Tennessee Valley Authority Kingston Fossil Plant Harriman, Roane County, Tennessee	**Perimeter Containment** **North Dredge Cell (Dike C) Segment 1** **Sta. A161+50 to A179+50**
Document Control Number RDP-0113-E *Calculation Package* FPGKIFFESCDX00030020100006	

Exhibit 13
Soil Strength Properties

Purpose:

- Document strength parameters for each soil and ash material to be used in subsequent analyses.

Methods:

- Derive parameters from field/laboratory test data, where available.
- Consider typical values for similar materials.

Results:

- Design strength parameters are tabulated for:
  - Static, drained, long-term conditions
  - Static, undrained, short-term conditions
  - Dynamic, undrained conditions
  - Residual strengths of liquefied materials

Calculation Performed by: Stantec Consulting Services, Inc.	
Prepared by: YW and AFR, Nov 2010	*Reviewed by:* Multiple reviews of prior drafts
Revisions:	

Soil Strength Properties

Table of Contents

Section **Page No.**

1. Drained Strengths for Static, Long Term Conditions 1
- 1.1. Earthen Berm 1
- 1.2. Landfilled Ash 1
- 1.3. Hydraulically Placed Ash 1
- 1.4. Sensitive Silt/Clay 2
- 1.5. Lean Clay Foundation Soil 2
- 1.6. Sandy Silt to Silty Sand 2
- 1.7. Coarse Stone and Shot Rock 3

2. Undrained Strengths for Static, Short Term Conditions 9
- 2.1. Earthen Berm 9
- 2.2. Landfilled Ash 10
- 2.3. Hydraulically Placed Ash 10
- 2.4. Sensitive Silt/Clay 11
- 2.5. Lean Clay Foundation Soil 11
- 2.6. Sandy Silt to Silty Sand 11
- 2.7. Coarse Stone and Shot Rock 12

3. Undrained Strengths for Dynamic Conditions 22

4. Residual Strengths for Liquefied Soils 24
- 4.1. Earthen Berm 25
- 4.2. Landfilled Ash 25
- 4.3. Hydraulically Placed Ash 25
- 4.4. Sensitive Silt/Clay 26
- 4.5. Lean Clay Foundation Soil 27
- 4.6. Sandy Silt to Silty Sand 27
- 4.7. Coarse Stone and Shot Rock 27

5. Additional Soil Properties used in *FLAC* 34
- 5.1. Tensile Strength 34
- 5.2. Dilation Angle 34
- 5.3. Progressive Liquefaction Model 34
- 5.4. Parameters for the Liquefaction Model 36

6. References 40

Table of Contents
(Continued)
List of Tables

Table **Page No.**

Table 1.1. Drained Soil Shear Strength Parameters for Static, Long Term Conditions 3

Table 2.1. Undrained Soil Shear Strength Parameters for Static, Short Term Conditions 13

Table 3.1. Shear Strength Parameters for Unliquefied Soil under Seismic Conditions 23

Table 4.1. Shear Strength Parameters for Liquefied Soil under Seismic Conditions 28

Table 5.1. Dilation Angles for Volumetric Response of Modeled Materials 37

List of Figures

Figure **Page No.**

Figure 1.1. Failure Stresses and Strength Envelope from Triaxial Compression Tests on Landfill Ash (Drained Condition)....4

Figure 1.2. Failure Stresses and Strength Envelopes from Triaxial Compression Tests on Hydraulically Placed Ash (Drained Condition)....5

Figure 1.3. Failure Stresses and Strength Envelope from Direct Simple Shear Tests on Sensitive Silt/Clay (Drained Condition) 6

Figure 1.4. Failure Stresses and Strength Envelope from Triaxial Compression Tests on Lean Clay (Drained Condition)....7

Figure 1.5. Failure Stresses and Strength Envelope from Triaxial Compression Tests on Sandy Silt to Silty Sand (Drained Condition)....8

Figure 2.1. Failure Stresses and Strength Envelope from Triaxial Compression Tests on Landfill Ash (Undrained Condition)....14

Figure 2.2. Failure Stresses and Strength Envelopes from Triaxial Compression Tests on Hydraulically Placed Ash (Undrained Condition)....15

Figure 2.3. Failure Stresses and Strength Envelopes from Triaxial Compression Tests on Hydraulically Placed Ash (Undrained Condition)....16

Figure 2.4. Failure Stresses and Strength Envelope from Direct Simple Shear Tests on Sensitive Silt/Clay (Undrained Condition) 17

Figure 2.5. Peak Shear Strength from Field Vane Shear Tests on Lean Clay....18

Table of Contents
(Continued)

Figure **Page No.**

Figure 2.6. Normalized Peak Shear Strength from Field Vane Shear Tests on Lean Clay 19

Figure 2.7. Failure Stresses and Strength Envelope from Triaxial Compression Tests on Lean Clay (Undrained Condition) 20

Figure 2.8. Failure Stresses and Strength Envelope from Triaxial Compression Tests on Sandy Silt to Silty Sand (Undrained Condition) 21

Figure 4.1. Residual Strength Ratio – Hydraulically Place Ash (SPT Data) 29

Figure 4.2. Residual Strength Ratio – Hydraulically Place Ash (CPT Data) 30

Figure 4.3. Residual Strength Ratio – Sandy Silt to Silty Sand (SPT Data) 31

Figure 4.4. Residual Strength Ratio – Sandy Silt to Silty Sand (CPT Data) 32

Figure 4.5. Critical void ratio and steady state line for hydraulically placed ash (from AECOM 2009) 33

Figure 5.1. SPT Blowcounts Measured in the Hydraulically Placed Ash 38

Figure 5.2. SPT Blowcounts Measured in the Sandy Silt to Silty Sand 39

List of Appendixes

Appendix A Test Data – Landfilled Ash

Appendix B Test Data – Hydraulically Placed Ash

Appendix C Test Data – Sensitive Silt/Clay

Appendix D Test Data – Lean Clay Foundation Soil

Appendix E Test Data – Sandy Silt to Silty Sand

Soil Strength Properties

1. Drained Strengths for Static, Long Term Conditions

Excess (or deficit) pore water pressures, generated by changes in mean stress or shearing stress, will dissipate under static, long term conditions. Pore pressures within a soil can then be computed assuming hydrostatic conditions or from a solution for steady state seepage. As long as the distribution of pore pressure within the cross section can be quantified, effective stresses can be computed and the drained shear strength (S_d) of the soil can be determined from effective stress strength parameters (c' and ϕ'):

$$S_d = c' + \sigma' \tan \phi' \qquad 1.1$$

Drained strength parameters for representing the Kingston materials under static, long term conditions are summarized in Table 1.1. These parameters were determined as described below. Where the parameters were derived from drained or undrained triaxial compression test data, failure was assumed at the point of maximum effective stress ratio (σ'_1/σ'_3). Drained strength parameters for the stabilized foundation zone are characterized in Exhibit 14.

1.1. Earthen Berm

The compacted embankment material to be used in the perimeter berm was conservatively assumed to have a friction angle of ϕ' = 28º and no effective cohesion. These values would be typical for a normally consolidated clay with plasticity index of about 35 (Terzaghi et al. 1996; Duncan and Wright 2005).

1.2. Landfilled Ash

Three consolidated-undrained (CU) triaxial compression tests with pore pressure measurements were performed by Stantec on ash obtained from the Kingston dredge cell (Appendix A). The test specimens were compacted to 90% of the standard Proctor maximum dry unit weight, backpressure saturated, then isotropically consolidated to effective pressures of 5, 15, and 30 psi. The results from these three tests, plotted in Figure 1.1, indicate a friction angle of ϕ' = 38º and c' = 0.

However, similar ash materials at other sites typically exhibit shear strengths that are significantly lower than indicated by these limited laboratory tests. Considering the material type (ML) and the anticipated density, correlations presented in NAVFAC (1986) suggest that a typical friction angle would be about 30º. Hence, values of ϕ' = 30º and c' = 0 were selected for the landfilled ash.

1.3. Hydraulically Placed Ash

Triaxial compression tests were performed by AECOM on reconstituted fly ash specimens, which were prepared by moist tamping, dry pluviation, or drained slurry methods, using ash recovered from borings in the Kingston dredge cell. Data (Appendix B) is available from 25 tests, including isotropically consolidated drained, isotropically

consolidated undrained, and K_o-consolidated undrained triaxial compression tests. Data is also available from six isotropically consolidated undrained tests performed in 1995 by Law Engineering, Inc. The combined results, plotted in Figure 1.2, indicate a friction angle of ϕ' = 25º and a cohesion of c' = 0.

The laboratory strength of the hydraulically placed ash is consistent with the SPT blowcounts measured in the field. Neglecting blowcounts greater than 10, the average $(N_1)_{60}$ in this material is 3.1 (see Section 5.4 and Figure 5.1). In a natural sand, this would correlate to a friction angle of about 25º, consistent with the value fit to the laboratory data.

In consideration of the results obtained from laboratory testing and SPT correlations, a friction angle of ϕ' = 25º was assigned for the hydraulically placed ash material.

1.4. Sensitive Silt/Clay

The University of Massachusetts (UMass) Amherst performed a total of 12 direct simple shear (DSS) tests on intact samples of the sensitive silt/clay layer, collected with a thin-walled piston sampler near the interface of the ash deposits and the natural alluvium. The UMass test report is included here as Appendix C. Because principal stresses are not measured in a DSS test, the maximum principal stress ratio cannot be quantified; instead, failure was assumed at the peak shear stress.

GeoTesting Express (GeoTesting) performed five DSS tests on similar materials encountered in borings at Dike C, located southeast of the dredge cell. Both the UMass and GeoTesting data sets are plotted in Figure 1.3, indicating ϕ' = 28° and c' = 0.

The strength of this material, under fully drained conditions, appears to be higher than the overlying ash, perhaps due to the silt content, higher average density, or laboratory stress path (DSS vs. triaxial compression). Note that this thin layer will influence the computed stability of the slope only if the assigned shear strength is less than the strength of the overlying material.

1.5. Lean Clay Foundation Soil

AECOM performed 20 CU triaxial compression tests (with pore pressure measurements) on the alluvial, lean clay soil (Appendix D). The intact test specimens were obtained using a thin-walled piston sampler, and were isotropically consolidated prior to vertical compression. During Stantec's exploration and investigation program for Dike C, GeoTesting Express performed 18 CU tests on a similar, lean clay deposit in the dike foundation.

This combined data set is plotted in Figure 1.4; the data from AECOM and GeoTesting Express are consistent. Assuming no effective cohesion, a friction angle of about ϕ' = 32º is obtained.

1.6. Sandy Silt to Silty Sand

Four CU tests on the alluvial sandy silt and silty sand soils were performed by AECOM (Appendix E). The intact test specimens were obtained using a thin-walled piston sampler, and were isotropically consolidated prior to vertical compression. Assuming no

effective cohesion, this limited data set (Figure 1.5) indicates a friction angle of about ϕ' = 30º.

Correlations between SPT blowcounts and friction angle were used to independently evaluate the strength of this deposit. Neglecting blowcounts greater than 20, the average $(N_1)_{60}$ in this material is 6.5 (see Section 5.4 and Figure 5.2). Based on typical correlations, this suggests a friction angle of about 30º, consistent with the laboratory test results. Hence, a friction angle of ϕ' = 30º was assigned for the sandy silt to silty sand deposit.

Thin layers of dense sand were encountered just above bedrock in some borings. SPT test results from this depth indicate a friction angle of about ϕ' = 39º. Because this material is encountered only in some borings and its thickness is relatively thin, it is conservatively grouped into the sandy silt and silty sand soils, and assigned the same friction angle of ϕ' = 30º.

1.7. Coarse Stone and Shot Rock

Both of these materials were assumed to be poorly graded, angular stone aggregates. Based on a typical range for materials of this type (US Navy 1986), a drained friction angle of ϕ' = 38º was assumed (with c' = 0). This strength estimate is somewhat conservative, as significantly higher friction angles are possible in these materials.

Table 1.1. Drained Soil Shear Strength Parameters for Static, Long Term Conditions

Material Name	Drained, Effective Stress Strength Parameters	
	ϕ' (deg.)	c' (psf)
Earthen Berm	28	0
Landfilled Ash	30	0
Hydraulically Placed Ash	25	0
Sensitive Silt/Clay	28	0
Lean Clay Foundation Soil	32	0
Sandy Silt to Silty Sand	30	0
Coarse Stone	38	0
Shot Rock	38	0

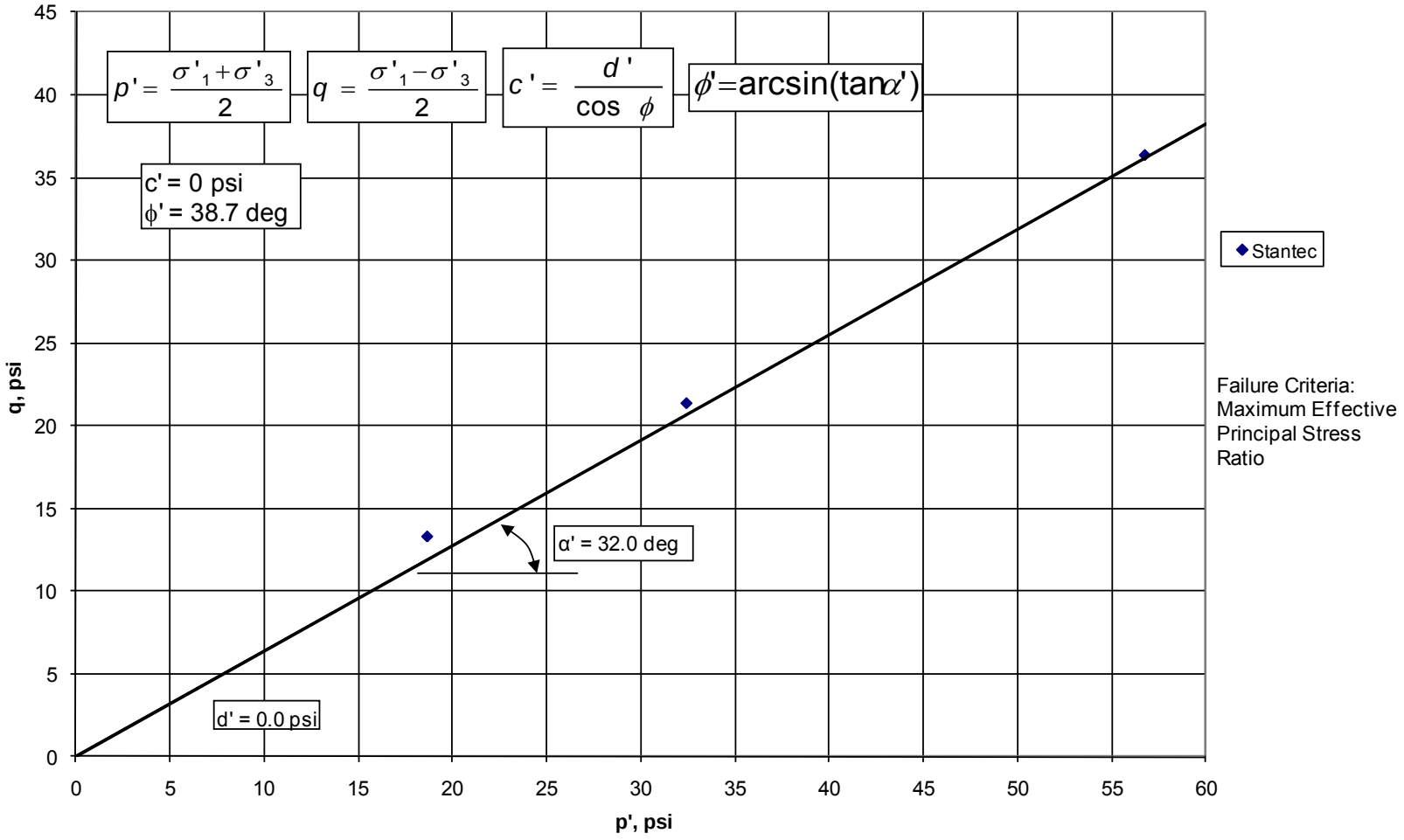


Figure 1.1. Failure Stresses and Strength Envelope from Triaxial Compression Tests on Landfill Ash (Drained Condition)

