section 15.3. *Soil 1* fiber reinforced specimen strengths (Figure 15.18) measured with the *Shear* gage were in the 0.25 to 0.75 bin 50% of the time, which is comparable from a discussion standpoint to the non-fiber reinforced specimens (Figure 15.8). The other 50% of the fiber reinforced *Shear* gage data was in the 0.75 to 1.25 bin, which also occurs in Figure 15.8, though the distribution between the 0.25 to 0.75 and 0.75 to 1.25 bins is different between the figures. The *Dial* and *Ring* readings from the non-fiber reinforced specimens (Figure 15.8) have a considerable amount of data in the 0.75 to 1.25 bin, with nearly all the rest of the data in the 0.25 to 0.75 bin. The *Dial* and *Ring* readings from the fiber reinforced specimens (Figure 15.18) contrasts this data as it has no data in the 0.25 to 0.75 bin, a moderate amount of data in the 0.75 to 1.25 bin, and more data in the 1.25 to 1.75 and 1.75 to 2.25 bins than in any other. The data suggests that fibers led to some amount of strength over prediction in the penetrometers. Fibers also tended to increase *Shear* gage readings relative to *UC* specimens, but not to an extent where strength over prediction occurred nor to an extent where the data was conclusive with the data available.

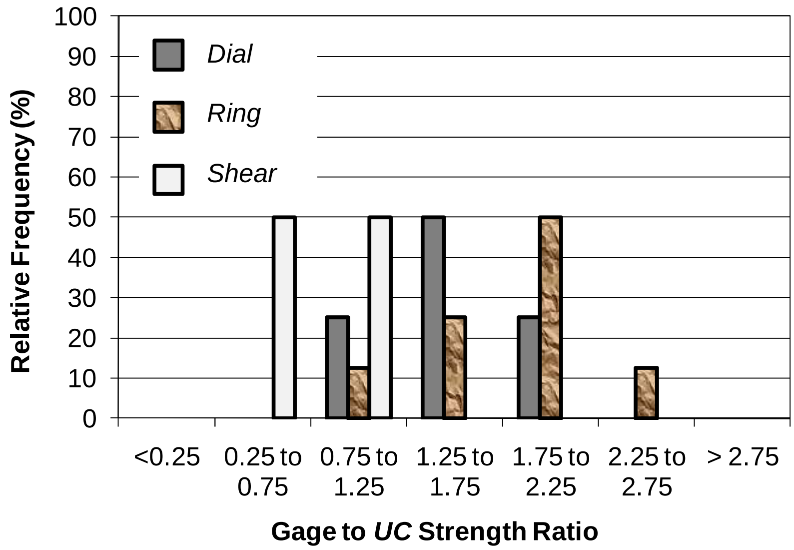


Figure 15.18. Gage Comparisons for Soil 1 and Fibers (5, 100) Using Trial Data

Soil 2 fiber reinforced specimen strengths (Figure 15.19) measured with the *Shear* gage were mostly in the 0.75 to 1.25 bin, with all remaining values in the 1.25 to 1.75 bin. In general this agrees with the non-fiber reinforced behavior presented in Figure 15.9 except in Figure 15.9 small amounts of readings were in the other bins. Fiber reinforced specimens were slightly more over predicted with the *Dial* and *Ring* gages than were Figure 15.9 non-fiber reinforced specimens. Overall, fibers did not seem to affect the hand held gage strength predictions relative to *UC* specimens.

Soil 3 fiber reinforced specimen strengths (Figure 15.20) measured with the *Shear* gage were all in the 0.75 to 1.25 bin, indicating the gage was able to describe the behavior the same as *UC* testing for practical purposes. The behavior of non-fiber reinforced *Soil 3* specimens measured with the *Shear* gage were dispersed more so than fiber reinforced specimens, though over 60% of the readings were in the 0.75 to 1.25 bin. Fiber reinforced

Soil 3 strengths were over predicted more with the *Dial* and *Ring* gages than non-fiber reinforced material, as its behavior was on either side of the 0.75 to 1.25 bin. Overall, observed differences between fiber and non-fiber reinforced specimens was not dramatic considering the amount of fiber reinforced data available. The data doesn't provide evidence that fibers prevent strength measurement with hand held gages, especially the *Shear* gage.

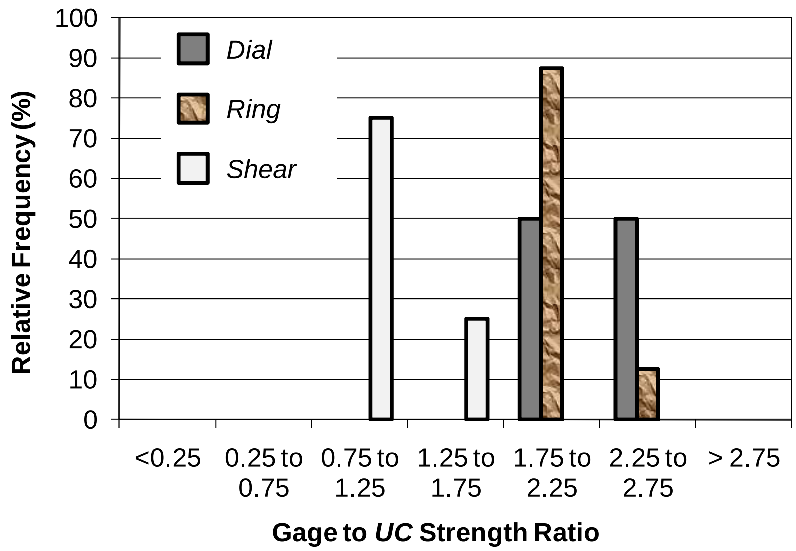


Figure 15.19. Gage Comparisons for Soil 2 and Fibers (5, 100) Using Trial Data

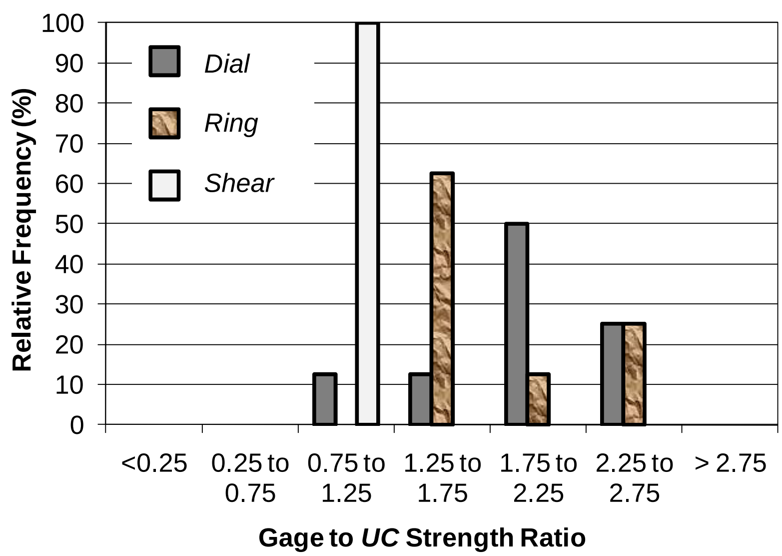
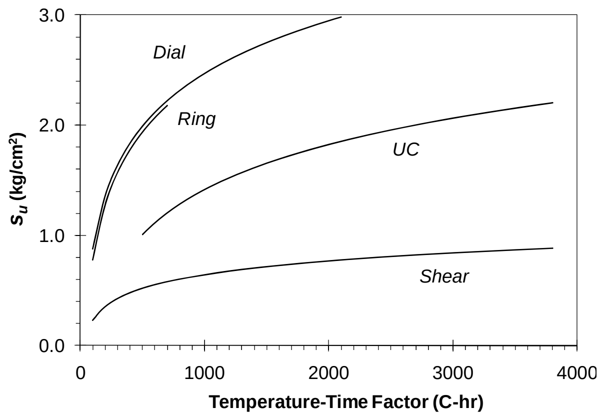


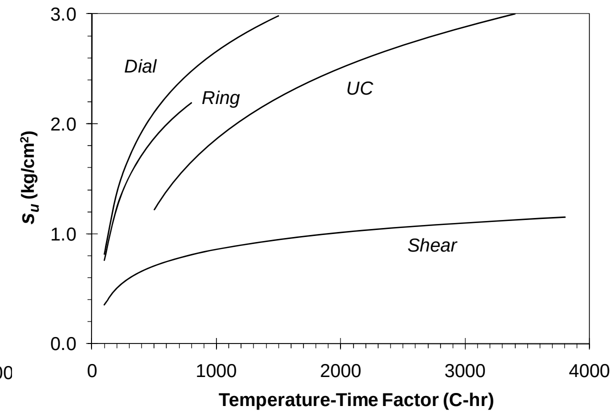
Figure 15.20. Gage Comparisons for Soil 3 and Fibers (5, 100) Using Trial Data

15.5 Hand Held Gage and *UC* Measured Strength (10, 100) Comparison

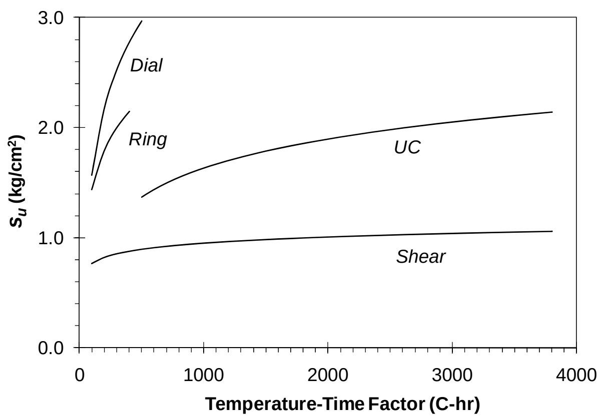
The *w/cm* ratio was 5 for all testing discussed in this section. The accuracy (proximity to true value) of the hand held gages was evaluated with gage to *UC* strength ratios as in Section 15.3, but no *Protocol 1* data was collected at (10, 100) so all *UC* measurements taken for the soil, blend, and cement type of interest were used to develop a trendline equation that served as the control for the experiment. Six trials (2 per soil type excluding 1 repeatability trial) and 5 variability slabs were used for the analysis (365 total readings). Figure 15.21 plots trendline results; *Dial* and *Ring* readings were highest in most instances.



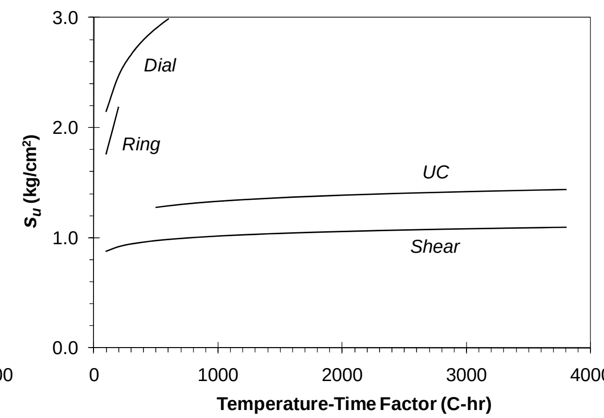
a) Th T III-Soil 1



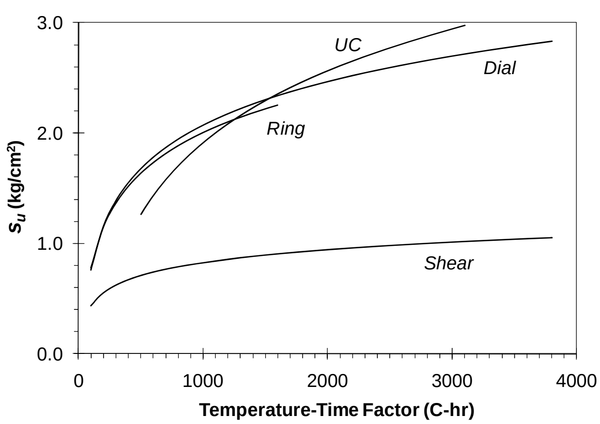
b) SC1-Soil 1



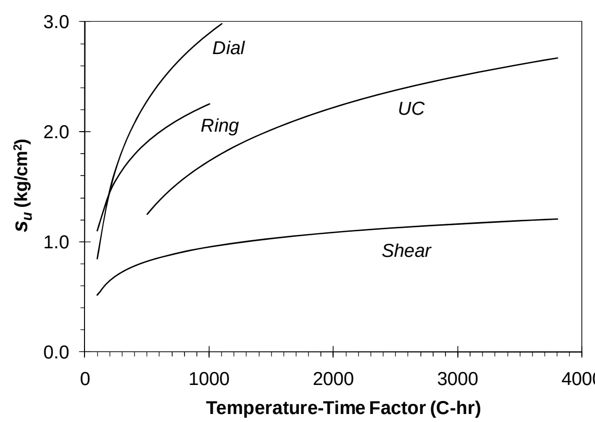
c) Th T III-Soil 2



d) SC1-Soil 2



e) Th T III-Soil 3



f) SC1-Soil 3

Figure 15.21. Shear Strength Test Results, (10,100), All Soils

With a modest amount of data available relative to (5, 100) testing, the comparisons using trial trendlines and those using variability slab means were combined into 1 data set. Also, since the penetrometers peaked relatively early in the curing period in many instances, the medium of comparison was adjusted relative to the (5, 100) testing. Trendlines were plotted up to the juncture where maximum readings were obtained, and it should be noted that readings after the gage peaked were not used to develop the trendline equations. The *UC* data was plotted beginning at 500 C-hr since all data used was collected with *Protocol 2*. To be as consistent as possible, *TTF* factors of 500, 1500, and 3500 C-hr were considered alongside any *TTF* factor where a gage peaked.

Figures 15.22 to 15.24 plot frequency histograms for all (10, 100) data. The amount of readings with the *Ring* gage were limited at 2, 4, and 7 readings in *Soils 2*, *1*, and *3*, respectively. *Ring* readings are therefore, not terribly informative in *Soil 2*.

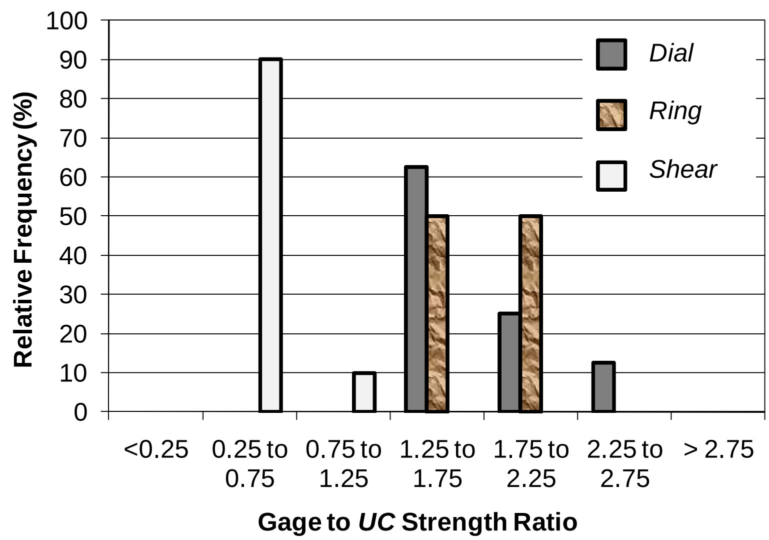


Figure 15.22. Gage Comparisons for Soil 1 (10, 100) Using All Data

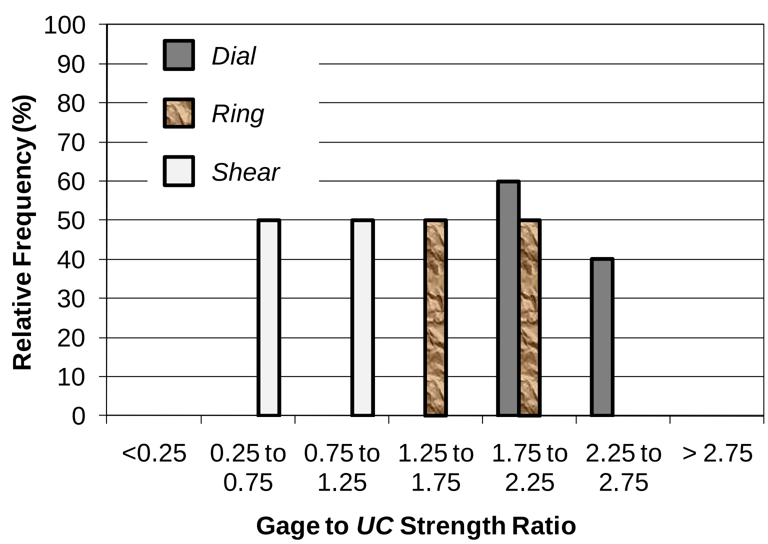


Figure 15.23. Gage Comparisons for Soil 2 (10, 100) Using All Data

Behavior of the *Shear* gage was similar in *Soil 1* between (10, 100) and (5, 100) testing (Figures 15.8 and 15.22). The percentage of values in the 0.25 to 0.75 bin was above 80%. *Dial* and *Ring* readings were noticeably higher at (10, 100) than at (5, 100).

Figure 15.23 (10, 100) and Figure 15.9 (5, 100) have approximately the same amount of *Shear* gage readings in the 0.75 to 1.25 bins for *Soil 2*. However, at (10, 100) all remaining readings (50%) were in the 0.75 to 1.25 bin, which was not the case at (5, 100) as only 26% of the readings were in the 0.75 to 1.25 bin. Penetrometer behavior was similar between (10, 100) and (5, 100) testing.

Figure 15.24 plots *Soil 3* data at (10, 100), which is much different for the *Shear* gage than (5, 100) data (Figure 15.10), as the (5, 100) had a distribution of values and the (10, 100) data had almost all values in the 0.25 to 0.75 bin. A high *Shear* gage percentage in the 0.25 to 0.75 bin was common at (10, 100) for all soils. Penetrometer data was somewhat similar between (10, 100) and (5, 100) as most of the data was in the same 3 bins.

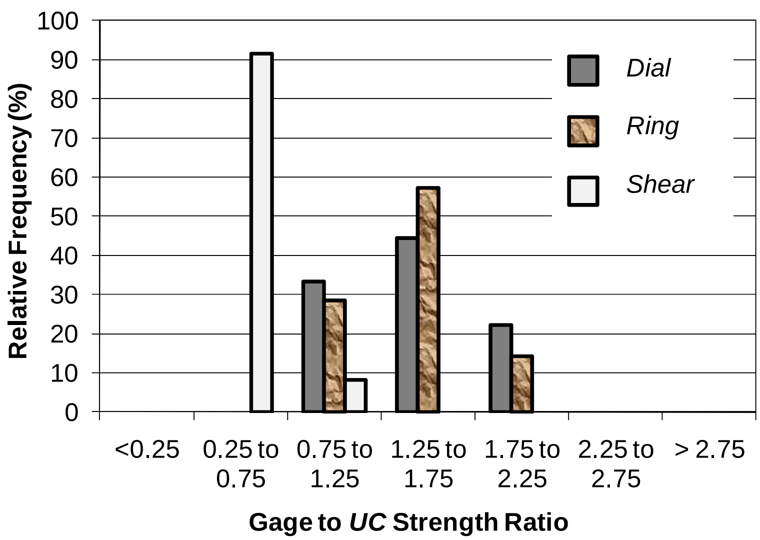
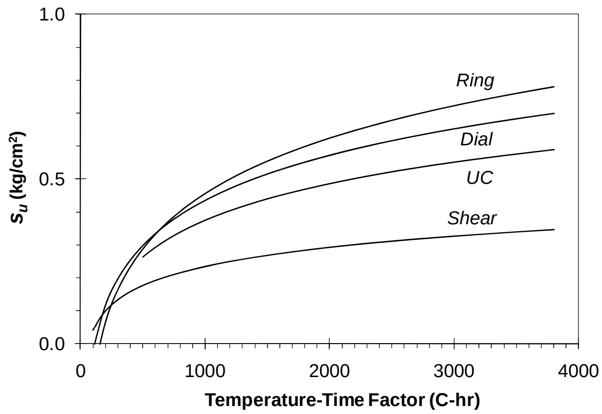


Figure 15.24. Gage Comparisons for Soil 3 (10, 100) Using All Data

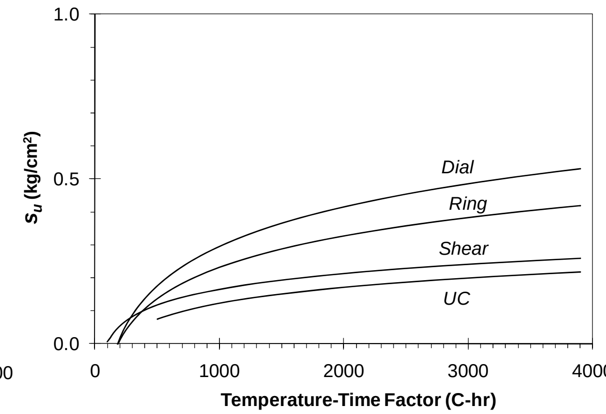
15.6 Hand Held Gage and *UC* Measured Strength (15, 233) Comparison

The *w/cm* ratio was 4.7 in this section. The hand held gage accuracy (proximity to true value) was evaluated with gage to *UC* strength ratios as in Section 15.3, but no *Protocol 1* data was collected at (15, 233); so, all *UC* measurements taken for the soil, blend, and cement type of interest were used to develop a trendline equation that served as the control for the experiment. Six trials (2 per soil type excluding 1 repeatability trial) and 5 variability slabs were used for the analysis (365 total readings). Figure 15.25 plots trendline test results; *Dial* and *Ring* readings were the highest in all plots.

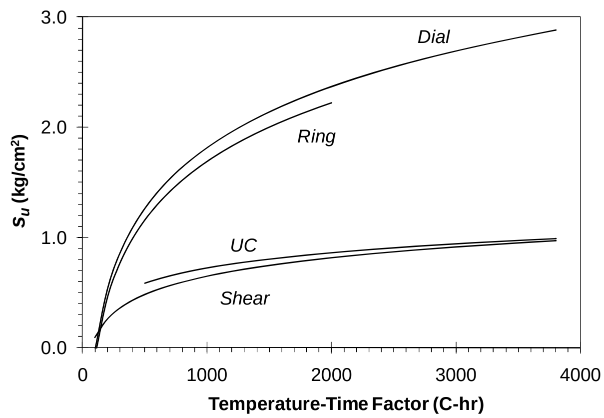
With a modest amount of data available relative to (5, 100) testing, the comparisons using trial trendlines and those using variability slab means were combined into 1 data set. To be consistent relative to (10, 100) analysis, trendlines were plotted up to the juncture where maximum readings were obtained, and it should be noted that readings after the gage peaked were not used to develop the trendline equations (at (15, 233) this only happened in 1 instance). The *UC* data was plotted beginning at 500 C-hr since all data used was collected with *Protocol 2*. To be as consistent as possible, *TTF* factors of 500, 1500, and 3500 C-hr were considered alongside any *TTF* factor where a gage peaked. Figures 15.26 to 15.28 plot frequency histograms for all (15, 233) data.



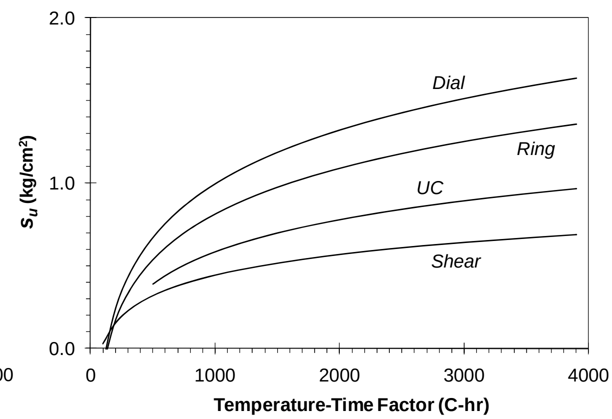
a) Soil 1-Th T III



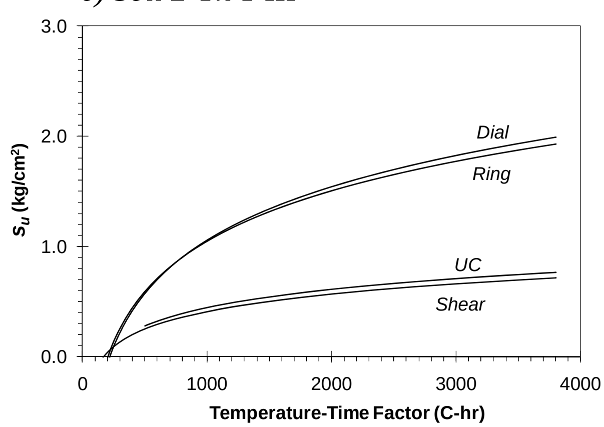
b) Soil 1-SC1



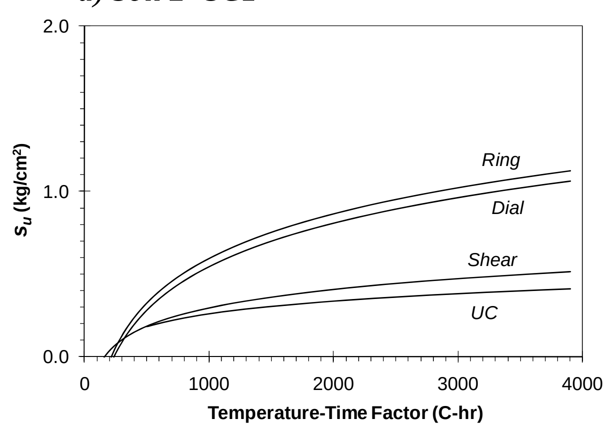
c) Soil 2-Th T III



d) Soil 2- SC1



e*) Soil 3-Th T III*



f) Soil 3-SC1

Figure 15.25. Shear Strength Test Results, (15, 233), All Soils

The *Shear* gage (Figure 15.26) over predicted strength in *Soil 1* more at (15, 233) with approximately 30% of the readings in the 1.25 to 1.75 bin, whereas no data was in this bin at (5, 100) as seen in Figure 15.8. The *Dial* and *Ring* readings similarly over predicted strength more at (15, 233) with no readings in the 0.25 to 0.75 bin, over 40% *Ring* readings in the 1.75 to 2.25 bin, and over 40% *Dial* readings in the 2.25 to 2.75 bin. The (15, 233) blend consistency may have affected results relative to the (5, 100) blend. Average gage to *UC* strength ratios were 2.24, 1.93, and 1.27 for *Dial*, *Ring*, and *Shear* gages, respectively.

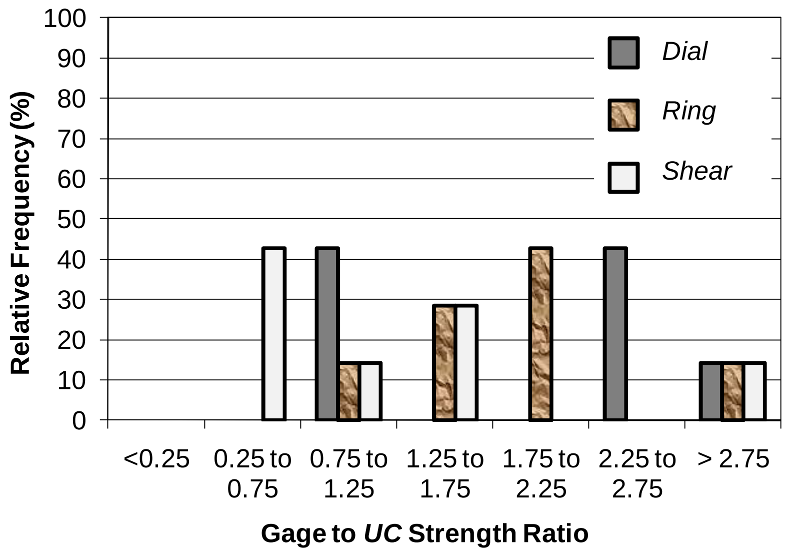


Figure 15.26. Gage Comparisons for Soil 1 (15, 233) Using All Data

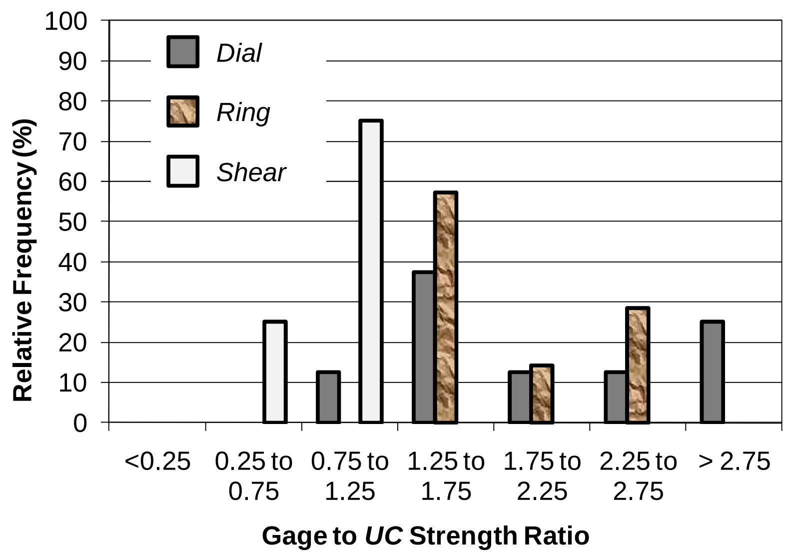


Figure 15.27. Gage Comparisons for Soil 2 (15, 233) Using All Data

The *Shear* gage (Figure 15.27) predicted strength for *Soil 2* in the 0.75 to 1.25 bin over 70% of the time at (15, 233) which is comparable in a general sense to (5, 100) data (Figure 15.9) where strength was predicted in the 0.75 to 1.25 bin over 50% of the time. The difference in behavior between (15, 233) and (5, 100) was that all remaining readings at (15,

233) were in the 0.25 to 0.75 bin, while all remaining readings at (5, 100) were in bins of 1.25 and higher. Penetrometer readings between (15, 233) and (5, 100) were comparable in the sense that they over predicted shear strength and were distributed throughout several bins. Average gage to *UC* ratios were 2.10, 1.80, and 0.85 for *Dial*, *Ring*, and *Shear* gages, respectively.

The *Shear* gage (Figure 15.28) predicted strength for *Soil 3* at (15, 233) in the 0.75 to 1.25 bin almost 90% of the time, which is comparable in a general sense to (5, 100) data (Figure 15.10) where strength was predicted in the 0.75 to 1.25 bin over 60% of the time. *Shear* gage prediction in the 0.25 to 0.75 bin was also comparable as (15, 233) and (5, 100) had values just over 10%. At (15, 233) the penetrometers over predicted strength more than at (5, 100), as the 2.25 to 2.75 bin had the highest amount of readings at just over 40% for each gage. Average gage to *UC* ratios were 1.93, 1.95, and 1.00 for *Dial*, *Ring*, and *Shear* gages, respectively.

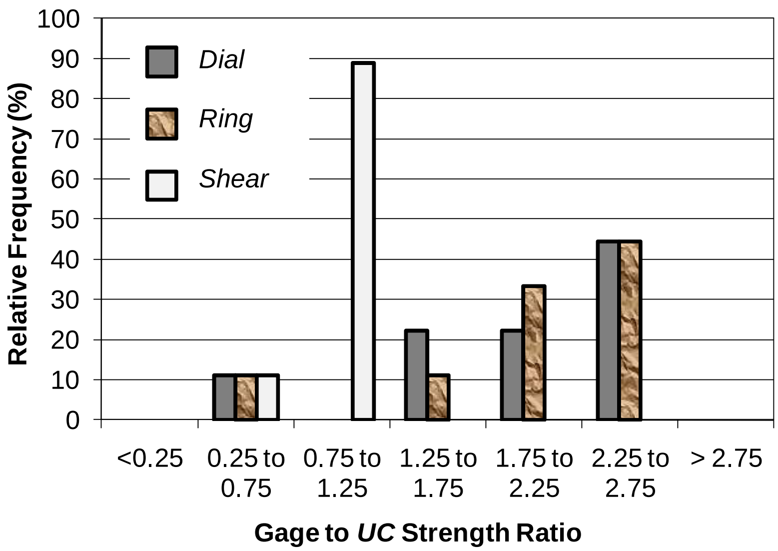


Figure 15.28. Gage Comparisons for Soil 3 (15, 233) Using All Data

15.7 Summary of Hand Held Gage and *UC* Strength Comparison

The overall observation of the chapter is that on site calibration with *UC* testing should be performed for any of the hand held gages if their readings are going to be used in a manner where considerable accuracy is needed. All of the gages measured shear strength that was noticeably different from *UC* testing in some instances, while accurate readings were recorded by some gages in some instances. Organic content appeared to be correlated to gage accuracy, at least for (5, 100) specimens. The relative consistency of the cementitious stabilized blends may have affected shear strength prediction, though the data collected did not allow detailed quantification of the extent to whichblend consistency affected hand held gage readings.

Table 15.1 provides average gage to *UC* strength ratios for all conditions considered. Average values are not necessarily fully representative of behavior, but they do provide some information. Brief summaries of test results have also been provided.

Table 15.1. Summary of Average Gage to *UC* to Strength Ratios

Condition	Data	Soil	Dial to *UC*	Ring to *UC*	Shear to *UC*
(5, 100)	Trendlines	1	0.74	0.78	0.53
		2	2.20	2.12	1.30
		3	1.45	1.28	0.98
(5, 100)	Variability	1	1.77	1.63	1.12
	Slab	2	2.70	2.33	1.35
	Means	3	1.60	1.45	1.04
(5, 100)	Fibers	1	1.57	1.84	0.70
		2	2.33	2.15	1.10
		3	1.94	1.81	1.13
(10, 100)	All	1	1.75	1.64	0.52
		2	2.19	1.74	0.68
		3	1.46	1.34	0.56
(15, 233)	All	1	2.24	1.93	1.27
		2	2.10	1.80	0.85
		3	1.93	1.95	1.00

- *Soil 1* (5, 100): Strength over prediction was not common with the *Shear* gage or either penetrometer. Most cases under predicted strength, especially the *Shear* gage.
- *Soil 2* (5, 100): Strength over prediction occurred with the penetrometers, while the *Shear* gage predicted at to slightly over *UC* measured values.
- *Soil 3* (5, 100): Strength was under predicted and over predicted by all gages. The *Shear* gage predicted reasonable strength in the majority of cases.
- *Soil 1* (5, 100) Fibers: Fibers led to some amount of strength over prediction in the penetrometers and may have increased *Shear* gage readings relative to *UC* readings.
- *Soil 2* (5, 100) Fibers: Fibers did not seem to affect behavior.
- *Soil 3* (5, 100) Fibers: Behavioral differences between fiber and non-fiber specimens were not dramatic considering the data available.
- *Soil 1* (10, 100): *Shear* gage behavior was similar (10, 100) and (5, 100). Penetrometers were noticeably higher at (10, 100) than (5, 100).
- *Soil 2* (10, 100): Similar behavior was observed between (10, 100) and (5, 100) with all 3 hand held gages.
- *Soil 3* (10, 100): At (5, 100) *Shear* gage had a distribution of values, yet at (10, 100) most data was in 1 bin. Penetrometers behavior was similar at (10, 100) and (5, 100).
- *Soil 1* (15, 233): All gages over predicted strength more at (15, 233) than at (5, 100). Consistency differences may have affected results.
- *Soil 2* (15, 233): All gages were comparable at (15, 233) and (5, 100).
- *Soil 3* (15, 233): Strength was comparable at (15, 233) and at (5, 100) with the *Shear* gage. Penetrometers over predicted strength at (15, 233) more so than at (5, 100).

CHAPTER 16 – SUMMARY CONCLUSIONS AND RECOMMENDATIONS

16.1 Summary

The research discussed in this report was undertaken to develop an emergency construction material for use in a disaster due to flooding. Approximately 3,300 unconfined compression tests were performed alongside approximately 5,500 readings with each of three hand held gages. In total, nearly 20,000 strength readings were taken. The majority of the testing was performed on three soils at three moisture contents using fourteen stabilization materials. This report revealed many suitable attributes that could be immediately useful in disaster recovery. Other techniques in this report are ready for a full scale demonstration, and provided it is successful, they should be used in a disaster environment.

A primary objective of this report was to develop strength, modulus, and ductility trends for a variety of soil types, cementitious materials, cementitious material contents, and moisture contents. Another primary objective of this report was to test specialty cements (either specialty grind portland cements or specialty blended calcium sulfoaluminate cements) and compare their characteristics to commercially and readily available products in the cements market with the intention of achieving better properties with the specialty cements.

Analysis protocols were established so that experimental data could be compared to control data while still preventing the variability of testing high moisture content soils from leading to conclusions that could be incorrect. Portland cements were compared within the same production facility in the majority of cases. Portland cement strengths were used as a reference for other cementitious materials. Considering all of the cases tested, sufficient replication for direct statistical comparisons was beyond the scope of this effort.

Properties achieved with high moisture content fine grained cementitiously stabilized soil slurries were compared to properties found during literature review. Use of fine grained soils with lower moisture than those considered for a disaster environment appears more common in traditional environments. Table 16.1 summarizes properties from soil mixing projects documented in Chapter 2. In general, shear strength for cementitiously stabilized soils in literature were 1.5 to 10 kg/cm^2 and maximum strains were 2% or less. Dosages of 60 to 80 kg/m^3 appear to be the lower bound used in the majority of soil mixing operations.

Table 16.1. Summary of Soil Mixing Properties

Table of Origin	Method	Mixes	*D* (kg/m^3)	72 hr s_u (kg/cm^2)	168 hr s_u (kg/cm^2)
2.3	*DSM*	16	125 to 400	0.5 to 5.4	0.5 to 7.8
2.4	*WSM*	16	130 to 225	0.5 to 5.4	1.0 to 9.5

A variety of soils were tested with DSM and WSM.

Compacted and unstabilized clay can have shear strength achieving 2 kg/cm^2, which is an excellent bearing material. Typical *CLSM* is 1.75 to 3.5 kg/cm^2 at 28 days. Low ground pressure equipment can typically be supported with a 0.2 kg/cm^2 shear strength.

Projects identified in literature review demonstrated the use of SGM where ≈ 1 kg/cm^2 design strengths (s_{ud}) are used.

The remainder of this section compares attributes of different cementitious materials that within the body of the report were separated by chapter. Figures 16.2 through 16.4 summarize strength information of all unconfined compression (*UC*) data collected and is intended to serve as a guide to the blend of materials suitable for a given application. The data in Figures 16.2 to 16.4 should not be used in absence of the information presented in the individual chapters. For example, fiber reinforced specimens were not tested with all cements so a direct comparison between the highest strengths achieved with portland cements shown is not warranted. Overall, very high moisture content blends were capable of producing strengths comparable to conventional stabilized fine grained materials with lower moisture.

Table 16.2. Overall Comparison of Strength Properties at (5, 100)

Soil	**s_u (kg/cm^2)**	**24 hr**				**72 hr**				**168 hr**			
1	0.2	☑	✓	×	Ø	☑	✓	×	Ø	☑	✓	×	Ø
	0.5	☑	✓	Ø		☑	✓	×		☑	✓	×	
	1.0	Ø	Ø			☑	Ø	Ø		☑	Ø	×	
	1.5					Ø				Ø		Ø	
2	0.2	☑	✓	---	Ø	☑	✓	---	¥	☑	✓	---	¥
	0.5	☑	✓			☑	✓		Ø	☑	✓		Ø
	1.0	Ø	Ø			Ø	Ø			Ø	Ø		
	1.5												
3	0.2	☑	✓	Ø	Ø	☑	✓	Ø	Ø	☑	✓	Ø	Ø
	0.5	☑	✓			☑	✓			☑	✓		
	1.0	Ø	Ø			Ø	Ø			Ø	Ø		
	1.5												

The data in the table indicates whether a given stabilization treatment was able to produce the shear strength shown at the appropriate curing level.

☑ *Portland Cement* ✓ *Portland Cement and Fibers*

× *GGBFS(50%) and Portland Cement (50%)* ¥ *Calcium Sulfoaluminate Cements*

--- *No testing conducted* Ø *Strength was not achieved*

Table 16.2 shows that no blend was able to achieve 1.5 kg/cm^2 with 5% cement dosage. Portland cement is most suitable for this combination of parameters. Use of fibers did not increase strength sufficiently to achieve a higher strength category than portland cement. *GGBFS* and portland cement were able to achieve the same strength category as portland cement in one instance, though this was with 50% *GGBFS* and 50% portland cement as opposed to 75% *GGBFS* and 25% portland cement that was most effective at other combinations of parameters. *GGBFS* is not recommended for this combination of parameters in a disaster environment. Calcium sulfoaluminate cements did not provide useful properties at this combination of moisture and cement dosage.