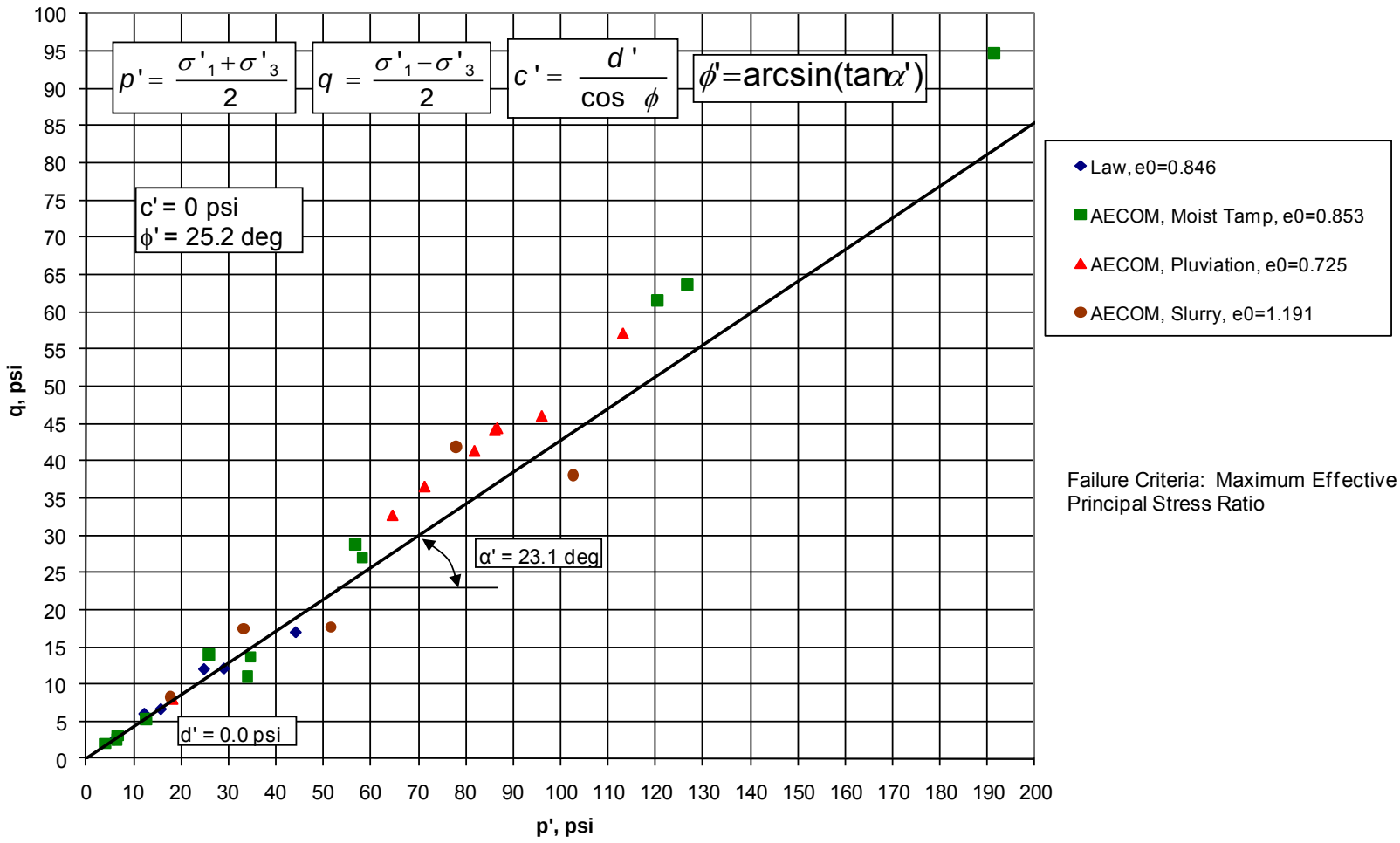


Figure 1.2. Failure Stresses and Strength Envelopes from Triaxial Compression Tests on Hydraulically Placed Ash (Drained Condition)

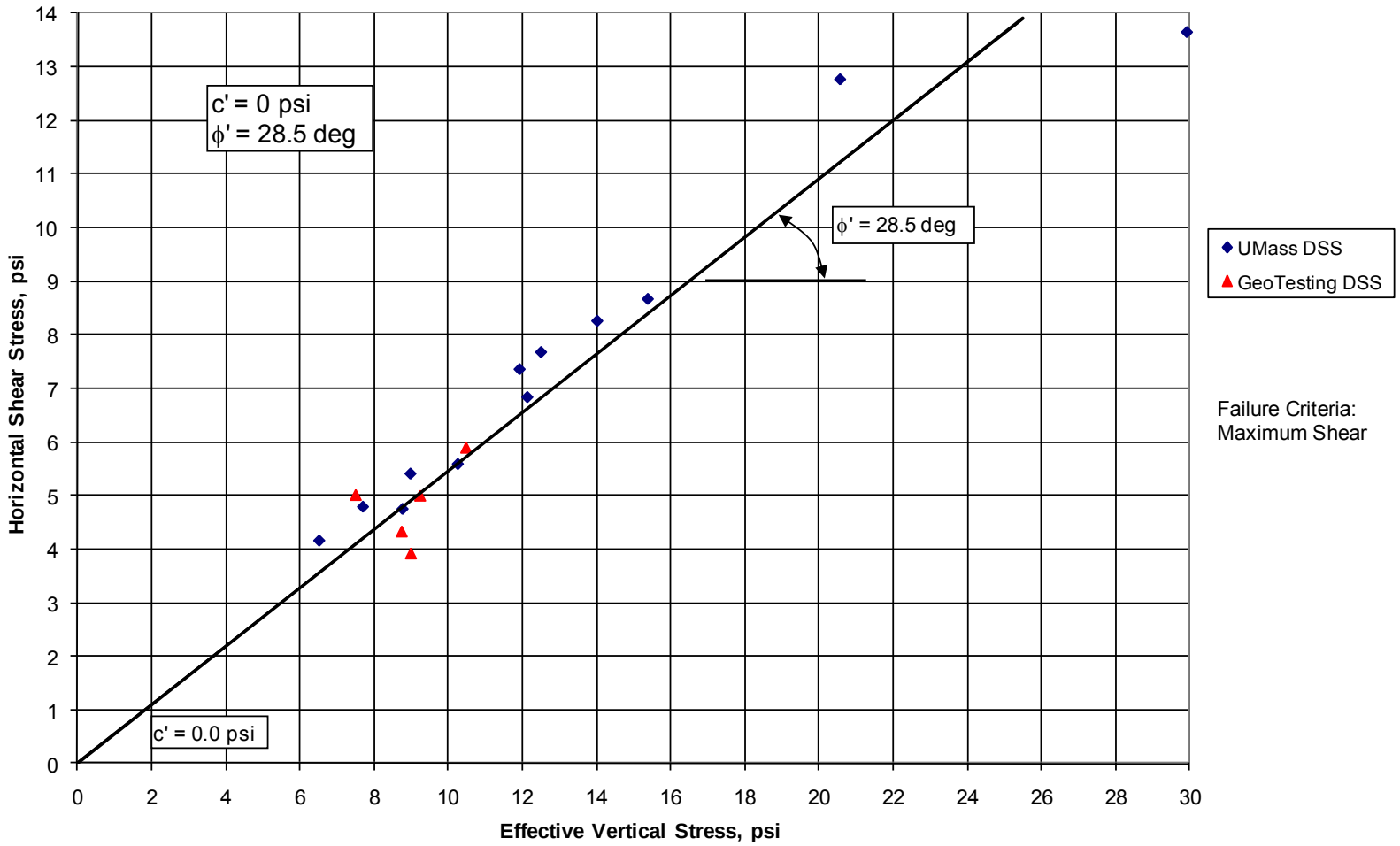
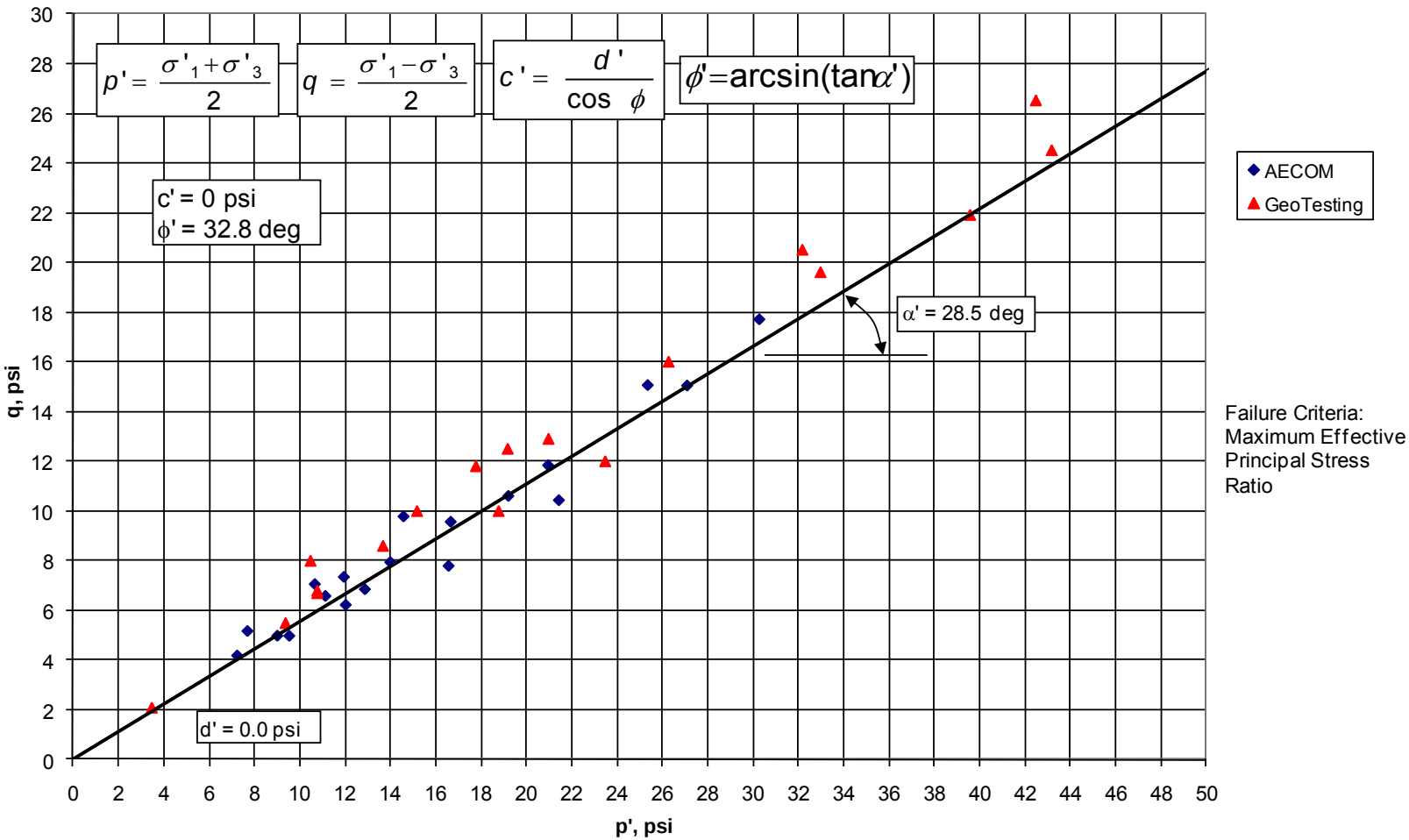


Figure 1.3. Failure Stresses and Strength Envelope from Direct Simple Shear Tests on Sensitive Silt/Clay (Drained Condition)



Failure Criteria:
Maximum Effective
Principal Stress
Ratio

Figure 1.4. Failure Stresses and Strength Envelope from Triaxial Compression Tests on Lean Clay (Drained Condition)

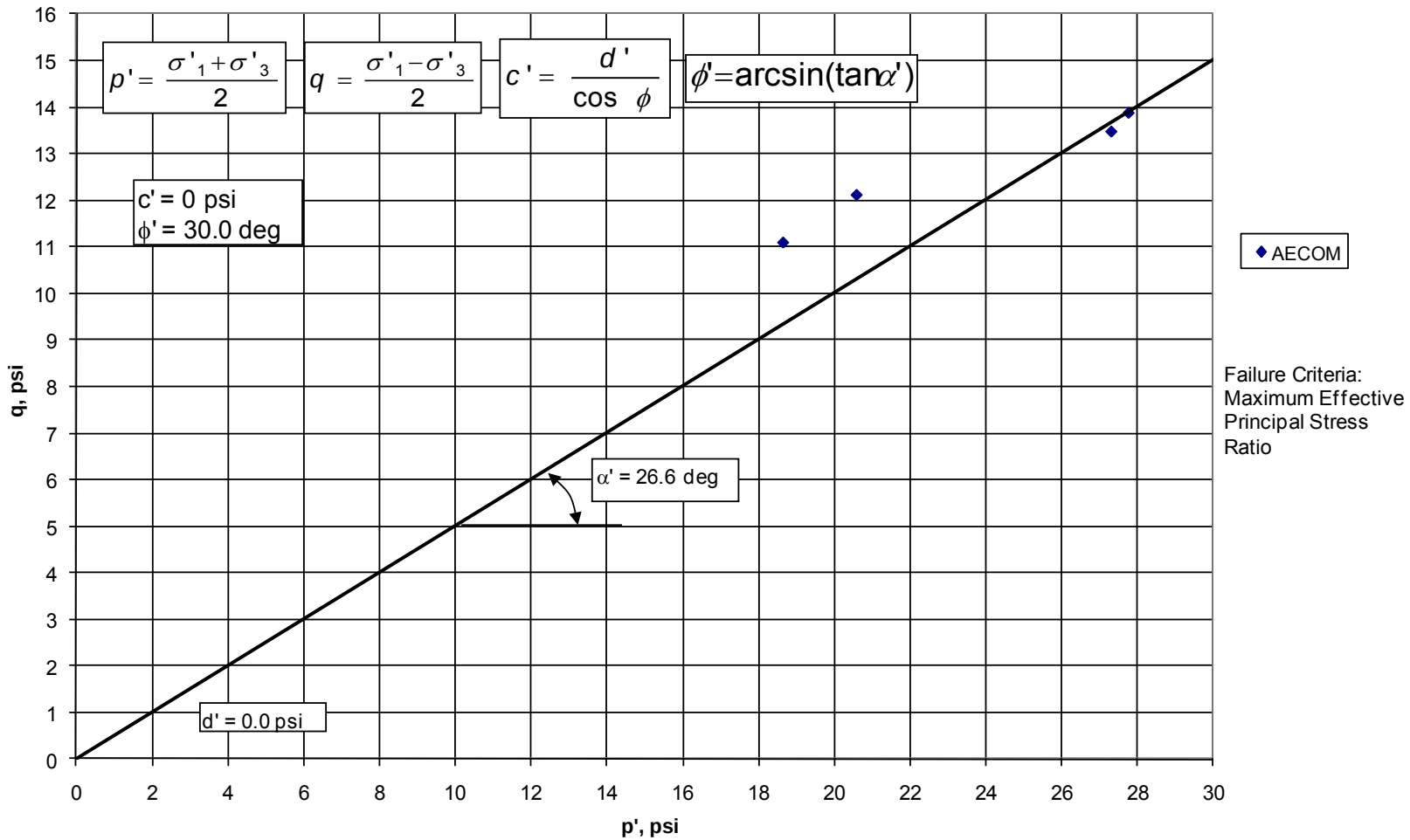


Figure 1.5. Failure Stresses and Strength Envelope from Triaxial Compression Tests on Sandy Silt to Silty Sand (Drained Condition)

2. Undrained Strengths for Static, Short Term Conditions

The undrained strength (S_u) of a soil can be expressed using the total stress parameters (c and ϕ):

$$S_u = c + \sigma'_{con} \tan\phi \qquad 2.1$$

where σ'_{con} is the effective consolidation pressure. Equation 2.1 (representing an "R-envelope") can be fit to the results of consolidated undrained (CU) tests. Neglecting the increase in strength with higher consolidation pressure, undrained strengths of a saturated soil may be modeled assuming the unconsolidated undrained (UU) case, where $\phi = 0$ and $S_u = c$. For critical soil layers, the preferred formulation for representing undrained strengths uses the c/p ratio:

$$S_u = \left(\frac{c}{p}\right)\sigma'_{1,con} \qquad 2.2$$

where $\sigma'_{1,con}$ is the major principal effective stress to which the soil has been consolidated. The c/p ratio varies with overconsolidation ratio (OCR), and is often represented using:

$$\left(\frac{c}{p}\right) = S \cdot OCR^{m} \qquad 2.3$$

where S is an intercept value (where $OCR = 1$) and m is an empirical exponent (typical values of 0.75 to 1).

Undrained, total stress parameters for the Kingston dredge cell soils are summarized in Table 2.1. These parameters, selected as discussed below, represent peak undrained strengths for each soil. Where the shear strength parameters were derived from consolidated undrained laboratory tests, failure was assumed at the point of maximum shear stress. For tests that did not exhibit a peak in the stress-strain curve, failure was assumed at 15% axial strain. Undrained strength parameters for the stabilized foundation zone are characterized in Exhibit 14.

Both the earthen berm and the restored ash embankment landfill will be constructed above the groundwater level and will be unsaturated. Shearing of unsaturated soils does not typically result in significant excess pore water pressure, such that drained strength parameters can often be used even for evaluating stability under short-term conditions. However, these compacted, fine-grained materials may be close to saturation, due to capillary rise of ground water and infiltration from the ground surface. Hence, it is prudent to estimate and apply undrained strength parameters for both the earthen berm and the landfilled ash.

2.1. Earthen Berm

The earthen berm was assumed to have a uniform, undrained strength of S_u = 1,500 psf, a typical value for stiff clay (Terzaghi et al. 1996).

2.2. Landfilled Ash

Three consolidated-undrained (CU) triaxial compression tests were performed by Stantec on ash obtained from the Kingston dredge cell (Appendix A). The test specimens were compacted to 90% of the standard Proctor maximum dry unit weight, backpressure saturated, then isotropically consolidated to effective pressures of 5, 15, and 30 psi. The results, plotted in Figure 2.1, indicate a friction angle of ϕ = 27º and a cohesion of c = 1,200 psf.

The recommended drained strength parameters for this material are ϕ' = 30º and c' = 0. For normal stress levels less than about 17,690 psf, the undrained strength envelope is above the drained strength envelope. In this stress regime, the higher undrained strength results from decreasing pore pressures (dilative response) during undrained shear. In the field, negative (suction) pore pressures generated by shearing may not be sustainable, as water will readily seep through this material and prevent underpressures of the magnitude that would occur in a sealed laboratory sample.

To avoid a reliance on suction pore water pressures for increased shearing resistance, a bilinear, undrained strength envelope is recommended for this material. At pressure levels below 17,690 psf, the fully drained strength should be used. The undrained envelope (ϕ = 27º and c = 1,200 psf) should only be used for stress levels of 17,690 psf or higher. Considering the height of the proposed landfill ash embankment, the stress level within the ash will be much lower than 17,690 psf; therefore, in practice, the fully drained strength parameters (ϕ' = 30º and c = 0 psf) can be used to represent this material for the undrained condition.

2.3. Hydraulically Placed Ash

Triaxial compression tests were performed by AECOM on reconstituted fly ash specimens, which were prepared by moist tamping, dry pluviation, or drained slurry methods, using ash recovered from borings in the Kingston dredge cell. Twenty CU (isotropically consolidated undrained and K_o-consolidated undrained) triaxial compression test data are available (Appendix B). The maximum deviator stress was used as the point of failure in each undrained test. The results, plotted in Figure 2.2, are very scattered. A strength envelope fit below about two-thirds of the data points gives a friction angle of ϕ = 13º and a cohesion of c = 0.

Considering the results in Figure 2.2 more closely, the method of preparing the test specimens can be seen to strongly impact both the soil density and the measured undrained strengths. The average void ratios of the specimens prepared by the moist tamping and dry pluviation methods were 0.843 and 0.725, respectively. Individual envelopes were fit to the test results for the moist tamped and dry pluviated samples in Figure 2.3; indicating ϕ = 10º and c = 0 (moist tamp) and ϕ = 21º and c = 0 (pluviation). The data obtained from the slurry preparation technique, which produced specimens with void ratios in the range of 0.917 to 1.459, are too scattered to develop an envelope.

The average void ratio of the hydraulically placed ash remaining in the dredge cell area is about 0.85 (see Exhibit 3). As the density of the moist-tamped specimens most closely match the field conditions, strength parameters of ϕ = 10º and c = 0 are recommended for the hydraulically placed ash.

Several of the tests in Appendix B show a substantial decrease in undrained shearing resistance at strains past the peak strength (strain softening). Thus, this material is susceptible to progressive failures, where overstressing at one locality can lead to redistribution of stress and retrogressive sliding of a larger mass. In the field, the peak undrained strength measured in the laboratory will not be mobilized at the same time along the full length of a failure surface. Peak undrained strength parameters for this material must be used with caution, in recognition of this behavior.

2.4. Sensitive Silt/Clay

The University of Massachusetts (UMass) Amherst performed a total of 12 direct simple shear (DSS) tests on intact samples of the sensitive silt/clay layer, collected by AECOM with a thin-walled piston sampler near the interface of the ash deposits and the natural alluvium. Consolidation tests were also performed by UMass to measure the maximum pre-consolidation pressure of the soils. The UMass laboratory report is included in Appendix C. Results from these DSS tests are plotted as $\log(S_u/\sigma'_{vc})$ vs. $\log(OCR)$ in Figure 2.4. The estimated S and m (Equation 2.3) are about 0.24 and 0.71, respectively.

Like the hydraulically placed ash, tests on the sensitive silt/clay indicate a significant loss in undrained shearing resistance past the peak undrained strength. Hence, peak undrained strength parameters for this material should be used with caution.

2.5. Lean Clay Foundation Soil

AECOM performed a total of 164 field vane shear tests on the alluvial lean clay material. Both peak and remolded shear strengths were measured, but only the peak strengths are considered here. The results (Figure 2.5 and Figure 2.6) are scattered, but can be characterized with S_u = 1,400 psf (ϕ = 0) or with c/p = 0.5 (Equation 2.2). Unfortunately, the overconsolidation ratio is difficult to quantify, because of the uncertainty in the pore water pressures (and effective stress) prior to the dredge cell failure and at the time of the vane shear testing. Coupled with the scatter evident in Figure 2.6, the c/p ratio parameters (S and m, Equation 2.3) cannot be determined with confidence from this data.

AECOM also performed 20 CU triaxial compression tests on the alluvial lean clay soil (Appendix D). In addition, GeoTesting performed 14 CU tests on samples of lean clay from beneath Dike C. The intact test specimens were obtained using a thin-walled piston sampler, and were isotropically consolidated prior to triaxial compression. A p-q plot of both AECOM and GeoTesting data, presented in Figure 2.7, indicates a friction angle of about ϕ = 24º and no cohesion.

The laboratory test results are consistent with the field vane shear data. Substituting c = 0 and ϕ = 24º into Equation 2.1 results in S_u/σ'_{con} = tan(24º) = 0.45, which is 10% less than the value of c/p = 0.5 fit to the field data. Hence, c = 0 and ϕ = 24º are recommended as design properties for this soil.

2.6. Sandy Silt to Silty Sand

This soil deposit is relatively pervious, and will exhibit undrained behavior only under very rapid loadings (such as seismic events). Under most static loading conditions, excess pore pressures (positive or negative) generated in this deposit will quickly

dissipate, such that drained, effective stress strength parameters can be used in a static slope stability evaluation. Undrained parameters were selected here primarily to support analyses for seismic loading conditions (see Section 3).

Four CU tests were performed by AECOM on the alluvial sandy silt and silty sand soils (Appendix E). The intact test specimens were obtained using a thin-walled piston sampler, and were isotropically consolidated prior to shearing. The *p-q* plot is presented in Figure 2.8, which indicates ϕ = 12º and c = 1,000 psf.

A closer examination of the laboratory test data shows that these soils exhibited significant decreases in pore pressure at failure (dilative response in undrained shear). In the field, negative pore water pressure generated by shearing would not be sustainable, as water would quickly seep through these sandy soils. The effective stresses and the shearing resistance would then drop, as the suction pressures in the shear zone equilibrate with the surrounding conditions.

To avoid a reliance on suction pressures for increased shearing resistance, a bilinear, undrained strength envelope is recommended for this soil deposit. At pressure levels below 2,740 psf, the fully drained strength (ϕ' = 30º) should be used. The undrained envelope (ϕ = 12º and c = 1,000 psf) should be used for stress levels of 2,740 psf or higher.

2.7. Coarse Stone and Shot Rock

These materials will have very high hydraulic conductivities, such that excess pore water pressures will quickly dissipate when the material is sheared under a static load. Hence, the static, undrained strength of these materials can be characterized using the drained strength parameters (ϕ = 38º, c = 0).

Table 2.1. Undrained Soil Shear Strength Parameters for Static, Short Term Conditions

Material Name	Undrained, Total Stress Strength Parameters			Notes on Usage
	ϕ (deg.)	c (psf)	c/p	
Earthen Berm	0	1,500	--	
Landfilled Ash	30 27	0 1,200	--	for σ < 17,690 psf for $\sigma \geq$ 17,690 psf
Hydraulically Placed Ash	10	0	--	
Sensitive Silt/Clay	--	--	S = 0.24 m = 0.71	Use to compute c/p as function of OCR
Lean Clay Foundation Soil	24	0	--	
Sandy Silt to Silty Sand	30 12	0 1,000	-- --	for σ < 2,740 psf for $\sigma \geq$ 2,740 psf
Coarse Stone	38	0	--	
Shot Rock	38	0	--	

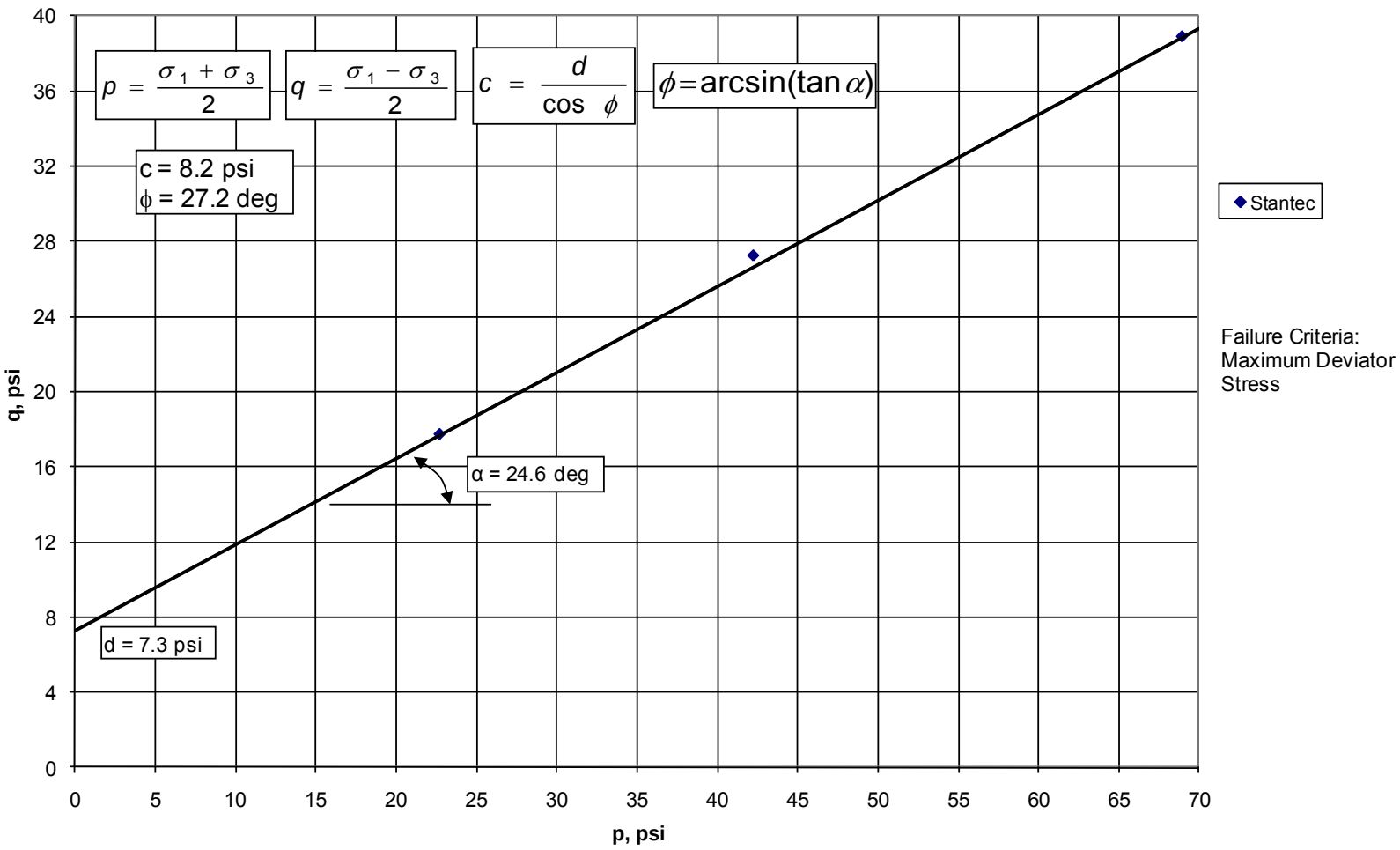


Figure 2.1. Failure Stresses and Strength Envelope from Triaxial Compression Tests on Landfill Ash (Undrained Condition)

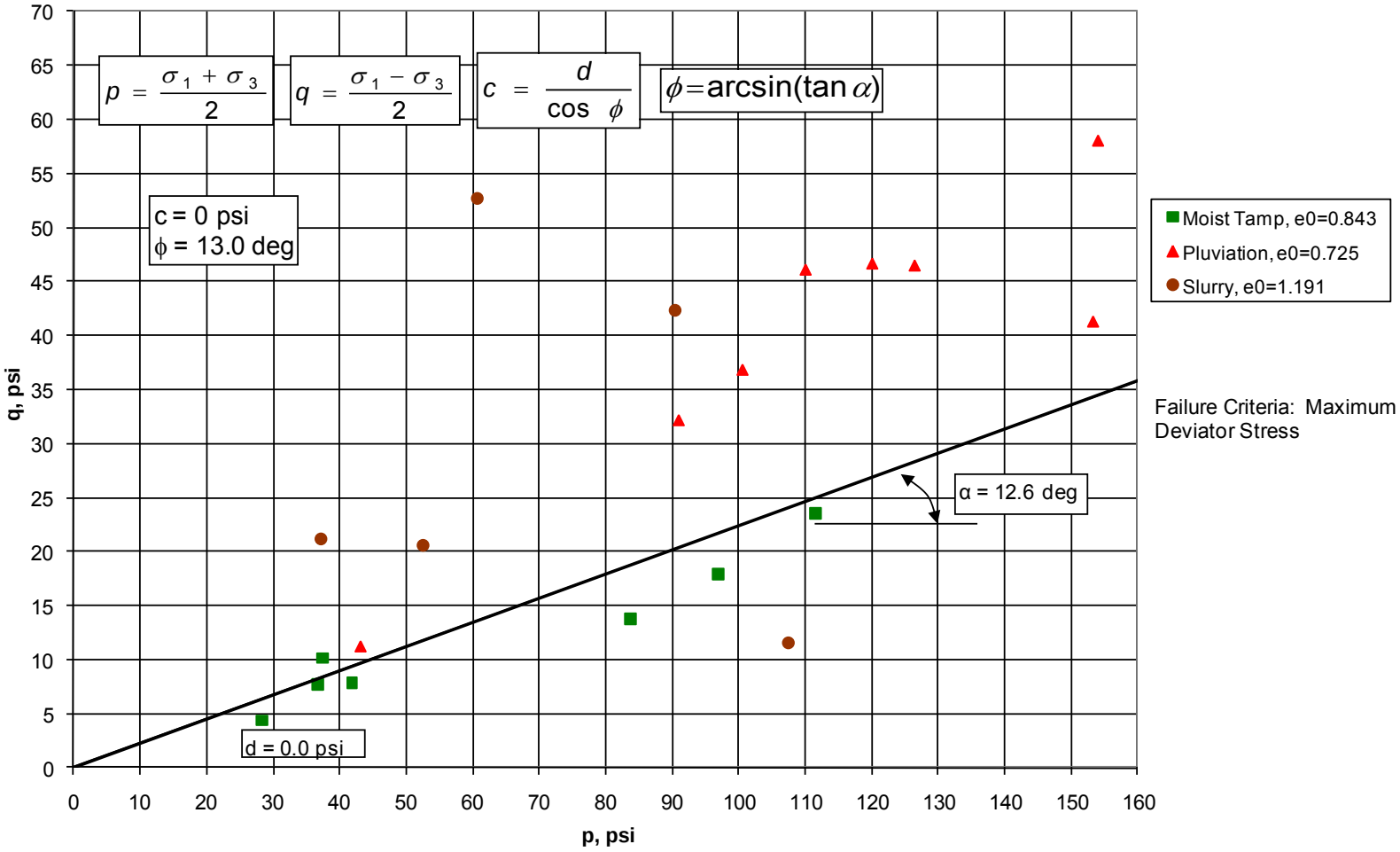


Figure 2.2. Failure Stresses and Strength Envelopes from Triaxial Compression Tests on Hydraulically Placed Ash (Undrained Condition)