Table 16.3 shows that significant strength can be achieved with this combination of cement dosage and moisture content. After 24 hr curing at room temperature (typical *TTF* of 500 to 600 C-hr), portland cement was the best stabilizing agent and was able to achieve 1.5 kg/cm^2 with all soils. Portland cement and fibers were able to achieve 2 kg/cm^2 with *Soil 2*, but otherwise were in the 1.5 kg/cm^2 category. *GGBFS* and *CS* cements were of no value at (10, 100) and 24 hr curing. Relative behaviors were similar, in general, after 72 hour curing at room temperature (typical *TTF* of 1,600 to 1,800 C-hr) as portland cement was the recommended product. Strength of 2 kg/cm^2 could be achieved at 72 hours by all soils. *GGBFS* was able to provide comparable properties at 72 hours in *Soil 1* to portland cement. *CS* cements were of no practical value at this combination of parameters for any soil.

Table 16.3. Overall Comparison of Strength Properties at (10, 100)

Soil	s_u (kg/cm^2)	24 hours				72 hours				168 hours			
1	0.2	☑	✓	×	¥	☑	✓	×	¥	☑	✓	×	¥
	0.5	☑	✓	Ø	¥	☑	✓	×	¥	☑	✓	×	¥
	1.0	☑	✓		Ø	☑	✓	×	Ø	☑	✓	×	Ø
	1.5	☑	✓			☑	✓	×		☑	✓	×	
	2.0	Ø	Ø			☑	✓	×		☑	✓	×	
	3.0					Ø	Ø	Ø		☑	Ø	×	
	4.0									☑		×	
	5.0									Ø		×	
	6.0											×	
	7.0											Ø	
2	0.2	☑	✓	---	¥	☑	✓	---	¥	☑	✓	---	¥
	0.5	☑	✓		Ø	☑	✓		Ø	☑	✓		Ø
	1.0	☑	✓			☑	✓			☑	✓		
	1.5	☑	✓			☑	✓			☑	✓		
	2.0	Ø	✓			☑	✓			☑	✓		
	3.0		Ø			Ø	Ø			Ø	✓		
	4.0										Ø		
	5.0												
	6.0												
	7.0												
3	0.2	☑	✓	Ø	¥	☑	✓	Ø	¥	☑	✓	Ø	¥
	0.5	☑	✓		¥	☑	✓		¥	☑	✓		¥
	1.0	☑	✓		Ø	☑	✓		Ø	☑	✓		Ø
	1.5	☑	✓			☑	✓			☑	✓		
	2.0	Ø	Ø			☑	✓			☑	✓		
	3.0					Ø	Ø			☑	✓		
	4.0									Ø	Ø		
	5.0												
	6.0												
	7.0												

The data in the table indicates whether a given stabilization treatment was able to produce the shear strength shown at the appropriate curing level.

☑ *Portland Cement* — ✓ *Portland Cement and Fibers*

× *GGBFS(75%) and Portland Cement (25%)* — ¥ *Calcium Sulfoaluminate Cements*

--- *No testing conducted* — Ø *Strength was not achieved*

After 168 hr room temperature curing (typical *TTF* of 3,800 to 4,200 C-hr), there were noticeable differences between soil types and relative to earlier cure times. Portland cement is easily the recommended product for *Soil 2* and *Soil 3*, as *GGBFS* and *CS* cements were ineffective. *CS* cements were also ineffective in *Soil 1*. *GGBFS*, however, was able to achieve 6 kg/cm^2 with *Soil 1*, while portland cement was only able to achieve 4 kg/cm^2. For longer cure times and select soils, *GGBFS* appears to be the best product. For applications where time is essential, though, there may not be sufficient time to evaluate *GGBFS* to ensure it will provide adequate strength, and portland cement provided good results with *Soil 1* as well. At 168 hours, strength was inversely proportional to organics as 4, 3, and 2 kg/cm^2 were achieved with *Soil 1* (5.7% organics), *Soil 3* (10.4% organics), and *Soil 2* (25.6% organics), respectively.

Table 16.4. Overall Comparison of Strength Properties at (15, 233)

Soil	s_u (kg/cm^2)	24 hours				72 hours				168 hours			
1	0.2	☑	Ø	---	¥	☑	✓	---	¥	☑	✓	---	¥
	0.5	Ø			¥	☑	Ø		¥	☑	Ø		¥
	1.0				¥	Ø			¥	Ø			¥
	1.5				¥				¥				¥
	2.0				¥				¥				¥
	3.0				¥				¥				¥
	4.0				Ø				¥				¥
	5.0								Ø				¥
	6.0												Ø
2	0.2	☑	✓	---	¥	☑	✓	---	¥	☑	✓	---	¥
	0.5	☑	✓		¥	☑	✓		¥	☑	✓		¥
	1.0	Ø	Ø		Ø	☑	✓		Ø	☑	✓		¥
	1.5					Ø	Ø			Ø	✓		Ø
	2.0										Ø		
	3.0												
	4.0												
	5.0												
	6.0												
3	0.2	☑	✓	---	¥	☑	✓	---	¥	☑	✓	---	¥
	0.5	Ø	Ø		¥	☑	✓		¥	☑	✓		¥
	1.0				¥	Ø	Ø		¥	☑	Ø		¥
	1.5				Ø				¥	Ø			¥
	2.0								Ø				¥
	3.0												Ø
	4.0												
	5.0												
	6.0												

The data in the table indicates whether a given stabilization treatment was able to produce the shear strength shown at the appropriate curing level.

☑ *Portland Cement* ✓ *Portland Cement and Fibers*

× *GGBFS(75%) and Portland Cement (25%)* ¥ *Calcium Sulfoaluminate Cements*

--- *No testing conducted* Ø *Strength was not achieved*

Table 16.4 clearly shows *CS* cements are the product of choice for *Soil 1* at all test times. Shear strength was 3, 4, and 5 kg/cm^2 after 24, 72, and 168 hours curing, respectively. All other products were either not tested or produced significantly weaker blends. In *Soil 2*, *CS* and portland cements were comparable and neither produced considerable strength considering the relatively high dosage rate of 15%. Materials similar to *Soil 2* at (15, 233) should be avoided in disaster applications as the benefit to cost ratio is not favorable. For *Soil 3*, *CS* cements were the superior product at all test times. Shear strength was 1, 1.5, and 2 kg/cm^2 after 24, 72, and 168 hr curing, respectively. Organic content had an inverse effect on *CS* cements as it did on portland cements (higher the organic content the lower the strength). Organics, however, did not inhibit performance, they merely affected performance. Review of literature indicated that organic content appeared to reduce shear strength in an overall sense, but useable strength could be produced with soils containing organics.

16.2 Conclusions

Specific conclusions are presented in this section in bullet form. Conclusions that can be readily drawn from the summary and tables in the previous section are not repeated in this section for brevity. The overall conclusion of the research is the high moisture content cementitiously stabilized slurries are a viable emergency construction material for use on a short term basis.

- Specialty grind portland cements were successfully produced at wet and dry process full-scale cement plants, demonstrating the feasibility of the on demand cement concept presented in Chapter 4. SO_3 content and *Blaine* Fineness were the properties adjusted. Test results indicated that a modified SO_3 content can change shear strength of stabilized soil slurries in many instances, but that there is not a specific SO_3 content that works for all applications.
- Increased *Blaine* Fineness was not shown advantageous except for *Soil 2*.
- Specialty grind portland cements outperformed *Type III* control cements in five of the nine cases investigated, as shown in Table 9.5. *Soil 3* did not benefit from the specialty cement property changes as the properties were the same as the controls.
- SO_3 contents tested were not low enough to cause incompatibility and very low early strengths that were mentioned in the literature review (discussed in terms of concrete applications); the exception was the case of cement *SC2*, produced with a very low SO_3 content and high fineness where strengths were lower in some soils. There does not appear to be a high risk in reducing the SO_3 content by moderate amounts for cementitiously stabilized high moisture content soils.
- *Type III* control cements were only slightly different when compared to each other in five of nine cases. Control cements were noticeably different in four cases with *Th T III* higher in three of these four cases.
- *GGBFS* appears to have a limited window of application in disaster recovery where high later age (e.g. 168 hours) strengths are desired.
- Literature review showed that shorter fibers were more effective and that cement may allow fibers to bond better to clay. Shorter (20 mm) fibers performed slightly better

than longer (70 mm) fibers at (5, 100) in this report; shorter fibers were used for all additional testing.

- Literature review indicated that a small to no strength gain occurred due to fibers in some instances and that ductility was improved considerably in most instances. Testing 20 mm fibers with three soils and three moisture contents in this report showed various levels of strength improvement due to fiber addition with the magnitude of improvement increasing with organic content. Ductility was markedly improved as a result of fiber addition.
- Hand held gages can be used as a quality control tool during construction, though they have some limitations. Shear strength should be calibrated using on site unconfined compression measurements with the same soil and moisture content.
- *Dial* gage R^2 values indicated it was the most precise, and variability slab testing indicated it was the least variable of the gages at (5, 100). At (10, 100) and (15, 233) the *Dial* gage was, in general, on par with the *Shear* gage in terms of variability.
- The *Dial* gage appears suitable for disaster recovery use for blends with lower shear strength as it peaked for stronger blends. The *Shear* gage did not peak for the blends tested.
- The higher the organic content, in general, the higher the measured strength with the hand held gages. Blend consistency may have affected shear strength prediction, though more detailed statements cannot be made.
- *CS* cements were able to encapsulate more water than portland cements, and preparation time affected the strength of *CS* cements with faster preparation time, in general, improving strength.
- A distinct set of conditions existed where *CS* cements were the superior performer (moisture contents in excess of 133% and cement contents of 15 to 20% in conjunction with a relatively low organic soil with a moderate liquid limit). *CS* cements provide no advantage below a 10% dosage rate. *SC3* performed best with *Soil 1*, while *CTS RS* and *SC4* performed the best with *Soil 3*.
- Fairly reliable correlations were developed between shear strength and elastic modulus that can be used for design purposes. The elastic modulus in units of MPa was 6.9 to 18.9 times the soil shear strength in units of kg/cm^2 for portland cement and did not appear sensitive to cement source. The highest modulus to strength behavior for portland cement occurred at (5, 100), while, in general, (15, 233) was on the lower end of the modulus behaviors. *GGBFS* correlations were 9.6 to 14.2, brackish and salt water correlations were 10.5 to 11.5, and fiber correlations were 6.4 to 14.1, all of which are, in general, within the portland cement range. *CS* cements had modulus to strength relationships between 9.6 to 12.2 for *Soil 1*, which were within the range of values tested for portland cement. *Soil 2* and *Soil 3* had modulus to strength relationships approximately twice as high as with portland cements at 19.4 and 17.3, respectively, when *CS* cement was used for the same combination of conditions.
- Maximum strains were less than 3% for nearly all testing in the absence of fibers. In some instances strains were 4% or higher for low strength specimens often tested at early cure times. Maximum strains of 1.5 to 2.5% were common.

- Confinement increased shear strength by ≈ 4 in one reference (which if not considered in design provides conservative results.) Temperature increase as stabilized slurry cures in a large mass should accelerate strength gain relative to what would be predicted from air temperature measurement and provide an additional, yet un-quantified, factor of safety.
- Water type appeared to have some effect on strength gain, and the effect was not consistent with soil type. Strength gain was observed in some cases while strength loss was observed in other cases. Overall it does not appear water type has a detrimental effect on strength gain, though the data collected indicates salt water is more problematic than brackish water.
- Heat generation could be detected during *Slab* testing. The finding led to more sophisticated testing to measure thermal profiles, which may be useful in the monitoring of property development and related quality control. Measurement of thermal profiles on site could be used to detect incompatibility or evaluate mixing effectiveness in large soil masses.
- Once the stabilized soils are used for disaster recovery, they should be useful for other applications. Examples were provided in the literature review (Section 2.11), which shows that re-use of these types of materials is a potentially viable option.

16.3 Recommendations

Specific recommendations are presented in this section in bullet form. Most recommendations are related to implementing the findings of this report, while one is related to additional research that would be useful. The overall recommendation of the research is to use high moisture content cementitiously stabilized slurries as an emergency construction material for use on a short term basis. Literature does not document noteworthy use of these materials for disaster recovery.

- Produce a series of specialty grind portland cements with progressively different SO_3 contents at a given *Blaine* Fineness. Produce a second series of specialty grind portland cements with a constant SO_3 content and progressively different *Blaine* Finenesses. Ideally these two series of materials would be performed at two or three different cement plants. Test three to five soils, classifying as CL to OH after 24 hours cure at (10, 100). Test replication should be sufficient to make statistical comparisons of all products. It is anticipated that 1,000 to 3,000 tests would be required for the experiment, but the results would advance the findings of this report tremendously.
- Do not use *CS* cements with highly organic soils.
- Test three *CS* cements with different blends on site after 24 hours curing to select the most compatible product for the material at the site.
- Perform on site calibration of hand held gages with *UC* testing to greatly increase accuracy. Use hand held gages to evaluate properties of the stabilized soil masses during construction.
- If *GGBFS* is to be used, it is recommended to use 75% *GGBFS* and 25% portland cement.

- Use thermal profile testing or *SAC* on site as a mixing efficiency quality control tool.
- The *F20* fiber should be used along with cementitious materials whenever economically viable to add considerable ductility and a probable slight strength gain.
- Design shear strength (s_{ud}) should not exceed 2.5 kg/cm^2 with any of the materials investigated in this report. It is recommended to use the design strength as 1/4th of the laboratory measured strength. The recommendation is based on Eq. 2.2 with $\gamma = \lambda = 0.5$. Using γ of 0.5 may be conservative when large areas of the emergency construction material are used, but in absence of additional data the value should remain at 0.5. Use the design shear strength and the plots provided in this report to select the design elastic modulus.
- Test onsite soil in conjunction with the cement plant of choice and adjust the SO_3 content to achieve the best strength performance using 24 hour testing. If SO_3 adjustment does not provide desired properties, adjust Blaine Fineness. Adjust SO_3 during the early stages of the project to arrive at the most suitable value. SO_3 changes within the bounds studied did not cause significant strength loss in any case (with possible exception of *SC2* in some instances), so progressively changing the value should work well on site. An initial recommendation from this research is to reduce the SO_3 content 25% relative to what the cement plant currently produces for routine concrete applications.

CHAPTER 17 - REFERENCES

ACI-229R (2005). *ACI Manual of Concrete Practice-Controlled Low-Strength Materials.* ACI Committee 229, American Concrete Institute, Farmington Hills, MI.

Anday, M. C. (1963). “Curing Lime-Stabilized Soils,” *Highway Research Record*, 29, 13-26.

Ang, E.C., and Loehr, J.E. (2003). “Specimen Size Effects For Fiber-Reinforced Silty Clay in Unconfined Compression,” *Geotechnical Testing Journal*, 26(2), 191-200.

Austin, D.N., Shrader, S.L., and Chill, D.S. (1993). “Soil Stabilization With Discrete Fibrillated Polypropylene Fibers,” *Geotechnical Fabrics Report,* Oct Issue, 4-10.

Bennert, T.A., Maher, M.H., Jafari, F., and Gucunski, N. (2000). “Use of Dredged Sediments from Newark Harbor for Geotechnical Applications,” *Geotechnics of High Water Content Materials*, ASTM STP 1374, 152-164.

Burke, G.K., and Shen, A.L. (2005). “An Analysis of Single Axis Wet Mix Performance,” *Proceedings of the International Conference on Deep Mixing Best Practices and Recent Advances, Deep Mixing ’05*, May 23 to 25, Stockholm, Sweden.

Carruth, W.D. (2011). *Laboratory Evaluation of Specialty Portland Cements and Polymer Fibers in Stabilization of Fine Grained Soils.* MS Thesis, Mississippi State University.

Carruth, W.D., and Howard, I.L. (2011). “Evaluation of Unconfined Compression Area Correction Methods for Cementitious and Fiber Stabilized Fine Grained Soils,” *Proceedings of Geo-Frontiers 2011*, Advances in Geotechnical Engineering-Geotechnical Special Publication No. 211, (CD-Rom), ASCE, Mar 13-16, Dallas, TX, 2564-2573.

Cerato, A.B., and Lutenegger A.J. (2005). “Activity, Relative Activity and Specific Surface Area of Fine-Grained Soils,” *Proceedings of the 16th International Conference on Soil Mechanics and Geotechnical Engineering (ICSMGE)*, Vol 2, Osaka, Japan, Sept 12-16, 325-328.

Chang, D.T., Liao, W., and Chen, S. (1996). “Effectiveness of Various Sludge Stabilization and Solidification Methods,” *Transportation Research Record: Journal of the Transportation Research Board*, 1546, 41-52.

Chew, S.H., Kamruzzaman, A.H.M., and Lee, F.H. (2004). “Physicochemical and Engineering Behavior of Cement Treated Clays,” *Journal of Geotechnical and Geoenvironmental Engineering*, 130(7), 696-706.

Circeo, J.J., Davidson, D.T., and David, H.T. (1962). “Strength-Maturity Relations of Soil-Cement Mixtures,” *Highway Research Board Bulletin*, 353, 84-97.

Clare, K.E. and Farrar, D.M. (1956). "The Use of Cements of Different Fineness in Soil-Cement Mixtures," *Magazine of Concrete Research*, 8(24), 137-144.

Cost, V.T. (2006). "Incompatibility of Common Concrete Materials-Influential Factors, Effects, and Prevention," *Proceedings of the 2006 Concrete Bridge Conference*, May 7-10, Reno, NV, 1-24.

Edil, T.B., and Wang, X. (2000). "Shear Strength and K_0 of Peats and Organic Soils," *Geotechnics of High Water Content Materials-ASTM STP 1374*, 209-225.

Emery, J.J. (1980). "Stabilizing Industrial Sludge for Fill Applications," *7th International Congress on the Chemistry of Cement*, Paris, France, 644-648.

FDOT (2000). *Soils and Foundation Handbook 2000.* Florida Department of Transportation, Gainesville, FL, 163.

Fletcher, C.S., and Humphries, W.K. (1991). "California Bearing Ratio Improvement of Remolded Soils by the Addition of Polypropylene Fiber Reinforcement," *Transportation Research Record: Journal of the Transportation Research Board*, No. 1295, 80-86.

Gassman, S.L., Pierce, C.E., and Schroeder, A.J. (2001). "Effects of Prolonged Mixing and Retempering on Properties of Controlled Low-Strength Material (CLSM)," *ACI Materials Journal*, 98(2), 194-199.

Gray, D.H., and Ohashi, H. (1983). "Mechanics of Fiber Reinforcement in Sand," *Journal of Geotechnical Engineering*, 109(3), 335-353.

Hernandez-Martinez, F.G. and Al-Tabbaa, A. (2009). "Effectiveness of Different Binders in the Stabilization of Organic Soils," *Proceedings of the International Symposium on Deep Mixing and Admixture Stabilization*, Okinawa, Japan, May 19-21.

Hirabayashi, H., Taguchi, H., Tokunaga, S., Shinkawa, N., Fujita, T., Inagawa, H., and Yasuoka, N. (2009). "Laboratory Mixing Tests on Cement Slurry Preparation, Specimen Preparation and Curing Temperature," *Proceedings of the Deep Mixing 2009 Okinawa Symposium and International Symposium on Deep Mixing & Admixture Stabilization,* May 19-21, Okinawa, Japan.

Horpibulsuk, S., Miura, N., and Nagaraj, T.S. (2005). "Clay-Water/Cement Ratio Identity for Cement Admixed Soft Clays," *Journal of Geotechnical and Geoenvironmental Engineering*, 131(2), 187-192.

Howard, I. (2006). *Full-Scale Field Study and Finite Element Modeling of a Flexible Pavement Containing Geosynthetics.* PhD Dissertation, University of Arkansas.

Kitazume, M., and Nishimura, S. (2009). "Influence of Specimen Preparation and Curing Conditions on Unconfined Compression Behavior of Cement-Treated Clay," *Proceedings of the Deep Mixing 2009 Okinawa Symposium and International Symposium on Deep Mixing & Admixture Stabilization,* May 19-21, Okinawa, Japan.

Kelly, D.S., and Diethelm, E.C. (1996). "Effects of Various Additives on the Solidification of Oily Sludges: A Bench-Scale Study," *Stabilization and Solidification of Hazardous, Radioactive, and Mixed Wastes: 3rd Volume, ASTM STP 1240,* 584-595.

Koerner, R.M. (1998). *Designing With Geosynthetics.* Prentice Hall, 4th Edition, New Jersey.

Kosmatka, S.H., Kerhoff, B., and Panarese, W.C. (2006). *Design and Control of Concrete Mixtures.* Portland Cement Association Engineering Bulletin 001, Stroke, IL, 358.

Lee, F., Lee, Y., Chew, S., and Yong, K. (2005). "Strength and Modulus of Marine Clay-Cement Mixes," *Journal of Geotechnical and Geoenvironmental Engineering,* 131(2), 178-186.

Li., S., Songyu, L., Yanjun, D., Fei, J., and Lei, F. (2008). "Experimental Study on the Stabilization of Organic Clay with Fly Ash and Cement Mixed Method," *ASCE GeoCongress 2008: The Challenge of Sustainability in the Geoenvironment,* New Orleans, Louisiana, March 9-12, 20-27.

Lin, S.L., Cross, W.H., Chain, E.S.K, Lai, J.S., Giabbai, M., Hung, C.H. (1996). "Stabilization and Solidification of Lead in Contaminated Soils," *Journal of Hazardous Materials,* 48, 95-110.

Lorenzo, G.A. and Bergado, D.T. (2006). "Fundamental Characteristics of Cement-Admixed Clay in Deep Mixing," *Journal of Materials in Civil Engineering*, 18(2), 161-174.

Maher, M.H. and Ho, Y.C. (1994). "Mechanical Properties of Kaolinite/Fiber Soil Composite," *Journal of Geotechnical Engineering*, 120(8), 1381-1393.

Maher, M.H. and Gray, D.H. (1990). "Static Response of Sands Reinforced with Randomly Distributed Fibers," *Journal of Geotechnical Engineering*, 116(11), 1661-1677.

MacKay and Emery (1994). "Stabilization and Solidification of Contaminated Soils and Sludges Using Cementitious Systems: Selected Case Histories," *Transportation Research Record: Journal of the Transportation Research Board*, 1458, 67-72.

Miller, G.A. and Azad, S. (2000). "Influence of Soil Type on Stabilization with Cement Kiln Dust," *Construction and Building Materials,* Vol. 14, 89-97.

Nakai, K.A., Shinsha, H., and Kakizaki, T. (2009). “Application of CDM and SGM for the Apron Construction in Tokyo International Airport Re-expansion Project,” *Proceedings of the Deep Mixing 2009 Okinawa Symposium and International Symposium on Deep Mixing & Admixture Stabilization*, May 19-21, Okinawa, Japan.

Newman, K., Rushing, J.F., and White, D.J. (2008). “Rapid Soil Stabilization for Contingency Airfield Construction,” *Proceedings of the DOD Transportation Systems Workshop*, Phoenix, AZ, April 21-24.

NRF (2008). *National Response Framework.* US Department of Homeland Security, Washington, DC, 82.

Oota, M., Mitarai, Y., and Iba, H. (2009). “Outline of Pneumatic Flow Mixing Method and Application for Artificial Island Reclamation Work,” *Proceedings of the Deep Mixing 2009 Okinawa Symposium and International Symposium on Deep Mixing & Admixture Stabilization*, May 19-21, Okinawa, Japan.

Prusinski, J.R., and Bhattacharja, S. (1999). “Effectiveness of Portland Cement and Lime in Stabilizing Clay Soils,” *Transportation Research Record: Journal of the Transportation Research Board*, 1652, 215-227.

Przewlocki, J. (2000). “Two-Dimensional Random Field of Mechanical Soil Properties,” *Journal of Geotechnical and Geoenvironmental Engineering*, 126(4), 373-377.

Puppala, A.J., and Musenda, C. (2000). “Effects of Fiber Reinforcement on Strength and Volume Change in Expansive Soils,” *Transportation Research Record: Journal of the Transportation Research Board*, No. 1736, 134-140.

Rafalko, S.D., Brandon, T.L., Filz, George M., and Mitchell, J.K. (2007). “Fiber Reinforcement for Rapid Stabilization of Soft Clay Soils,” *Transportation Research Record: Journal of the Transportation Research Record*, 2026, 21-29.

Ranjan, G., Vasan, R.M., and Charan, H.D. (1996). “Probabilistic Analysis of Randomly Distributed Fiber-Reinforced Soil,” *Journal of Geotechnical Engineering*, 122(6), 419-426.

Santoni, R.L., Tingle, J.S., and Webster S.L. (2001). “Engineering Properties of Sand- Fiber Mixtures For Road Construction,” *Journal of Geotechnical and Geoenvironmental Engineering*, 127(3), 258-268.

Santoni, R.L., and Webster, S.L. (2001). “Airfields and Roads Construction Using Fiber Stabilization of Sands,” *Journal of Transportation Engineering*, 127(2), 96-104.

Sing, W.L., Hashim, R., and Ali, F.H. (2008). “Engineering Behaviour of Stabilized Peat Soil,” *European Journal of Scientific Research,* 21(4), 581-591.

Skempton, A.W. (1953). "The Colloidal Activity of Clays," *Proceedings of the 3rd International Conference of Soil Mechanics and Foundation Engineering,* Vol 1, 57-60.

Sehn, A.L. (1998). *Properties of Grout Mixes and Grouted Soils for the Ham Marine Project.* Specialty Study Report Submitted to HBI Inc, Odenton, MD, 58.

Tanaka, Y., Taguchi, H., and Mitarai, Y. (2009). "Application of Foam-mixed Light-weight Stabilized Geo-material for Port and Airport Constructions," *Proceedings of the Deep Mixing 2009 Okinawa Symposium and International Symposium on Deep Mixing & Admixture Stabilization,* May 19-21, Okinawa, Japan.

Terzaghi, K., Peck, R.B., and Mesri, G. (1996). *Soil Mechanics in Engineering Practice.* John Wiley & Sons, Inc, New York, NY.

Tingle, J.S., Webster, S.L., and Santoni, R.L. (1999). "Discrete Fiber Reinforcement of Sands for Expedient Road Construction," Technical Report GL-99-3, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

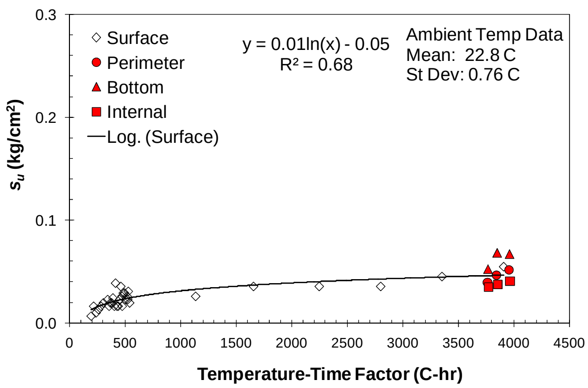
Tremblay, H., Duchesne, J., Locat, J., and Leroueil, S. (2002). "Influence of the Nature of Organic Compounds on Fine Soil Stabilization with Cement," *Canadian Geotechnical Journal*, 39(2), 535-546.

Tripathi, H., Pierce, C.E., Gassman, S.L., and Brown, T.W. (2004). "Methods for Field and Laboratory Measurement of Flowability and Setting Time of Controlled Low-Strength Materials," *Journal of ASTM International*, 1(6), 1-15

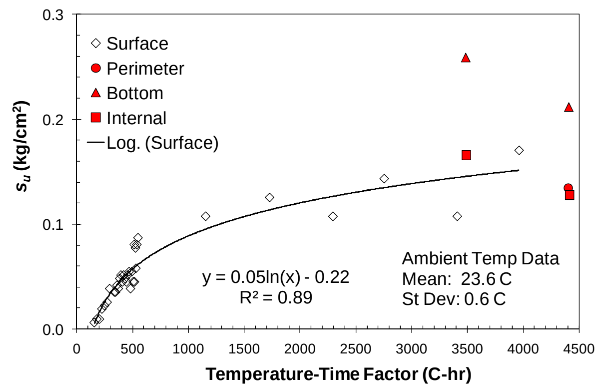
Vaghar, S., Donovan, J. Dobosz, K., and Clary, J. (1997). "Treatment and Stabilization of Dredged Harbor Bottom Sediments; Central Artery/Tunnel Project, Boston, Massachusetts," *Proceedings of GeoLogan '97,* Dredging and Management of Dredged Materials-Geotechnical Special Publication No. 65, July 16-17, Logan, UT, 105-121.

Zheng, J., and Qin, W. (2003). "Performance Characteristics of Soil-Cement from Industry Waste Binder," *Journal of Materials in Civil Engineering,* 15(6), 616-618.

APPENDIX A – SLAB TRIAL TEST RESULTS



a) *Th T III (5, 233) Soil 1-Dial*



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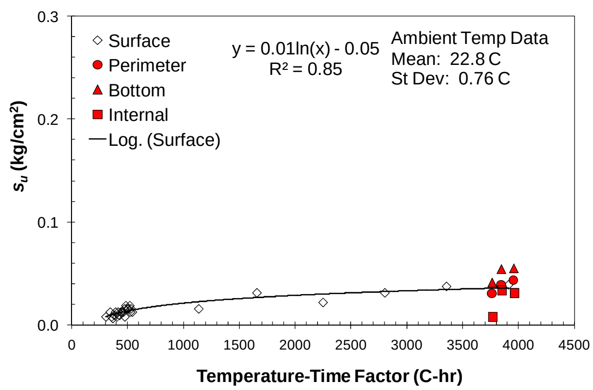
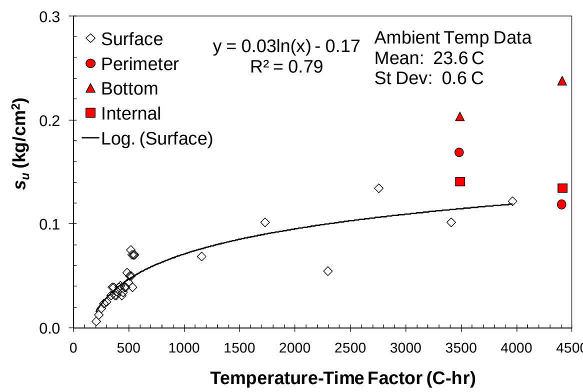


b) *Th T III (5, 233) Soil 1-Ring*



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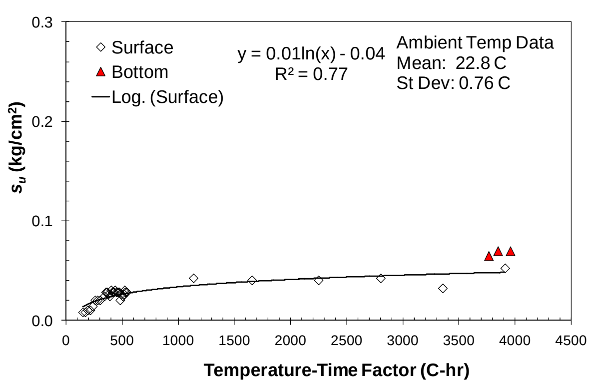
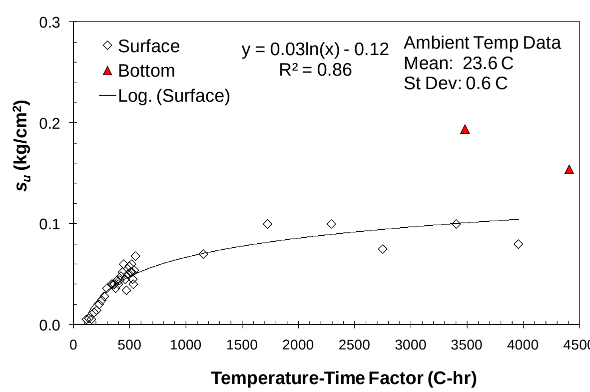
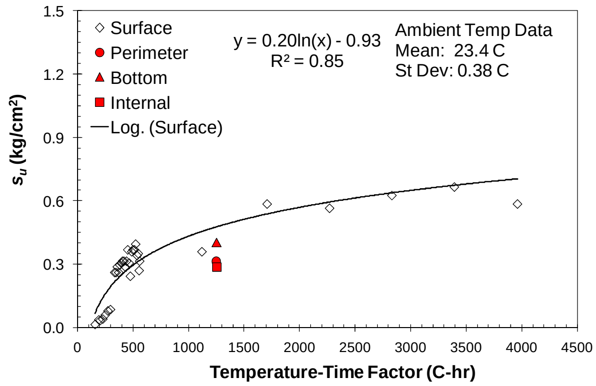


c) *Th T III (5, 233) Soil 1-Shear*

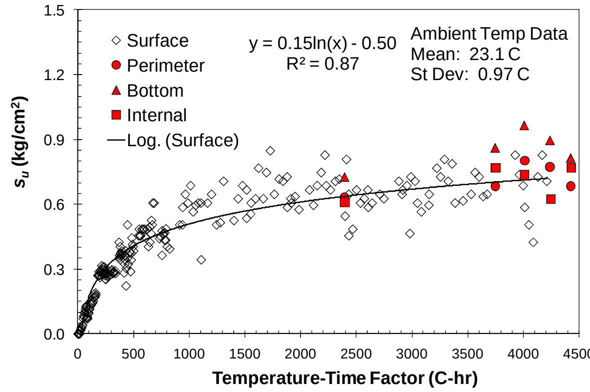


f) *Th T III (10, 233) Soil 1-Shear*

Figure A.1. Th T III (5, 233) Soil 1 and Th T III (10, 233) Soil 1



a) *Th T III (15, 233) Soil 1-Dial*



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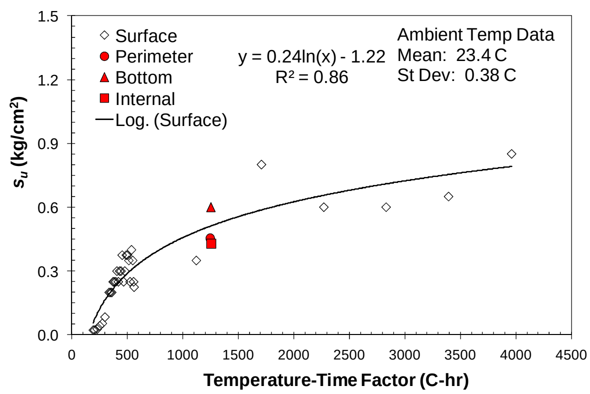
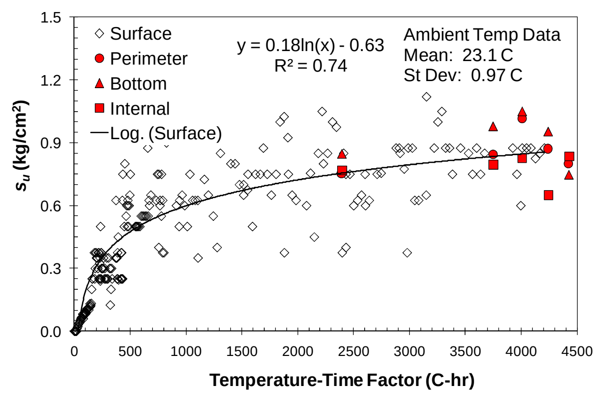
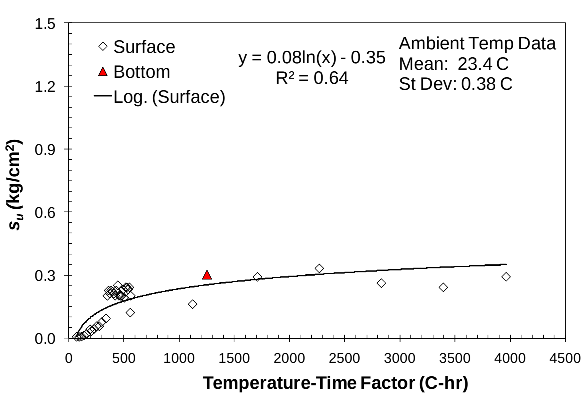


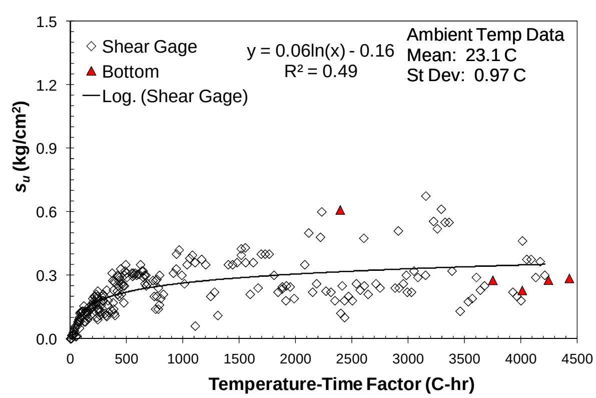
b) *Th T III (15, 233) Soil 1-Ring*



e) *Th T III (5, 100) Soil 1-Ring*

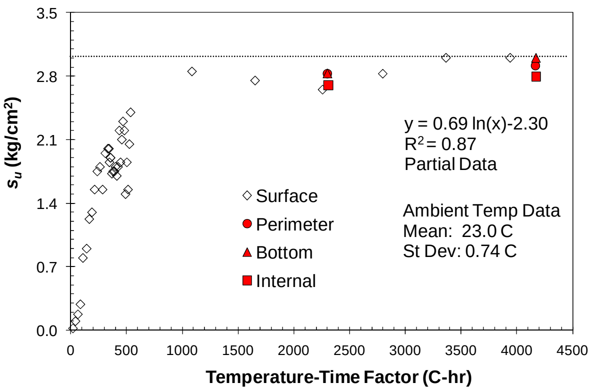


c) *Th T III (15, 233) Soil 1-Shear*

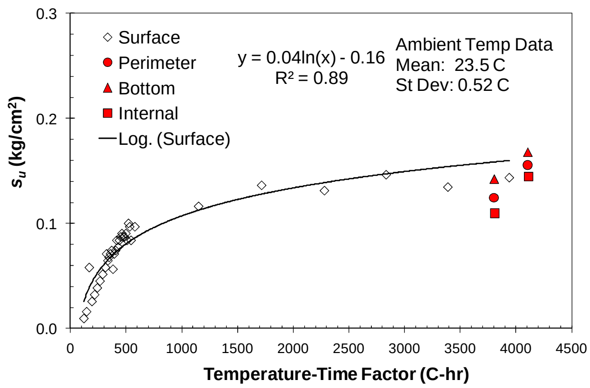


f) *Th T III (5, 100) Soil 1-Shear*

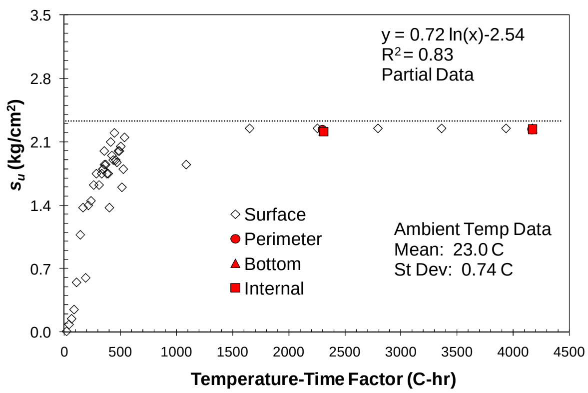
Figure A.2. Th T III (15, 233) Soil 1 and Th T III (5, 100) Soil 1



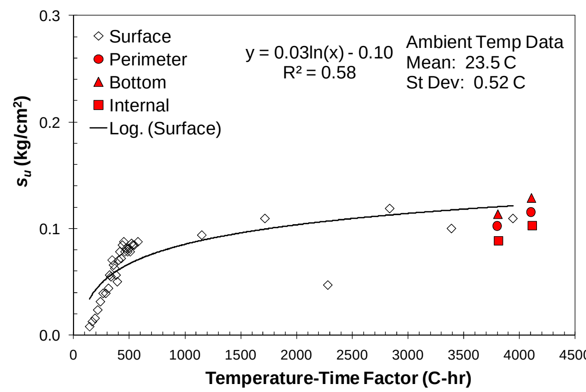
a) *Th T III (10, 100) Soil 1-Dial*



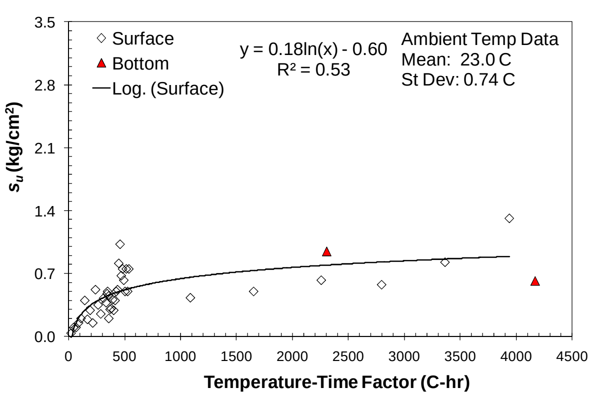
d) *Th T III (5, 233) Soil 2-Dial*



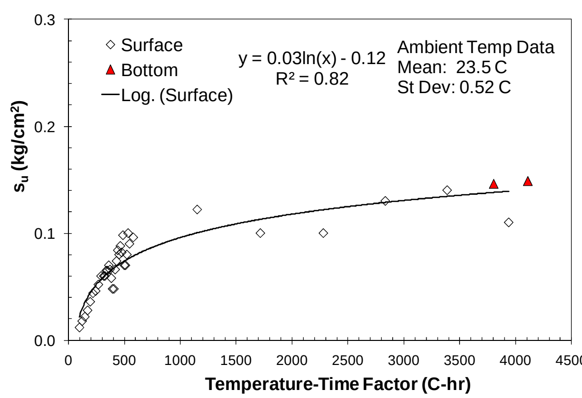
b) *Th T III (10, 100) Soil 1-Ring*



e) *Th T III (5, 233) Soil 2-Ring*

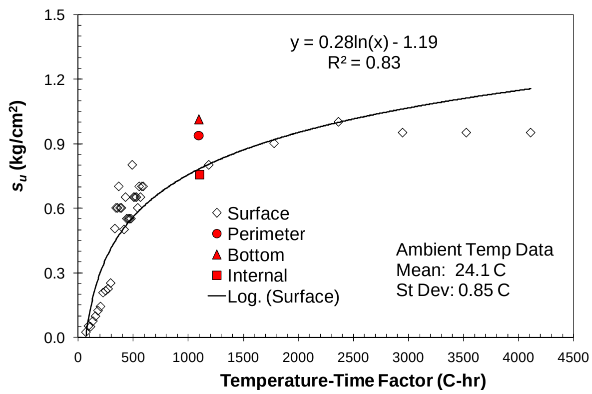


c) *Th T III (10, 100) Soil 1-Shear*

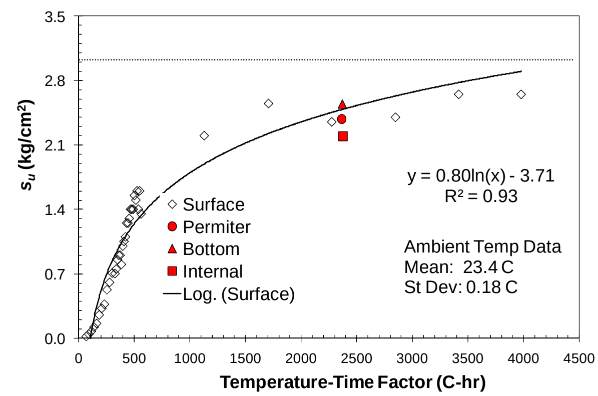


f) *Th T III (5, 233) Soil 2-Shear*

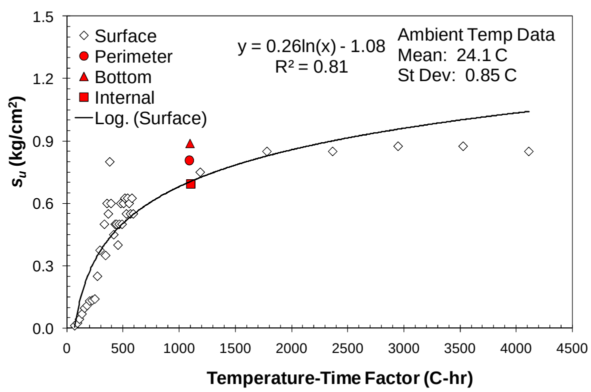
Figure A.3. Th T III (10, 100) Soil 1 and Th T III (5, 233) Soil 2



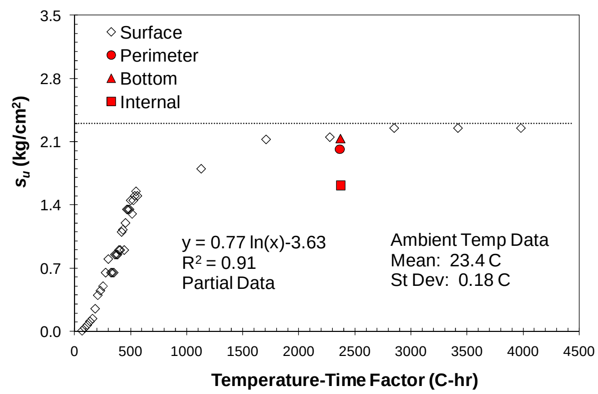
a) *Th T III (10, 233) Soil 2-Dial*



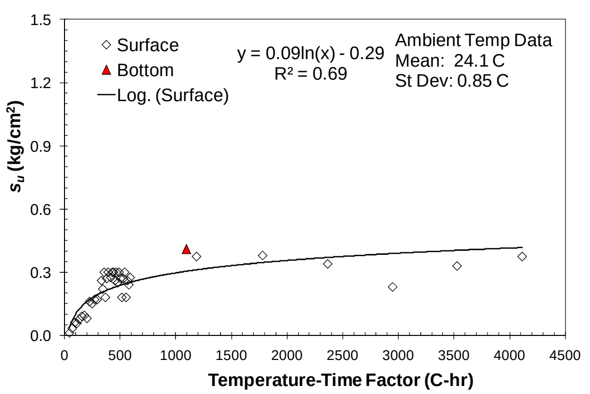
d) *Th T III (15, 233) Soil 2-Dial*



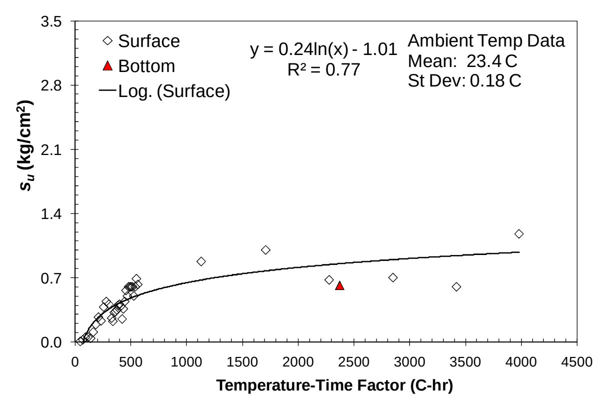
b) *Th T III (10, 233) Soil 2-Ring*



e) *Th T III (15, 233) Soil 2-Ring*

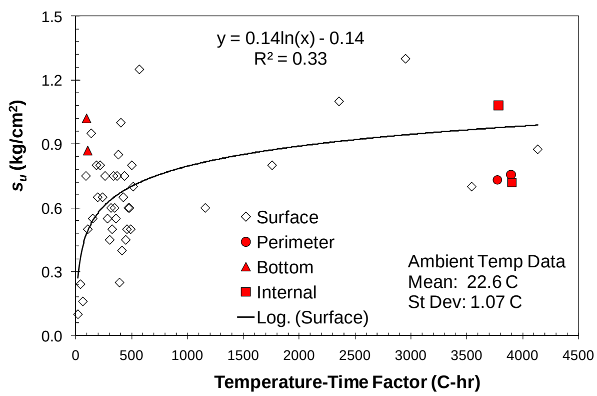


c) *Th T III (10, 233) Soil 2-Shear*

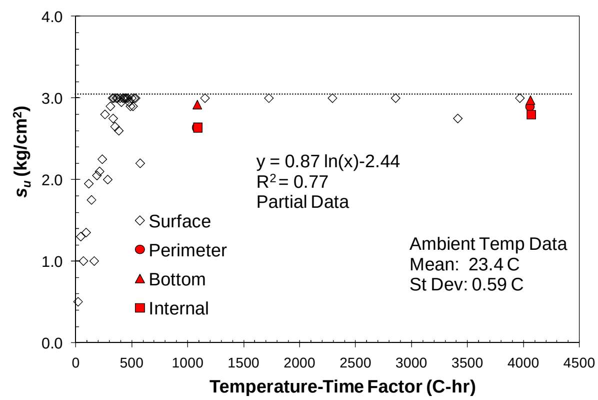


f) *Th T III (15, 233) Soil 2-Shear*

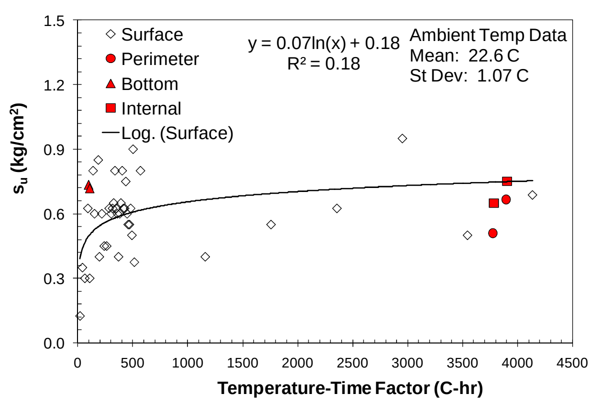
Figure A.4. Th T III (10, 233) Soil 2 and Th T III (15, 233) Soil 2



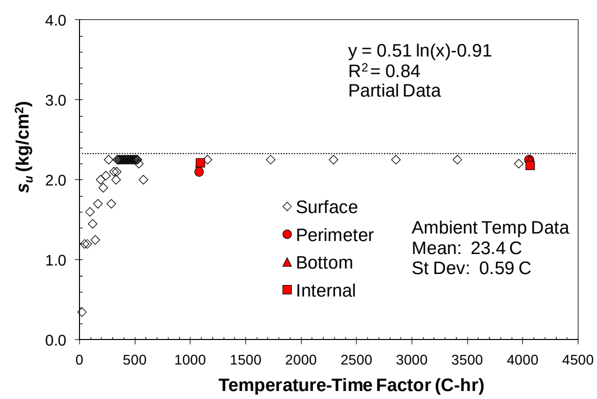
a) *Th T III (5, 100) Soil 2-Dial*



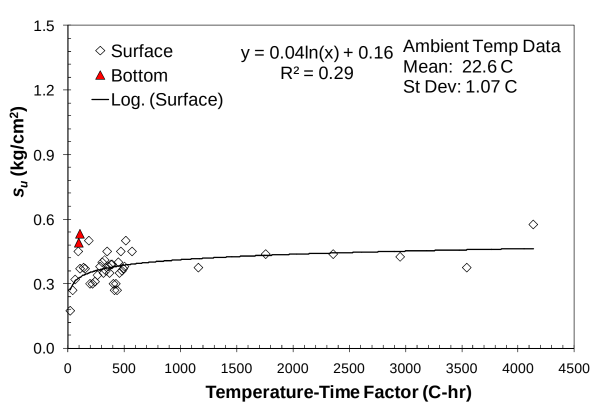
d) *Th T III (10, 100) Soil 2-Dial*



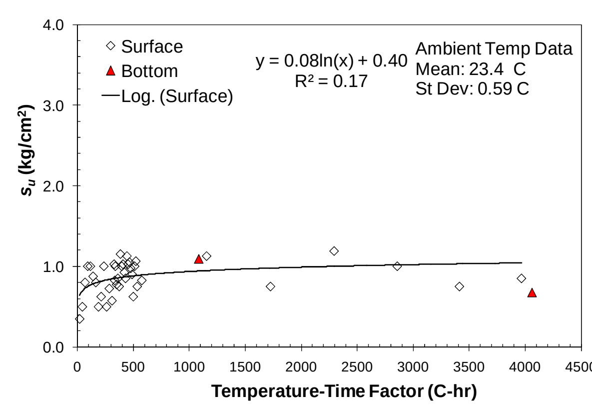
b) *Th T III (5, 100) Soil 2-Ring*



e) *Th T III (10, 100) Soil 2-Ring*

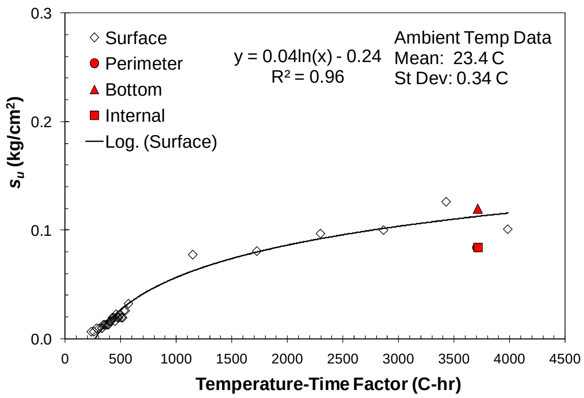


c) *Th T III (5, 100) Soil 2-Shear*

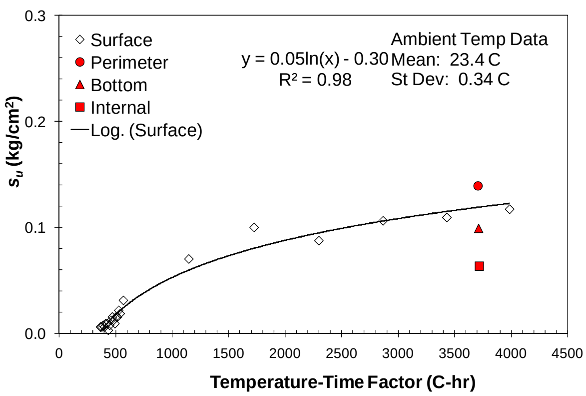


f) *Th T III (10, 100) Soil 2-Shear*

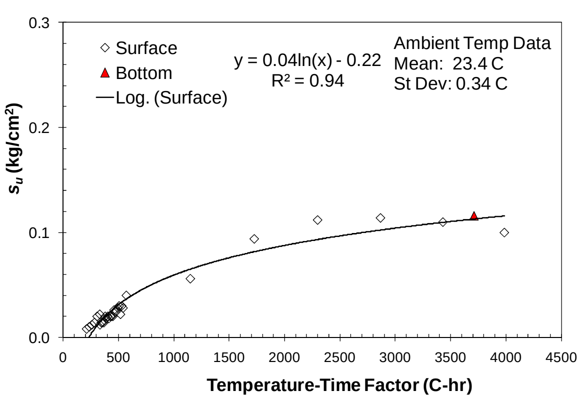
Figure A.5. Th T III (5, 100) Soil 2 and Th T III (10, 100) Soil 2



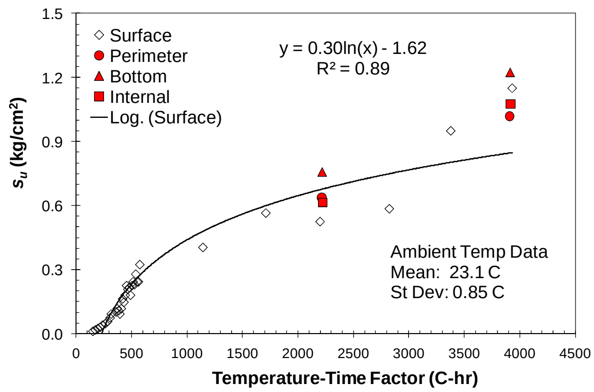
a) *Th T III (5, 233) Soil 3-Dial*



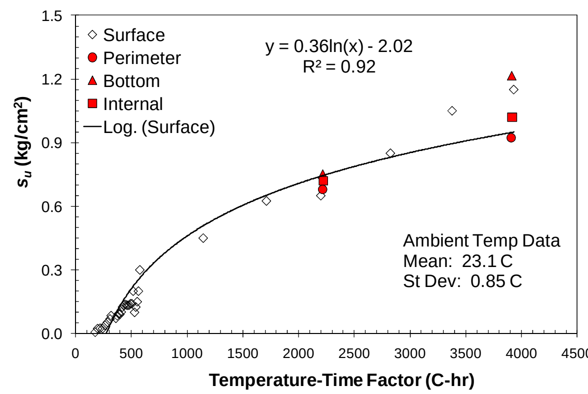
b) *Th T III (5, 233) Soil 3-Ring*



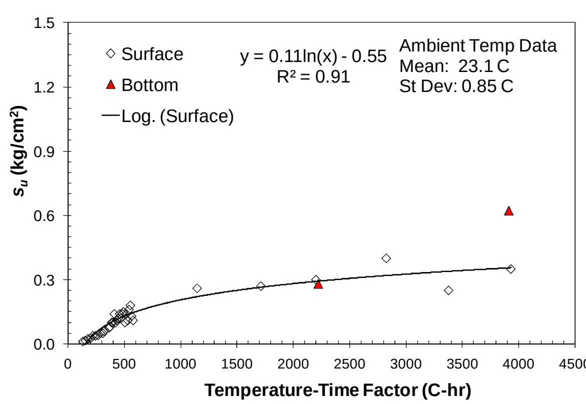
c) *Th T III (5, 233) Soil 3-Shear*



d) *Th T III (10, 233) Soil 3-Dial*

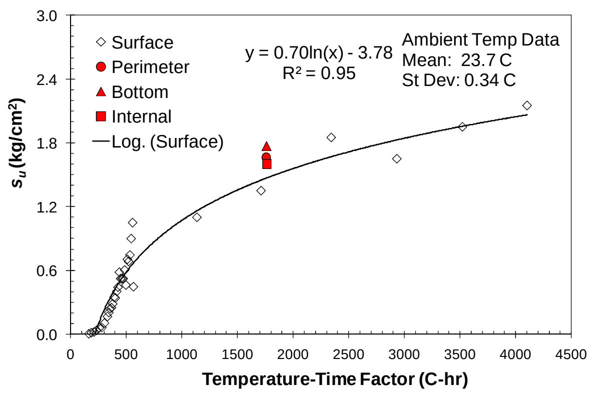


e) *Th T III (10,233) Soil 3-Ring*

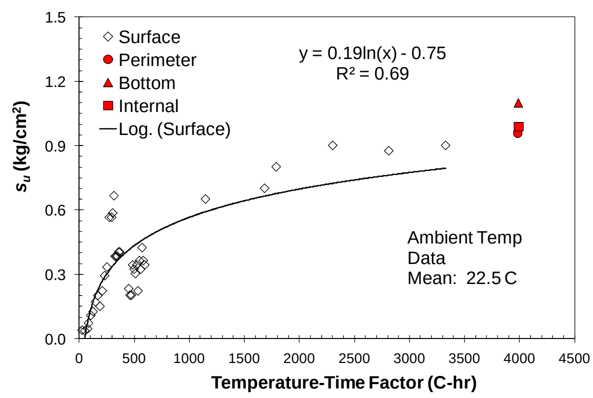


f) *Th T III (10, 233) Soil 3-Shear*

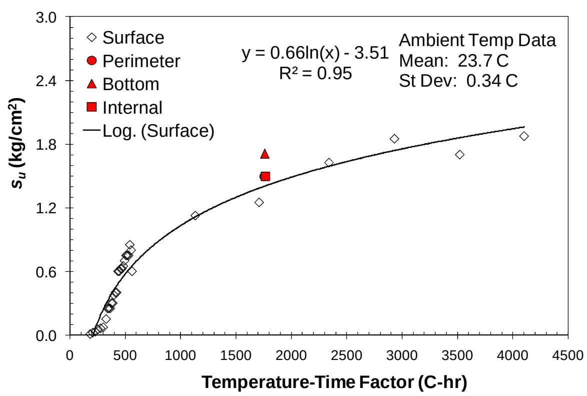
Figure A.6. Th T III (5, 233) Soil 3 and Th T III (10, 233) Soil 3



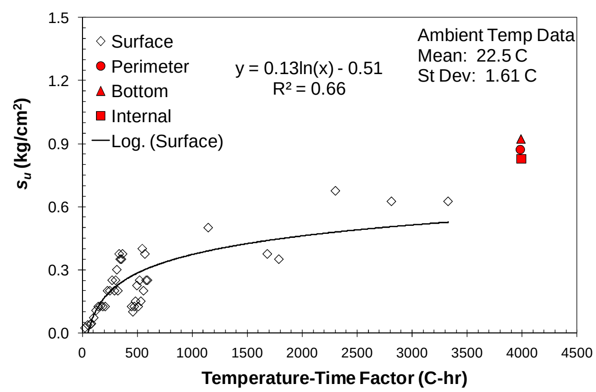
a) *Th T III (15, 233) Soil 3-Dial*



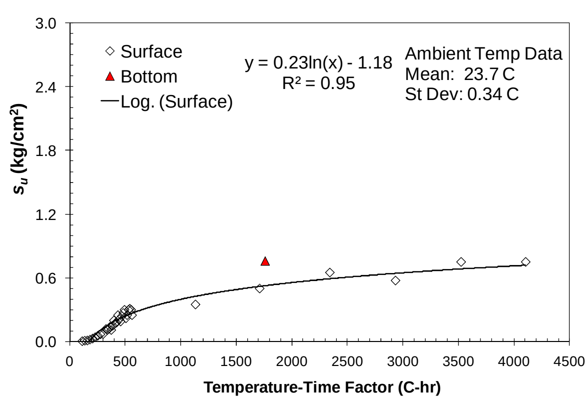
d) *Th T III (5, 100) Soil 3-Dial*



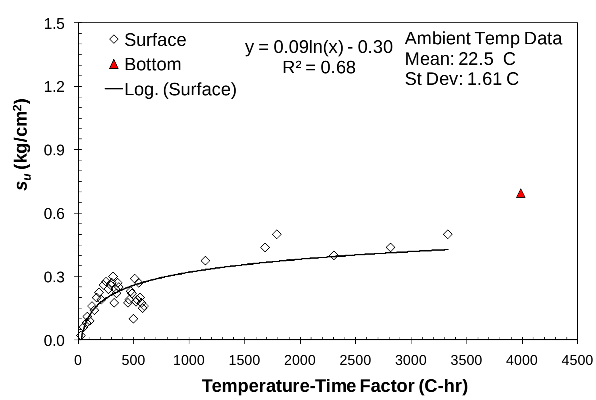
b) *Th T III (15, 233) Soil 3-Ring*



e) *Th T III (5, 100) Soil 3-Ring*

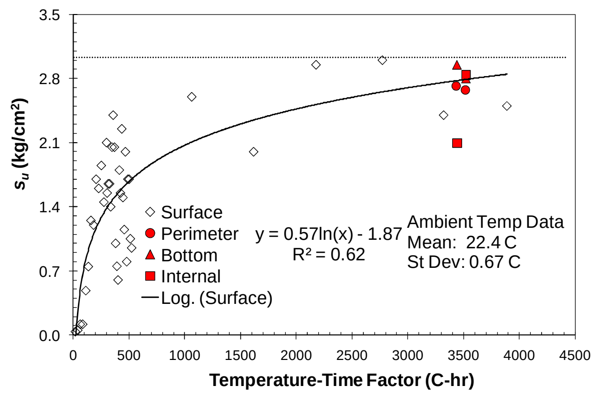


c) *Th T III (15, 233) Soil 3-Shear*

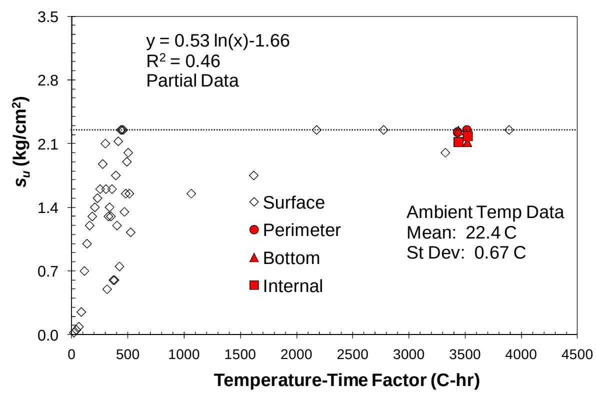


f) *Th T III (5, 100) Soil 3-Shear*

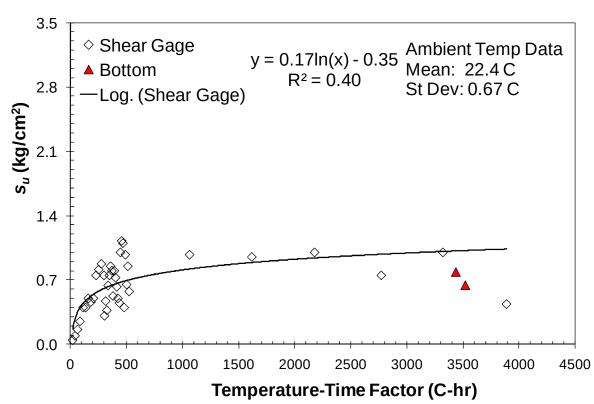
Figure A.7. Th T III (15, 233) Soil 3 and Th T III (5, 100) Soil 3



a) *Th T III (10, 100) Soil 3-Dial*

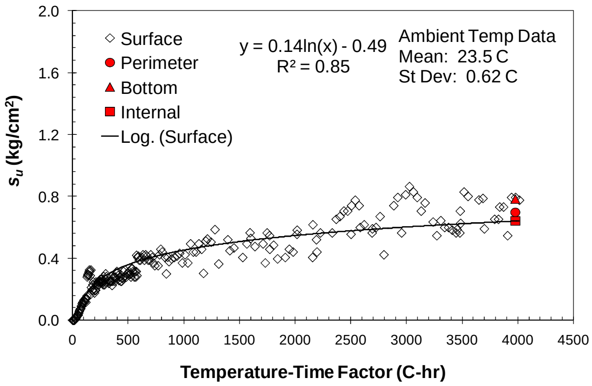


b) *Th T III (10, 100) Soil 3-Ring*

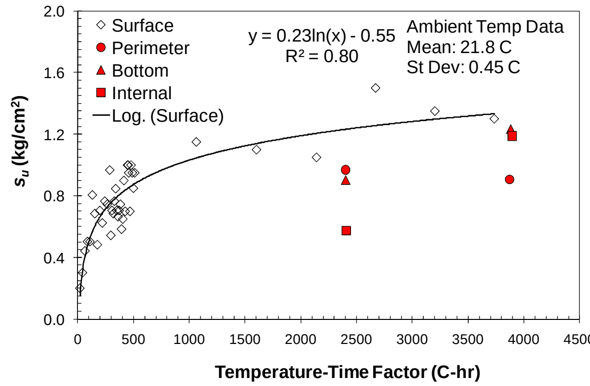


c) *Th T III (10, 100) Soil 3-Shear*

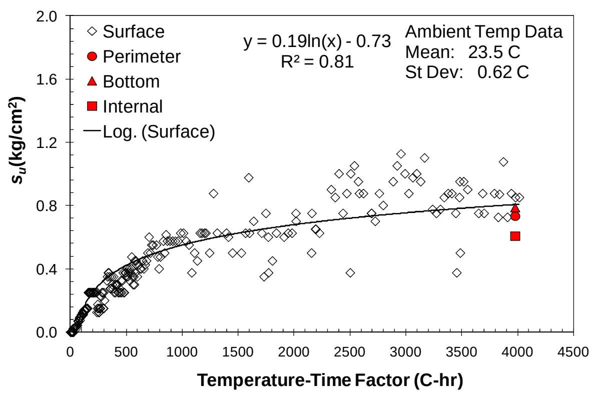
Figure A.8. Th T III (10, 100) Soil 3



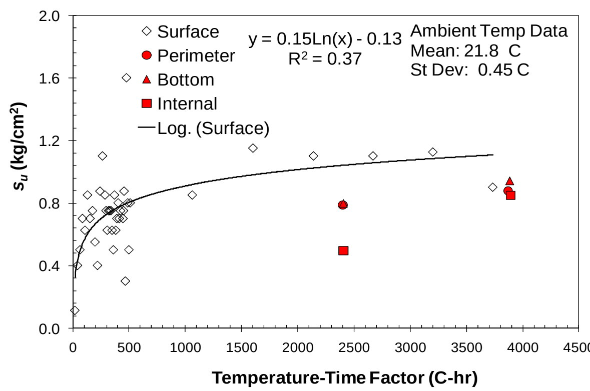
a) *A T I (5, 100) Soil 1-Dial*



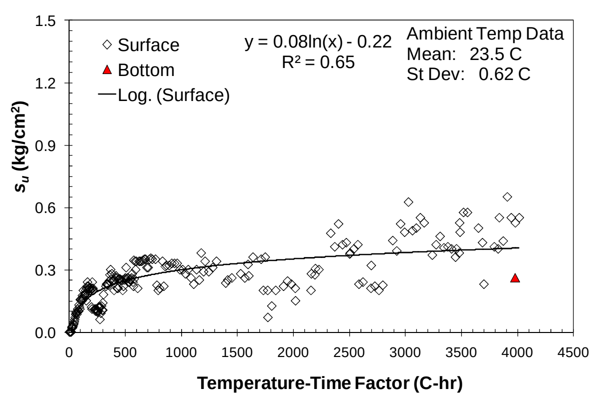
d) *A T I (5, 100) Soil 2-Dial*



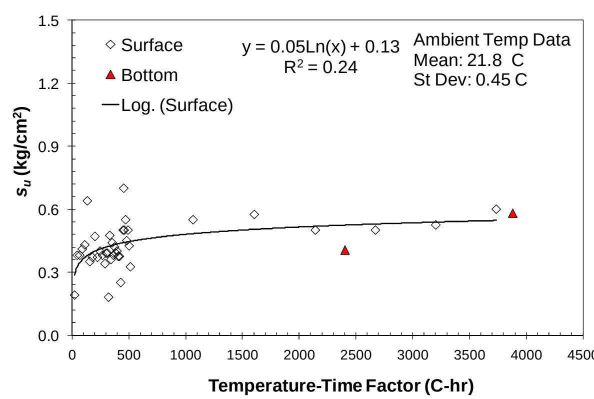
b) *A T I (5, 100) Soil 1-Ring*



e) *A T I (5, 100) Soil 2-Ring*

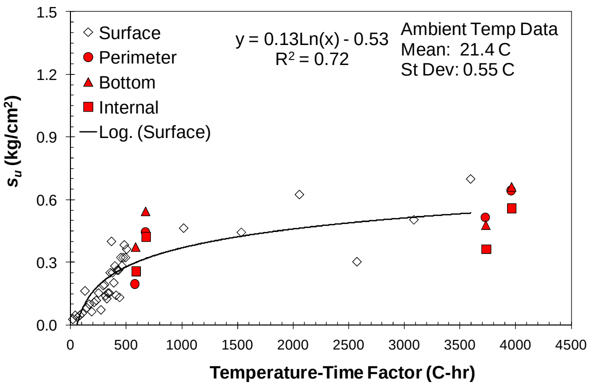


c) *A T I (5, 100) Soil 1-Shear*

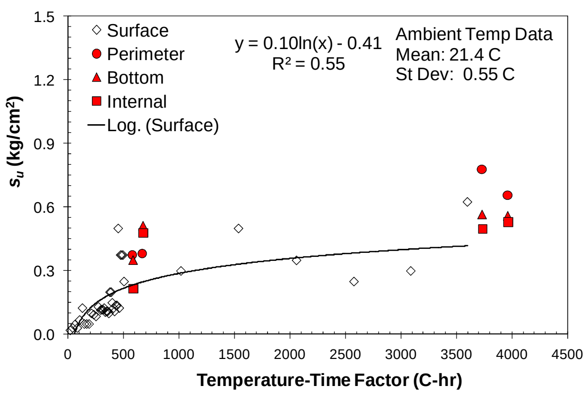


f) *A T I (5, 100) Soil 2-Shear*

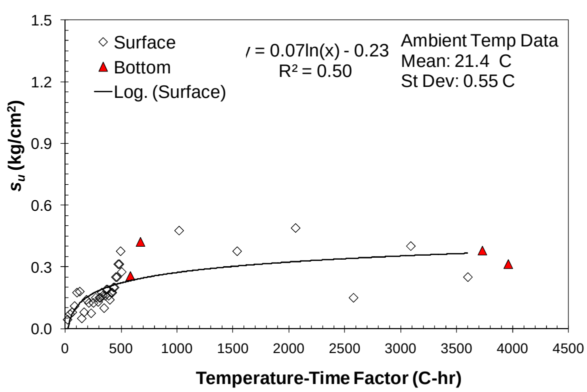
Figure A.9. A T I (5, 100) Soil 1 and A T I (5, 100) Soil 2



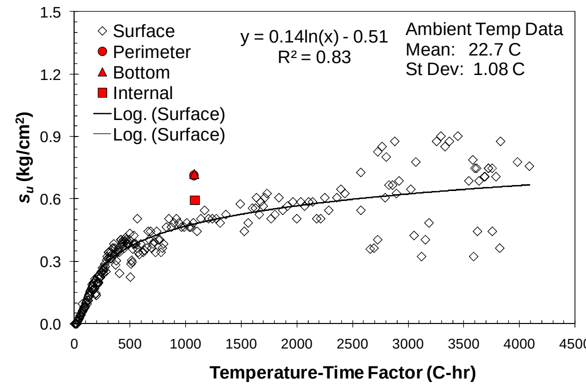
a) *A T I (5, 100) Soil 3-Dial*



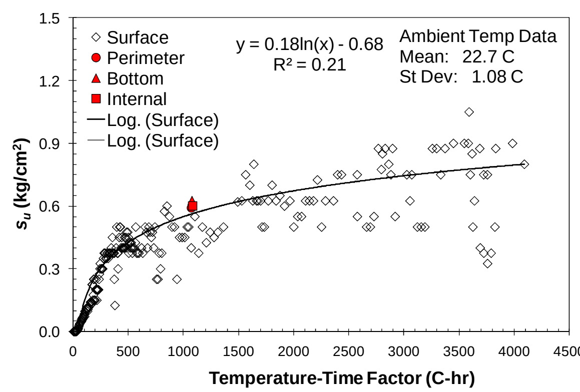
b) *A T I (5, 100) Soil 3-Ring*



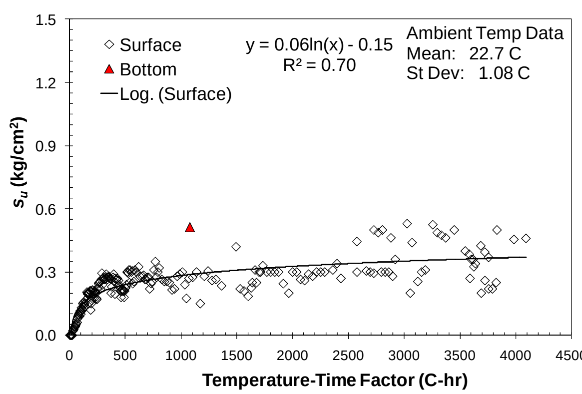
c) *A T I (5, 100) Soil 3-Shear*



d) *Th T I/II (5, 100) Soil 1-Dial*

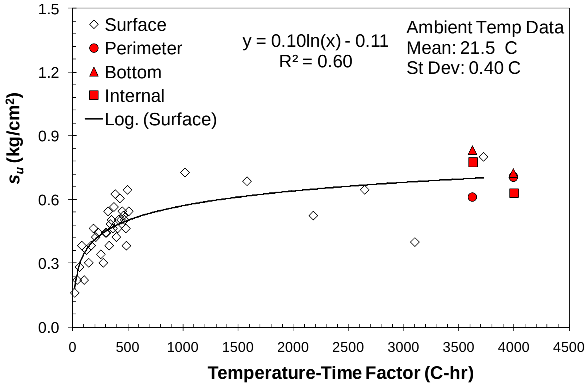


e) *Th T I/II (5, 100) Soil 1-Ring*

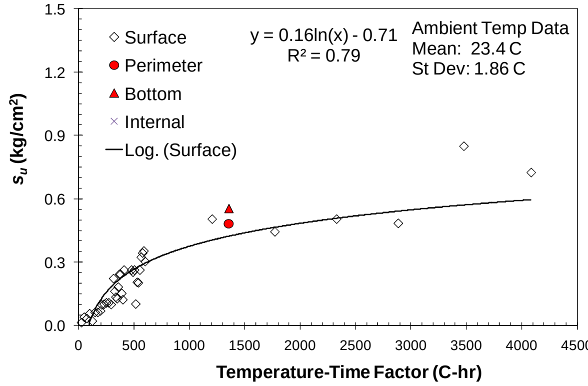


f) *Th T I/II (5, 100) Soil 1-Shear*

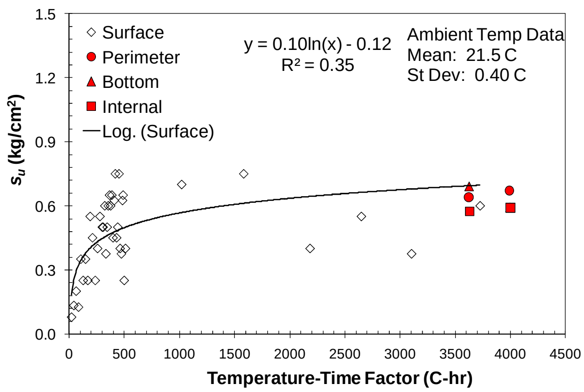
Figure A.10. A T I (5, 100) Soil 3 and Th T I/II (5, 100) Soil 1



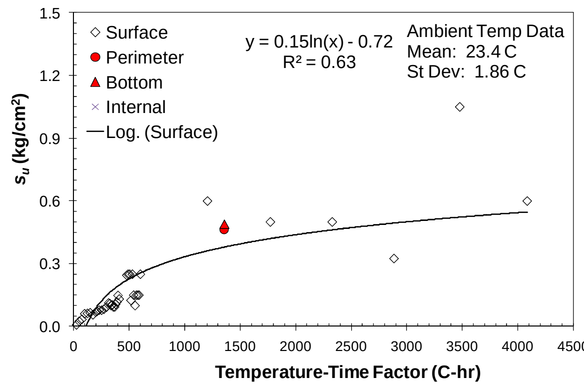
a) *Th T I/II (5, 100) Soil 2-Dial*



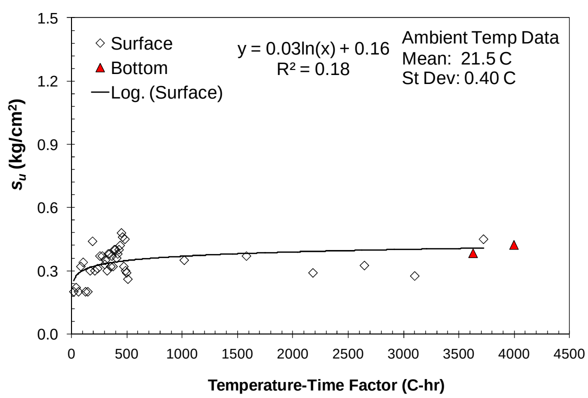
d) *Th T I/II (5, 100) Soil 3-Dial*



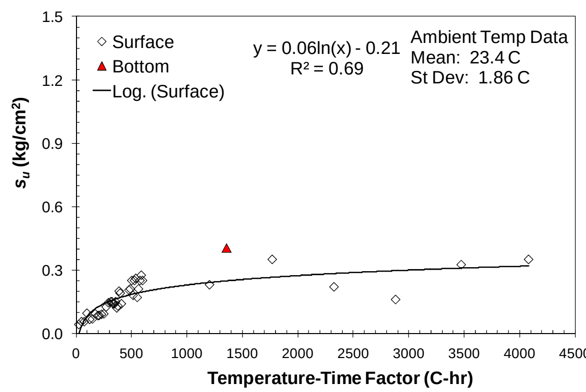
b) *Th T I/II (5, 100) Soil 2-Ring*



e) *Th T I/II (5, 100) Soil 3-Ring*

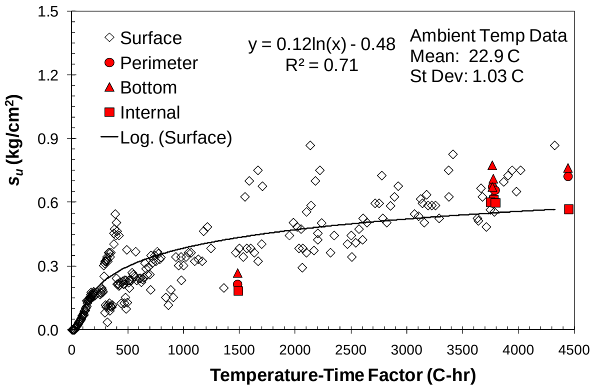


c) *Th T I/II (5, 100) Soil 2-Shear*

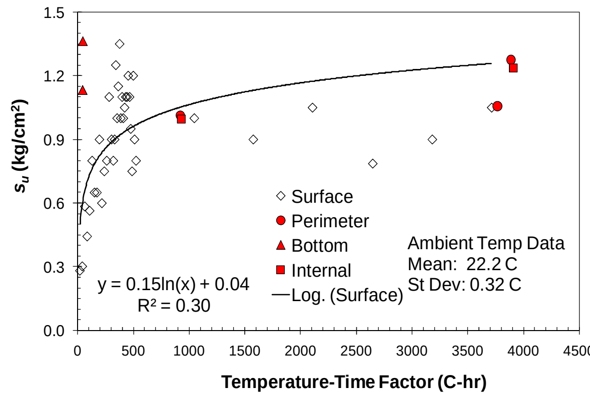


f) *Th T I/II (5, 100) Soil 3-Shear*

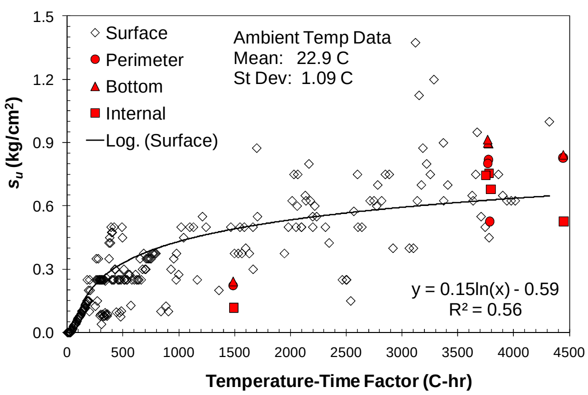
Figure A.11. Th T I/II (5, 100) Soil 2 and Th T I/II (5, 100) Soil 3