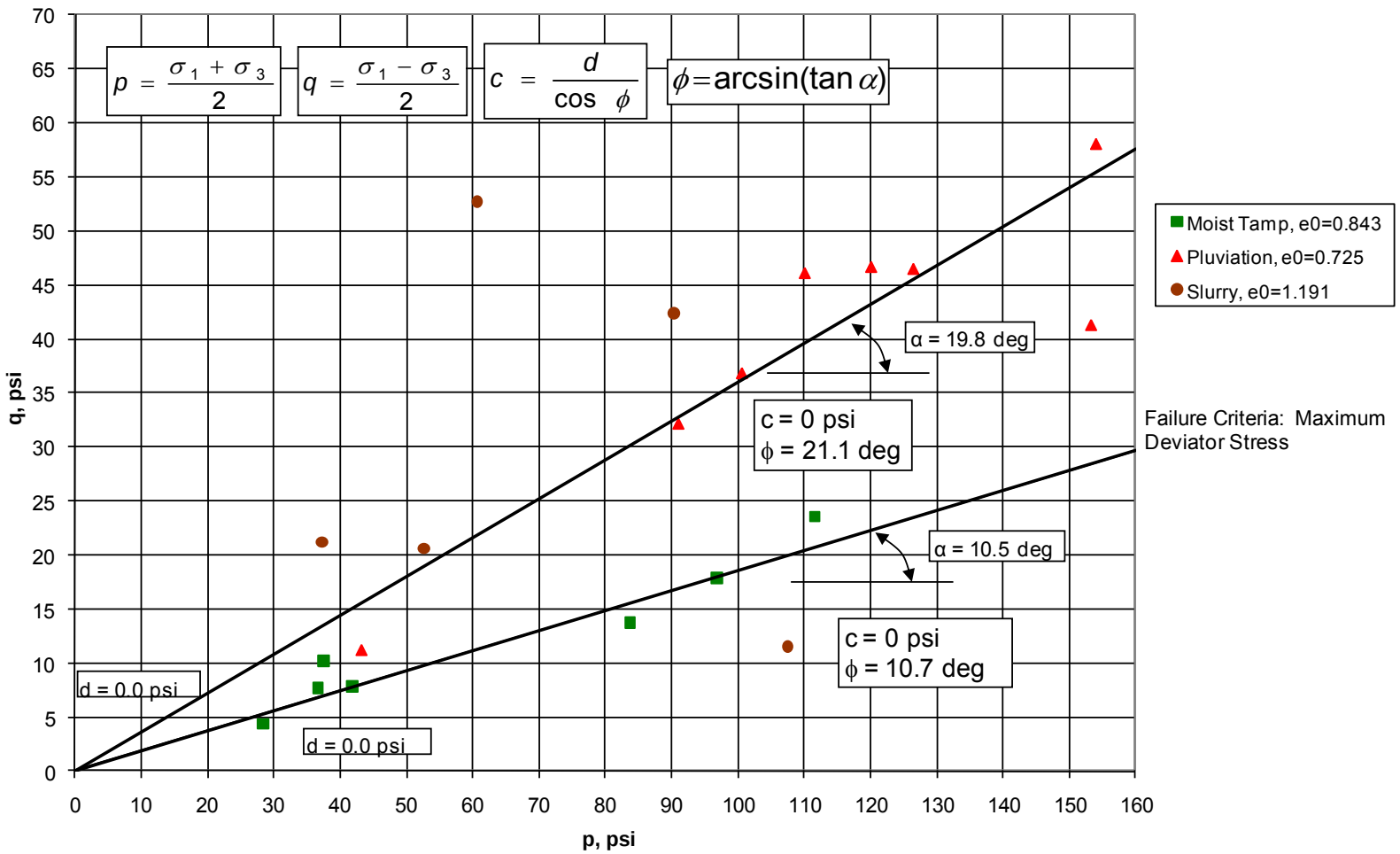


Figure 2.3. Failure Stresses and Strength Envelopes from Triaxial Compression Tests on Hydraulically Placed Ash (Undrained Condition)

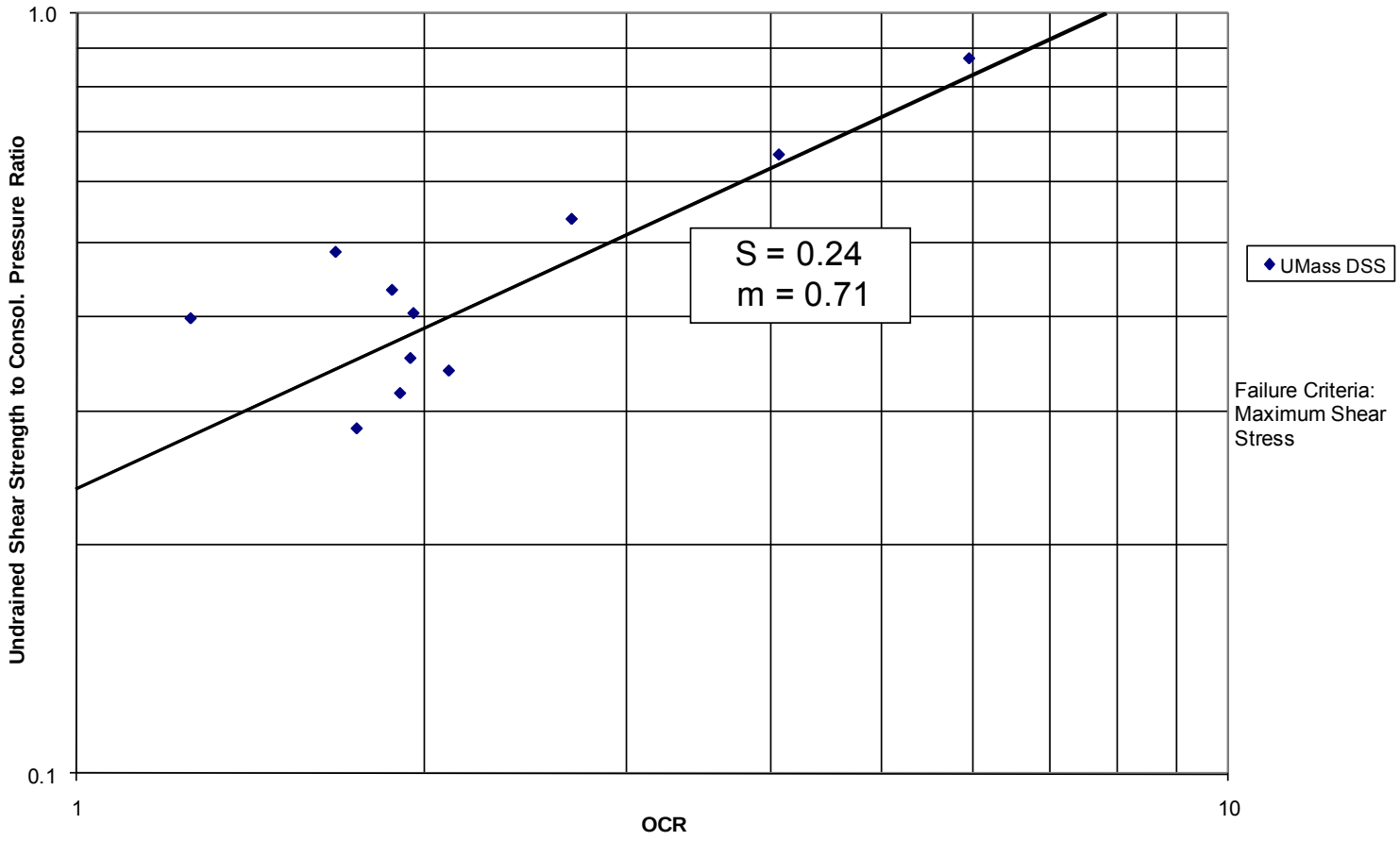


Figure 2.4. Failure Stresses and Strength Envelope from Direct Simple Shear Tests on Sensitive Silt/Clay (Undrained Condition)

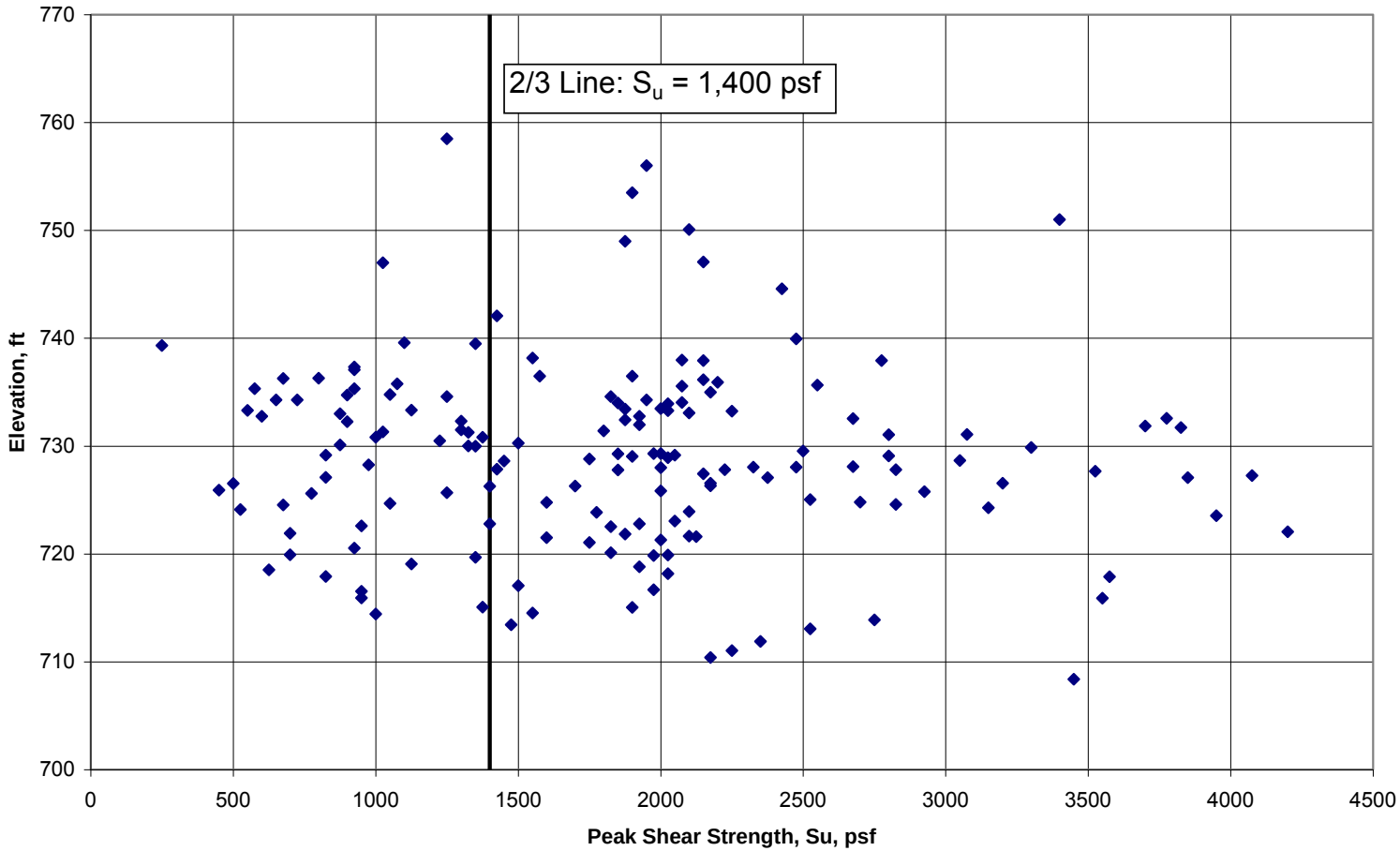


Figure 2.5. Peak Shear Strength from Field Vane Shear Tests on Lean Clay

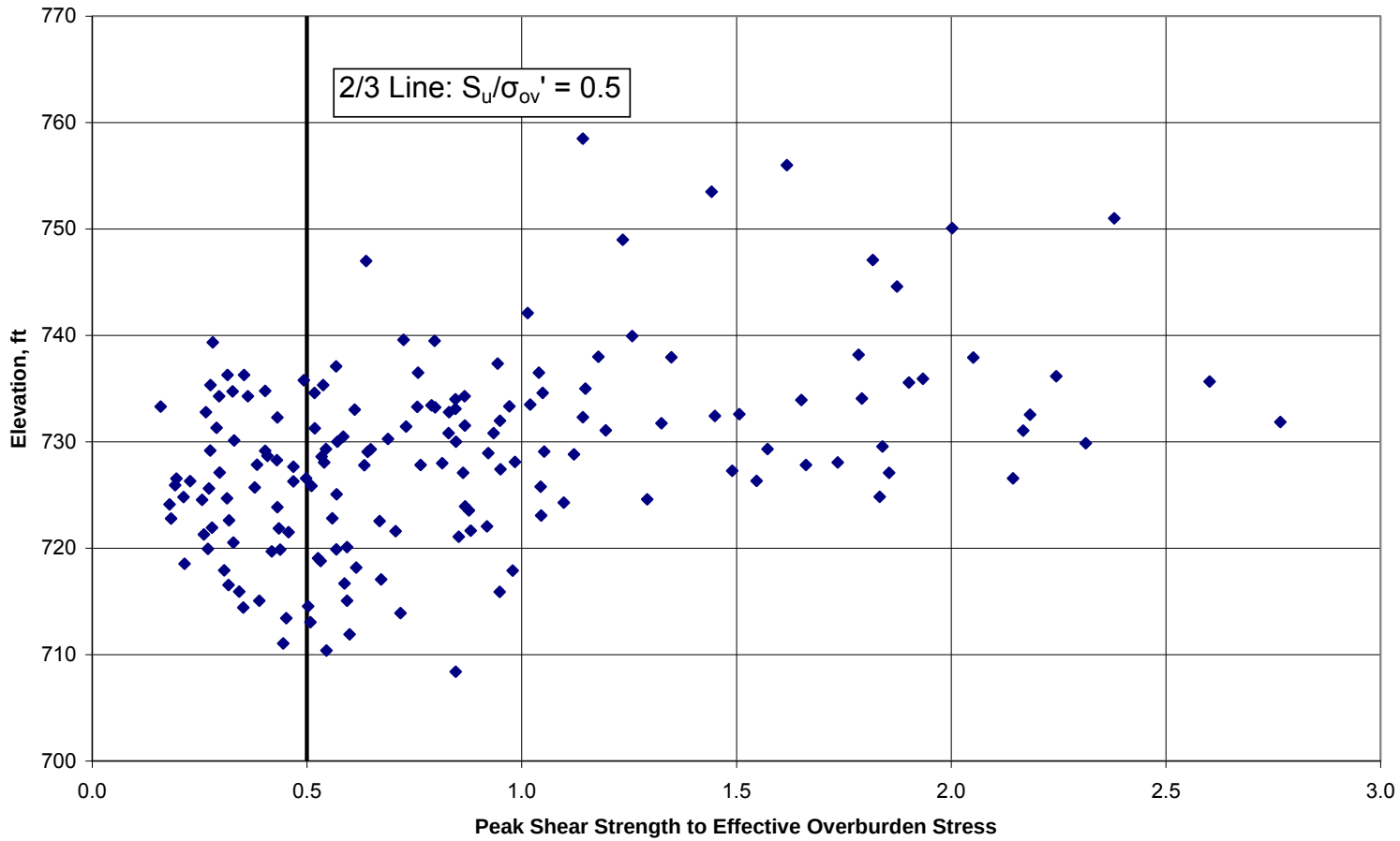


Figure 2.6. Normalized Peak Shear Strength from Field Vane Shear Tests on Lean Clay