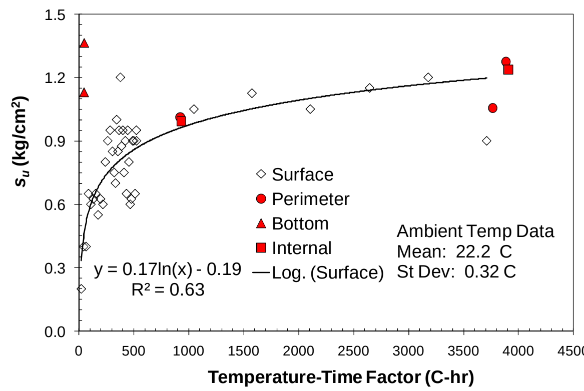
a) *A T III (5, 100) Soil 1-Dial*



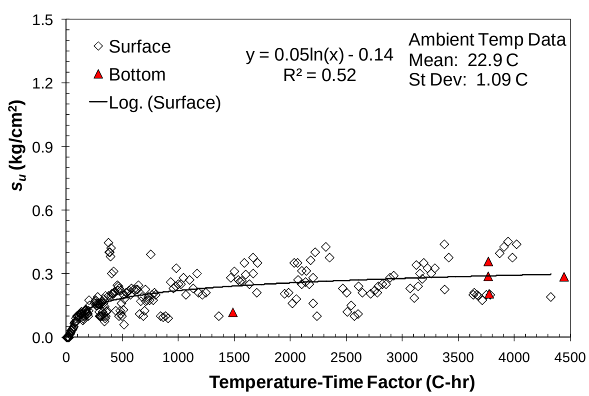
d) *A T III (5, 100) Soil 2-Dial*



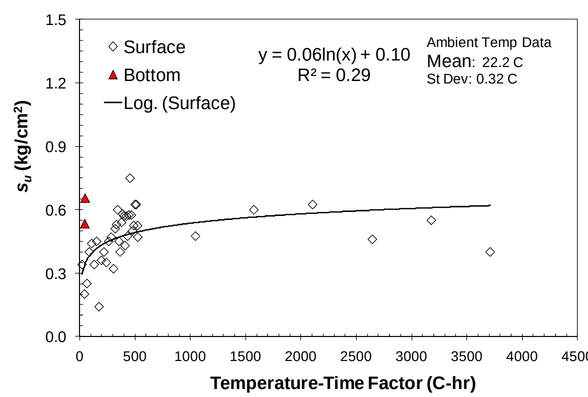
b) *A T III (5, 100) Soil 1-Ring*



e) *A T III (5, 100) Soil 2-Ring*

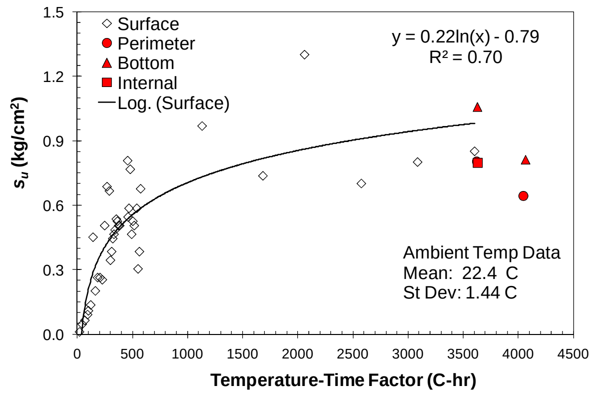


c) *A T III (5, 100) Soil 1-Shear*

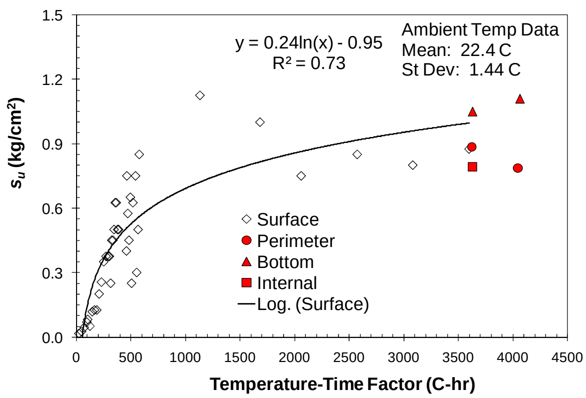


f) *A T III (5, 100) Soil 2-Shear*

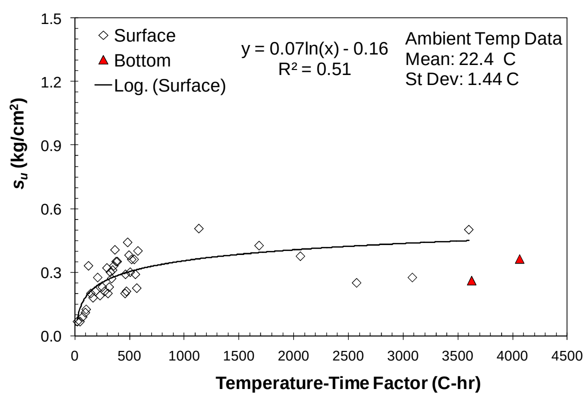
Figure A.12. A T III (5, 100) Soil 1and A T III (5, 100) Soil 2



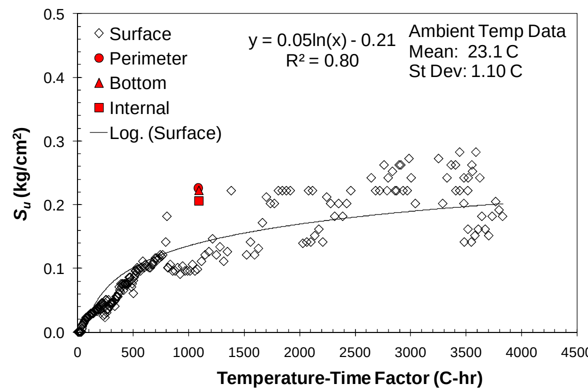
a) *A T III (5, 100) Soil 3-Dial*



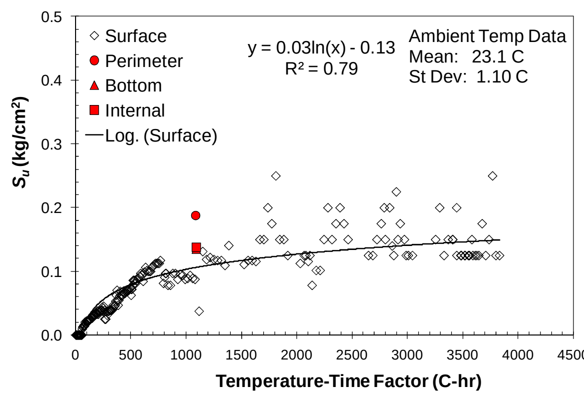
b) *A T III (5, 100) Soil 3-Ring*



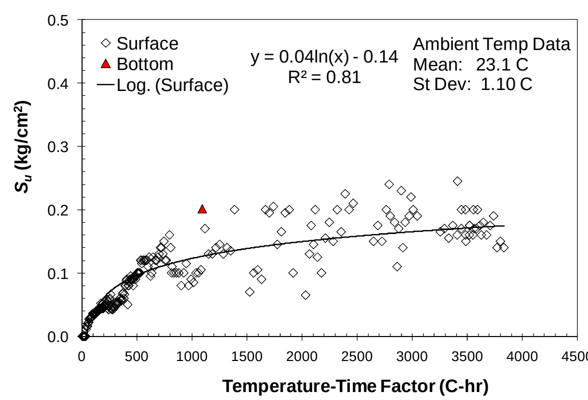
c) *A T III (5, 100) Soil 3-Shear*



d) *CTS RS (5, 100) Soil 1-Dial*

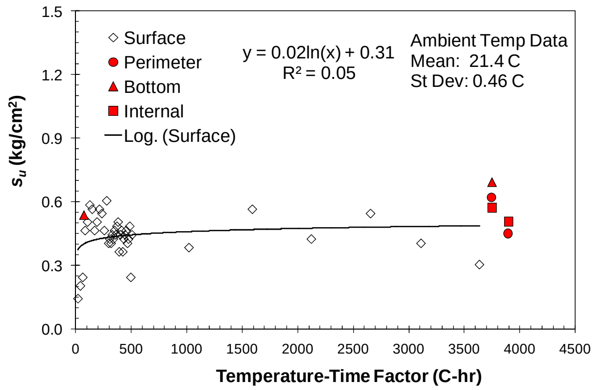


e) *CTS RS (5, 100) Soil 1-Ring*

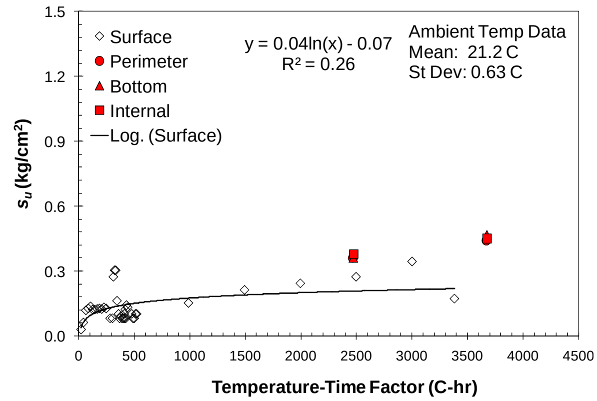


f) *CTS RS (5, 100) Soil 1-Shear*

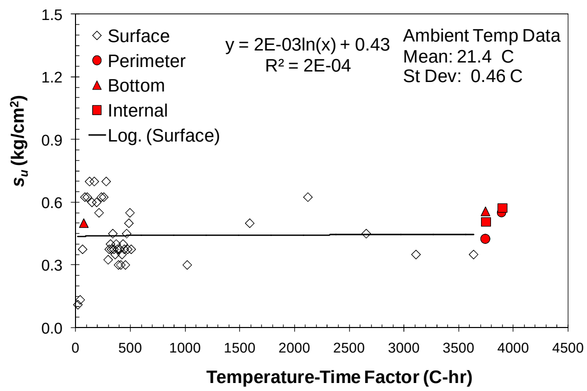
Figure A.13. A T III (5, 100) Soil 3and CTS RS (5, 100) Soil 1



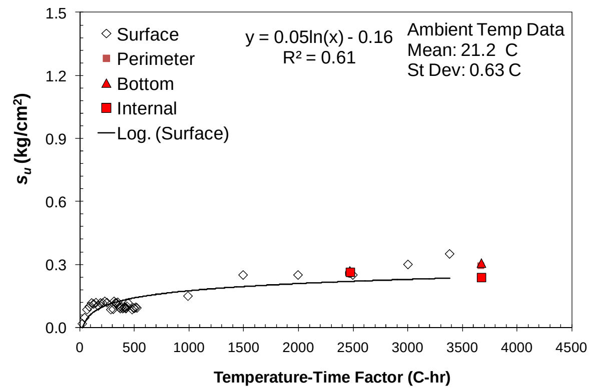
a) *CTS RS (5, 100) Soil 2-Dial*



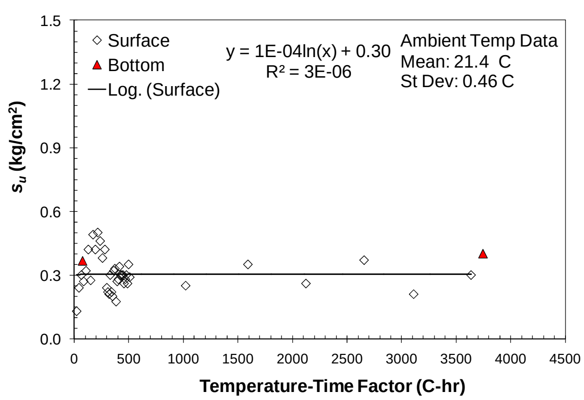
d) *CTS RS (5, 100) Soil 3-Dial*



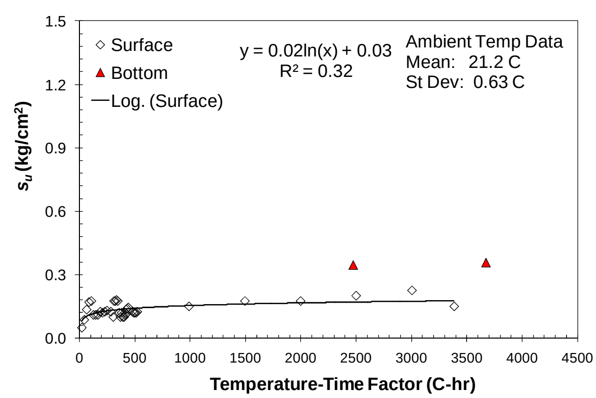
b) *CTS RS (5, 100) Soil 2-Ring*



e) *CTS RS (5, 100) Soil 3-Ring*

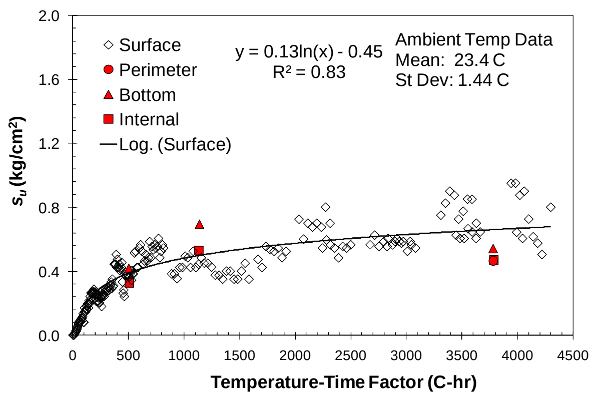


c) *CTS RS (5, 100) Soil 2-Shear*

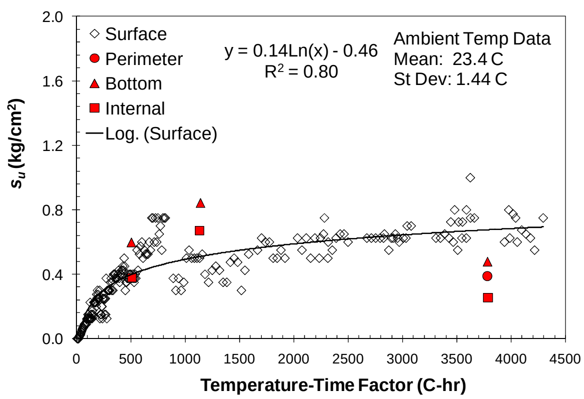


f) *CTS RS (5, 100) Soil 3-Shear*

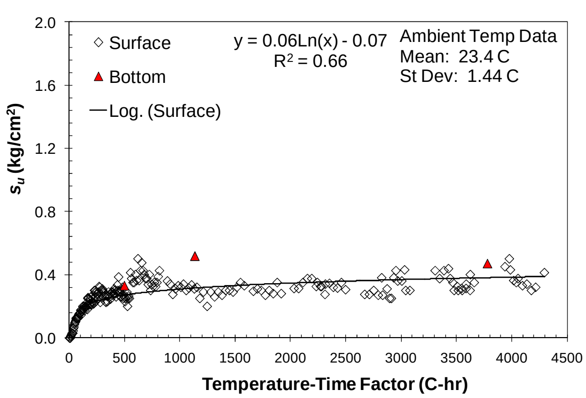
Figure A.14. CTS RS (5, 100) Soil 2 and CTS RS (5, 100) Soil 3



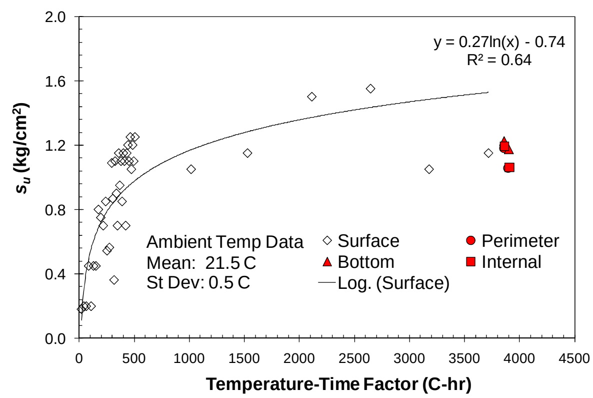
a) *SC1 (5, 100) Soil 1-Dial*



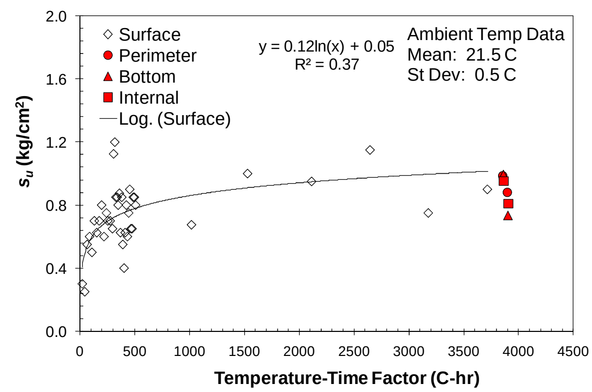
b) *SC1 (5, 100) Soil 1-Ring*



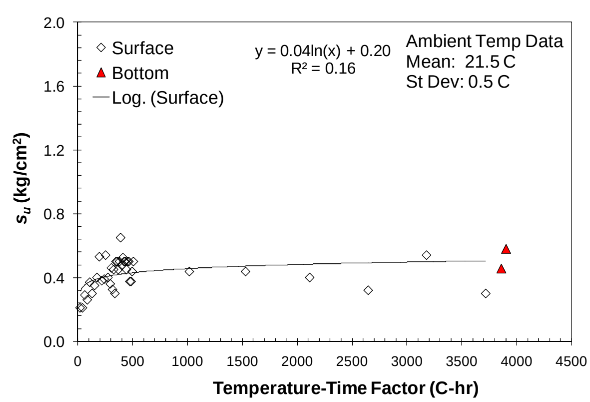
c) *SC1 (5, 100) Soil 1-Shear*



d) *SC1 (5, 100) Soil 2-Dial*

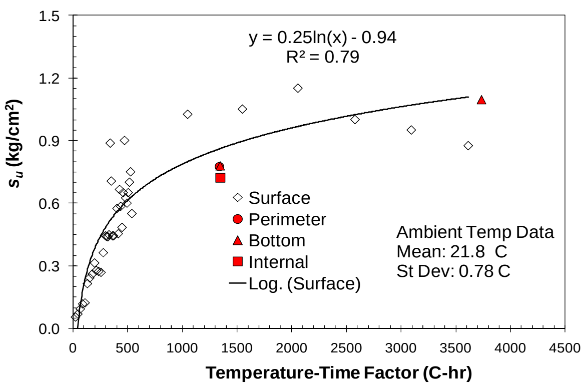


e) *SC1 (5, 100) Soil 2-Ring*

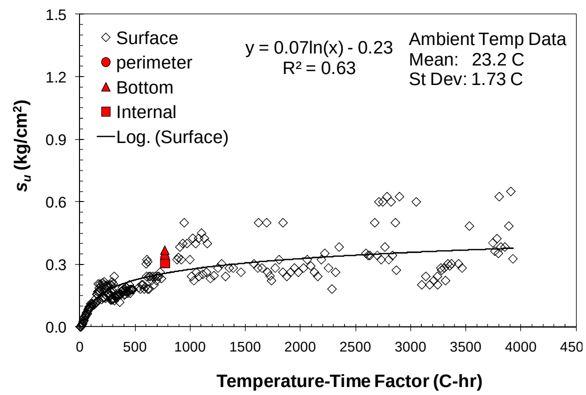


f) *SC1 (5, 100) Soil 2-Shear*

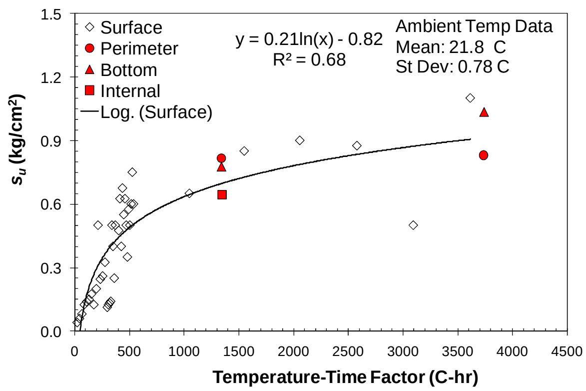
Figure A.15. SC1 (5, 100) Soil 1 and SC1 (5, 100) Soil 2



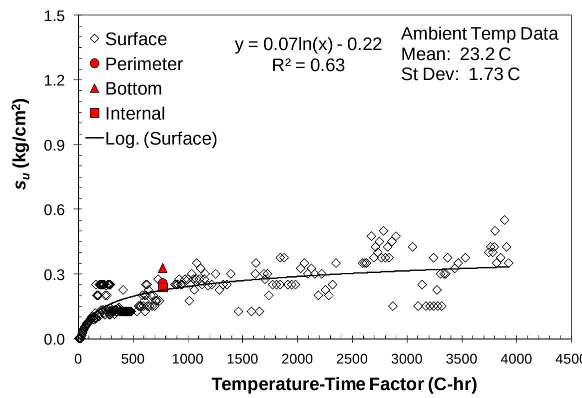
a) *SC1 (5, 100) Soil 3-Dial*



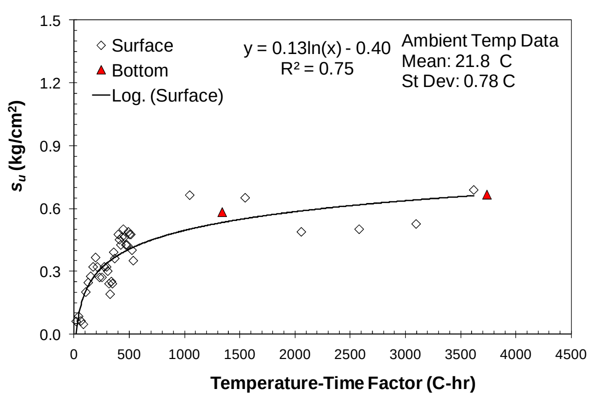
d) *SC2 (5, 100) Soil 1-Dial*



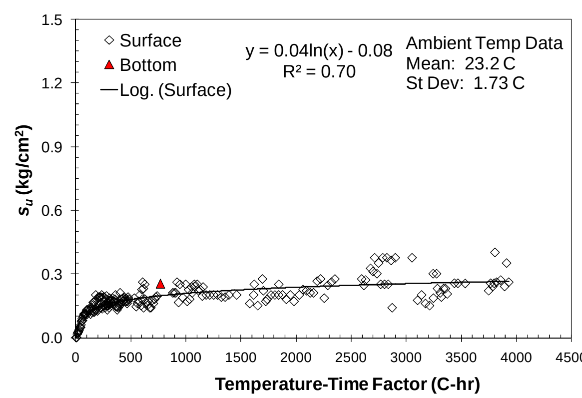
b) *SC1 (5, 100) Soil 3-Ring*



e) *SC2 (5, 100) Soil 1-Ring*

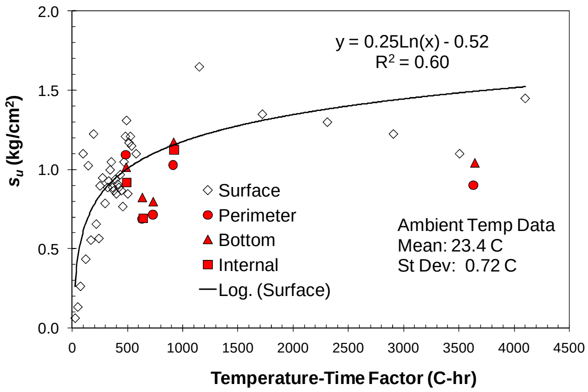


c) *SC1 (5, 100) Soil 3-Shear*

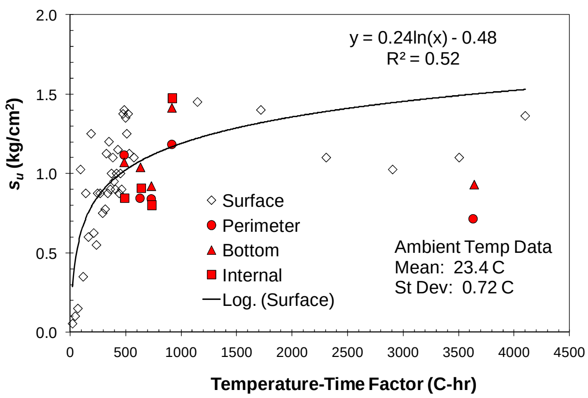


f) *SC2 (5, 100) Soil 1-Shear*

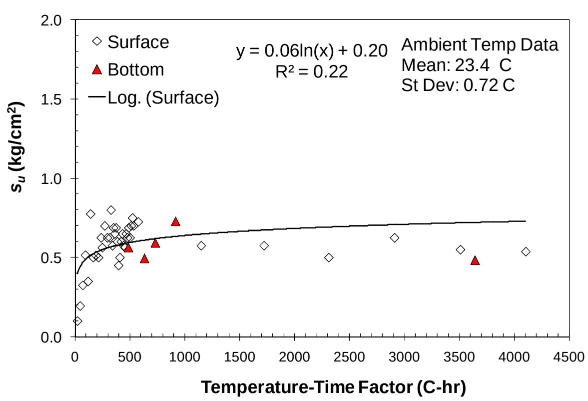
Figure A.16. SC1 (5, 100) Soil 3 and SC2 (5, 100) Soil 1



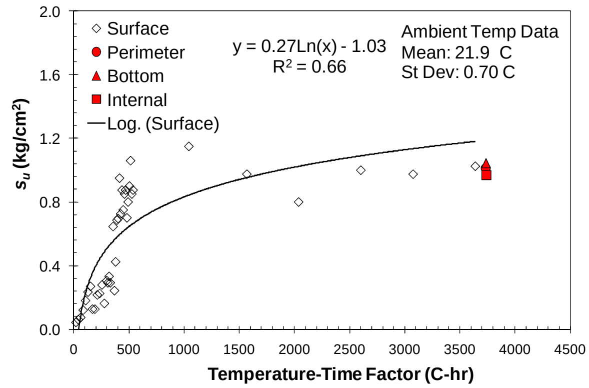
a) *SC2 (5, 100) Soil 2-Dial*



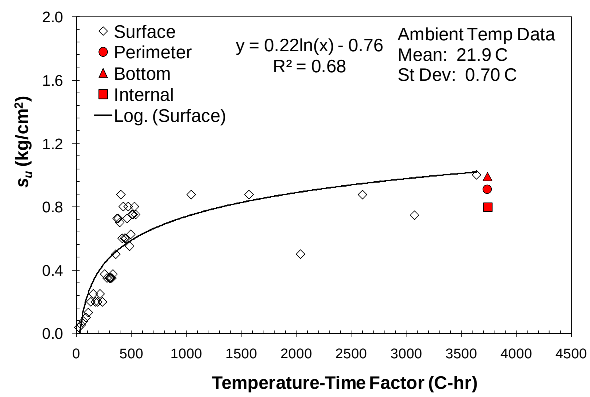
b) *SC2 (5, 100) Soil 2-Ring*



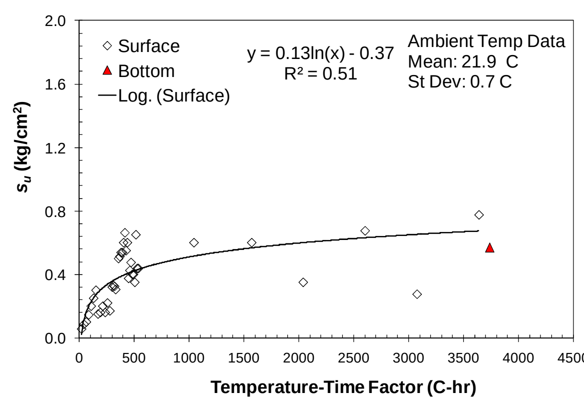
c) *SC2 (5, 100) Soil 2-Shear*



d) *SC2 (5, 100) Soil 3-Dial*

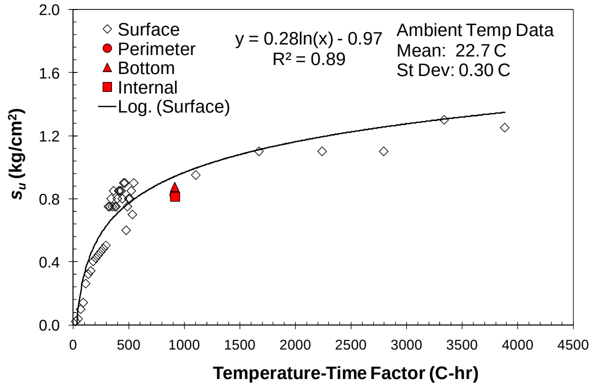


e) *SC2 (5, 100) Soil 3-Ring*

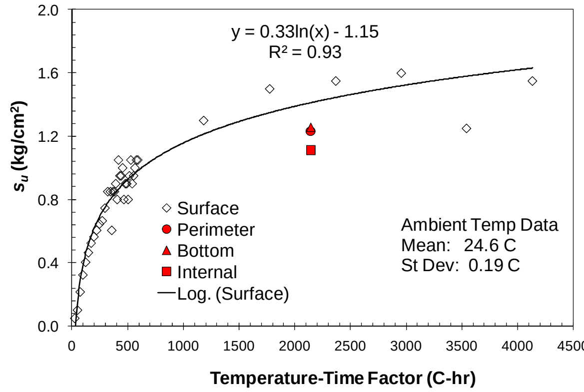


f) *SC2 (5, 100) Soil 3-Shear*

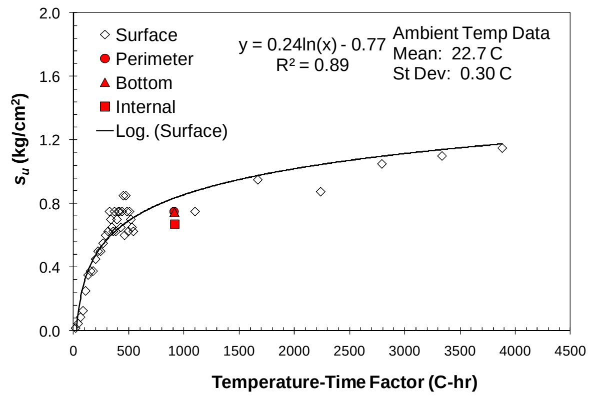
Figure A.17. SC2 (5, 100) Soil 2 and SC2 (5, 100) Soil 3



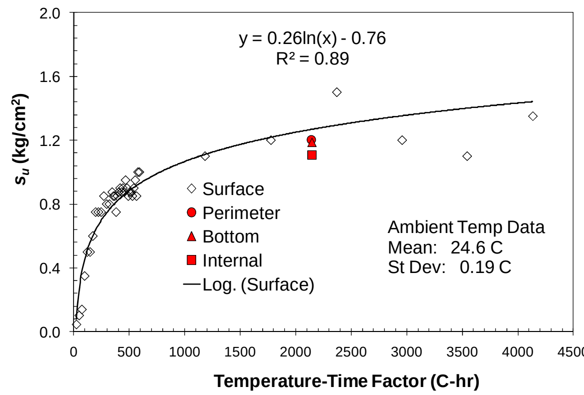
a) *SC1 (5, 100) Soil 1 Repeat-Dial*



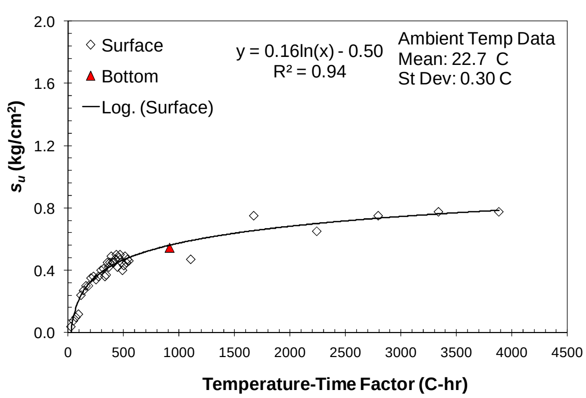
d) *SC1 (5, 100) Soil 1 Repeat-Dial*



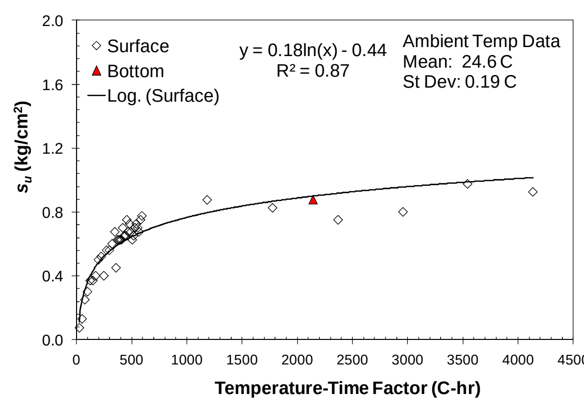
b) *SC1 (5, 100) Soil 1 Repeat -Ring*



e) *SC1 (5, 100) Soil 1 Repeat-Ring*

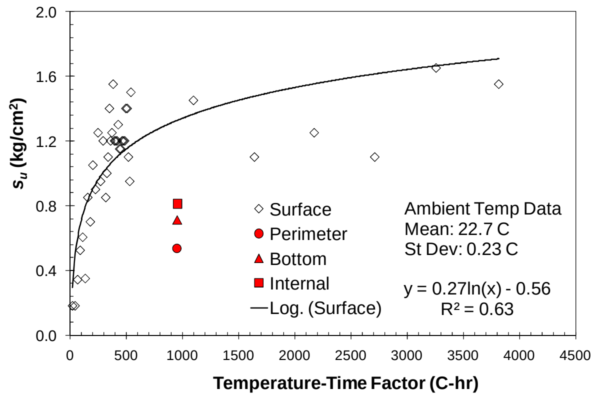


c) *SC1 (5, 100) Soil 1 Repeat -Shear*

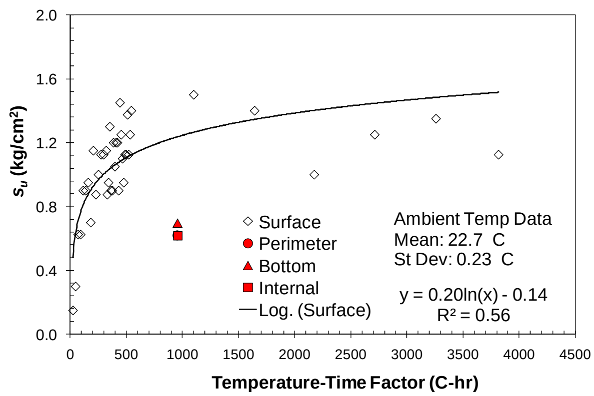


f) *SC1 (5, 100) Soil 1 Repeat-Shear*

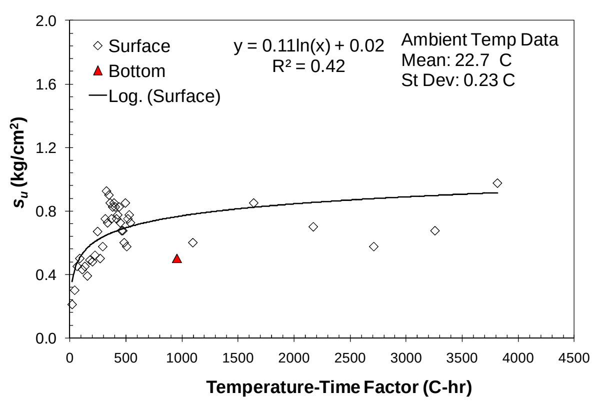
Figure A.18. SC1 (5, 100) Soil 1 Repeat and SC1 (5, 100) Soil 1 Repeat



a) *SC1 (5, 100) Soil 2 Repeat-Dial*

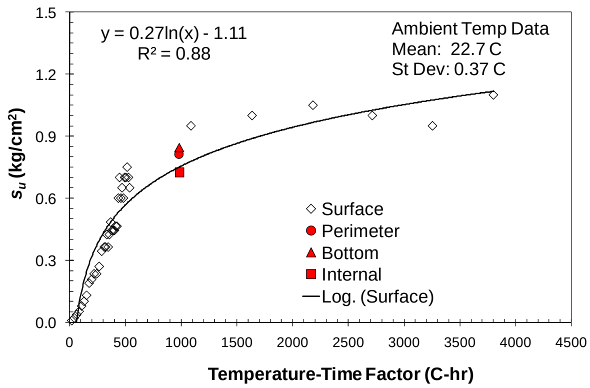


b) *SC1 (5, 100) Soil 2 Repeat-Ring*

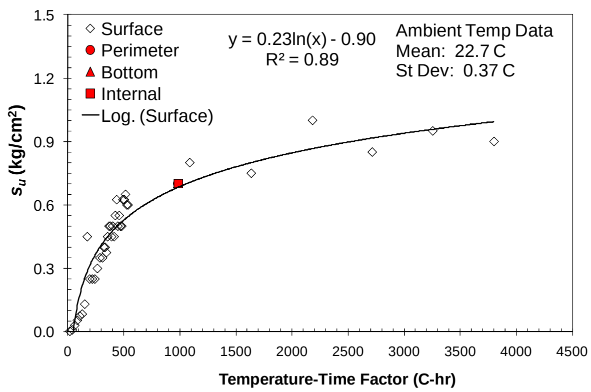


c) *SC1 (5, 100) Soil 2 Repeat-Shear*

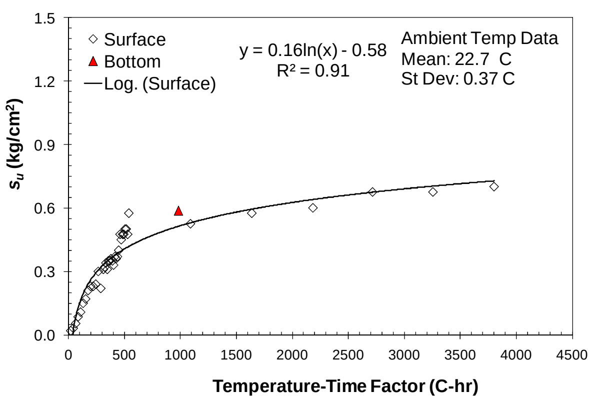
Figure A.19. SC1 (5, 100) Soil 2 Repeat



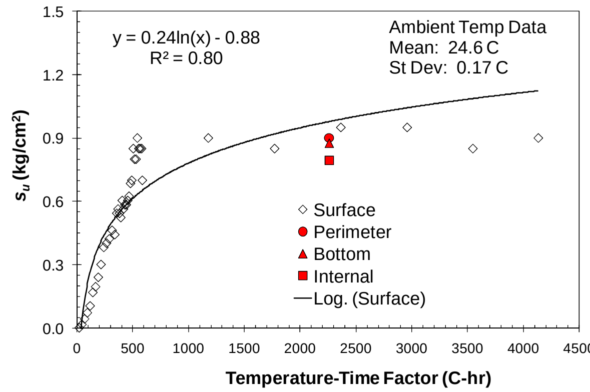
a) *SC1 (5, 100) Soil 3 Repeat-Dial*



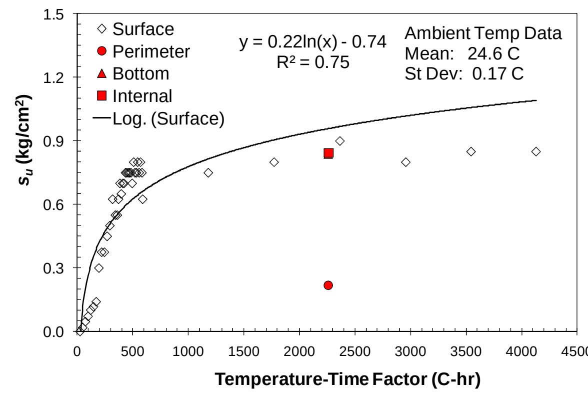
b) *SC1 (5, 100) Soil 3 Repeat-Ring*



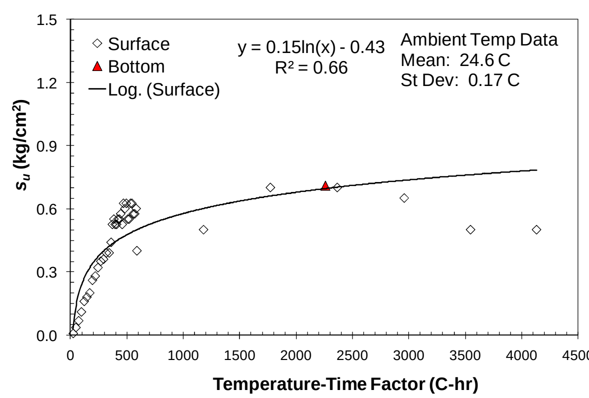
c) *SC1 (5, 100) Soil 3 Repeat-Shear*



d) *SC1 (5, 100) Soil 3 Repeat-Dial*

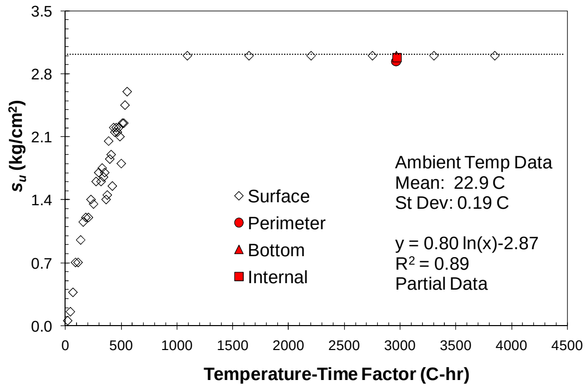


e) *SC1 (5, 100) Soil 3 Repeat-Ring*

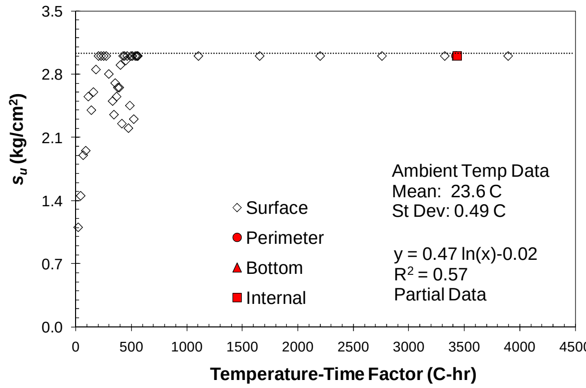


f) *SC1 (5, 100) Soil 3 Repeat-Shear*

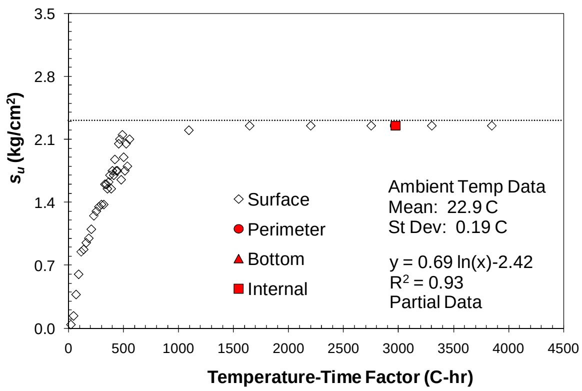
Figure A.20. SC1 (5, 100) Soil 3 Repeat and SC1 (5, 100) Soil 3 Repeat



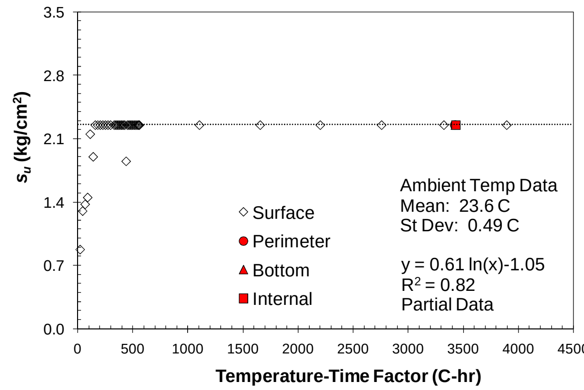
a) *SC1 (10, 100) Soil 1-Dial*



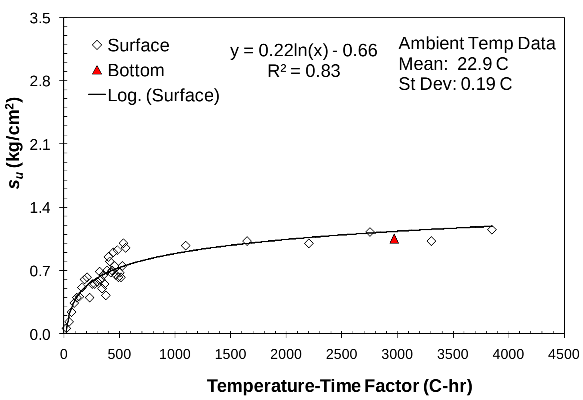
d) *SC1 (10, 100) Soil 2-Dial*



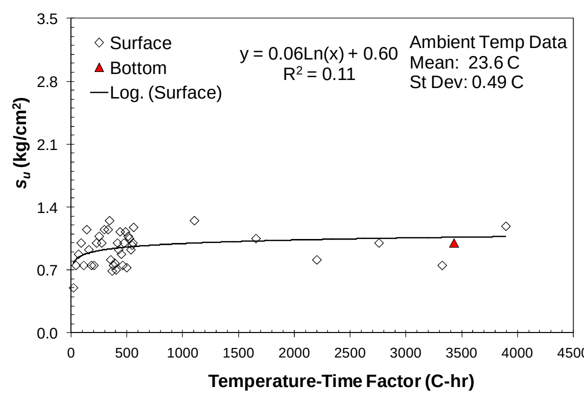
b) *SC1 (10, 100) Soil 1-Ring*



e) *SC1 (10, 100) Soil 2-Ring*

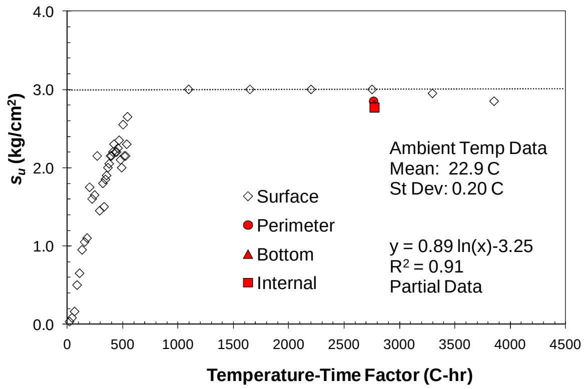


c) *SC1 (10, 100) Soil 1-Shear*

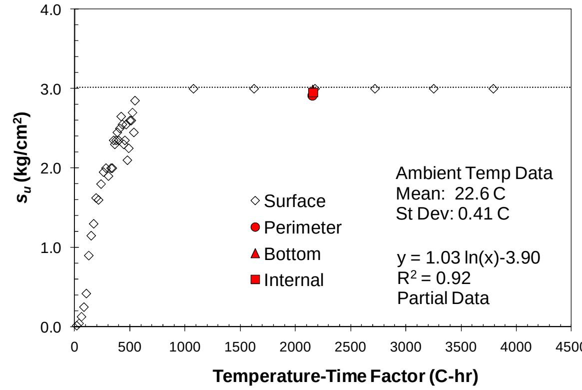


f) *SC1 (10, 100) Soil 2-Shear*

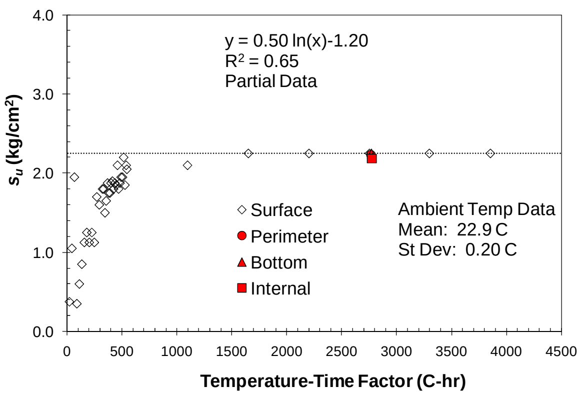
Figure A.21. SC1 (10, 100) Soil 1 and SC1 (10, 100) Soil 2



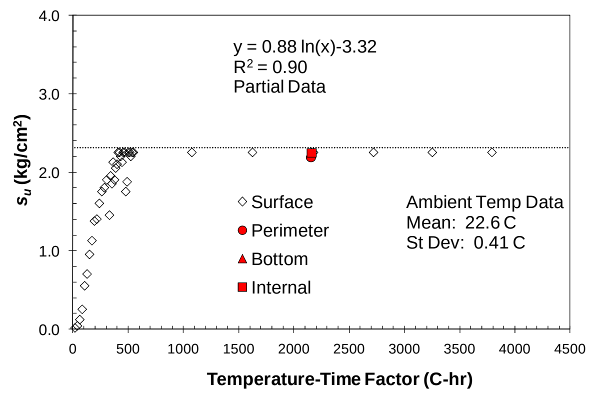
a) *SC1 (10, 100) Soil 3-Dial*



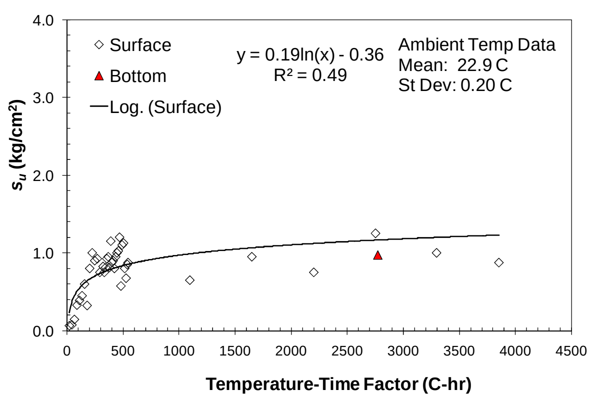
d) *SC1 (10, 100) Soil 3 Repeat-Dial*



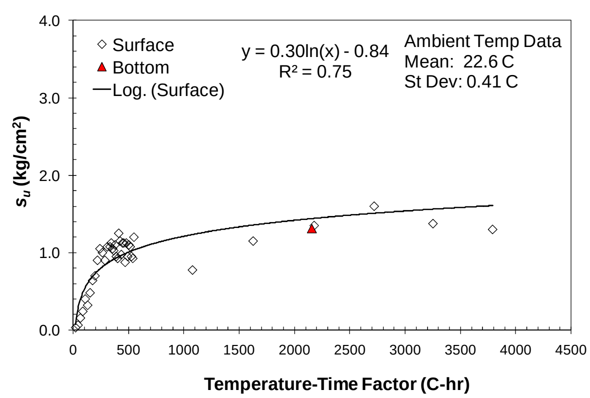
b) *SC1 (10, 100) Soil 3-Ring*



e) *SC1 (10, 100) Soil 3 Repeat-Ring*

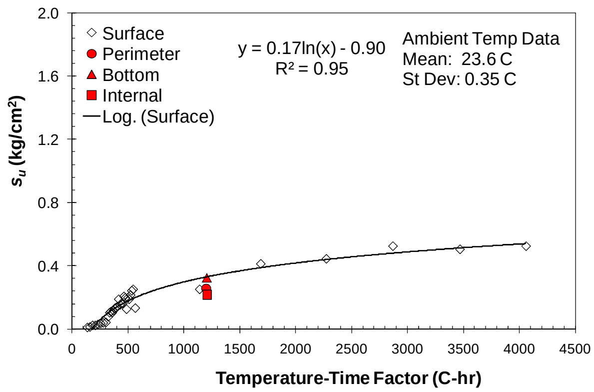


c) *SC1 (10, 100) Soil 3-Shear*

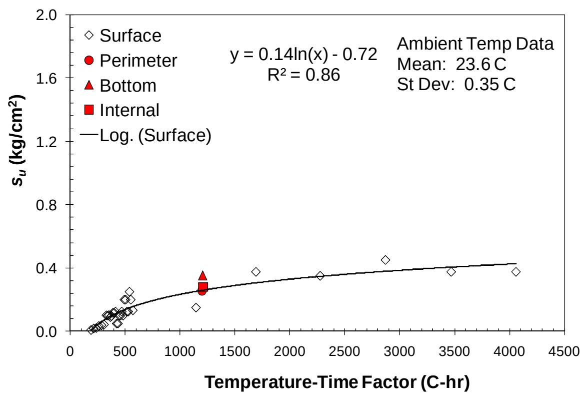


f) *SC1 (10, 100) Soil 3 Repeat-Shear*

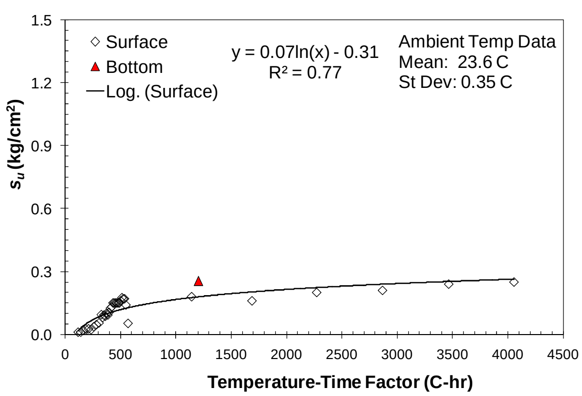
Figure A.22. SC1 (10, 100) Soil 3 and SC1 (10, 100) Soil 3 Repeat



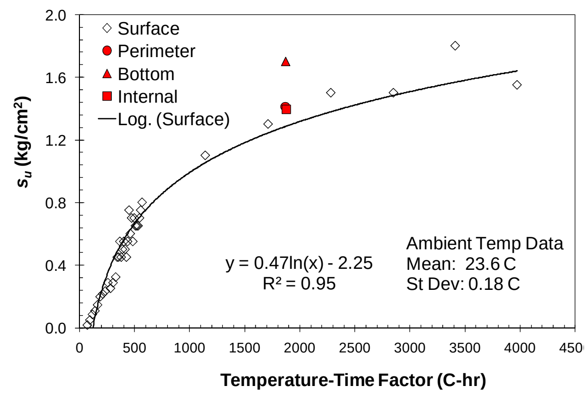
a) *SC1 (15, 233) Soil 1-Dial*



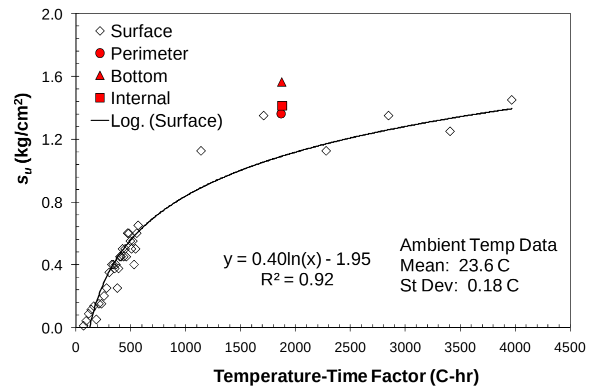
b) *SC1 (15, 233) Soil 1-Ring*



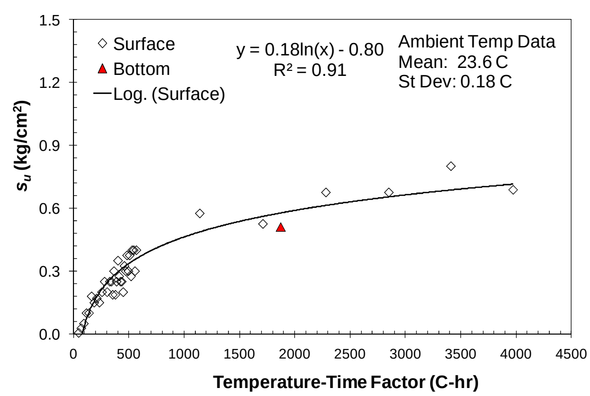
c) *SC1 (15, 233) Soil 1-Shear*



d) *SC1 (15, 233) Soil 2-Dial*

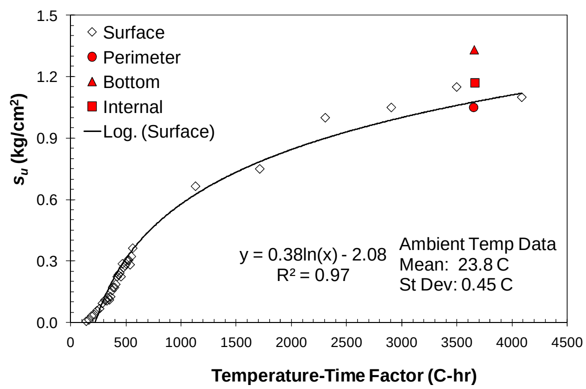


e) *SC1 (15, 233) Soil 2-Ring*

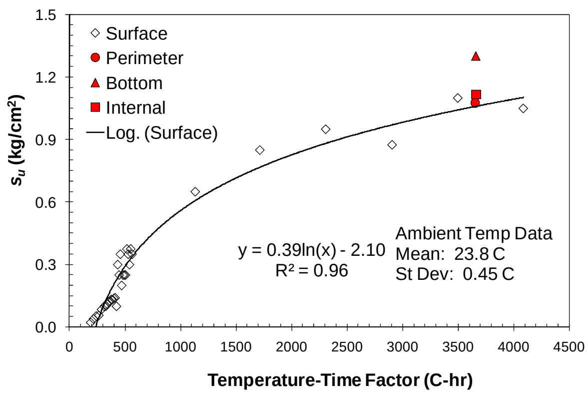


f) *SC1 (15, 233) Soil 2-Shear*

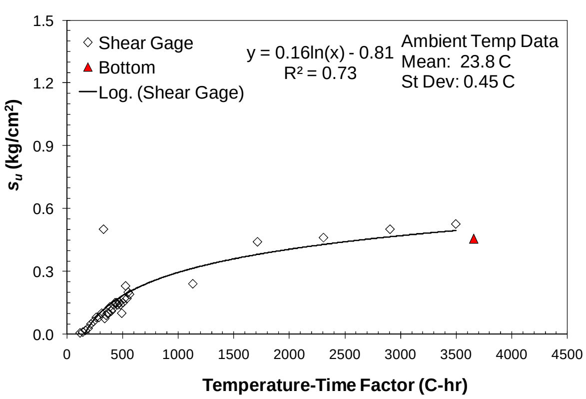
Figure A.23. SC1 (15, 233) Soil 1 and SC1 (15, 233) Soil 2



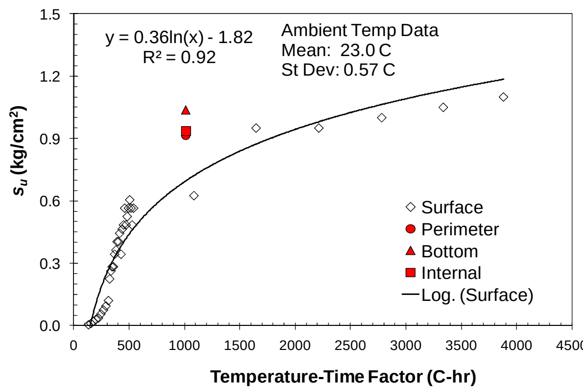
a) *SC1 (15, 233) Soil 3-Dial*



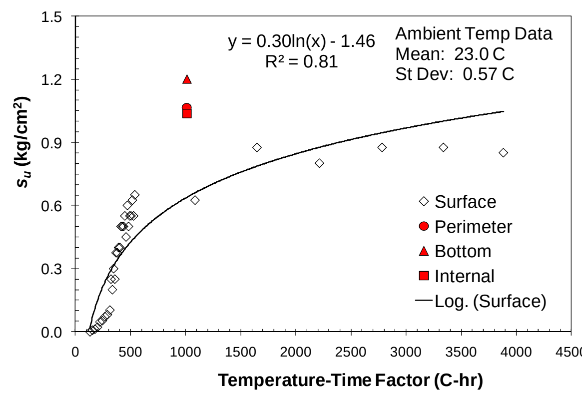
b) *SC1 (15, 233) Soil 3-Ring*



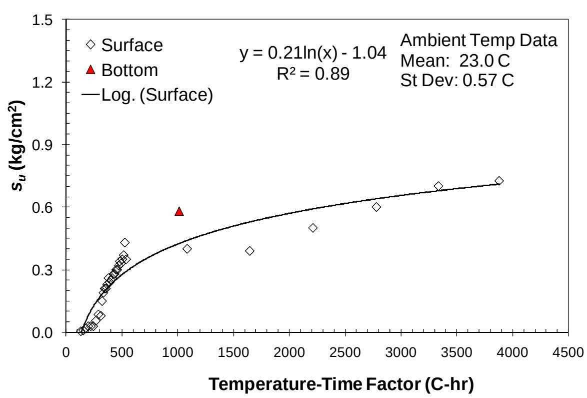
c) *SC1 (15, 233) Soil 3-Shear*



d) *SC1 (15, 233) Soil 3 Repeat-Dial*

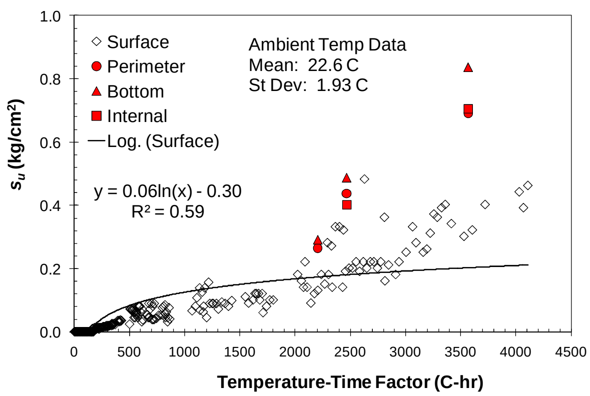


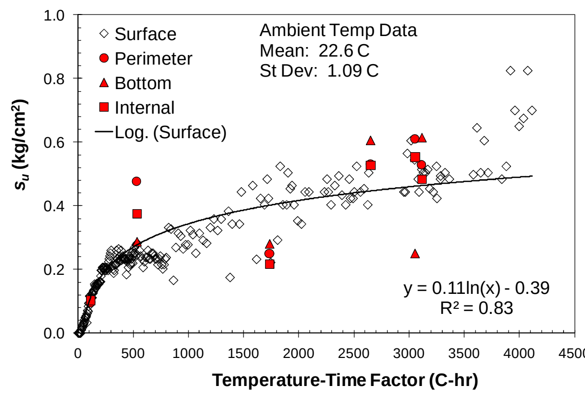
e) *SC1 (15, 233) Soil 3 Repeat-Ring*



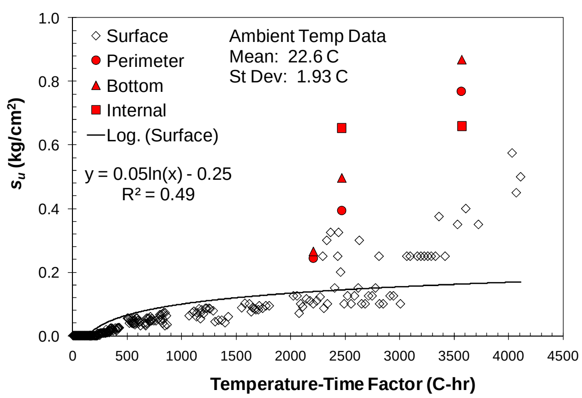
f) *SC1 (15, 233) Soil 3 Repeat-Shear*

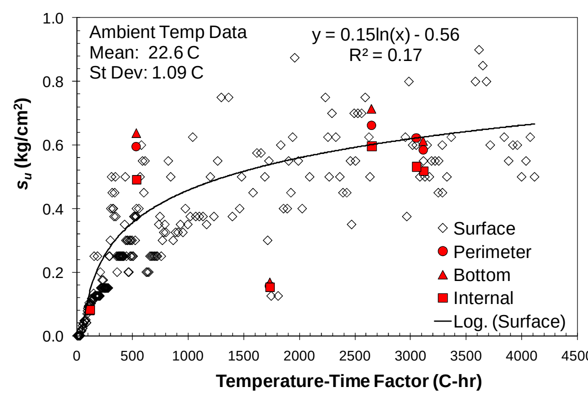
Figure A.24. SC1 (15, 233) Soil 3 and SC1 (15, 233) Soil 3 Repeat



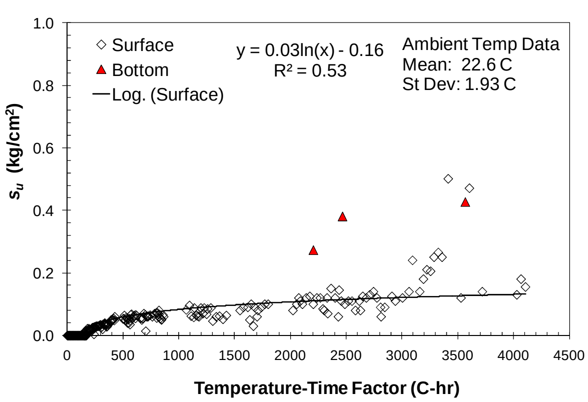


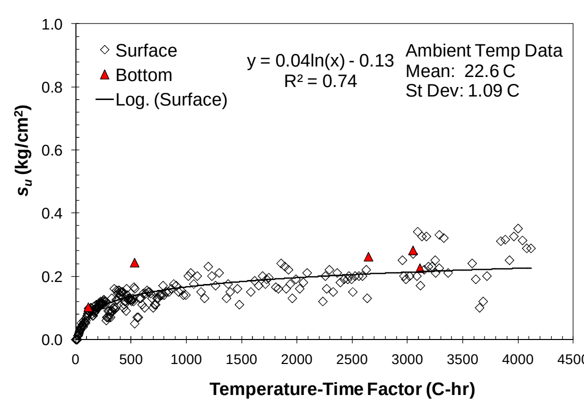
a) *A T I (GGBFS) (1.25, 3.75, 100) Soil 1-Dial* d) *SC2 (PoP) (5, 0.28, 100) Soil 1-Dial*





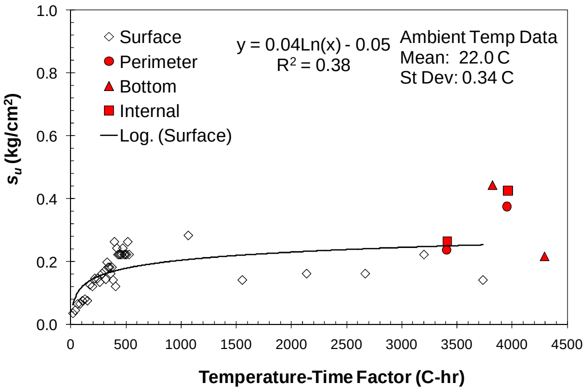
b) *A T I (GGBFS) (1.25, 3.75, 100) Soil 1-Ring* e) *SC2 (PoP) (5, 0.28, 100) Soil 1-Ring*

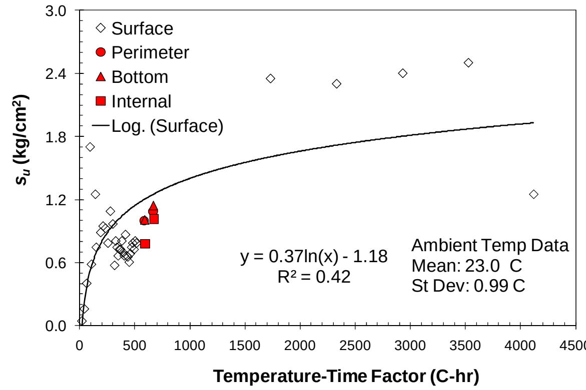




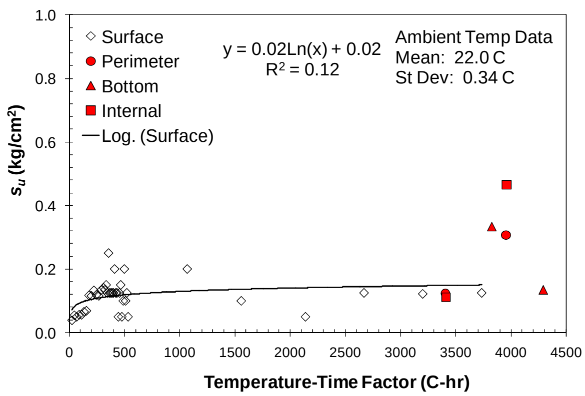
c) *A T I (GGBFS) (1.25, 3.75, 100) Soil 1-Shear* f) *SC2 (PoP) (5, 0.28, 100) Soil 1-Shear*

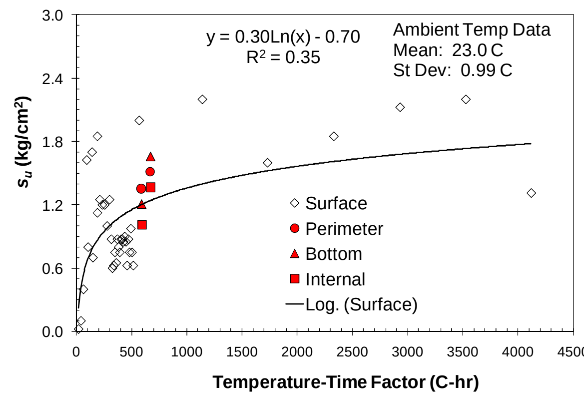
Figure A.25. A T I (GGBFS) (1.25, 3.75, 100) Soil 1 and SC2 (PoP) (5, 0.28, 100) Soil 1



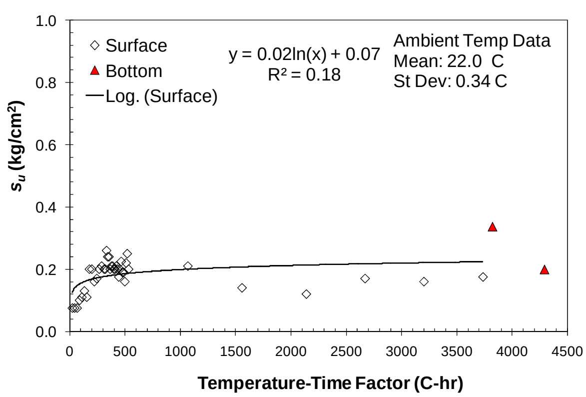


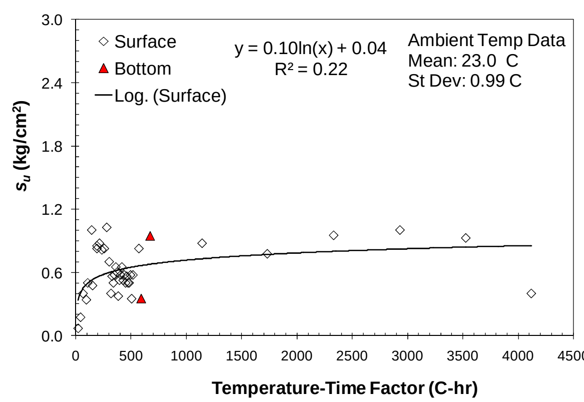
a) *A T I (GGBFS) (1.25, 3.75, 100) Soil 2-Dial* d) *SC2 (PoP) (5, 0.28, 100) Soil 2-Dial*





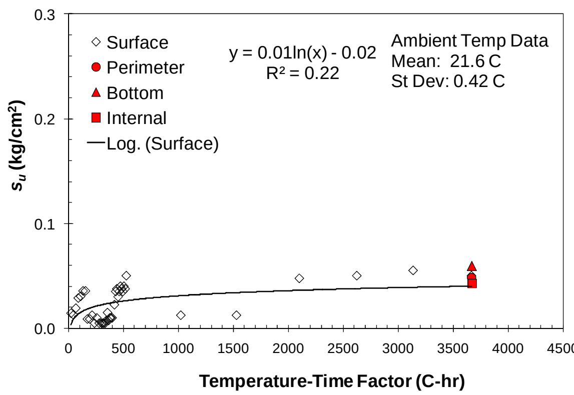
b) *A T I (GGBFS) (1.25, 3.75, 100) Soil 2-Ring* e) *SC2 (PoP) (5, 0.28, 100) Soil 2-Ring*

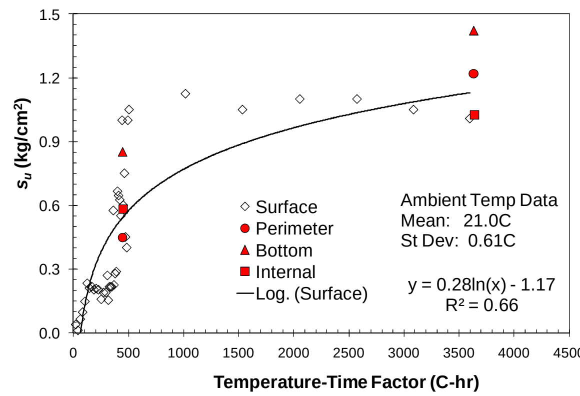




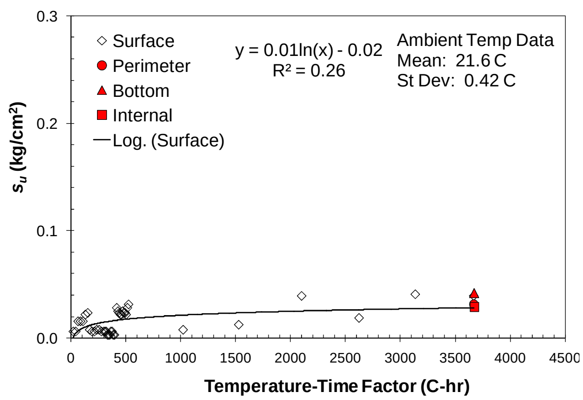
c) *A T I (GGBFS) (1.25, 3.75, 100) Soil 2-Shear* f) *SC2 (PoP) (5, 0.28, 100) Soil 2-Shear*

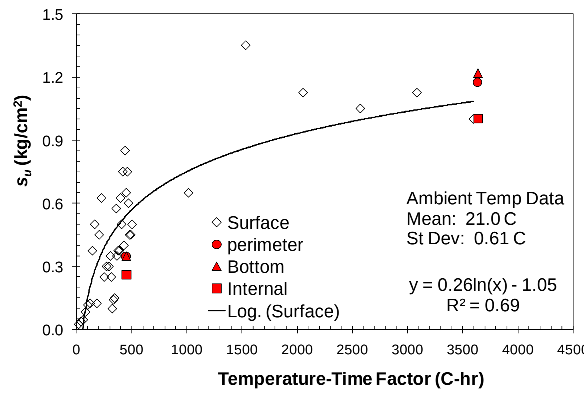
Figure A.26. A T I (GGBFS) (1.25, 3.75, 100) Soil 2 and SC2 (PoP) (5, 0.28, 100) Soil 2



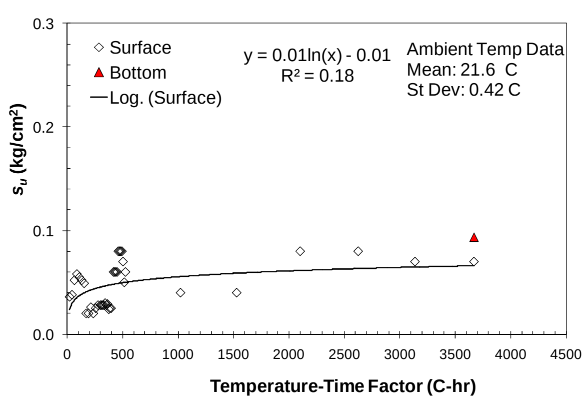


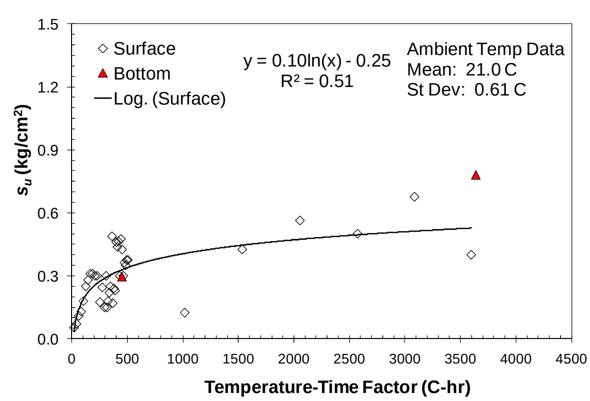
a) *A T I (GGBFS) (1.25, 3.75, 100) Soil 3-Dial* d) *SC2 (PoP) (5, 0.28, 100) Soil 3-Dial*





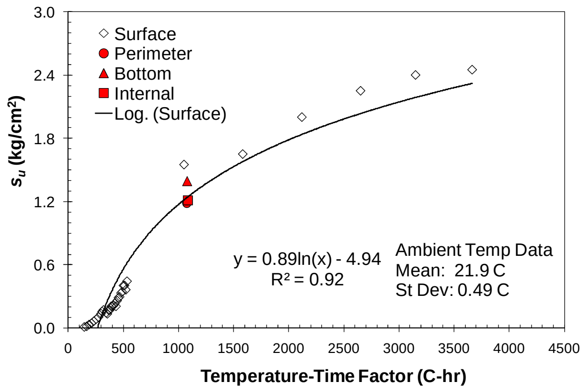
b) *A T I (GGBFS) (1.25, 3.75, 100) Soil 3-Ring* e) *SC2 (PoP) (5, 0.28, 100) Soil 3-Ring*



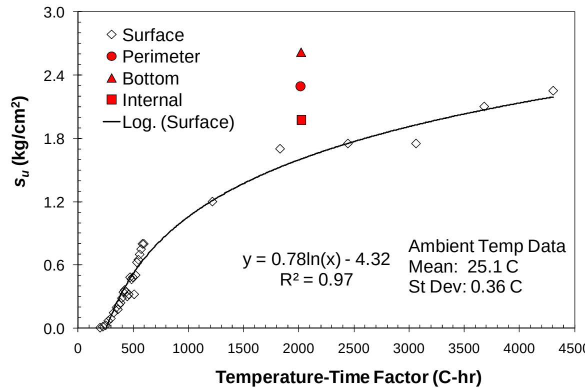


c) *A T I (GGBFS) (1.25, 3.75, 100) Soil 3-Shear* f) *SC2 (PoP) (5, 0.28, 100) Soil 3-Shear*

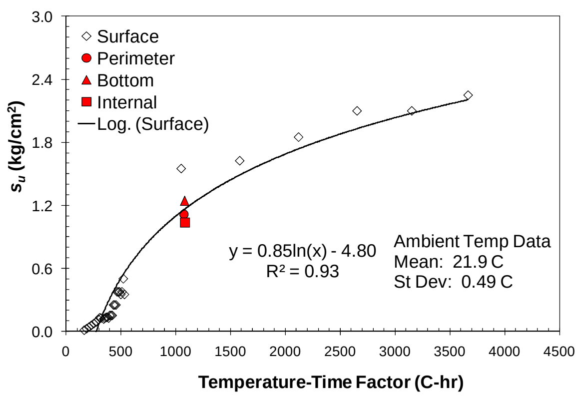
Figure A.27. A T I (GGBFS) (1.25, 3.75, 100) Soil 3 and SC2 (PoP) (5, 0.28, 100) Soil 3



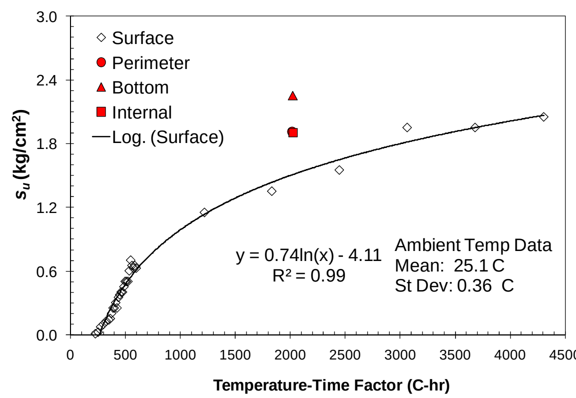
a) *Th T III Brackish (15, 233) Soil 3-Dial*



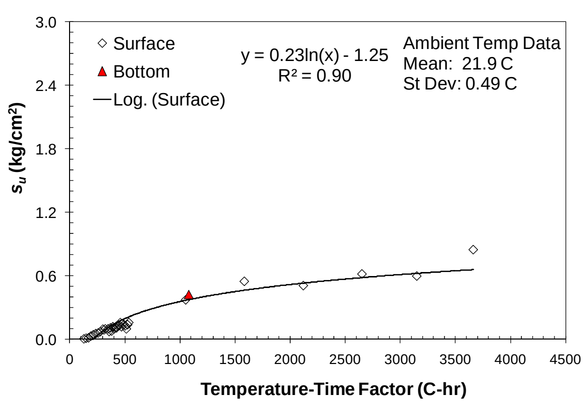
d) *Th T III Salt (15, 233) Soil 3-Dial*