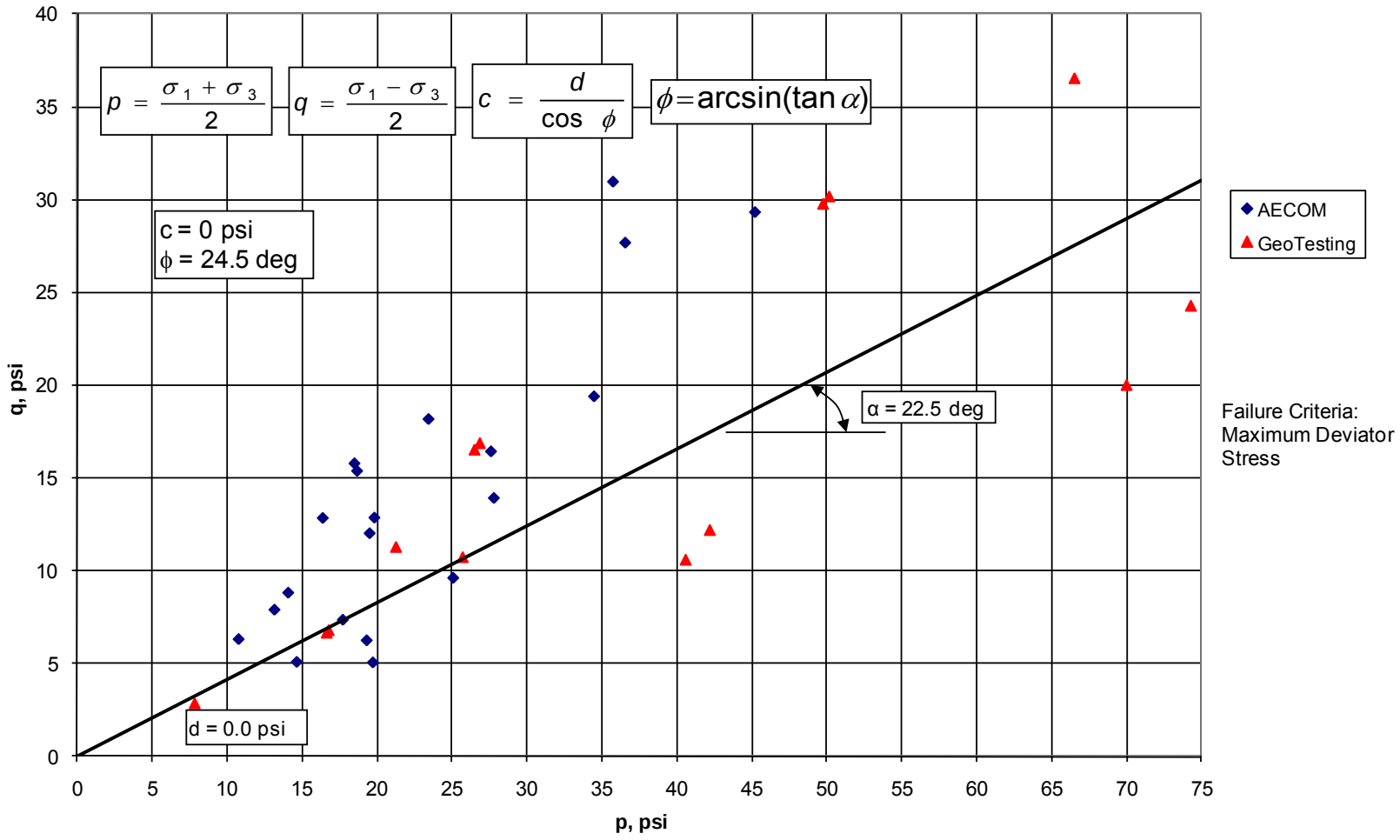


Figure 2.7. Failure Stresses and Strength Envelope from Triaxial Compression Tests on Lean Clay (Undrained Condition)

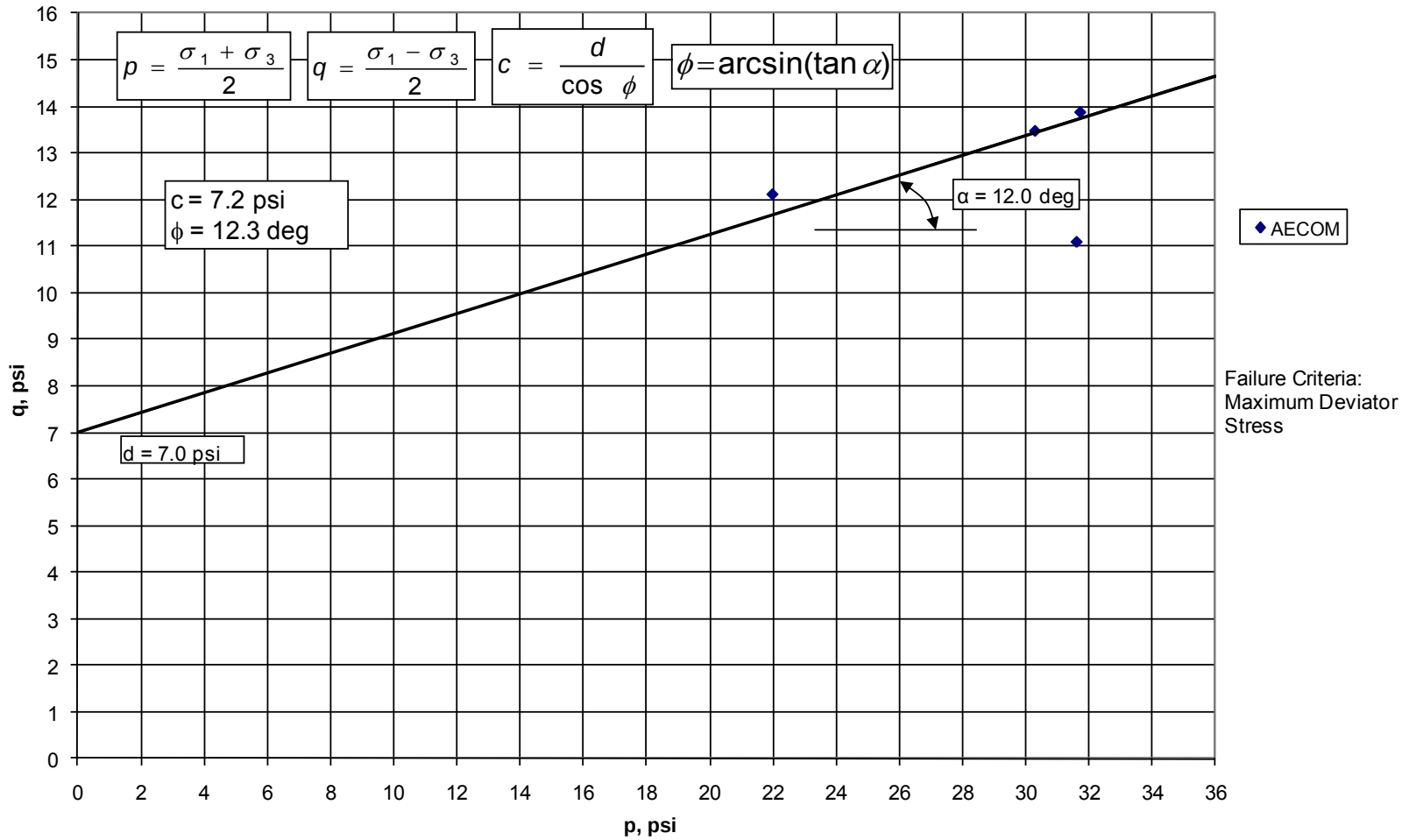


Figure 2.8. Failure Stresses and Strength Envelope from Triaxial Compression Tests on Sandy Silt to Silty Sand (Undrained Condition)

3. Undrained Strengths for Dynamic Conditions

The project site is subject to potentially damaging earthquake ground motions. Dynamic loading during a seismic event will generate excess pore water pressures that could destabilize slopes and trigger an undrained failure. Soils that do not liquefy will generally retain at least 80% of their pre-earthquake undrained shear strength (Makdisi and Seed 1978). Hence, undrained strengths for dynamic, earthquake loading can be conservatively estimated using:

$$c_{EQ} = 0.8c \qquad 3.1$$

$$\tan(\phi_{EQ}) = 0.8\tan(\phi) \qquad 3.2$$

$$\left(\frac{c}{p}\right)_{EQ} = 0.8\left(\frac{c}{p}\right) \qquad 3.3$$

These estimates are applicable only for saturated soils that do not liquefy, such as most clayey soils. In coarse grained soils, using 80% of the static undrained strength is applicable only if the factor of safety against liquefaction is greater than about 1.4.

In unsaturated soils, shear induced volumetric changes are accommodated by the compressibility of the pore air, such that cyclic shearing does not generate significant pore water pressures. Hence, the full undrained strength is assumed to apply for unsaturated soils subjected to seismic loading.

To analyze seismic conditions for the dredge cell closure, the strength parameters listed in Table 3.1 can be used for unliquefied soil zones. These values were computed using Equations 3.1 through 3.3 and the estimates for undrained strength in Table 2.1. Where the earthen berm and landfilled ash are unsaturated (above the phreatic surface), the static undrained strength is not reduced, as indicated in Table 3.1.

Considering the very large particle sizes within the shot rock, elevated pore pressures are not expected during an earthquake. Hence, no reduction in the undrained strength parameters was assumed for this material.

Table 3.1. Shear Strength Parameters for Unliquefied Soil under Seismic Conditions

Material Name	Total Stress Strength Parameters for Non-Liquefied Soil		
	ϕ_{EQ} (deg.)	c_{EQ} (psf)	Notes
Earthen Berm *(above phreatic surface)*	0	1,500	
Earthen Berm *(below phreatic surface)*	0	1,200	
Landfilled Ash *(above phreatic surface)*	30 27	0 1,200	for σ < 17,690 psf for $\sigma \geq$ 17,690 psf
Landfilled Ash *(below phreatic surface)*	25 22	0 960	for σ < 17,690 psf for $\sigma \geq$ 17,690 psf
Hydraulically Placed Ash	8	0	
Sensitive Silt/Clay	S = 0.19, m = 0.71		Use to compute c/p ratio
Lean Clay Foundation Soil	20	0	
Sandy Silt to Silty Sand	25 10	0 800	for σ < 2,760 psf for $\sigma \geq$ 2,760 psf
Coarse Stone	32	0	
Shot Rock	38	0	

4. Residual Strengths for Liquefied Soils

Liquefied soils exhibit a lower undrained shearing resistance due to the generation of excess pore water pressures during cyclic loading. Also known as the steady state strength, the residual shear strength is often quite low, but greater than zero. The shear strength of a liquefied soil is difficult to accurately measure through laboratory testing, and empirical correlations with in situ penetration resistance are often preferred.

Olson and Stark (2002) used a case history dataset to develop correlations between the residual strength ratio and the in situ penetration (SPT or CPT) resistance. The average trend lines through the data are (Olson and Stark 2002):

$$\frac{S_{us}}{\sigma'_v} = 0.03 + 0.0075\left[(N_1)_{60}\right] \pm 0.03 \quad \text{for } (N_1)_{60} \le 12 \qquad 4.1$$

$$\frac{S_{us}}{\sigma'_v} = 0.03 + 0.0143(q_{c1}) \pm 0.03 \quad \text{for } (q_{c1}) \le 6.5 \text{ MPa} \qquad 4.2$$

where $(N_1)_{60}$ is the normalized, corrected SPT blowcount, and q_{c1} is the normalized CPT tip resistance [$q_{c1} = 1.8 \cdot q_c / (0.8 + \sigma'_v/P_a)$]. A significant shortcoming of these correlations is the lack of field data for moderate to large confining pressures, corresponding to $(N_1)_{60} > 12$ and $q_{c1} > 6.5$ MPa.

Idriss and Boulanger (2007) examined the available case history data with the goal of providing better guidance on selecting strength ratios at higher penetration resistance. Among the challenges in developing these correlations, Idriss and Boulanger (2007) note the influence of "void redistribution" in a liquefied deposit, resulting from the migration of pore water within the soil during a liquefaction failure. Using the results from laboratory studies as a guide in extrapolating the case history data, Idriss and Boulanger (2007) proposed the two bounding relationships.

Where significant void redistribution might be expected, Idriss and Boulanger (2007) recommend a lower-bound estimate of residual strength ratio given by:

$$\frac{S_{us}}{\sigma'_v} = \exp\left[\frac{(N_1)_{60cs}}{16} + \left(\frac{(N_1)_{60cs} - 16}{21.2}\right)^3 - 3.0\right] \le \tan\phi' \qquad 4.3$$

Here, $(N_1)_{60CS}$ is the normalized, corrected SPT blowcount with an adjustment for fines content. To avoid inappropriately inflating the liquefied strength, the fines content adjustment should not be added to zero blowcounts ("weight of hammer" or "weight of rod"). The strength ratio is limited by the drained strength (tan ϕ'), based on the assumption that negative pore pressures (inferred by $S_{us} > \sigma' \tan\phi'$) cannot be sustained. Significant void redistribution might be expected at ". . . sites with relatively thick layers of liquefiable soil that are overlain by lower-permeability soils that would impede the post-earthquake dissipation of earthquake-induced excess pore water pressures" (Idriss and Boulanger 2007). For example, Equation 4.3 may be appropriate for a thick layer of liquefied soil overlain by a clayey embankment.

Where void redistribution is judged to be unlikely, Idriss and Boulanger (2007) recommend an upper-bound estimate of residual strength ratio given by:

$$\frac{S_{us}}{\sigma'_v} = \exp\left[\frac{(N_1)_{60cs}}{16} + \left(\frac{(N_1)_{60cs} - 16}{21.2}\right)^3 - 3.0\right]\left[1 + \exp\left(\frac{(N_1)_{60cs}}{2.4} - 6.6\right)\right] \leq \tan\phi' \qquad 4.4$$

This relationship is appropriately used for sites where void redistribution effects will be negligible. “This could include sites where the stratigraphy would not impede post-earthquake dissipation of excess pore water pressures, [which] would be accompanied by densification of the soils at all depths” (Idriss and Boulanger 2007). Where a rockfill dam is built over a relatively thin layer of liquefiable soil, for example, Equation 4.4 may be appropriate.

Residual strengths for the liquefied materials at the Kingston dredge cell are summarized in Table 4.1. These parameters were estimated as follows.

4.1. Earthen Berm

Since the compacted earthen berm will be constructed with cohesive soils and compacted in a controlled manner, it is not considered liquefiable.

4.2. Landfilled Ash

The embankment of landfilled ash will be constructed above the groundwater level and most of this material will remain unsaturated. Suction pore pressures and greater pore fluid compressibility in these zones will increase the liquefaction resistance of the ash, such that most of the landfilled ash will not liquefy in the design earthquake.

The lower reaches of the compacted ash may become saturated due to capillary rise or infiltration. The potential for liquefaction within the zone just above the ground water table is evaluated separately, in Exhibit 12. The residual strength of the hydraulically placed ash (Table 4.1) can be used to represent the compacted ash where it is predicted to liquefy.

4.3. Hydraulically Placed Ash

Results from liquefaction triggering analyses, performed using the available SPT and CPT data, indicate that the hydraulically placed ash will liquefy during the design earthquakes. Thus, residual strengths are needed for this material; estimates were developed using correlations based on SPT and CPT data. Where the prior analysis indicated that liquefaction was unlikely (depths where the factor of safety against liquefaction was greater than 1.4), the measured SPT or CPT penetration resistance was ignored in estimating residual strengths for the liquefied ash.

Correlations between residual strength ratio (S_{us}/σ'_v) and SPT or CPT data developed by Olson and Stark (2002) and Idriss and Boulanger (2007) were used to estimate the residual strength of the hydraulically placed ash material. Void redistribution was judged to be unlikely, so the upper bound Idriss and Boulanger (2007) correlation (Equation 4.4) was used. The results are plotted in Figure 4.1 and Figure 4.2. Taking an average through the data points, these three correlations indicate:

Idriss and Boulanger, upper bound correlation: $S_{us}/\sigma'_v = 0.072$
Olson and Stark, SPT correlation: $S_{us}/\sigma'_v = 0.043$
Olson and Stark, CPT correlation: $S_{us}/\sigma'_v = 0.057$

An average of these results gives a reasonable estimate for a representative, residual strength ratio $S_{us}/\sigma'_v = 0.06$ (Table 4.1).

Application of this ratio results in low to very low shear strengths (S_{us} generally 50 to 320 psf) within the cross section of the landfill closure. Considering the vertical effective stress at three locations, S_{us} = 50 psf at the base of the ash under the toe of the perimeter berm, S_{us} = 140 psf at the top of ash under the centerline of the perimeter berm, and S_{us} = 320 psf at the base of ash deposits under the landfill center (crest elevation 790 ft).

In their investigation of the root cause of the dredge cell failure, AECOM (2009) back-calculated estimates of the residual (steady state) strength of the liquefied ash. Assuming an infinite slope failure in saturated material, the strength ratio can be computed as (Jefferies and Been 2006):

$$\frac{S_{us}}{\sigma'_v} = \frac{\gamma_t}{\gamma_b} \cos\theta \sin\theta \qquad 4.5$$

where γ_t is the total unit weight, γ_b is the buoyant (submerged) unit weight, and θ is the slope angle (measured from horizontal). Based on the difference in surface elevation over two reaches covered by failed ash, AECOM estimated that the final surface slope was in the range of θ = 0.15 to 0.3 degrees. With γ_t = 107 pcf (see Exhibit 3) and γ_b = 45 pcf for the hydraulically placed ash, the strength ratio would be in the range of S_{us}/σ'_v = 0.006 to 0.01. These values are somewhat too low, however, because momentum in the failed mass results in a final slope geometry that is less steep than would be indicated by the soil strength.