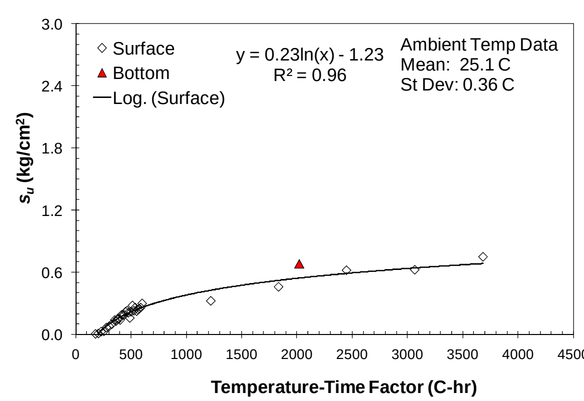
b) *Th T III Brackish (15, 233) Soil 3-Ring*



e) *Th T III Salt (15, 233) Soil 3-Ring*

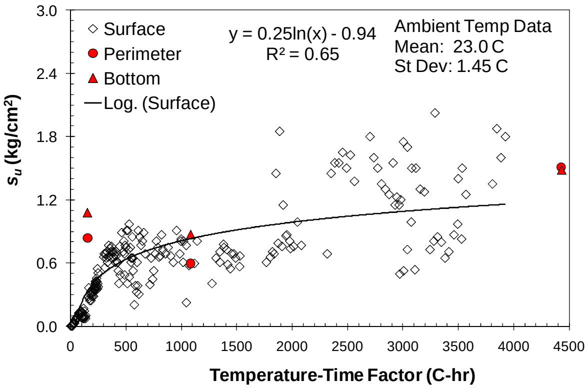


c) *Th T III Brackish (15, 233) Soil 3-Shear*

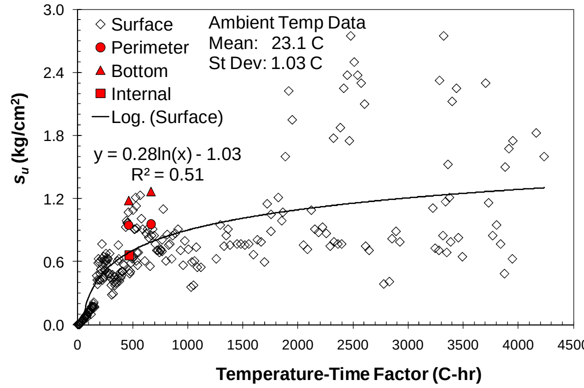


f) *Th T III Salt (15, 233) Soil 3-Shear*

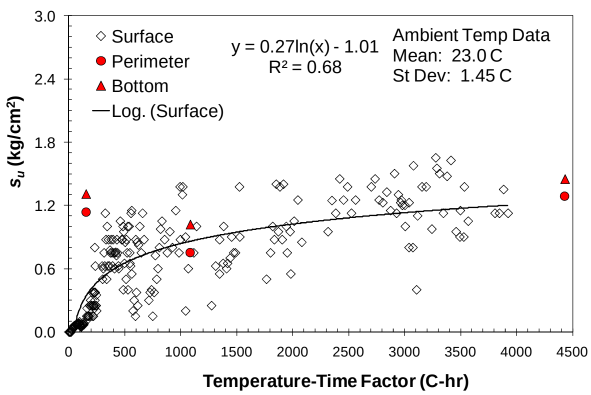
Figure A.28. Th T III Brackish (15, 233) Soil 3 and Th T III Salt (15, 233) Soil 3



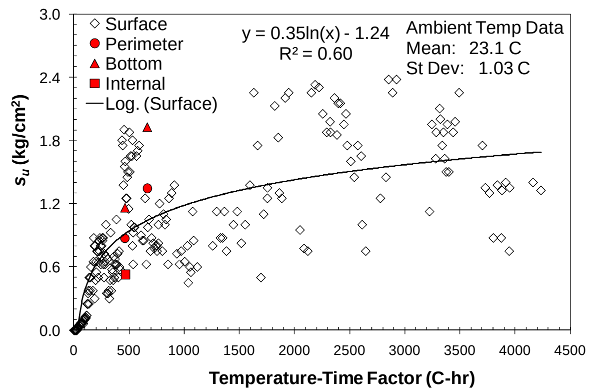
a) *SC1 (F70) (5, 0.5, 100) Soil 1-Dial*



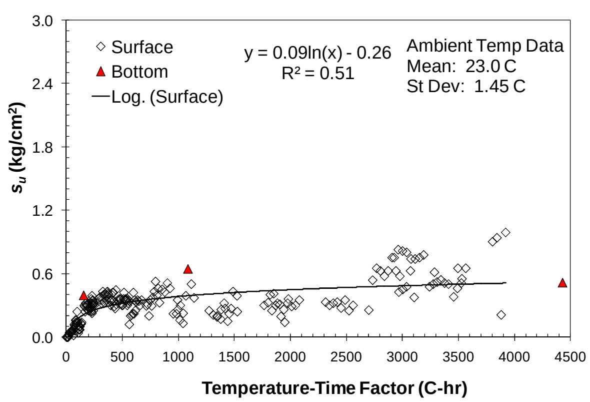
d) *SC1 (F20) (5, 0.5, 100) Soil 1-Dial*



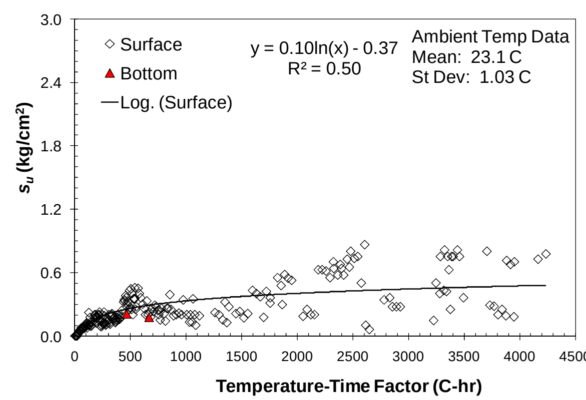
b) *SC1 (F70) (5, 0.5, 100) Soil 1-Ring*



e) *SC1 (F20) (5, 0.5, 100) Soil 1-Ring*

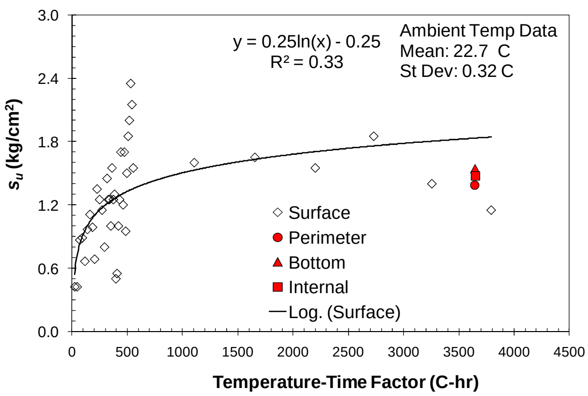


c) *SC1 (F70) (5, 0.5, 100) Soil 1-Shear*

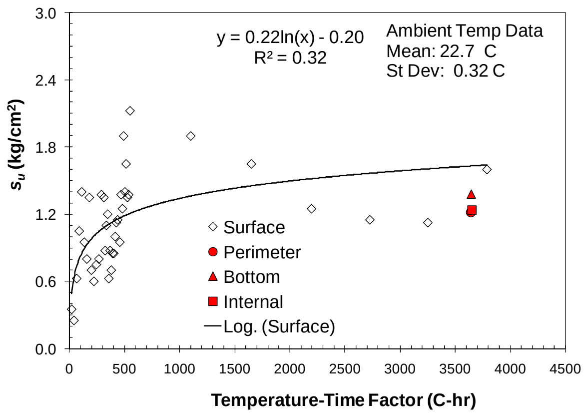


f) *SC1 (F20) (5, 0.5, 100) Soil 1-Shear*

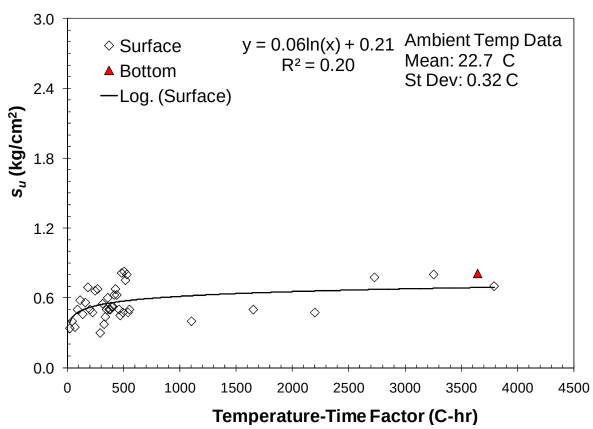
Figure A.29. SC1 (F70) (5, 0.5, 100) Soil 1and SC1 (F20) (5, 0.5, 100) Soil 1



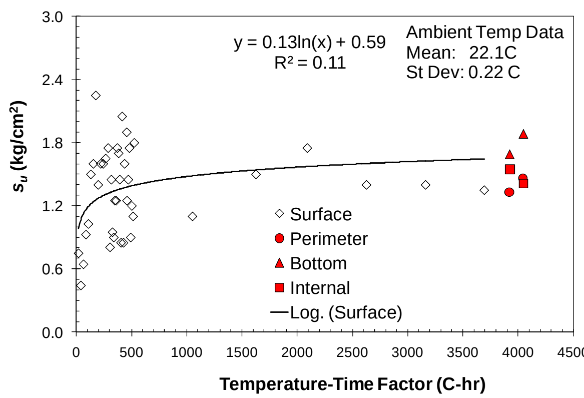
a) *SC1 (F70) (5, 0.5, 100) Soil 2-Dial*



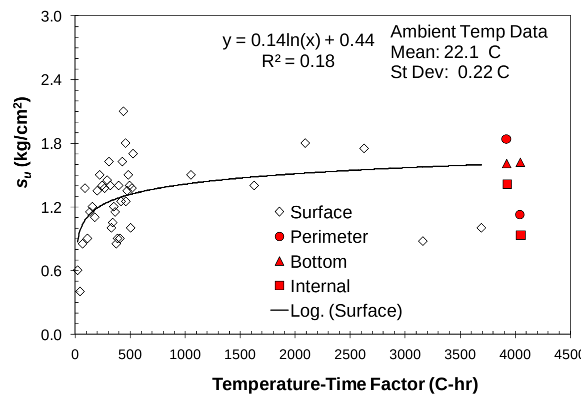
b) *SC1 (F70) (5, 0.5, 100) Soil 2-Ring*



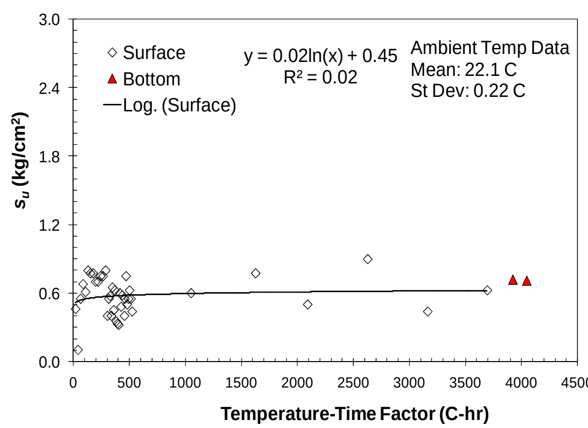
c) *SC1 (F70) (5, 0.5, 100) Soil 2-Shear*



d) *SC1 (F20) (5, 0.5, 100) Soil 2-Dial*

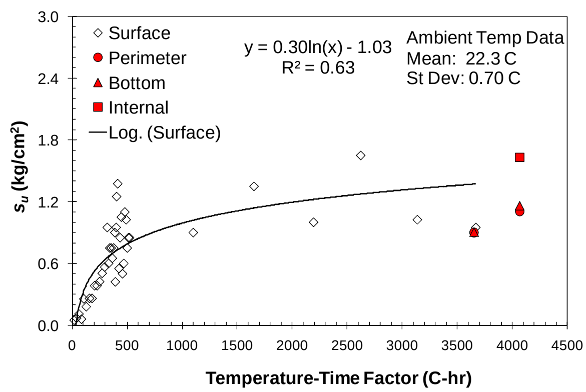


e) *SC1 (F20) (5, 0.5, 100) Soil 2-Ring*

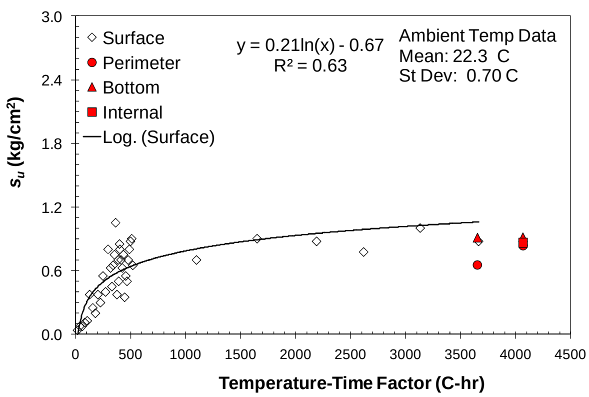


f) *SC1 (F20) (5, 0.5, 100) Soil 2-Shear*

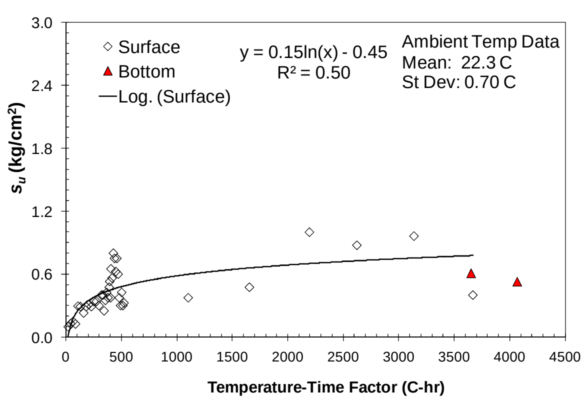
Figure A.30. SC1 (F70) (5, 0.5, 100) Soil 2 and SC1 (F20) (5, 0.5, 100) Soil 2



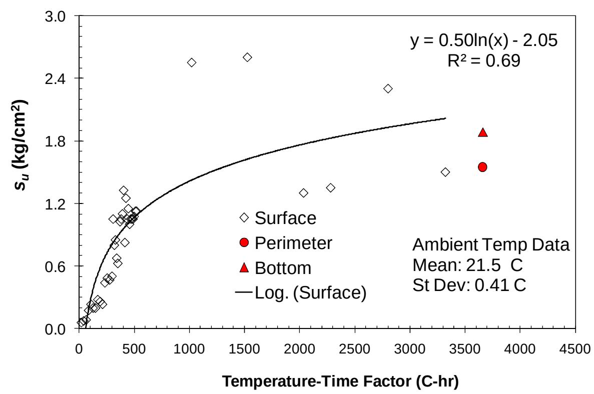
a) *SC1 (F70) (5, 0.5, 100) Soil 3-Dial*



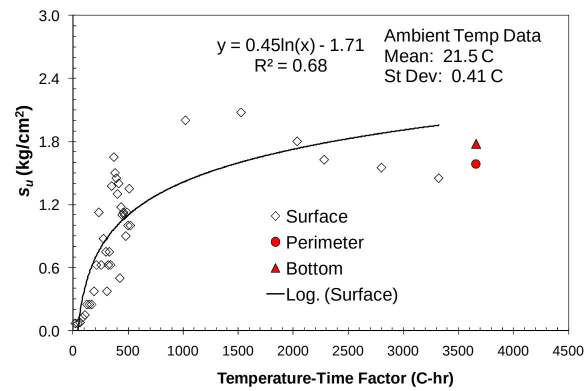
b) *SC1 (F70) (5, 0.5, 100) Soil 3-Ring*



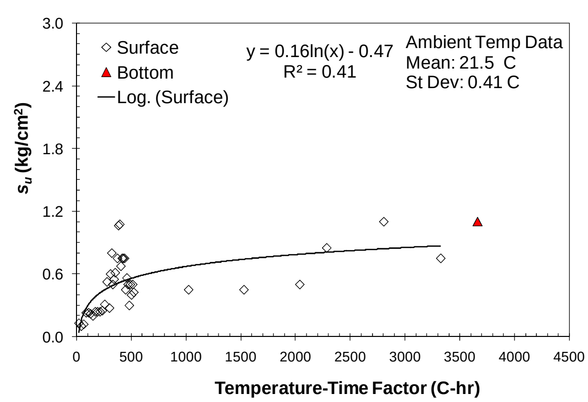
c) *SC1 (F70) (5, 0.5, 100) Soil 3-Shear*



d) *SC1 (F20) (5, 0.5, 100) Soil 3-Dial*



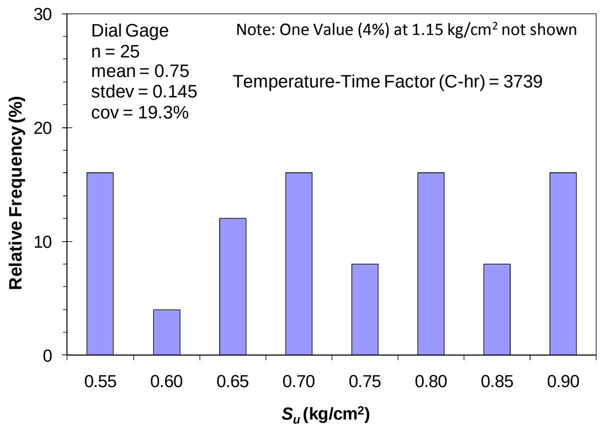
e) *SC1 (F20) (5, 0.5, 100) Soil 3-Ring*



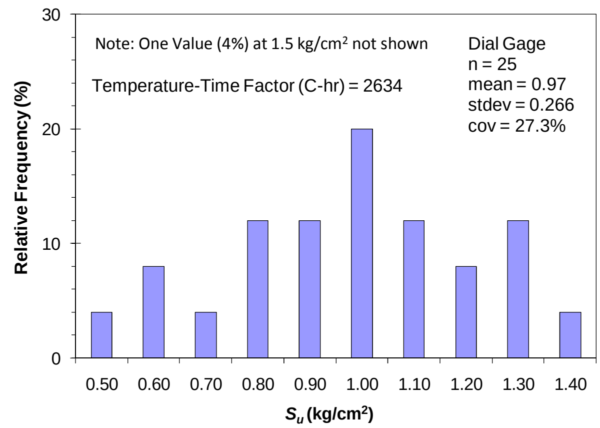
f) *SC1 (F20) (5, 0.5, 100) Soil 3-Shear*

Figure A.31. SC1 (F70) (5, 0.5, 100) Soil 3 and SC1 (F20) (5, 0.5, 100) Soil 3

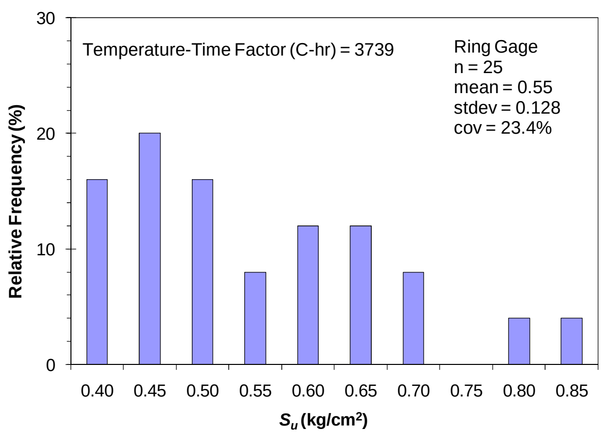
APPENDIX B – VARIABILITY SLAB TEST RESULTS



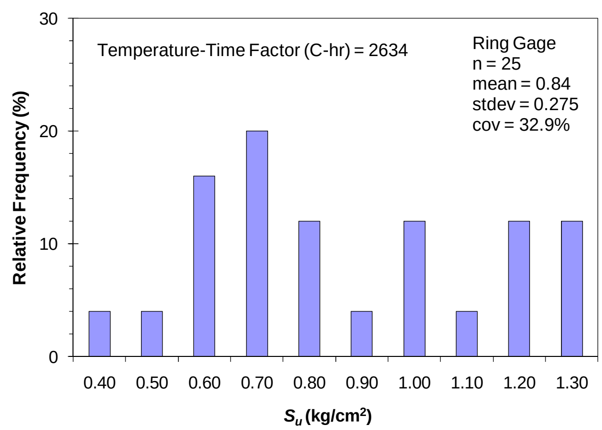
(a) Th T III (5, 100) Soil 2 168 hr-Dial



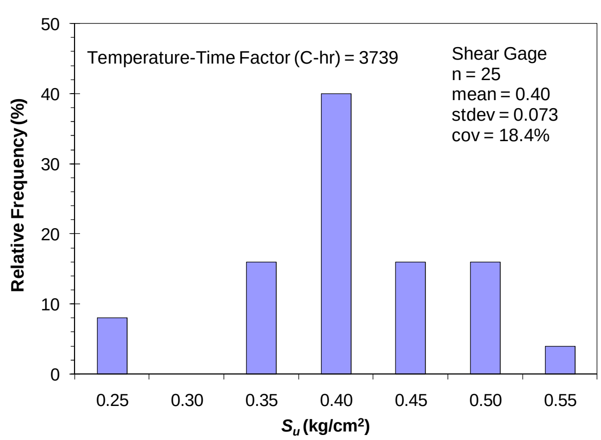
(d) Th T III (5, 100) Soil 3 120 hr-Dial



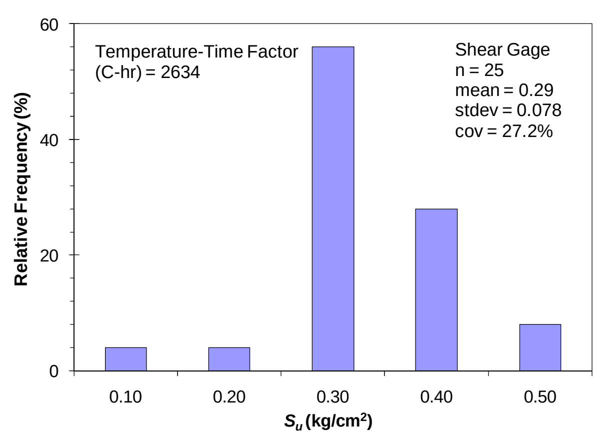
(b) Th T III (5, 100) Soil 2 168 hr-Ring



(e) Th T III (5, 100) Soil 3 120 hr-Ring

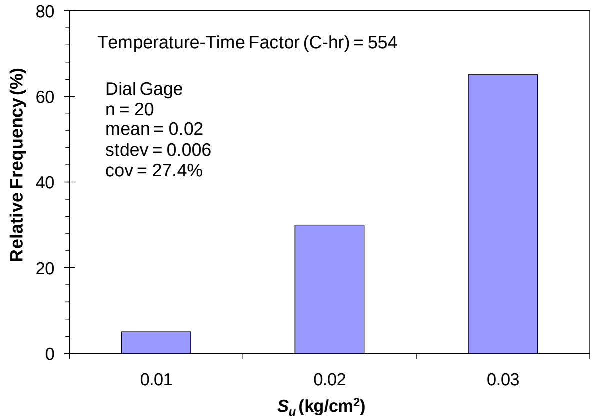


(c) Th T III (5, 100) Soil 2 168 hr-Shear

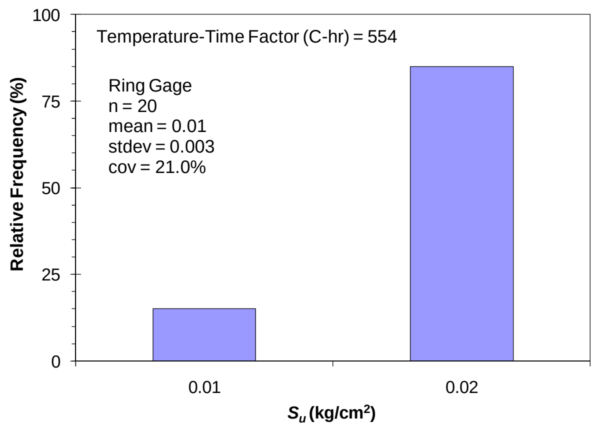


(f) Th T III (5, 100) Soil 3 120 hr-Shear

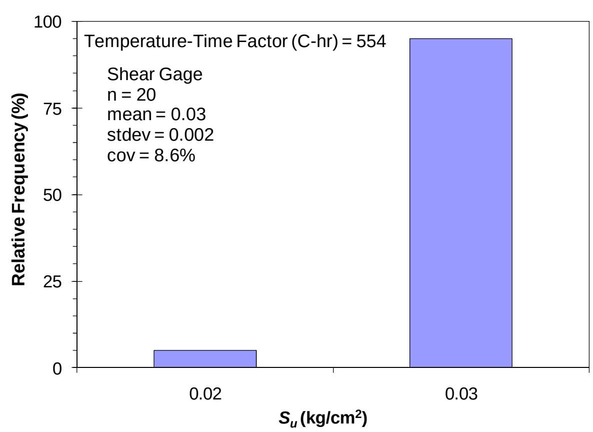
Figure B.1. Th T III (5, 100) Soil 2 168 hr and Th T III (5, 100) Soil 3 120 hr



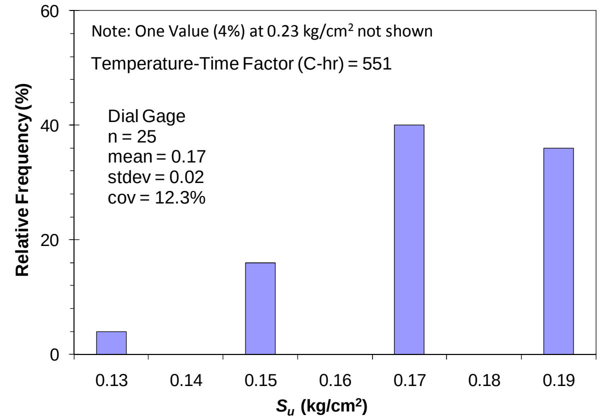
(a) Th T III (5, 233) Soil 1 24 hr-Dial



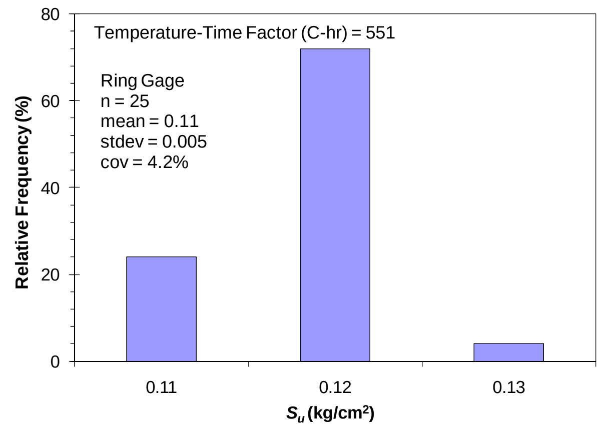
(b) Th T III (5, 233) Soil 1 24 hr-Ring



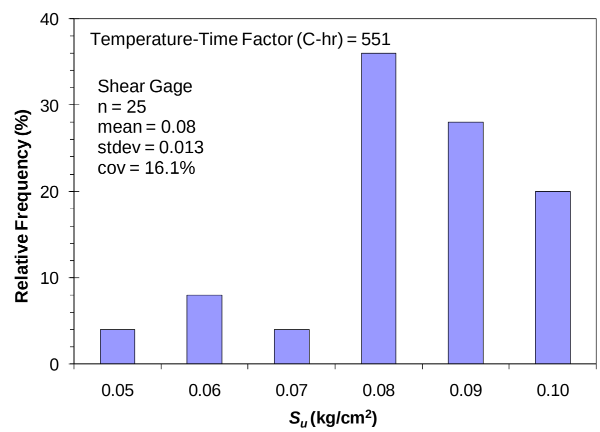
(c) Th T III (5, 233) Soil 1 24 hr-Shear



(d) Th T III (10, 233) Soil 3 24 hr-Dial

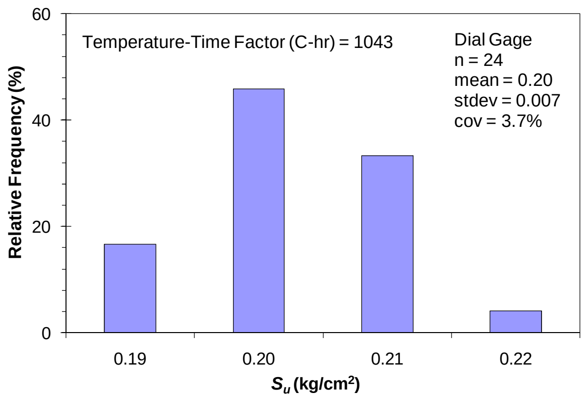


(e) Th T III (10, 233) Soil 3 24 hr-Ring

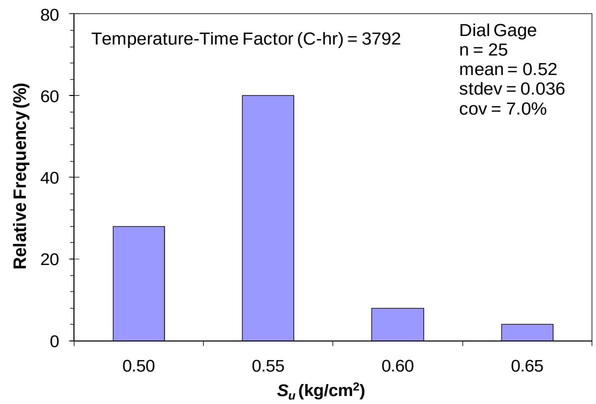


(f) Th T III (10, 233) Soil 3 24 hr-Shear

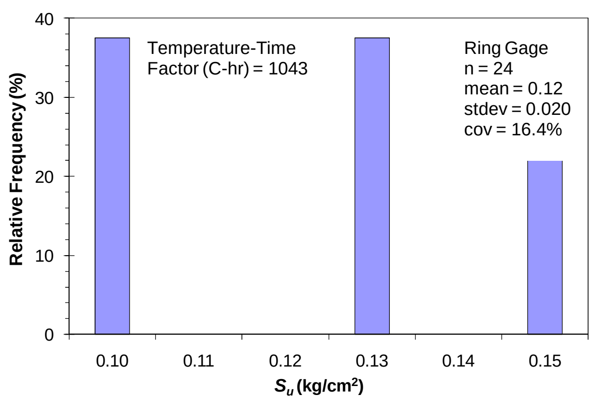
Figure B.2. Th T III (5, 233) Soil 1 24 hr and Th T III (10, 233) Soil 3 24 hr



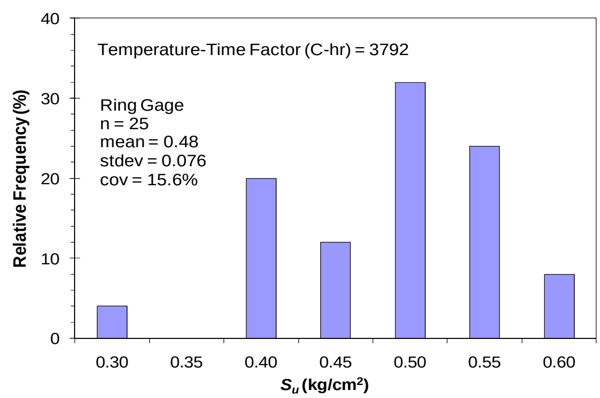
(a) CTS RS (5, 100) Soil 1 48 hr-Dial



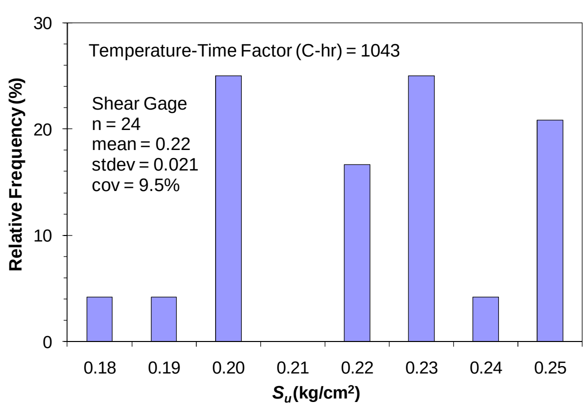
(d) CTS RS (5, 100) Soil 2 168 hr-Dial



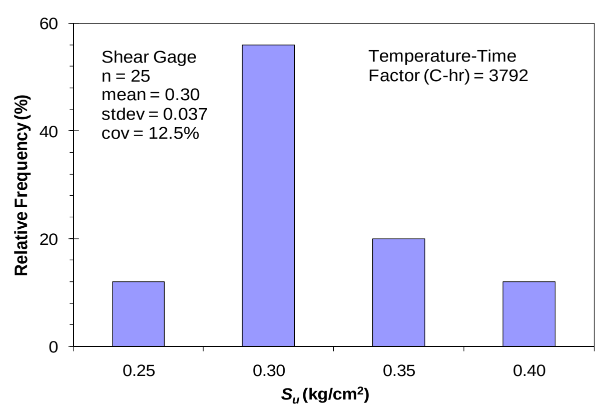
(b) CTS RS (5, 100) Soil 1 48 hr-Ring



(e) CTS RS (5, 100) Soil 2 168 hr-Ring

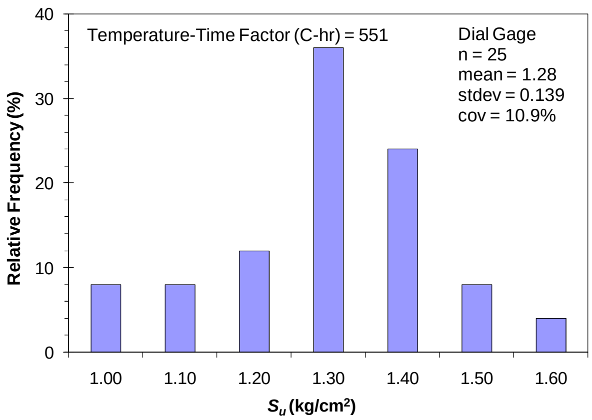


(c) CTS RS (5, 100) Soil 1 48 hr-Shear

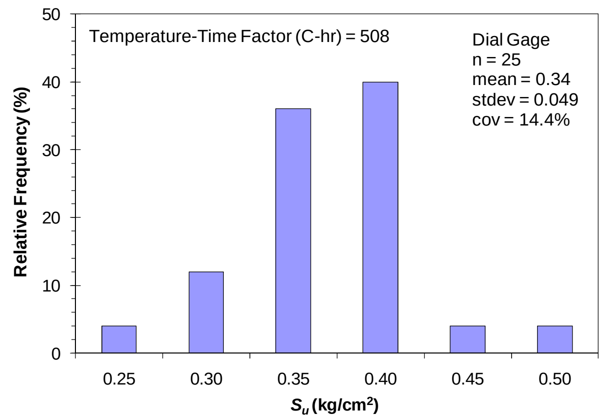


(f) CTS RS (5, 100) Soil 2 168 hr-Shear

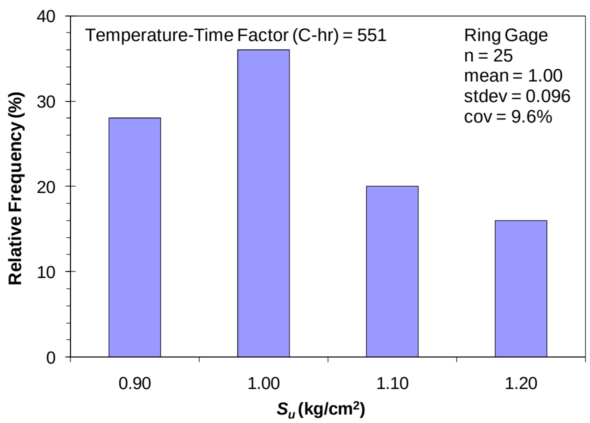
Figure B.3. CTS RS (5, 100) Soil 1 48 hr and CTS RS (5, 100) Soil 2 168 hr



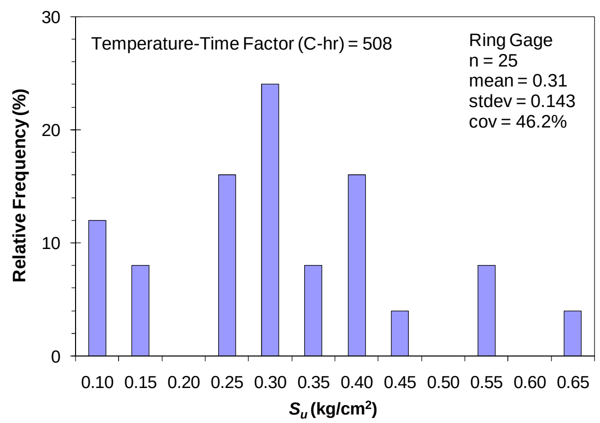
(a) A T III (5, 100) Soil 2 24 hr-Dial



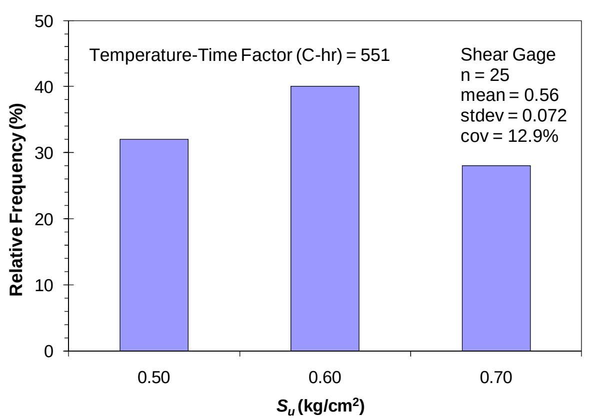
(d) A T III (5, 100) Soil 3 24 hr-Dial



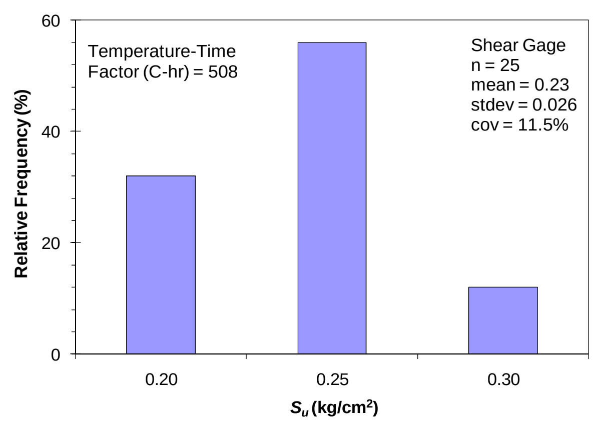
(b) A T III (5, 100) Soil 2 24 hr-Ring



(e) A T III (5, 100) Soil 3 24 hr-Ring

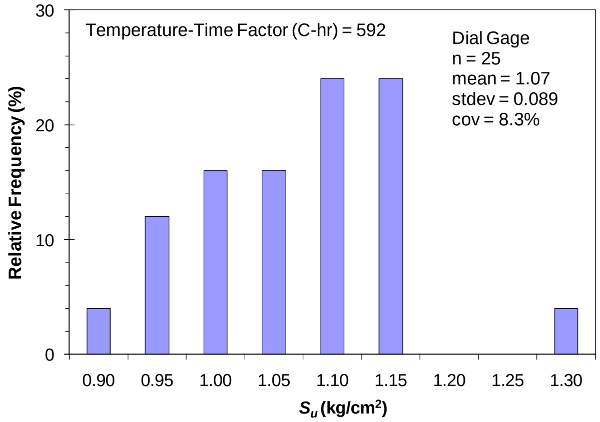


(c) A T III (5, 100) Soil 2 24 hr-Shear

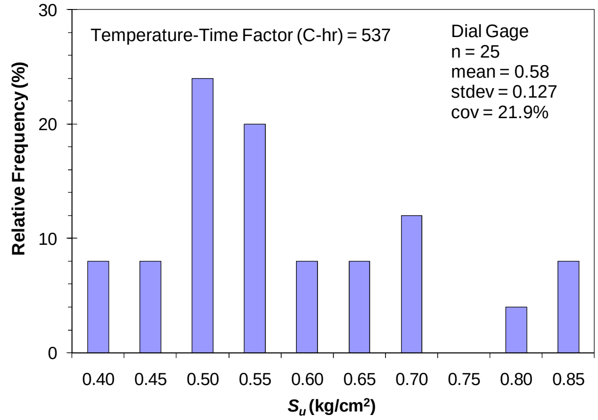


(f) A T III (5, 100) Soil 3 24 hr -Shear

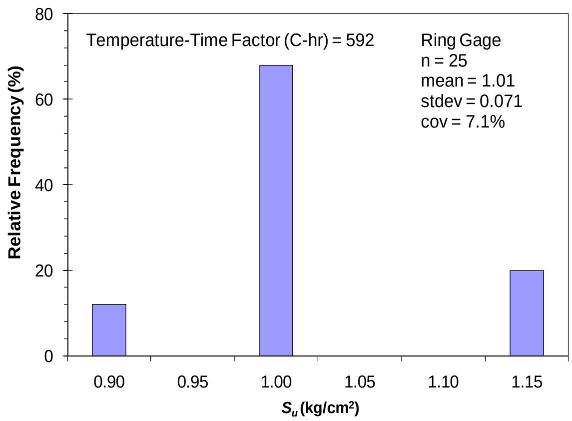
Figure B.4. A T III (5, 100) Soil 2 24 hr and A T III (5, 100) Soil 3 24 hr



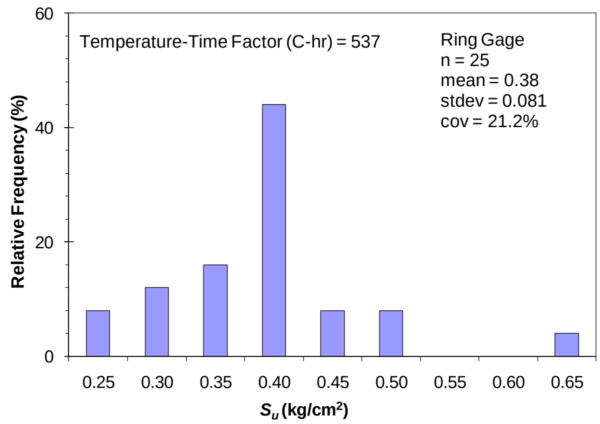
(a) A T I (5, 100) Soil 2 24 hr-Dial



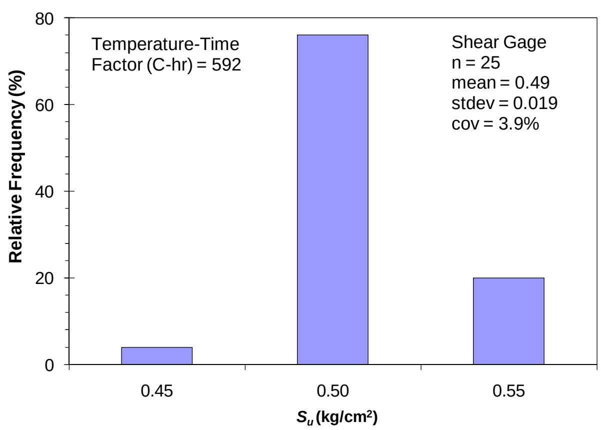
(d) A T I (5, 100) Soil 3 24 hr-Dial



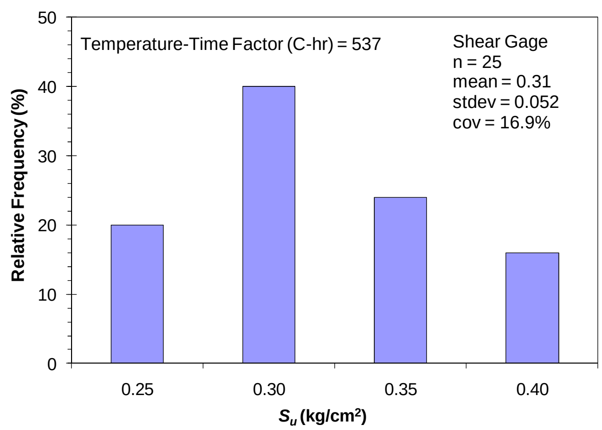
(b) A T I (5, 100) Soil 2 24 hr-Ring



(e) A T I (5, 100) Soil 3 24 hr-Ring

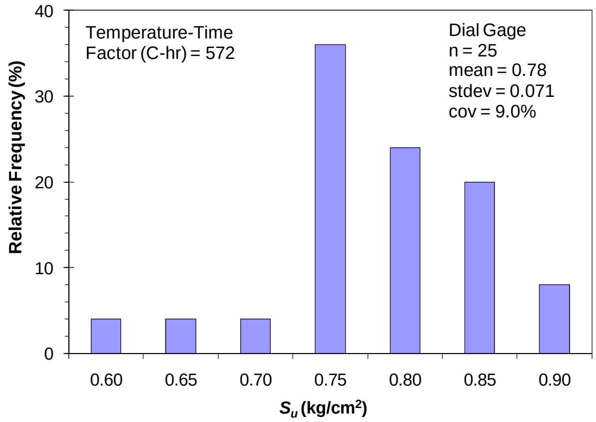


(c) A T I (5, 100) Soil 2 24 hr-Shear

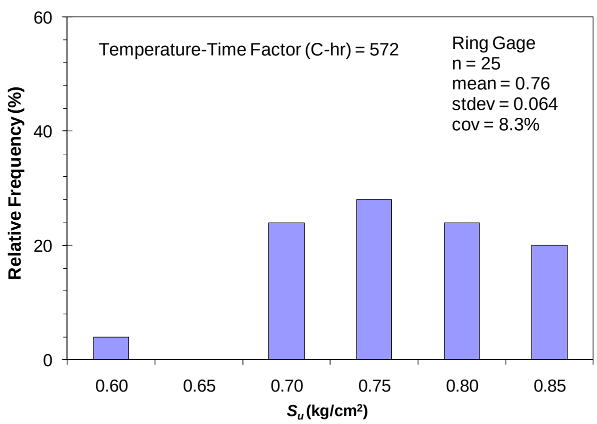


(f) A T I (5, 100) Soil 3 24 hr-Shear

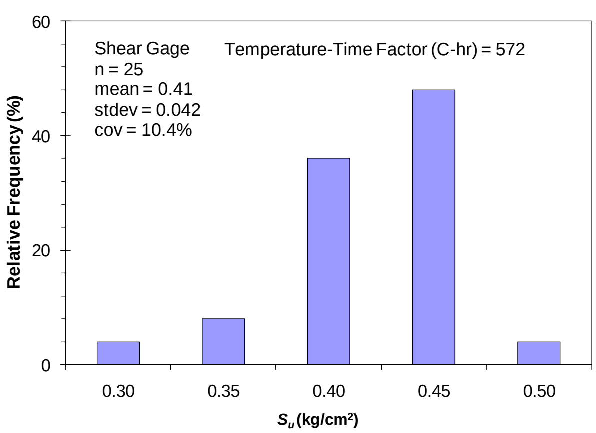
Figure B.5. A T I (5, 100) Soil 2 24 hr and A T I (5, 100) Soil 3 24 hr



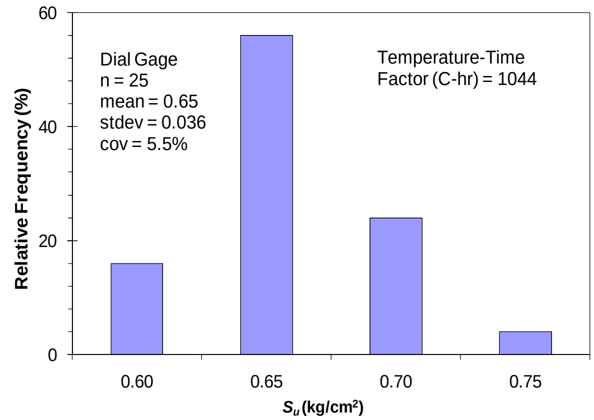
(a) Th T I/II (5, 100) Soil 1 24 hr-Dial



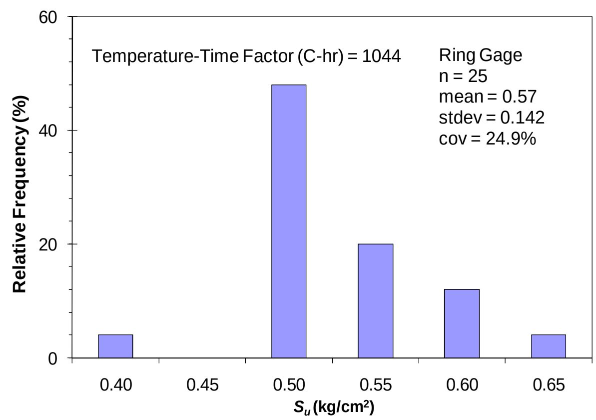
(b) Th T I/II (5, 100) Soil 1 24 hr-Ring



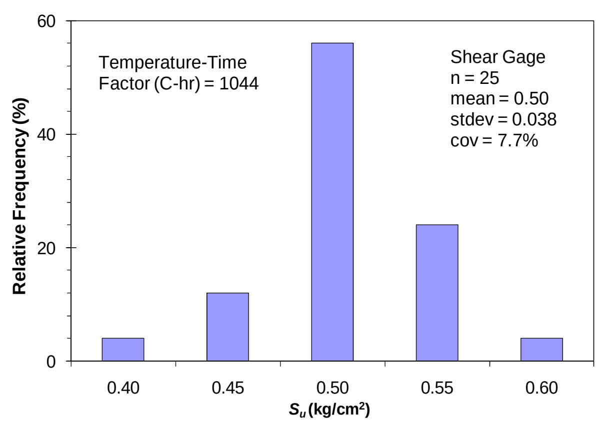
(c) Th T I/II (5, 100) Soil 1 24 hr-Shear



(d) Th T I/II (5, 100) Soil 1 48 hr-Dial

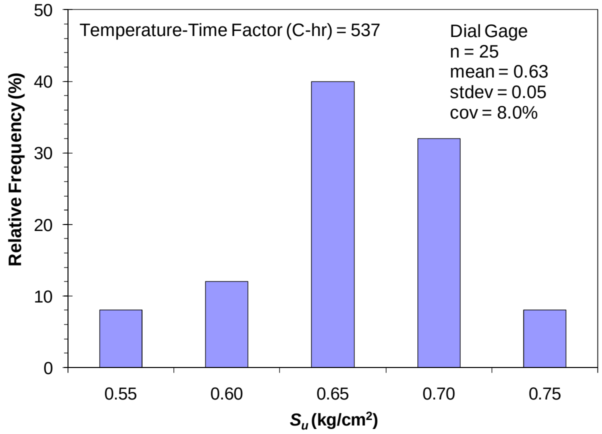


(e) Th T I/II (5, 100) Soil 1 48 hr -Ring

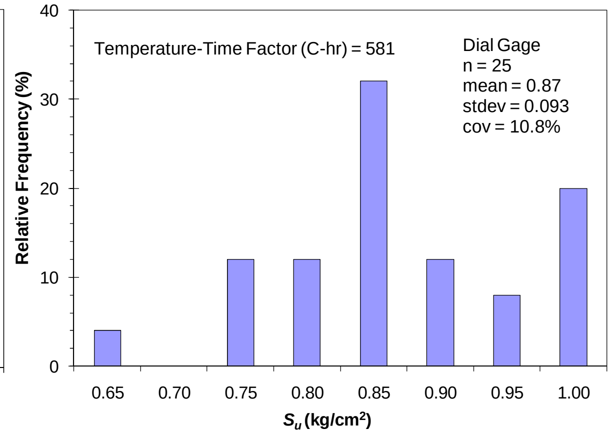


(f) Th T I/II (5, 100) Soil 1 48 hr-Shear

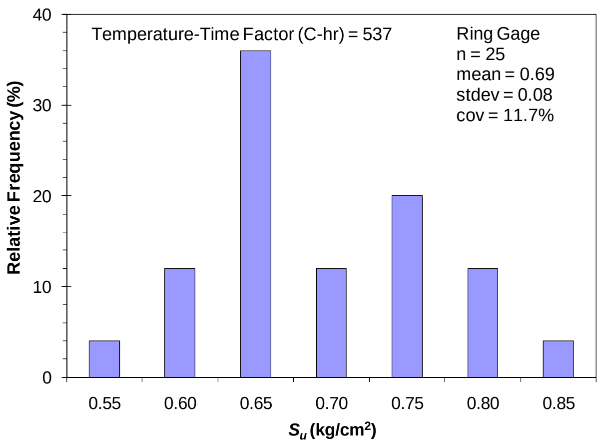
Figure B.6. Th T I/II (5, 100) Soil 1 24 hr and Th T I/II (5, 100) Soil 1 48 hr



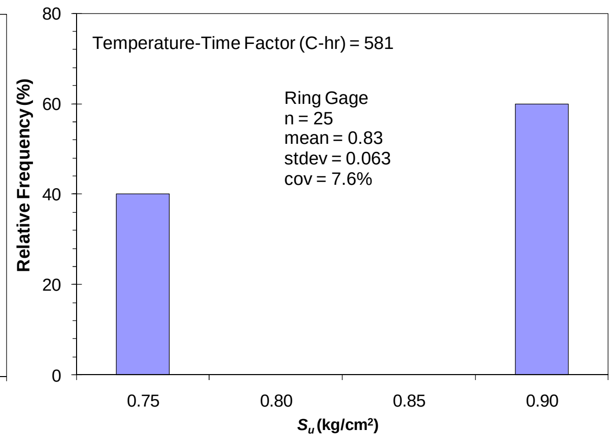
(a) *Th T I/II (5, 100) Soil 2 24 hr-Dial*



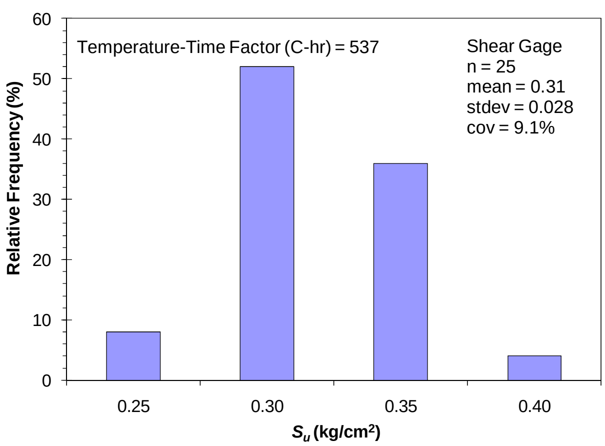
(d) *SC1 (5, 100) Soil 1 24 hr-Dial*



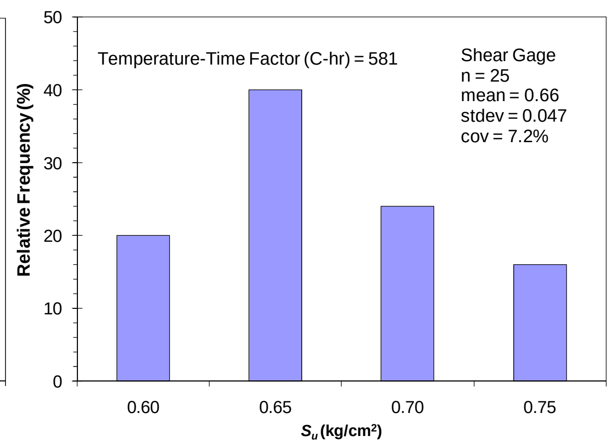
(b) *Th T I/II (5, 100) Soil 2 24 hr-Ring*



(e) *SC1 (5, 100) Soil 1 24 hr -Ring*

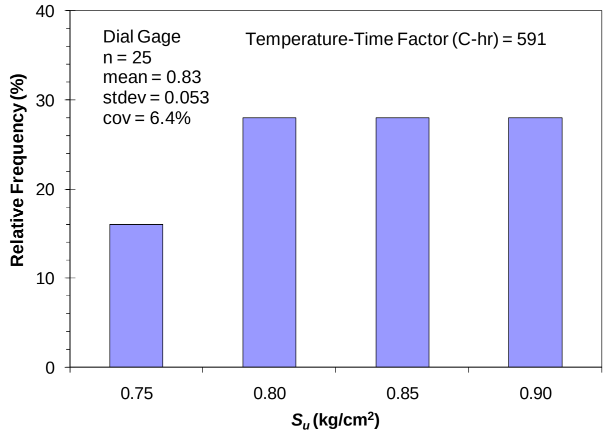


(c) *Th T I/II (5, 100) Soil 2 24 hr-Shear*

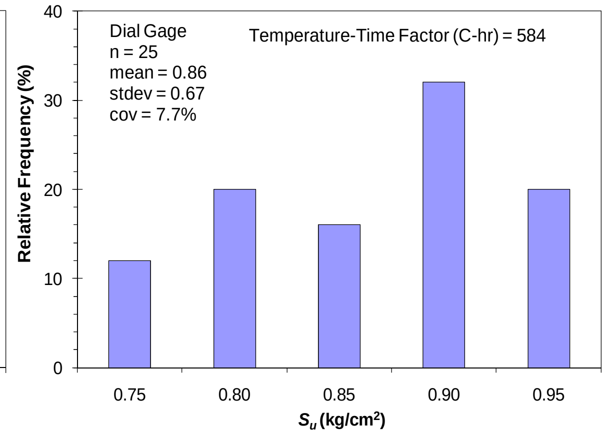


(f) *SC1 (5, 100) Soil 1 24 hr-Shear*

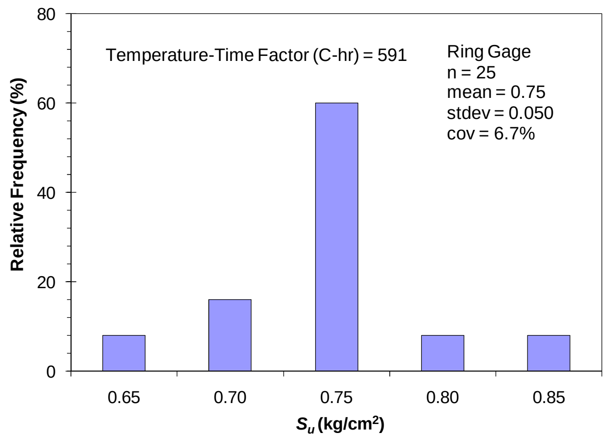
Figure B.7. Th T I/II (5, 100) Soil 2 24 hr and SC1 (5, 100) Soil 1 24 hr



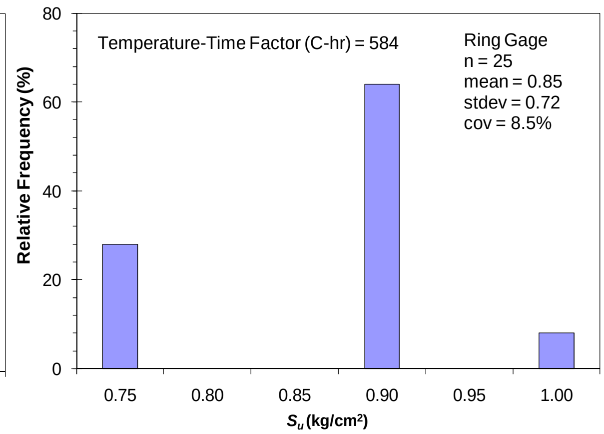
(a) SC1 (5, 100) Soil 1 24 hr-Dial



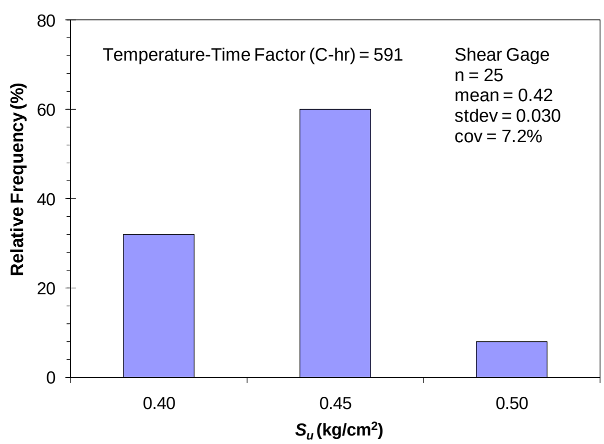
(d) SC1 (5, 100) Soil 1 24 hr-Dial



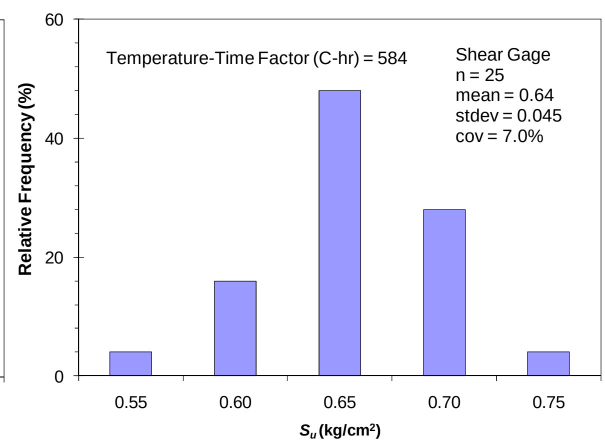
(b) SC1 (5, 100) Soil 1 24 hr-Ring



(e) SC1 (5, 100) Soil 1 24 hr-Ring

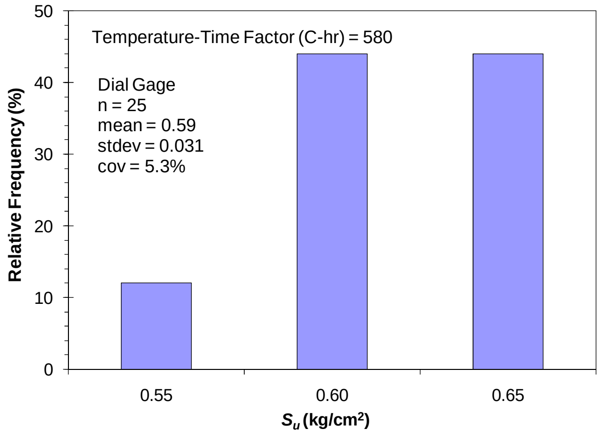


(c) SC1 (5, 100) Soil 1 24 hr-Shear

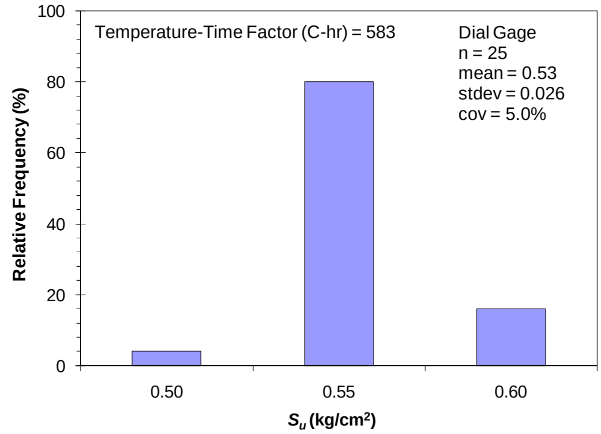


(f) SC1 (5, 100) Soil 1 24 hr-Shear

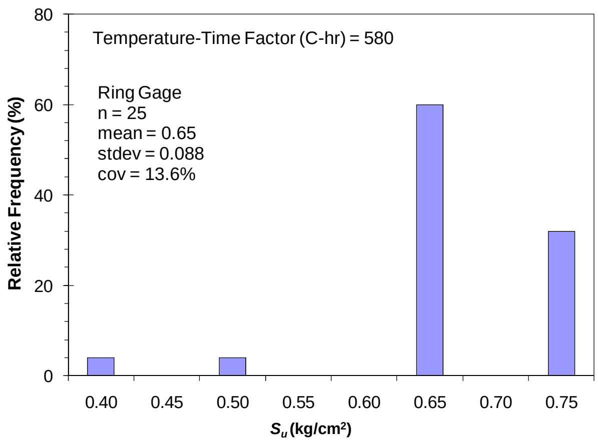
Figure B.8. SC1 (5, 100) Soil 1 24 hr and SC1 (5, 100) Soil 1 24 hr



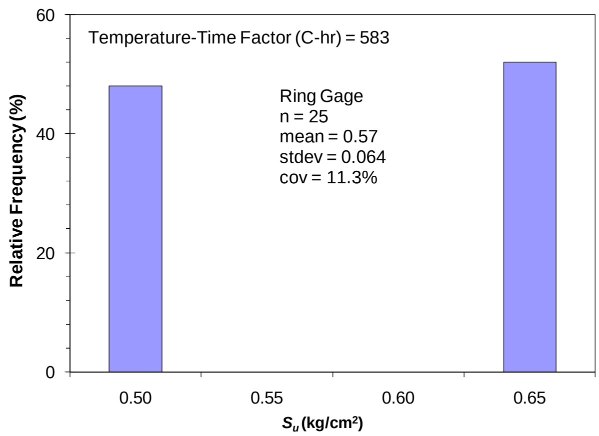
(a) SC1 (5, 100) Soil 3 24 hr-Dial



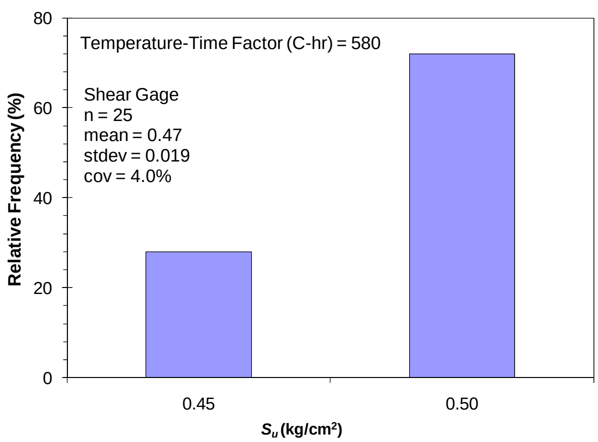
(d) SC1 (5, 100) Soil 3 24 hr-Dial



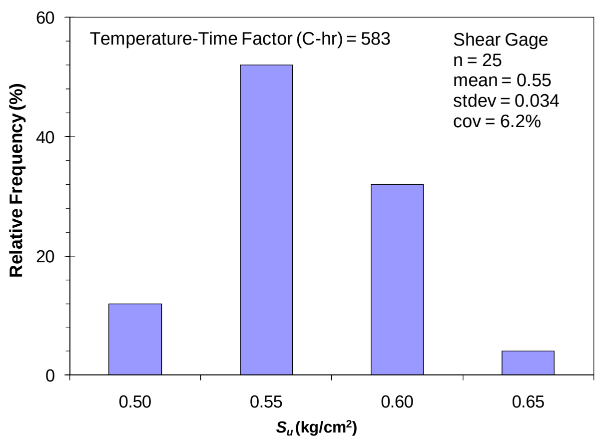
(b) SC1 (5, 100) Soil 3 24 hr-Ring



(e) SC1 (5, 100) Soil 3 24 hr-Ring

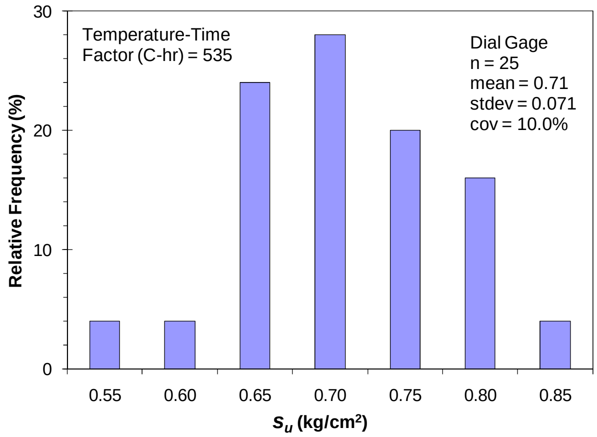


(c) SC1 (5, 100) Soil 3 24 hr-Shear

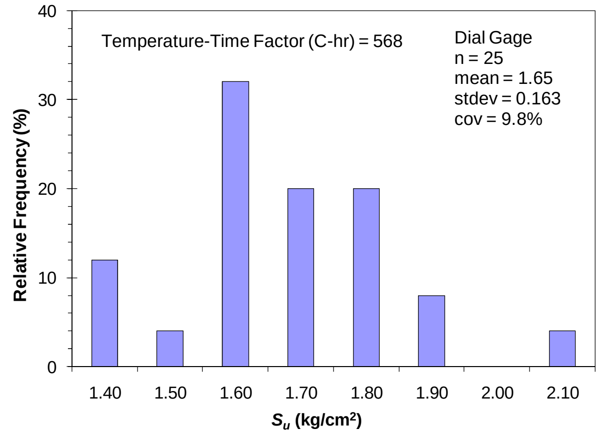


(f) SC1 (5, 100) Soil 3 24 hr-Shear

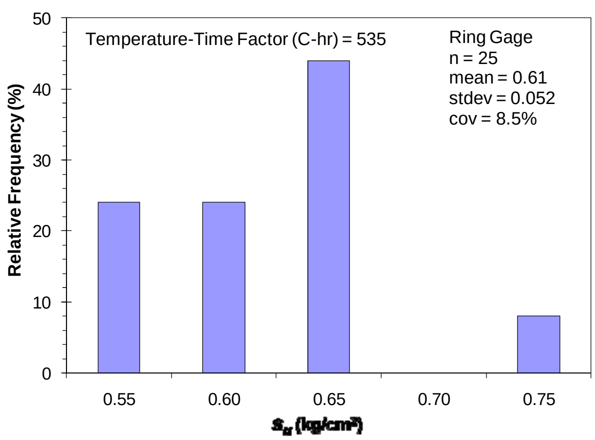
Figure B.9. SC1 (5, 100) Soil 3 24 hr and SC1 (5, 100) Soil 3 24 hr



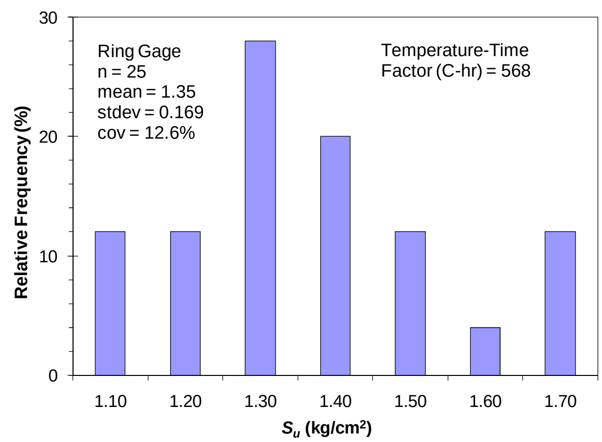
(a) SC2 (5, 100) Soil 1 24 hr-Dial



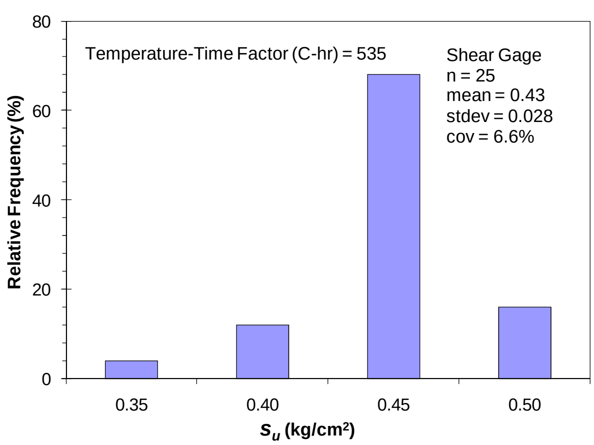
(d) SC2 (5, 100) Soil 2 24 hr-Dial



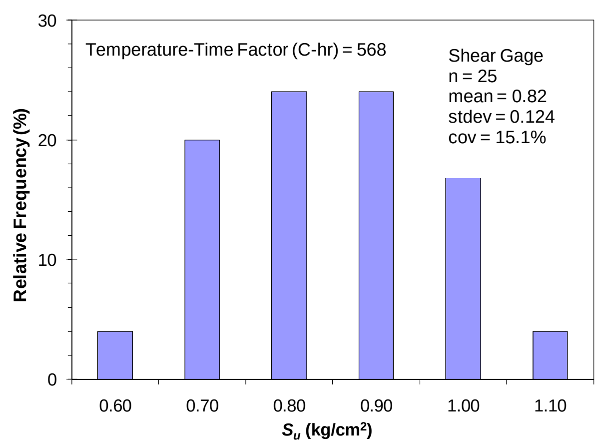
(b) SC2 (5, 100) Soil 1 24 hr-Ring



(e) SC2 (5, 100) Soil 2 24 hr-Ring

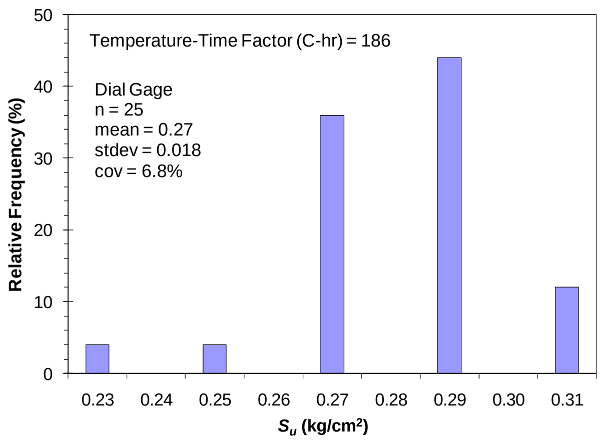


(c) SC2 (5, 100) Soil 1 24 hr-Shear

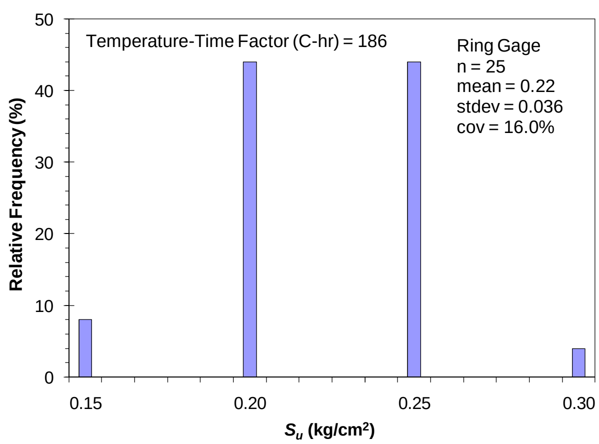


(f) SC2 (5, 100) Soil 2 24 hr-Shear

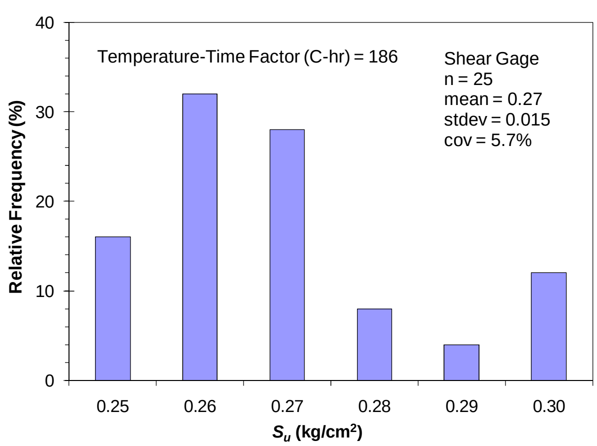
Figure B.10. SC2 (5, 100) Soil 1 24 hr and SC2 (5, 100) Soil 2 24 hr



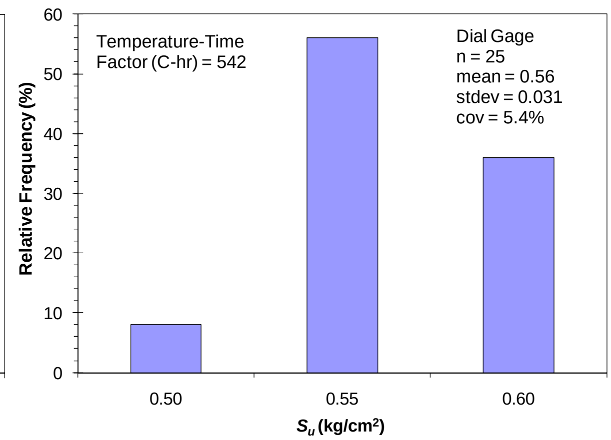
(a) SC2 (5, 100) Soil 3 8 hr-Dial



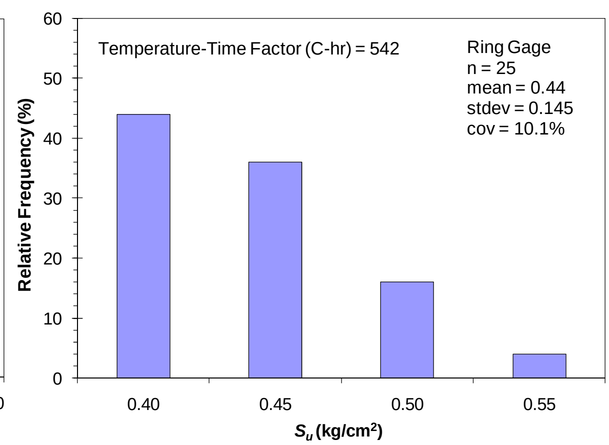
(b) SC2 (5, 100) Soil 3 8 hr-Ring



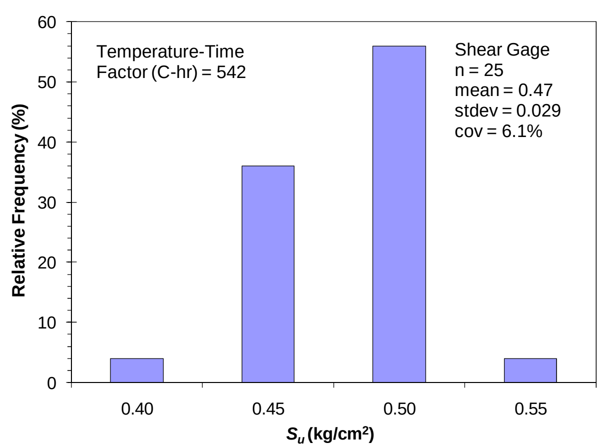
(c) SC2 (5, 100) Soil 3 8 hr-Shear



(d) SC2 (5, 100) Soil 3 24 hr-Dial

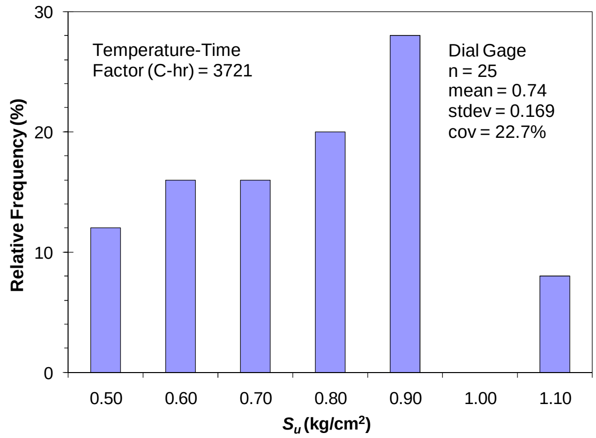


(e) SC2 (5, 100) Soil 3 24 hr-Ring

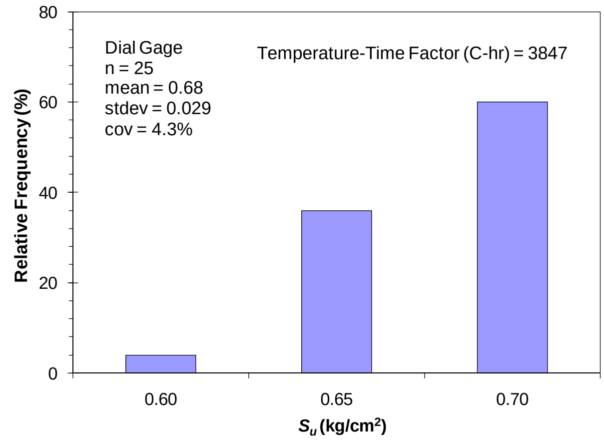


(f) SC2 (5, 100) Soil 3 24 hr-Shear

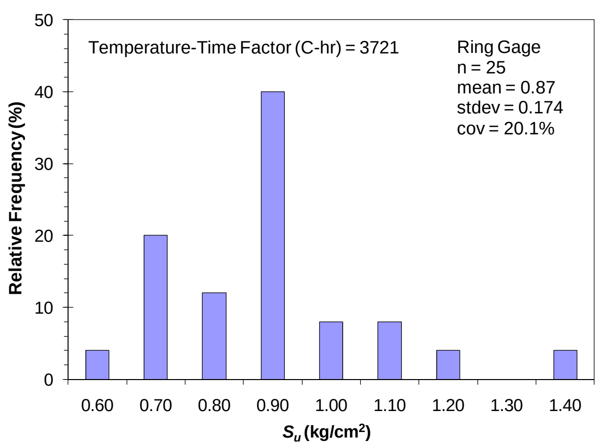
Figure B.11. SC2 (5, 100) Soil 3 8 hr and SC2 (5, 100) Soil 3 24 hr