AECOM (2009) also estimated the steady state strength of the ash using data from laboratory testing, as show in Figure 4.5. AECOM estimated S_{us} = 100 psf from the steady state line at a void ratio of 0.88. Using a void ratio of 0.85, as selected here for the hydraulically placed ash (Exhibit 3), the steady state line in Figure 4.5 indicates a residual strength of about 350 psf. Both of these numbers are in very good agreement with the values computed above, using $S_{us}/\sigma'_v = 0.06$ and the expected vertical effective stresses.

4.4. Sensitive Silt/Clay

The residual strength ratio in this material is assumed to be the same as in the overlying, hydraulically placed ash ($S_{us}/\sigma'_v = 0.06$). Since the sensitive silt/clay is encountered only in some borings, is relatively thin (< 1 foot), and has a residual strength equal to the overlying soil, this material will have a negligible effect on the post-earthquake stability.

Based on the laboratory data presented in Appendix C, the sensitive silt/clay exhibits plasticity and, thus, may not liquefy in the design earthquake. The unliquefied sensitive silt/clay would have an undrained strength equal to the values from Section 2.4, reduced to account for pore pressure generation. However, rather than try to prove this thin layer does not liquefy, it is simpler to conservatively assume liquefaction and assign residual

strength properties. As long as the shearing resistance assigned to the sensitive silt/clay is not less than that in the overlying ash, the sensitive layer will not affect stability. That is, deep failure surfaces will simply pass through the liquefied ash above the thin sensitive layer.

4.5. Lean Clay Foundation Soil

A liquefaction screening assessment was performed for the lean clay foundation soil based on measured Atterberg limits and natural water contents. The results indicate that the lean clay is unlikely to liquefy.

4.6. Sandy Silt to Silty Sand

Liquefaction triggering analyses indicate the sandy silt to silty sand material will likely liquefy during the design earthquakes. A residual strength ratio was estimated from the SPT and CPT data, using the correlations from Olson and Stark (2002) and Idriss and Boulanger (2007). Where prior analyses indicated liquefaction was unlikely (factor of safety against liquefaction greater than 1.4), the data were ignored in estimating liquefied strengths. Because this deposit is overlain with a relatively impervious soil, significant void redistribution effects are possible; hence, the lower bound correlation (Equation 4.3) from Idriss and Boulanger (2007) was used.

The results are plotted in Figure 4.3 and Figure 4.4. Taking an average through the data points, these three correlations indicate:

Idriss and Boulanger, lower bound correlation:	$S_{us}/\sigma'_v = 0.072$
Olson and Stark, SPT correlation:	$S_{us}/\sigma'_v = 0.050$
Olson and Stark, CPT correlation:	$S_{us}/\sigma'_v = 0.056$

An average of these results gives a reasonable estimate for a representative, residual strength ratio $S_{us}/\sigma'_v = 0.06$ (Table 4.1).

4.7. Coarse Stone and Shot Rock

These materials will consist of angular, quarried rock. Even with moderate to little compactive effort, liquefaction is highly unlikely in these materials.

Table 4.1. Shear Strength Parameters for Liquefied Soil under Seismic Conditions

Material Name	Total Stress Strength of Liquefied Soil
Earthen Berm	Not liquefiable
Landfilled Ash	Not liquefiable
Hydraulically Placed Ash	$S_{us}/\sigma'_v = 0.06$
Sensitive Silt/Clay	$S_{us}/\sigma'_v = 0.06$
Lean Clay Foundation Soil	Not liquefiable
Sandy Silt to Silty Sand	$S_{us}/\sigma'_v = 0.06$
Coarse Stone	Not liquefiable
Shot Rock	Not liquefiable

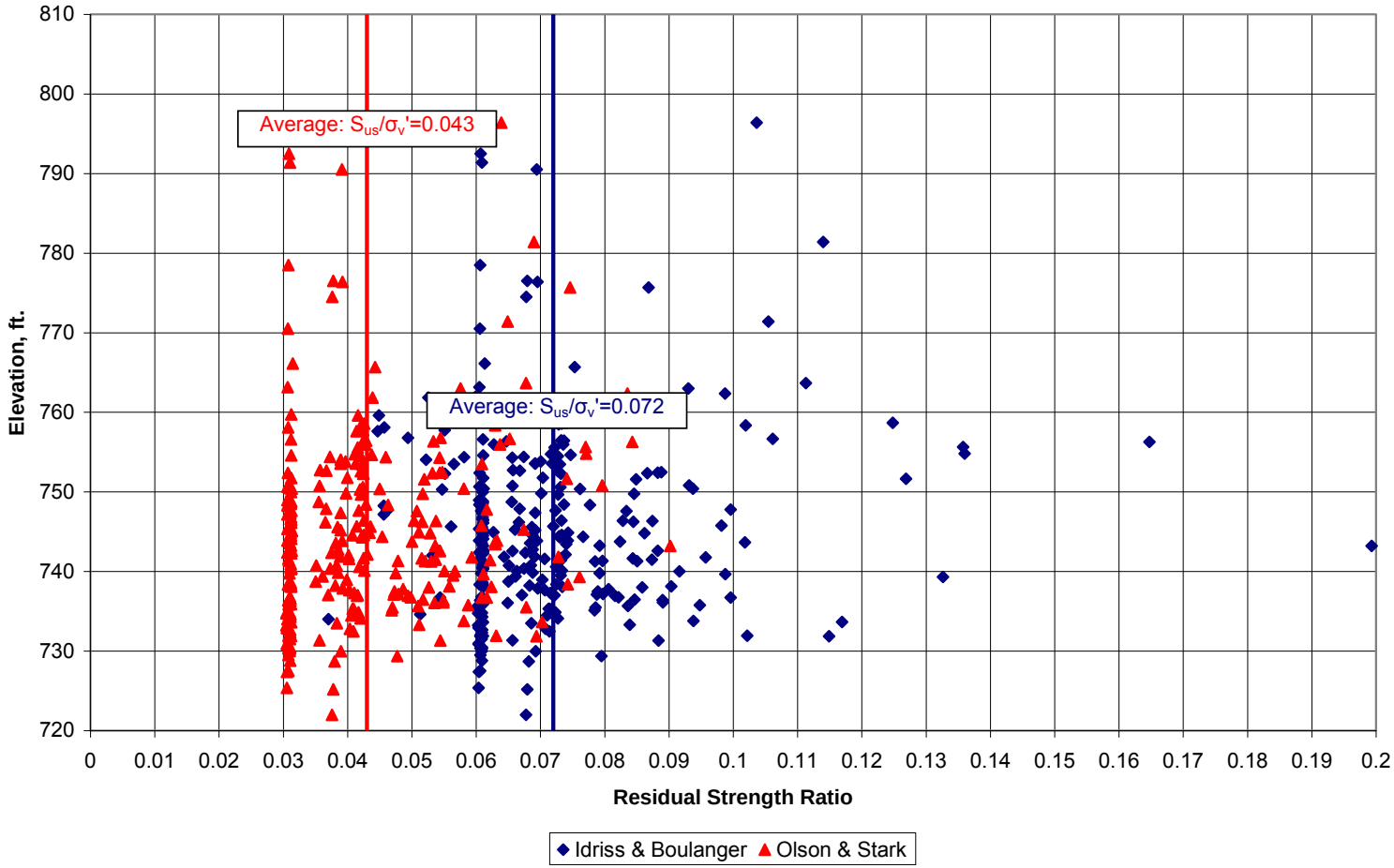


Figure 4.1. Residual Strength Ratio – Hydraulically Place Ash (SPT Data)

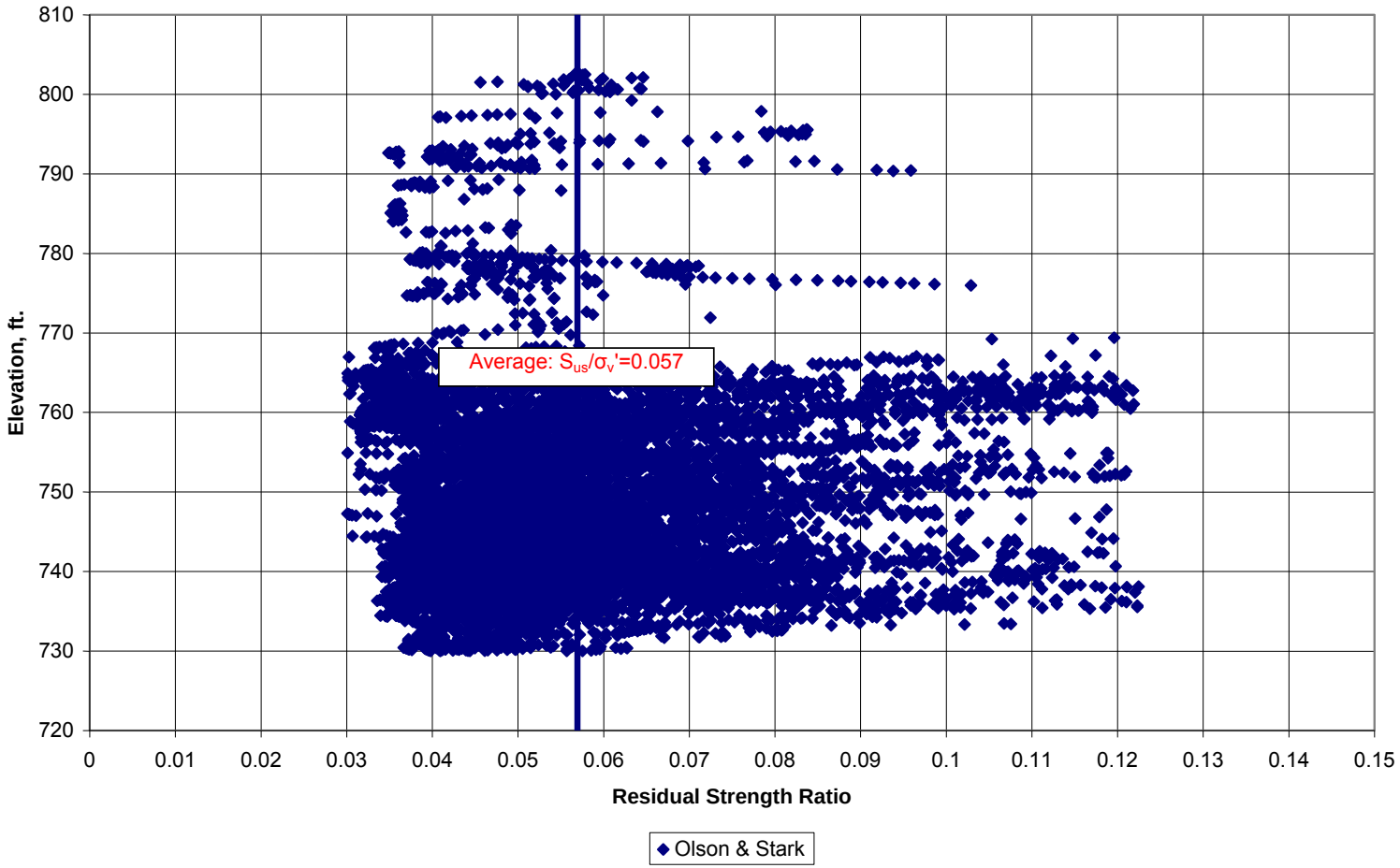


Figure 4.2. Residual Strength Ratio – Hydraulically Place Ash (CPT Data)

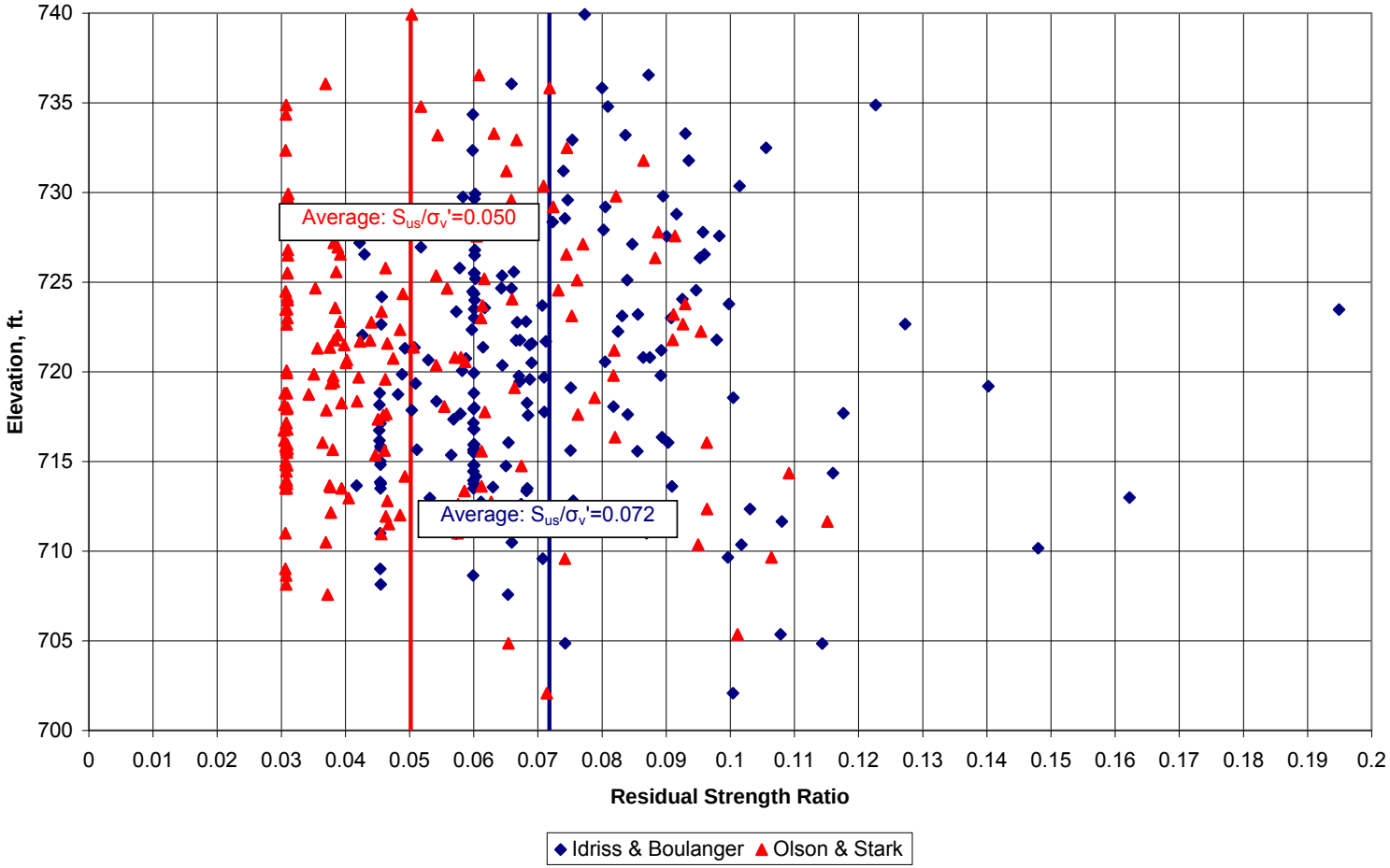


Figure 4.3. Residual Strength Ratio – Sandy Silt to Silty Sand (SPT Data)