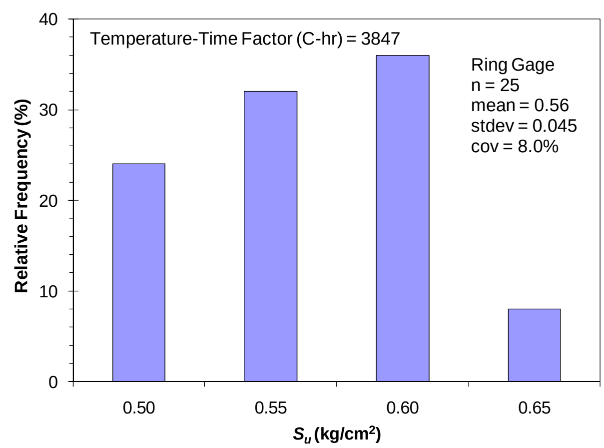
(a) SC2 (5, 100) Soil 3 168 hr-Dial



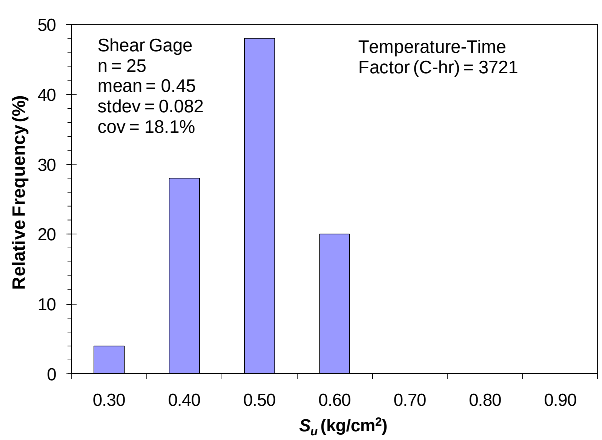
(d) SC2 (5, 100) Soil 3 168 hr-Dial



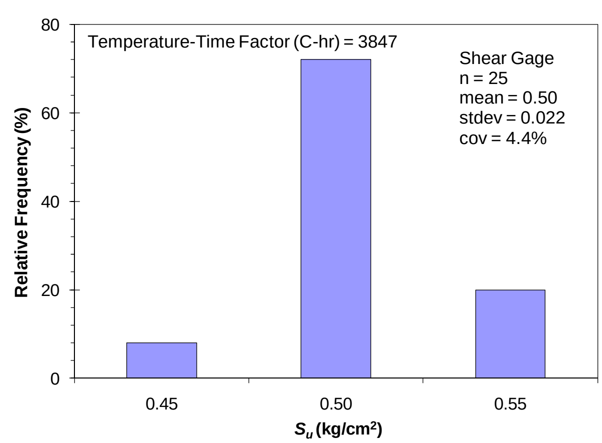
(b) SC2 (5, 100) Soil 3 168 hr-Ring



(e) SC2 (5, 100) Soil 3 168 hr-Ring

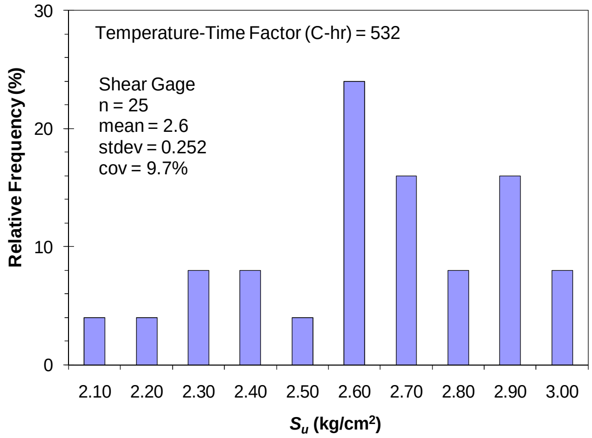


(c) SC2 (5, 100) Soil 3 168 hr-Shear

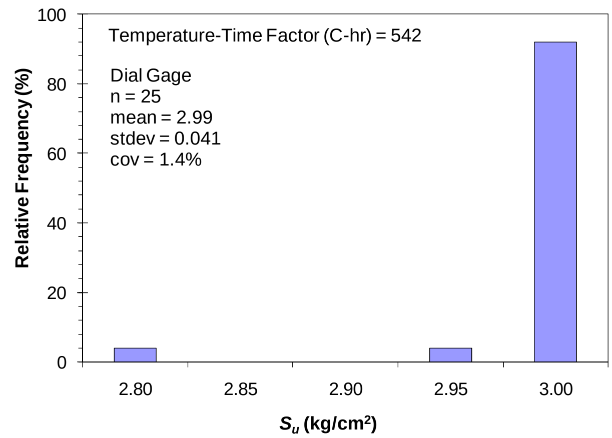


(f) SC2 (5, 100) Soil 3 168 hr-Shear

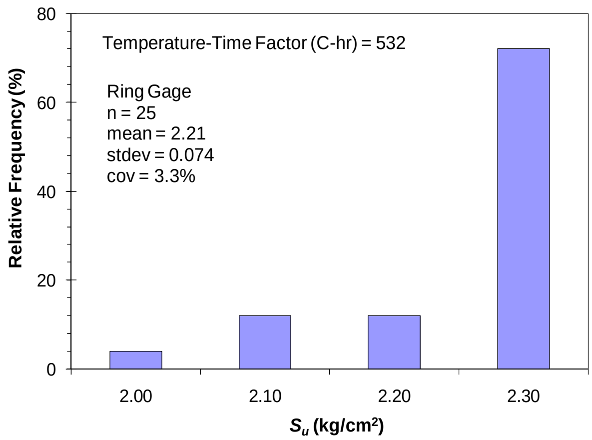
Figure B.12. SC2 (5, 100) Soil 3 168 hr and SC2 (5, 100) Soil 3 168 hr



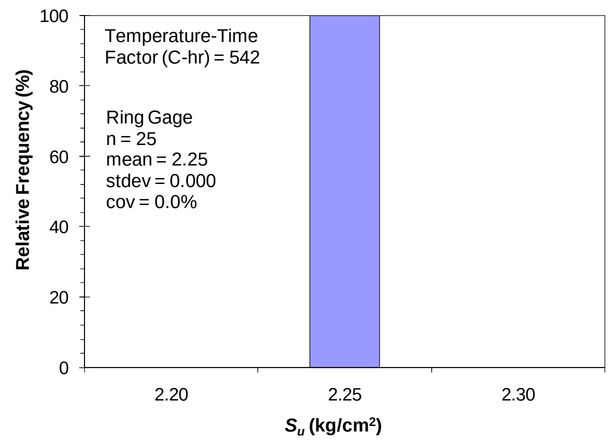
(a) SC2 (10, 100) Soil 1 24 hr-Dial



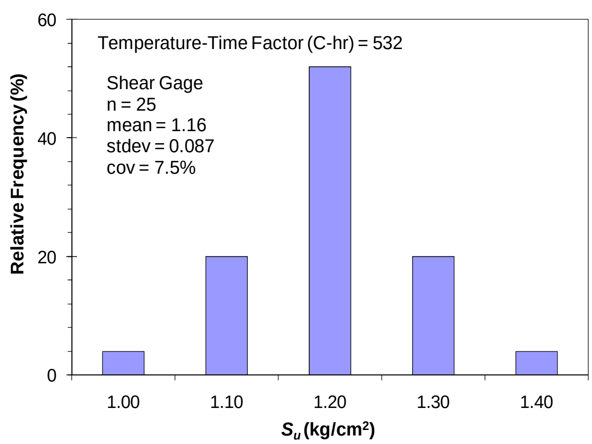
(d) SC2 (10, 100) Soil 2 24 hr-Dial



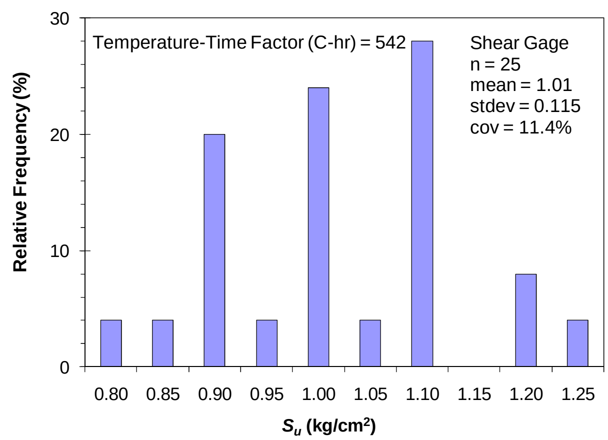
(b) SC2 (10, 100) Soil 1 24 hr-Ring



(e) SC2 (10, 100) Soil 2 24 hr-Ring

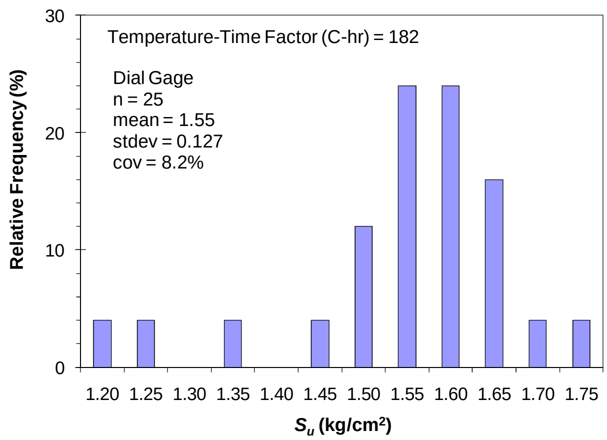


(c) SC2 (10, 100) Soil 1 24 hr-Shear

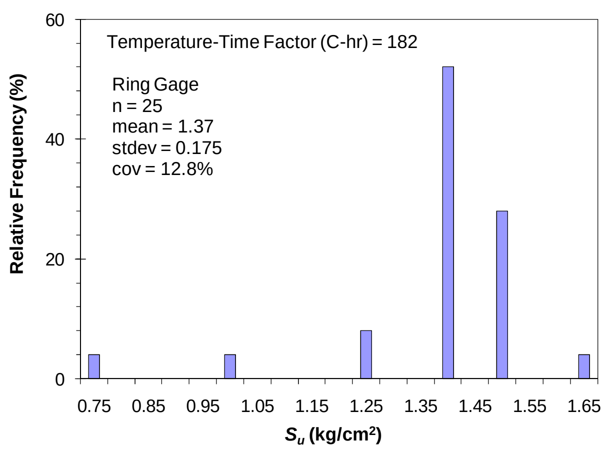


(f) SC2 (10, 100) Soil 2 24 hr-Shear

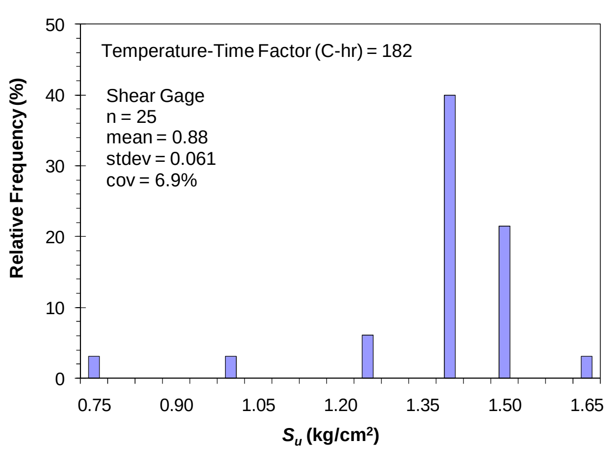
Figure B.13. SC2 (10, 100) Soil 1 24 hr and SC2 (10, 100) Soil 2 24 hr



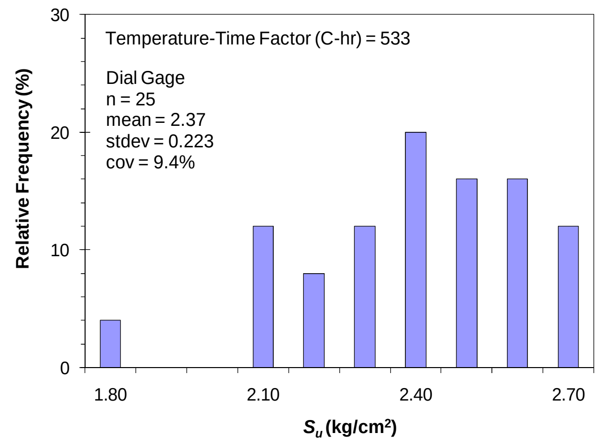
(a) SC2 (10, 100) Soil 3 8 hr-Dial



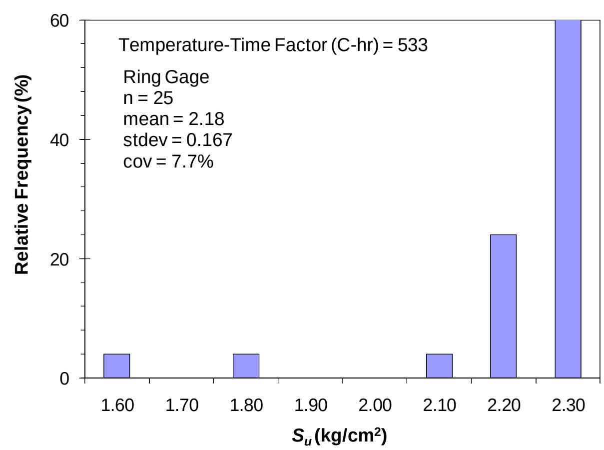
(b) SC2 (10, 100) Soil 3 8 hr-Ring



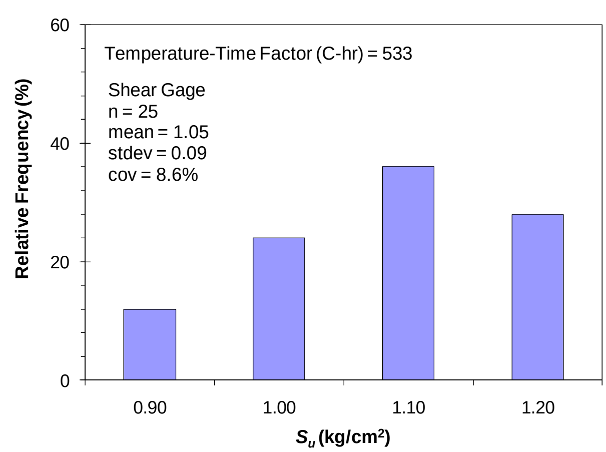
(c) SC2 (10, 100) Soil 3 8 hr-Shear



(d) SC2 (10, 100) Soil 3 24 hr-Dial

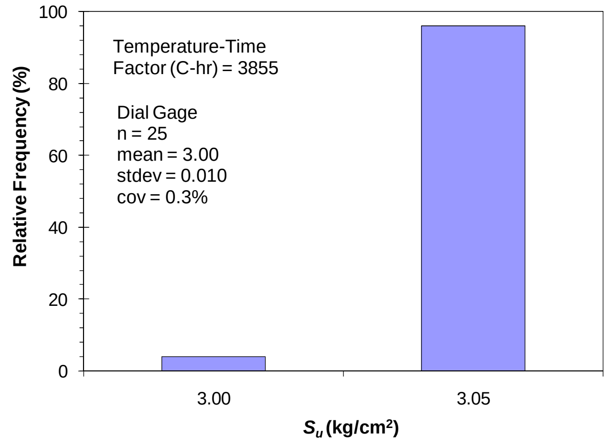


(e) SC2 (10, 100) Soil 3 24 hr-Ring

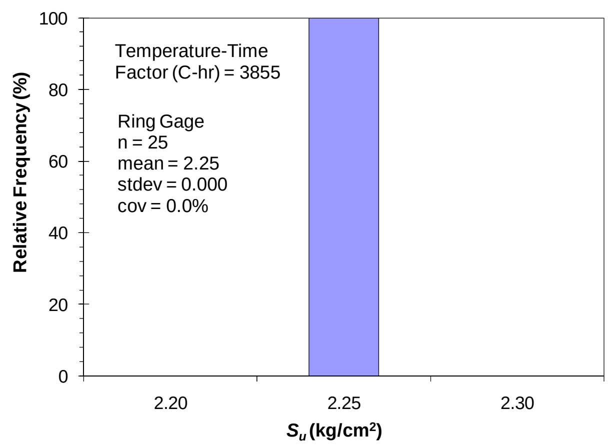


(f) SC2 (10, 100) Soil 3 24 hr-Shear

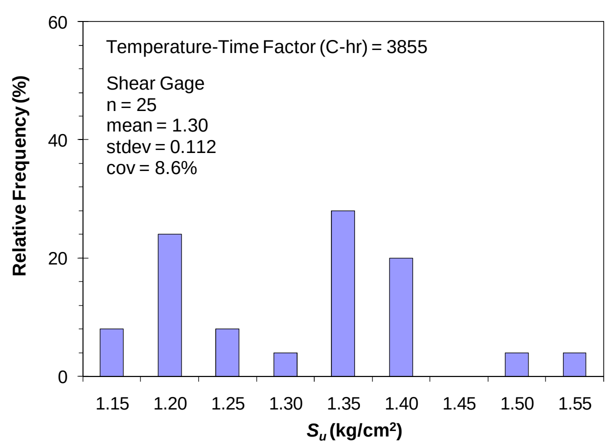
Figure B.14. SC2 (10, 100) Soil 3 8 hr and SC2 (10, 100) Soil 3 24 hr



(a) SC2 (10, 100) Soil 3 168 hr-Dial

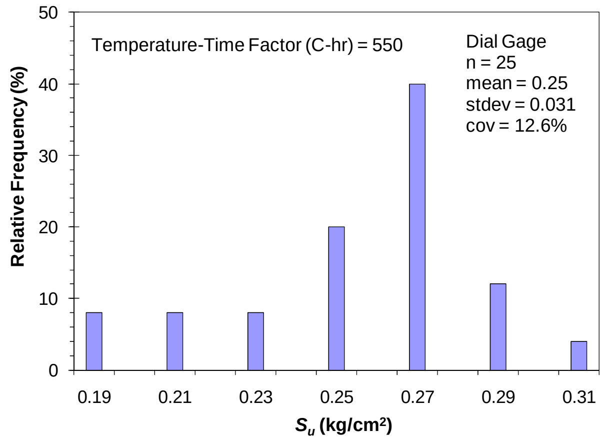


(b) SC2 (10, 100) Soil 3 168 hr-Ring

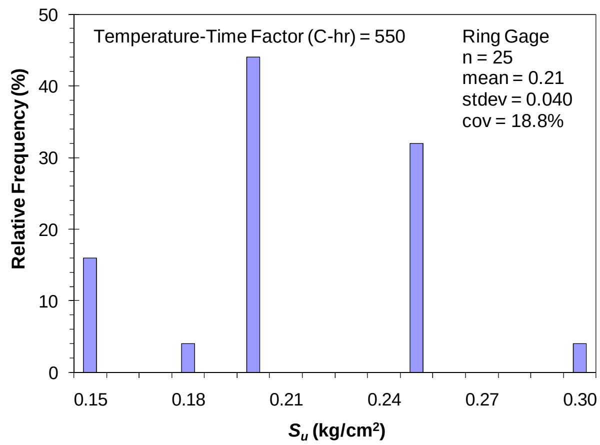


(c) SC2 (10, 100) Soil 3 168 hr-Shear

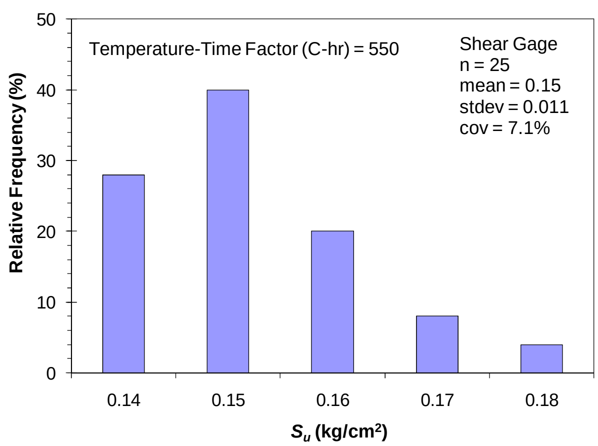
Figure B.15. SC2 (10, 100) Soil 3 168 hr



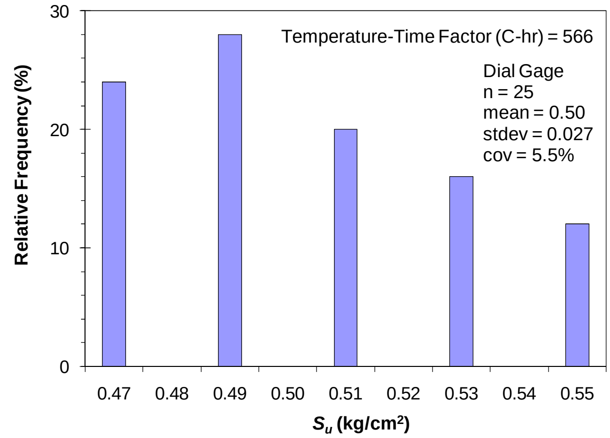
(a) SC2 (15, 233) Soil 1 24 hr-Dial



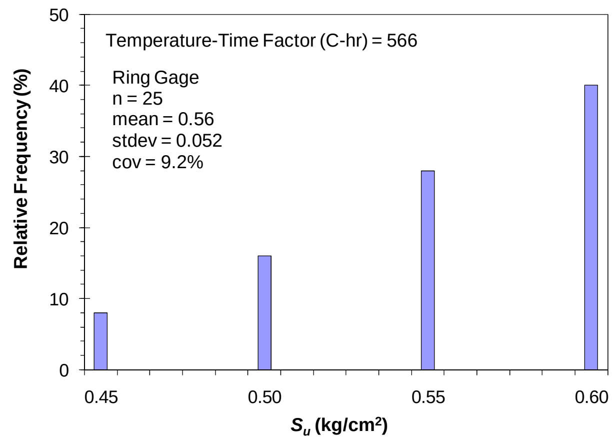
(b) SC2 (15, 233) Soil 1 24 hr-Ring



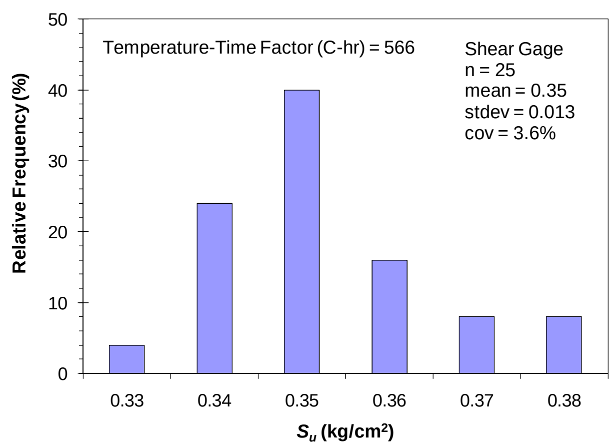
(c) SC2 (15, 233) Soil 1 24 hr-Shear



(d) SC2 (15, 233) Soil 2 24 hr-Dial

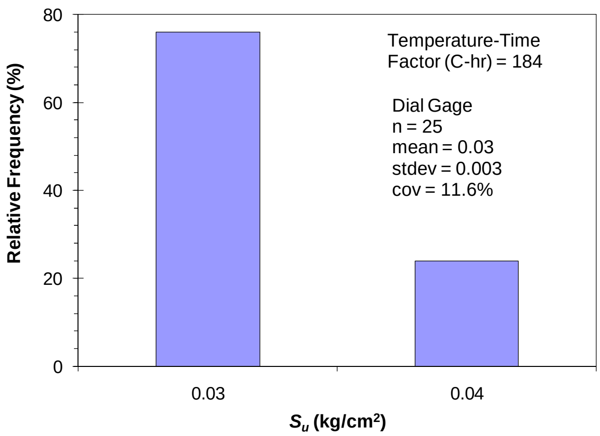


(e) SC2 (15, 233) Soil 2 24 hr-Ring

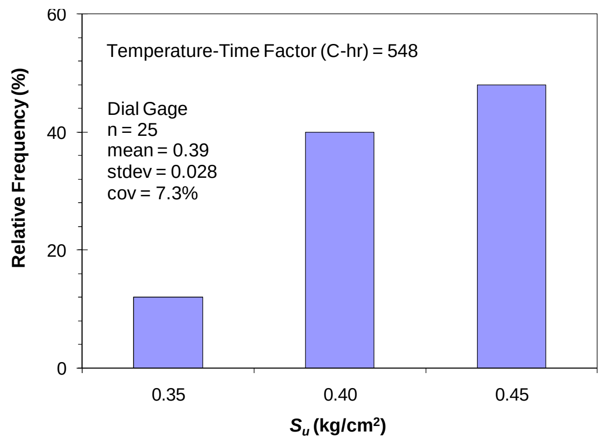


(f) SC2 (15, 233) Soil 2 24 hr-Shear

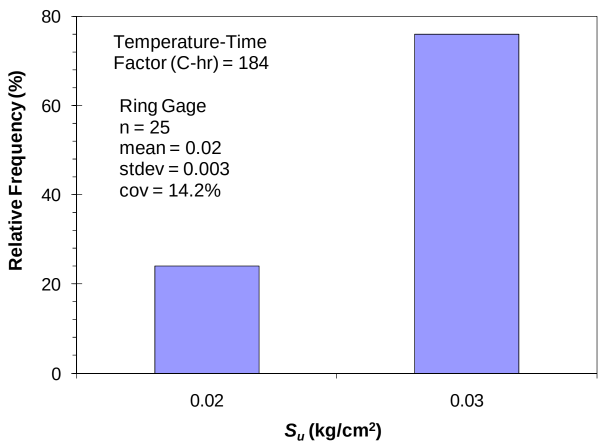
Figure B.16. SC2 (15, 233) Soil 1 24 hr and SC2 (15, 233) Soil 2 24 hr



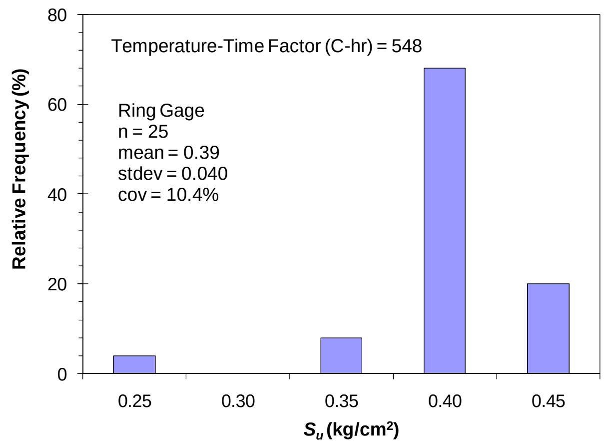
(a) SC2 (15, 233) Soil 3 8 hr-Dial



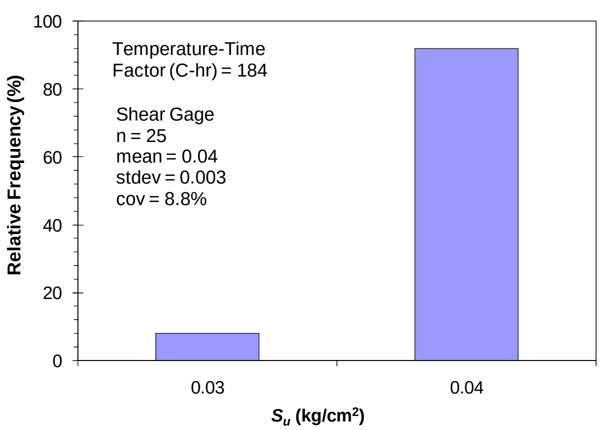
(d) SC2 (15, 233) Soil 3 24 hr-Dial



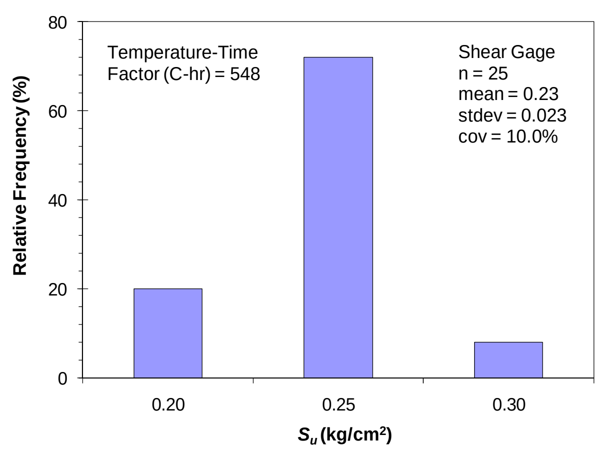
(b) SC2 (15, 233) Soil 3 8 hr-Ring



(e) SC2 (15, 233) Soil 3 24 hr-Ring

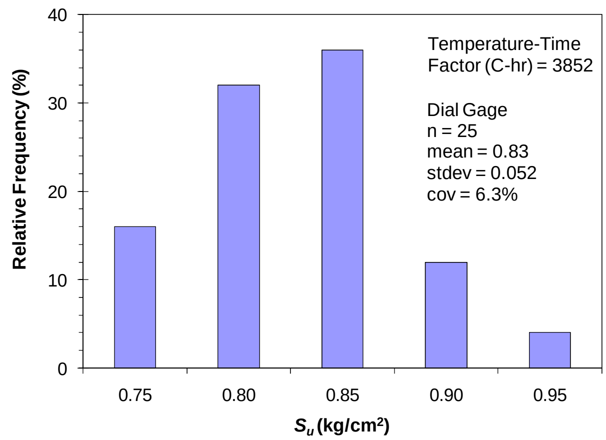


(c) SC2 (15, 233) Soil 3 8 hr-Shear

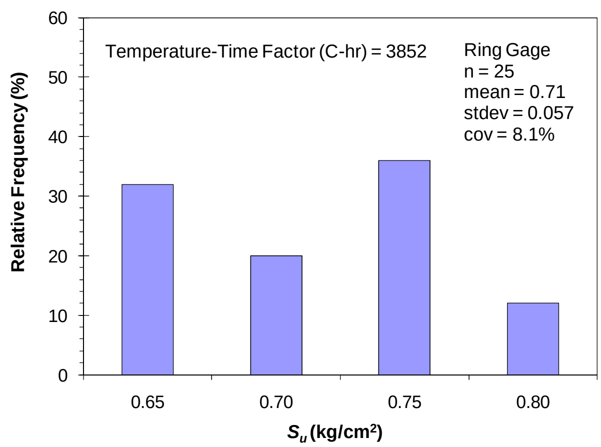


(f) SC2 (15, 233) Soil 3 24 hr-Shear

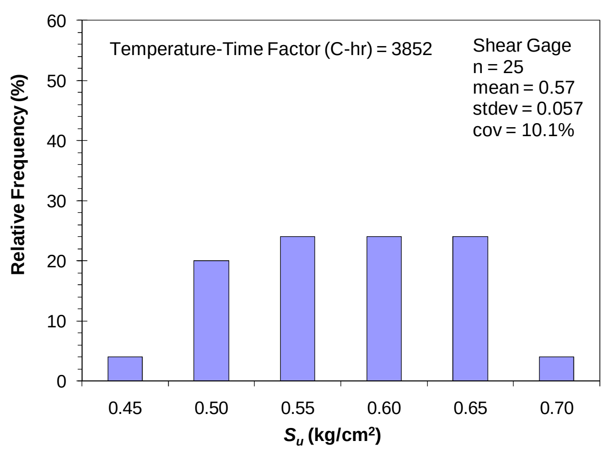
Figure B.17. SC2 (15, 233) Soil 3 8 hr and SC2 (15, 233) Soil 3 24 hr



(a) SC2 (15, 233) Soil 3 168 hr-Dial

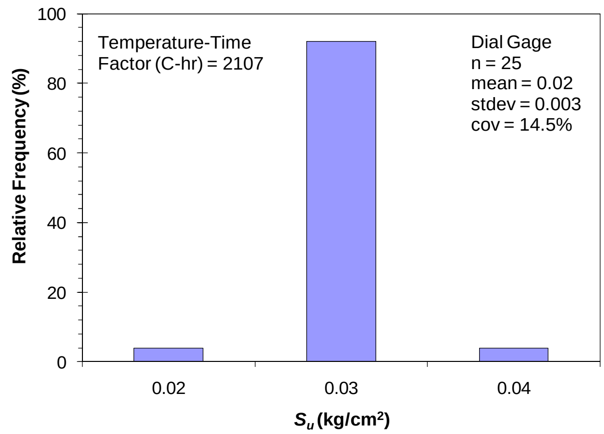


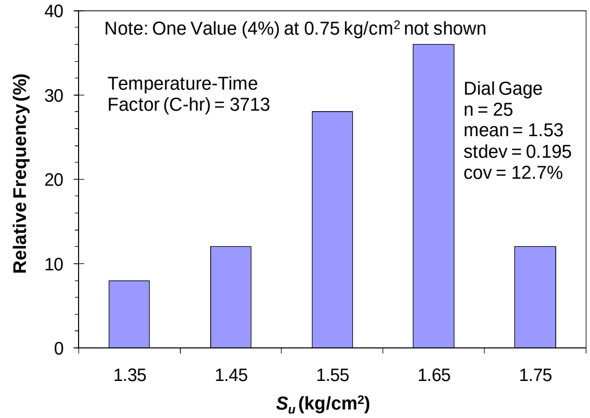
(b) SC2 (15, 233) Soil 3 168 hr-Ring



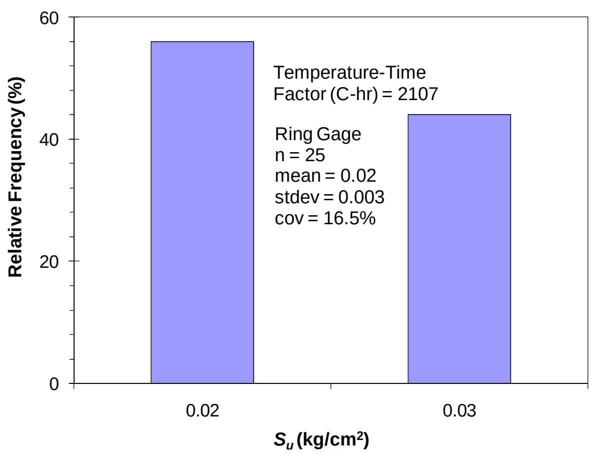
(c) SC2 (15, 233) Soil 3 168 hr-Shear

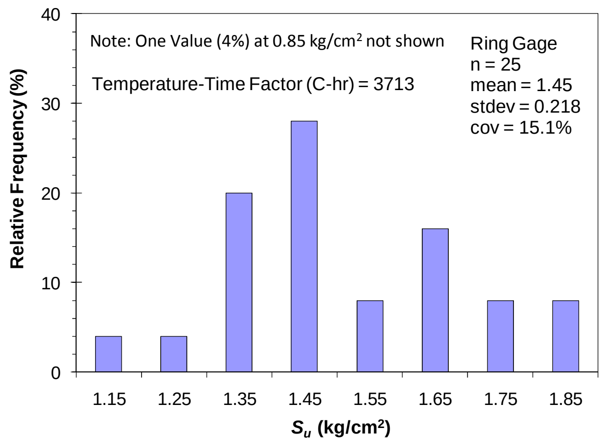
Figure B.18. SC2 (15, 233) Soil 3 168 hr



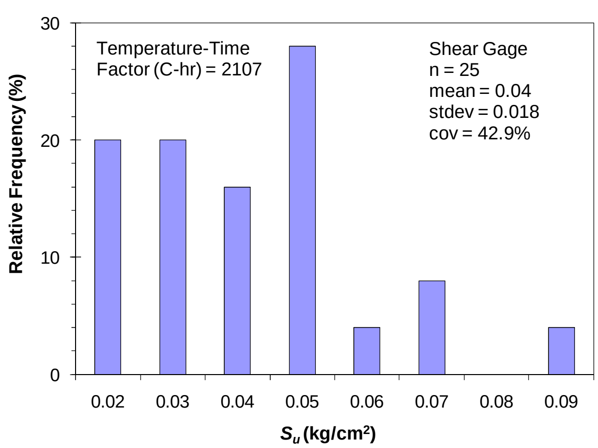


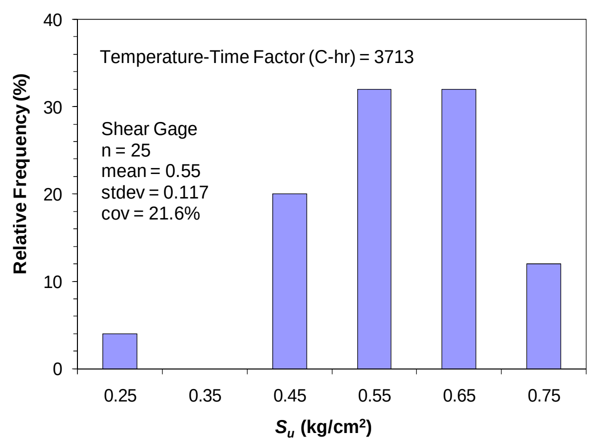
(a) A T I (GGBFS) (1.25, 3.75, 100) Soil 3 96 hr-Dial *(d) SC2 (PoP) (5, 100) Soil 3 168 hr-Dial*





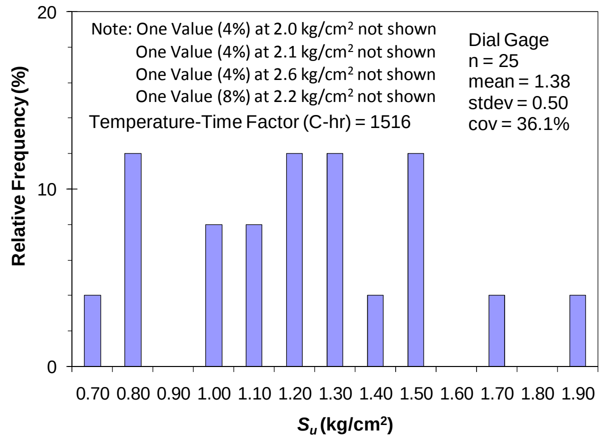
(b) A T I (GGBFS) (1.25, 3.75, 100) Soil 3 96 hr-Ring *(e) SC2 (PoP) (5, 100) Soil 3 168 hr-Ring*



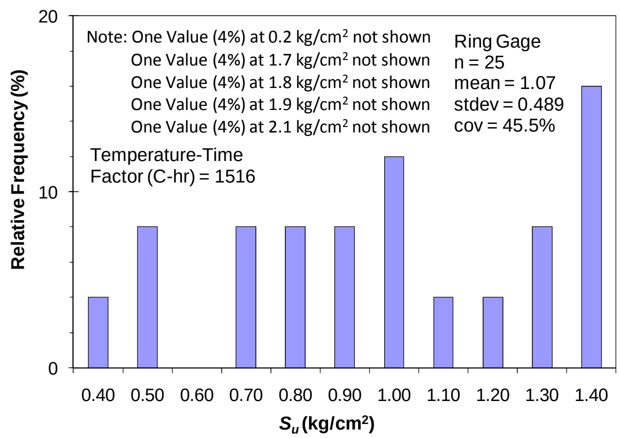


(c) A T I (GGBFS) (1.25, 3.75, 100) Soil 3 96 hr-Shear *(f) SC2 (PoP) (5, 100) Soil 3 168 hr-Shear*

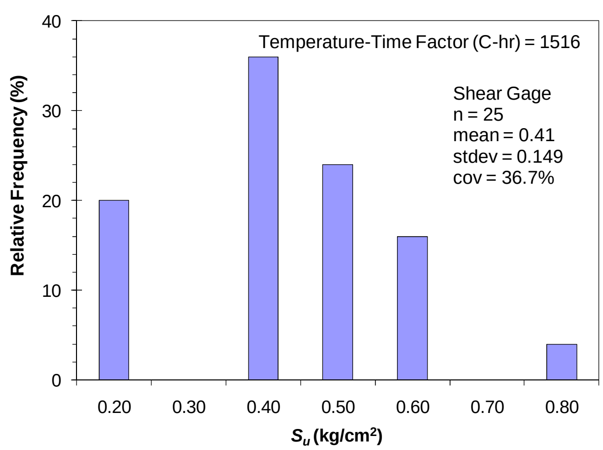
Figure B.19. A T I (GGBFS) (5, 100) Soil 3 96 hr and SC2 (PoP) (5, 100) Soil 3 168 hr



(a) SC1 (F20) (5, 0.5, 100) Soil 3 72 hr-Dial



(b) SC1 (F20) (5, 0.5, 100) Soil 3 72 hr-Ring



(c) SC1 (F20) (5, 0.5, 100) Soil 3 72 hr-Shear

Figure B.20. SC1 (F20) (5, 100) Soil 3 72 hr

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