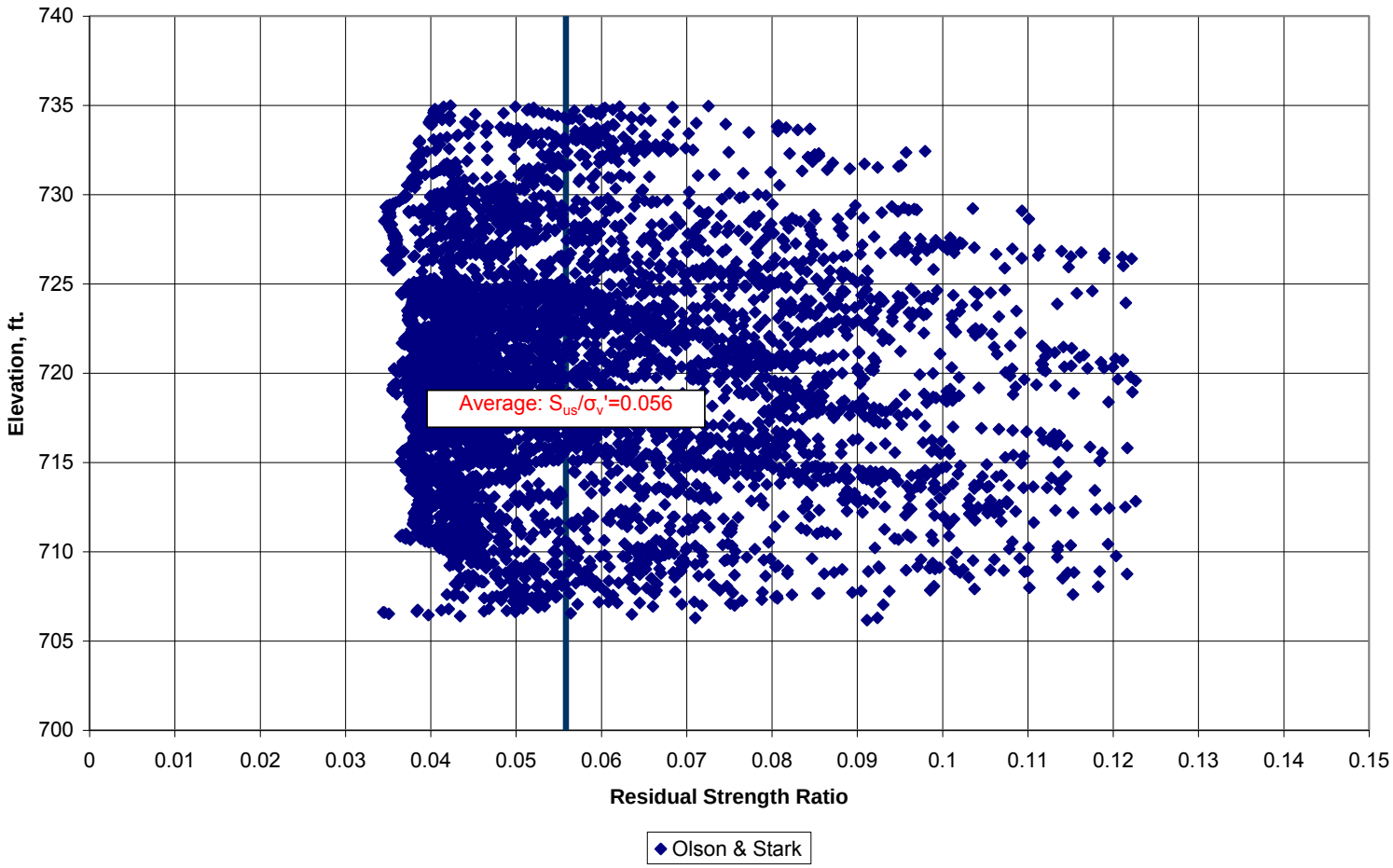


Figure 4.4. Residual Strength Ratio – Sandy Silt to Silty Sand (CPT Data)

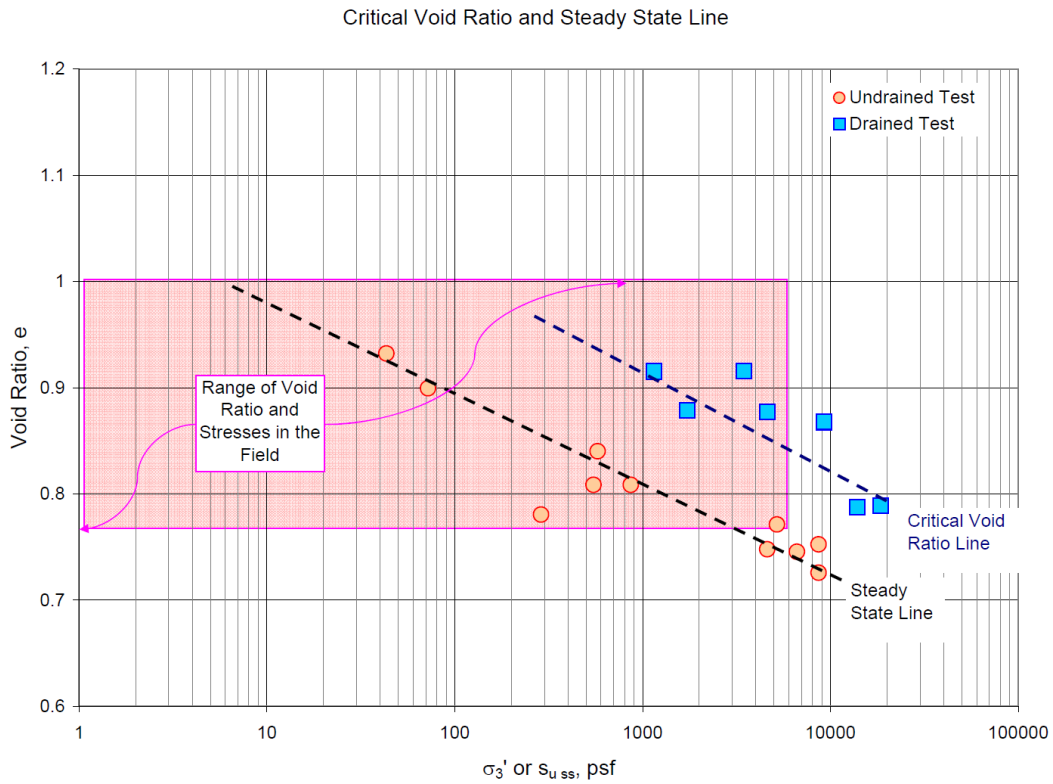


Figure 4.5. Critical void ratio and steady state line for hydraulically placed ash (from AECOM 2009)

5. Additional Soil Properties used in *FLAC*

5.1. Tensile Strength

A zero tensile strength is specified for all soil and ash materials in the *FLAC* model. In the Mohr-Coulomb model, a zero tension cutoff limits the minor principal effective stress to a minimum value of zero. If a grid zone in the *FLAC* model is subjected to a tensile strain, the minor principal stress is reduced to zero and remains constant. The other principal stresses are not modified.

5.2. Dilation Angle

The dilation angle is a parameter used in the Mohr-Coulomb constitutive model to predict volumetric strains caused by shearing. Relating the resulting volumetric strain to the induced, plastic shear strain at failure, the dilation angle is defined as:

$$\sin\psi = \frac{-\delta\varepsilon_s}{\delta\varepsilon_t} \qquad 5.1$$

where ψ is the angle of dilation, $\delta\varepsilon_s$ is the increment of volumetric strain, and $\delta\varepsilon_t$ is the increment of shear strain. A value of $\psi = \phi$ corresponds to an associated flow rule, which generally predicts excessive volumetric expansion (dilation) in particulate materials. With a value of $\psi = 0$, the model would predict no volume change when the material is failed in pure shear (no change in mean stress). Smaller values of ψ are appropriate for modeling less dilative (more contractive) soils.

Dilation angles selected for the dredge cell materials are summarized in Table 5.1. For the hydraulically placed ash, the dilation angle was determined from the laboratory data in Appendix B. The ratio of the incremental volume strain and the incremental shear strain at the peak shear stress was computed using data from each of the triaxial compression tests. The calculated angle of dilation for four of the five tests was -1.7°. The calculated value for the other test at the peak stress was -4.8°, but this result was deemed unreliable and ignored.

Lacking drained triaxial and direct shear test data for the remaining soils, estimated values of dilation angle were obtained using direct shear test data published by Lamb and Whitman (1979). Equivalent values of the dilation angle can be estimated from direct shear testing by equating the dilation angle to the difference of the peak (ϕ) and ultimate (ϕ_{cv}) friction angles:

$$\psi = \phi - \phi_{cv} \qquad 5.2$$

5.3. Progressive Liquefaction Model

When a contractive, saturated soil is subjected to cyclic loading, the tendency of the soil particles to move into a denser arrangement generates excess pore water pressures. Stresses on the soil matrix are transferred to the pore water, with a corresponding drop in effective stress. Liquefaction occurs when these excess pore pressures become large enough for the mean effective stress to approach zero, with a loss of shearing resistance to a residual value.

From a modeling perspective, this behavior is difficult to capture without the use of complex constitutive models; most such models are not widely accepted and require extensive dynamic laboratory testing to determine input parameters. The Mohr-Coulomb model used in the *FLAC* dynamic simulations does not account for pore pressure generation directly, but an add-on feature (FINN model) is available in *FLAC* to complement the constitutive model. The FINN model simulates pore pressure generation based on volumetric strain, as a function of cyclic shear strain and standard penetration test values. While this model is simple in principal and does not directly model the mechanism that causes pore pressure generation, the FINN model can predict the progressive liquefaction of a soil through the correlation of pore pressure generation to volumetric strain.

The Byrne (1991) formulation is used to implement the FINN model in *FLAC* and is defined as follows:

$$\frac{\Delta\varepsilon_{vd}}{\gamma} = C_1 \exp\left[-C_2\left(\frac{\varepsilon_{vd}}{\gamma}\right)\right] \qquad 5.3$$

where ε_{vd} is irrecoverable volume strain, γ is the cyclic shear-strain amplitude, and C_1 and C_2 are constants:

$$C_1 = 7600\left(D_r\right)^{-2.5} \qquad 5.4$$

$$C_2 = \frac{0.4}{C_1} \qquad 5.5$$

The relative density (D_r) can be estimated using an empirical relationship with normalized standard penetration test values, $(N_1)_{60}$.

$$D_r = 15\ \left[\left(N_1\right)_{60}\right]^{0.5} \qquad 5.6$$

The constant C_1 can then be expressed as:

$$C_1 = 8.7\ \left[\left(N_1\right)_{60}\right]^{-1.25} \qquad 5.7$$

The model assumes an undrained condition (i.e., the duration of the earthquake is much shorter than the time required to dissipate pore pressures) and therefore the change in pore pressure is directly related to the change in volumetric strain.

$$\Delta p = -\alpha\, M\, \Delta\varepsilon \qquad 5.8$$

where Δp is the change in pore pressure, α is the Biot coefficient (= 1 for soil). M is the Biot modulus, equal to the bulk modulus of water (2,000 MPa or 42,000 ksf) divided by the porosity of the soil.

The FINN model does not directly account for loss in shear strength as a result of pore pressure generation. Itasca Consulting Inc. (developers of the *FLAC* software) was retained as a sub-consultant to assist in generating code that would reduce the shear strength as a function of excess pore water pressure. This program stores the initial mean effective stress

(static state) for each zone, and compares it to the excess pore water pressure in that zone. When the ratio of excess pore water pressure to initial mean effective stress reaches 0.5, the post liquefaction strength is assigned to that zone. By definition, liquefaction occurs when this ratio reaches one (i.e., the excess pore water pressure is equal to the initial mean effective stress). However, due to limitations in the approach used to model pore water pressure generation, this threshold was set to 0.5, as recommended by Itasca, to ensure a conservative result.

5.4. Parameters for the Liquefaction Model

To implement the FINN model for progressive liquefaction, representative SPT blowcounts are needed for each liquefiable soil. Only three soils (hydraulically placed ash, sensitive silt/clay, and the sandy silt to silty sand) are treated as liquefiable in the analysis of the dredge cell closure.

SPT blowcounts measured in the hydraulically placed ash are plotted in Figure 5.1. This data, obtained from 46 borings completed by AECOM in the dredge cell, do not exhibit a trend with respect to elevation. Most of the $(N_1)_{60}$ values are less than about 10, with sporadic values in excess of 30, as indicated in the histogram at the top of Figure 5.1. The higher values are not representative of the general response of the ash deposits, and should be ignored in selecting design parameters. Neglecting data in the ash where $(N_1)_{60} > 10$, the average blowcount in this deposit is 3.1, as shown on Figure 5.1. For the FINN model, a value of $(N_1)_{60} = 3$ is selected for the hydraulically placed ash.

The sensitive silt/clay is too thin to obtain SPT blowcount data. A value of $(N_1)_{60} = 3$ is assumed, based on the value selected for the hydraulically placed ash.

SPT blowcounts in the sandy silt to silty sand, obtained from 44 AECOM borings, are plotted in Figure 5.2. The data in this deposit show a trend of increasing maximum blowcounts with depth, but also a significant number of very low blowcounts. Hence, the higher measured values will unduly influence a calculated average without representing the overall behavior of the deposit. Here, values of $(N_1)_{60} > 20$ were ignored, resulting in an average blowcount of 6.5 computed in this deposit (Figure 5.2). For the FINN model, a value of $(N_1)_{60} = 7$ is selected for the sandy silt to silty sand.

These values are consistent with the densities (Exhibit 3) and drained strength parameters (Section 1) selected to represent these three materials.

Table 5.1. Dilation Angles for Volumetric Response of Modeled Materials

Material Name	Angle of Dilation, Ψ (deg.)
Earthen Berm	2.0
Landfilled Ash	2.0
Hydraulically Placed Ash	-1.7
Sensitive Silt/Clay	2.0
Lean Clay Foundation Soil	2.0
Sandy Silt to Silty Sand	3.0
Stabilized Foundation Ash	2.0
Stabilized Foundation Clay	2.0
Stabilized Foundation Sand	3.0
Coarse Stone	6.0
Shot Rock	6.0

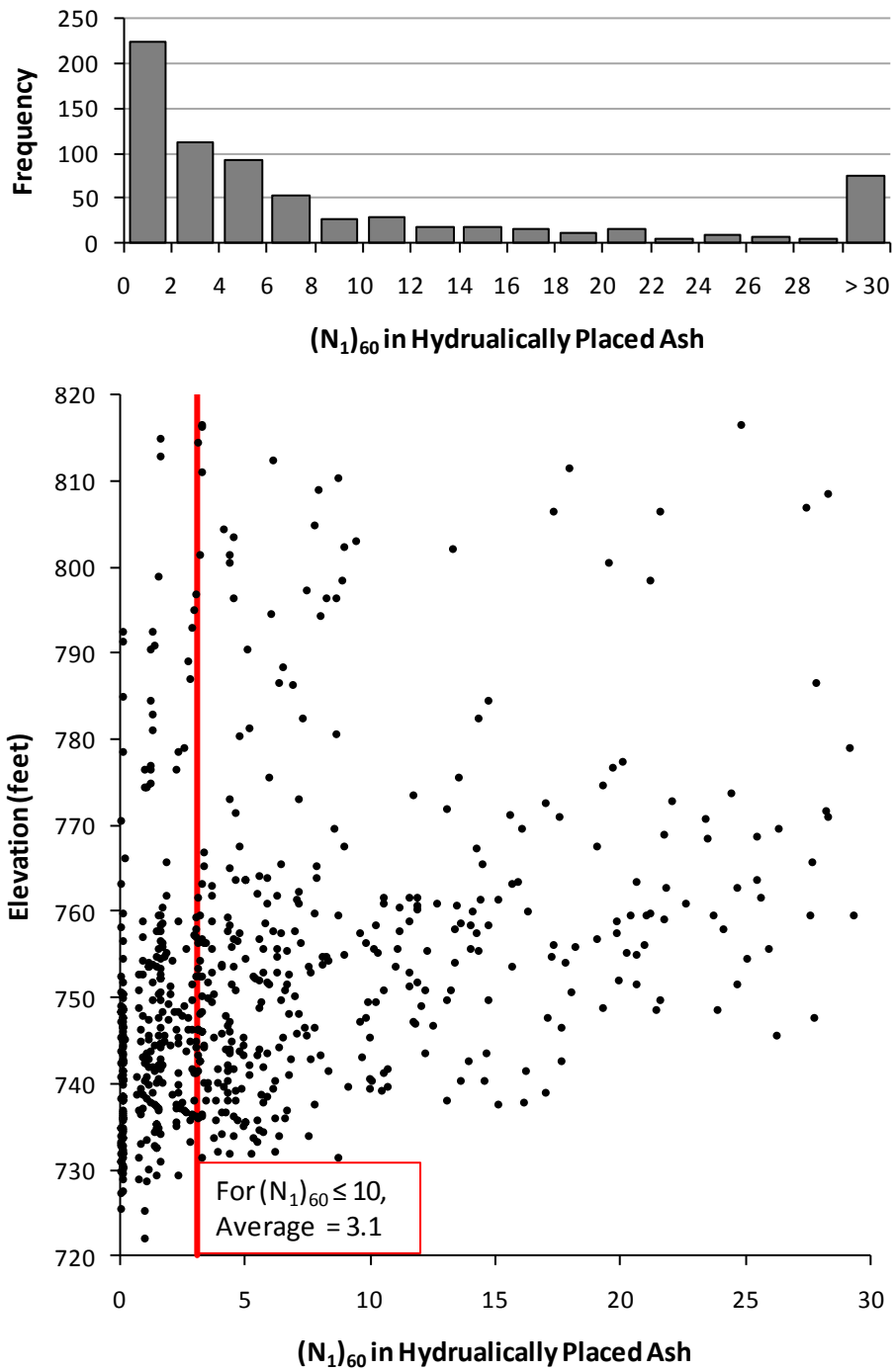


Figure 5.1. SPT Blowcounts Measured in the Hydraulically Placed Ash

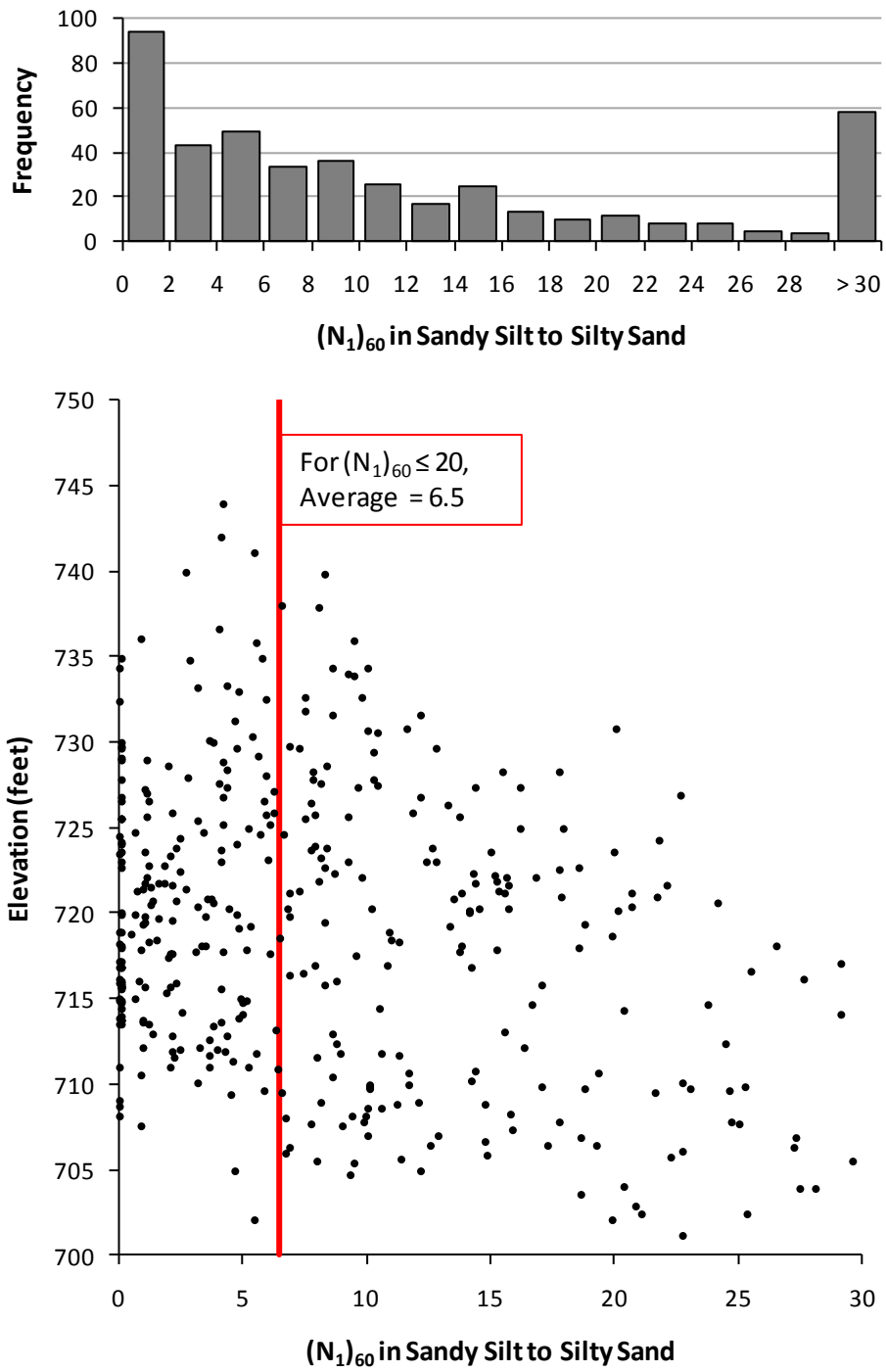


Figure 5.2. SPT Blowcounts Measured in the Sandy Silt to Silty Sand

6. References

AECOM (2009). "Root Cause Analysis of TVA Kingston Dredge Pond Failure on December 22, 2008." Four volumes, June 25.

Byrne, P. M. (1991). "A Cyclic Shear-Volume Coupling and Pore-Pressure Model for Sand." *Proc., Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, Paper No. 1.24,* St. Louis, Missouri, March, pp. 47-55.

Duncan, J. M, and Wright, S. G. (2005). *Soil Strength and Slope Stability*. John Wiley and Sons, Hoboken, New Jersey.

Idriss, I. M., and Boulanger, R. W. (2007). "Residual Shear Strength of Liquefied Soils." *Modernization and Optimization of Existing Dams and Reservoirs, Proc., 27th Annual USSD Conf,* United States Society on Dams, Philadelphia, March 5-9, pp. 621-634.

Ishihara, K. (1996). *Soil Behavior in Earthquake Geotechnics.* Clarendon Press, Oxford.

Jefferies, M., and Been, K. (2006). *Soil Liquefaction: A Critical State Approach.* Taylor and Francis, New York.

Lamb, W. T., and Whitman, R. V. (1979). *Soil Mechanics, SI Version*. John Wiley & Sons.

Makdisi, F. I., and Seed, H. B. (1978). "Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations." *J. Geotechnical Engineering Div.*, ASCE, Vol. 104, No. GT7, July, pp. 849-867.

Olson, S. M., and Stark, T. D. (2002). "Liquefied Strength Ratio from Liquefaction Flow Failure Case Histories." *Canadian Geotechnical J.*, Vol. 39, pp. 629-647.

Terzaghi, K., Peck, R. B., and Mesri, G. (1996). *Soil Mechanics in Engineering Practice.* 3rd ed., John Wiley and Sons, Hoboken, New Jersey.

U. S. Navy (1986). "Soil Mechanics." Design Manual 7.01, Naval Facilities Engineering Command, Alexandria, Virginia.

Appendix A

Test Data – Landfilled Ash

Consolidated Undrained Triaxial Test

ASTM D4767-04

Project Name	TVA Kingston Plant Dike Failure	Project Number	171468117
Sample Identification	Kingston Fly Ash @ 90%	Test Number	CU-2A
Visual Description	Silt (ML), gray	Prepared By	JAM
		Date	4-2-2009

Compacted — Maximum Dry Density (pcf) 82.8 — Compacted MC% 26.9

Specific Gravity 2.31 — Liquid Limit NP — Plastic Limit NP — Plasticity Index NP

Initial Specimen Data

Specimen Diameter (in.)		Specimen Height (in.)		Volumes (in^3)		Specimen	
Top	2.889	1	6.043	Sample	39.3513 (V_o)	Wet Weight (g)	963.05
Middle	2.879	2	6.033	Solids	20.1718 (VS_o)	Dry Weight (g)	763.63
Bottom	2.879	3	6.020	Water	12.1686 (Vw_o)	Wet Unit Weight (pcf)	93.2
Avg.	2.8823 (D_o)	4	6.029	Voids	19.1795 (Vv_o)	Dry Unit Weight (pcf)	73.9
Area (in^2)	6.5250 (A_o)	Avg. (H_o)	6.0309	Degree of Saturation (%)	63.4 (S_o)	Percent of Maximum	89.3
Moisture Content (%)	26.1	Final Trimmings		Void Ratio	0.951		

Saturation

Set Up & Saturated: Wet xx Dry ____ — Set up By KDG

Back Pressure Saturated to: 85 (psi) — Final Pore Pressure Parameter B 0.96 — Date 4-6-09

Panel Board Number C

Height Readings (in.)		Back Pressure Burette		Chamber Burette			
Initial	0.2799	Initial	17.34 (in.)	Initial	14.85 (in.)	Specimen Height (in.)	6.0337 (H_s)
Final	0.2771	Final	8.31 (in.)	Final	11.78 (in.)	Area (in^2) Method A	6.5310 (A_s)
Change	0.0028 (ΔH_o)	Change	-9.03 (in.)	Change	-3.07 (in.)	Specimen Volume (in^3)	39.41 (Vs)

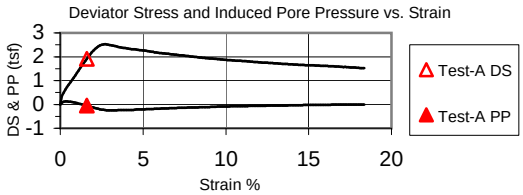
Consolidation

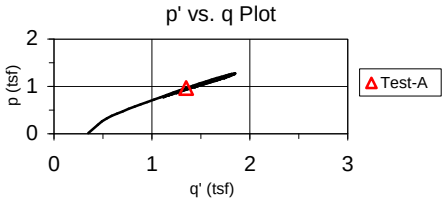
Height Readings (in.)		Back Pressure Burette Readings		Chamber Burette Readings		Pressures (psi)	
Initial	0.2771	Initial	1.19 (in.)	Initial	17.75 (in.)	Chamber	90
Final	0.2812	Final	2.42 (in.)	Final	13.59 (in.)	Back	85
Change	-0.0041 (ΔH_c)	Change	-1.23 (in.)	Change	-4.16 (in.)	Lateral	5 (σ_3)
Height (in.)	6.0296 (H_c)			Volume (in^3)	37.4568 (V_c)		
Area (in^3) Method B	6.2122 (A_c)			Volume - Water (in^3)	17.2851 (VWc)	t_{50} (min.)	0.033
Diameter (in.)	2.8124 (D_c)			Water Content (%)	37.1		
Dry Density (pcf)	77.7			Degree of Saturation (%)	100.0 (S_c)	Void Ratio	0.857

After Test

Final Measurements			Final Moisture Content		Stresses (membrane corrected) at Failure (psi)		
Maximum Diameter	3.477	(in.)	Wet Weight (g)	1046.90	Corrected Deviator	1.91	σ_d (tsf)
Wet weight (g)	1046.9	(WWf)	Dry Weight (g)	763.63	Major Principal	2.32	$\sigma_{1'f}$ (tsf)
Corrected Diameter	3.453	(in.)	Tare Weight (g)	0.00	Minor Principal	0.39	$\sigma_{3'f}$ (tsf)
					Rate of Strain (% / min.)	0.275	
Youngs Modulus for Membrane (psi)	200				Axial Strain at Failure (%)	1.60	
Membrane Thickness (in.)	0.012				Failure Criterion:	Maximum Effective Principal Stress Ratio	

Failure Sketch





Comments:

Consolidated Undrained Triaxial Test

ASTM D4767-04

Project Name	TVA Kingston Plant Dike Failure	Project Number	171468117
Sample Identification	Kingston Fly Ash @ 90%	Test Number	CU-2B
Visual Description	Silt (ML), gray	Prepared By	JAM
		Date	4-2-2009

Compacted — Maximum Dry Density (pcf) 82.8 — Compacted MC% 26.9

Specific Gravity 2.31 — Liquid Limit NP — Plastic Limit NP — Plasticity Index NP

Initial Specimen Data

Specimen Diameter (in.)		Specimen Height (in.)		Volumes (in3)		Specimen	
Top	2.901	1	6.023	Sample	39.4745 (V_o)	Wet Weight (g)	967.87
Middle	2.883	2	6.031	Solids	20.2013 (VS_o)	Dry Weight (g)	764.75
Bottom	2.883	3	5.986	Water	12.3944 (Vw_o)	Wet Unit Weight (pcf)	93.4
Avg.	2.8890 (D_o)	4	6.049	Voids	19.2732 (Vv_o)	Dry Unit Weight (pcf)	73.8
Area (in^2)	6.5552 (A_o)	Avg. (H_o)	6.0219	Degree of Saturation (%)	64.3 (S_o)	Percent of Maximum	89.1
Moisture Content (%)	26.6	Final Trimmings		Void Ratio	0.954		

Saturation

Set Up & Saturated: Wet xx Dry ____ — Set up By KDG

Back Pressure Saturated to: 75 (psi) — Final Pore Pressure Parameter B 0.98 — Date 4-6-09

Panel Board Number E

Height Readings (in.)		Back Pressure Burette		Chamber Burette			
Initial	0.2435	Initial	16.91 (in.)	Initial	12.91 (in.)	Specimen Height (in.)	6.0214 (H_s)
Final	0.244	Final	6.55 (in.)	Final	9.85 (in.)	Area (in^2) Method A	6.5541 (A_s)
Change	-0.0005 (ΔH_o)	Change	-10.36 (in.)	Change	-3.06 (in.)	Specimen Volume (in^3)	39.46 (V_s)

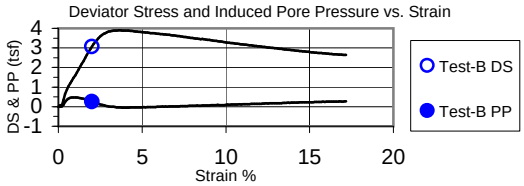
Consolidation

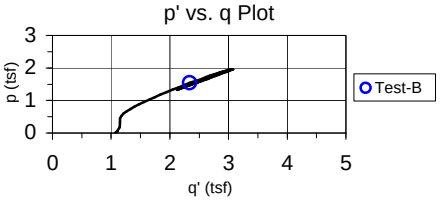
Height Readings (in.)		Back Pressure Burette Readings		Chamber Burette Readings		Pressures (psi)	
Initial	0.244	Initial	0.95 (in.)	Initial	17.87 (in.)	Chamber	90
Final	0.2448	Final	2.31 (in.)	Final	16.61 (in.)	Back	75
Change	-0.0008 (ΔH_c)	Change	-1.36 (in.)	Change	-1.26 (in.)	Lateral	15 (σ_3)
Height (in.)	6.0206 (H_c)			Volume (in^3)	37.7873 (V_c)		
Area (in^3) Method B	6.2764 (A_c)			Volume - Water (in^3)	17.5860 (VWc)	D_{50} (min.)	0.033
Diameter (in.)	2.8269 (D_c)			Water Content (%)	37.7		
Dry Density (pcf)	77.1			Degree of Saturation (%)	100.0 (S_c)	Void Ratio	0.871

After Test

Final Measurements			Final Moisture Content		Stresses (membrane corrected) at Failure (psi)		
Maximum Diameter	3.341	(in.)	Wet Weight (g)	1052.95	Corrected Deviator	3.07	σ_d (tsf)
Wet weight (g)	1052.95	(WWf)	Dry Weight (g)	764.75	Major Principal	3.88	$\sigma_1'_f$ (tsf)
Corrected Diameter	3.317	(in.)	Tare Weight (g)	0.00	Minor Principal	0.80	$\sigma_3'_f$ (tsf)
					Rate of Strain (% / min.)	0.219	
Youngs Modulus for Membrane (psi)	200				Axial Strain at Failure (%)	2.00	
Membrane Thickness (in.)	0.012				Failure Criterion:	Maximum Effective Principal Stress Ratio	

Failure Sketch





Comments:

Consolidated Undrained Triaxial Test

ASTM D4767-04

Project Name	TVA Kingston Plant Dike Failure	Project Number	171468117
Sample Identification	Kingston Fly Ash @ 90%	Test Number	CU-2C
Visual Description	Silt (ML), gray	Prepared By	JAM
		Date	4-1-2009

Compacted — Maximum Dry Density (pcf) 82.8 — Compacted MC% 26.9

Specific Gravity 2.31 — Liquid Limit NP — Plastic Limit NP — Plasticity Index NP

Initial Specimen Data

Specimen Diameter (in.)		Specimen Height (in.)		Volumes (in^3)		Specimen	
Top	2.886	1	5.978	Sample	38.9619 (V_o)	Wet Weight (g)	958.68
Middle	2.881	2	5.999	Solids	19.9771 (VS_o)	Dry Weight (g)	756.26
Bottom	2.873	3	5.973	Water	12.3516 (Vw_o)	Wet Unit Weight (pcf)	93.7
Avg.	2.8800 (D_o)	4	5.975	Voids	18.9848 (Vv_o)	Dry Unit Weight (pcf)	73.9
Area (in^2)	6.5144 (A_o)	Avg. (H_o)	5.9809	Degree of Saturation (%)	65.1 (S_o)	Percent of Maximum	89.3
Moisture Content (%)	26.8	Final Trimmings		Void Ratio	0.950		

Saturation

Set Up & Saturated: Wet xx Dry _____ — Set up By KDG

Back Pressure Saturated to: 60 (psi) — Final Pore Pressure Parameter B 0.97 — Date 4-7-09

Panel Board Number D

Height Readings (in.)		Back Pressure Burette		Chamber Burette			
Initial	0.2585	Initial	16.86 (in.)	Initial	15.08 (in.)	Specimen Height (in.)	5.9858 (H_s)
Final	0.2536	Final	9.55 (in.)	Final	13.06 (in.)	Area (in^2) Method A	6.5251 (A_s)
Change	0.0049 (ΔH_o)	Change	-7.31 (in.)	Change	-2.02 (in.)	Specimen Volume (in^3)	39.06 (V_s)

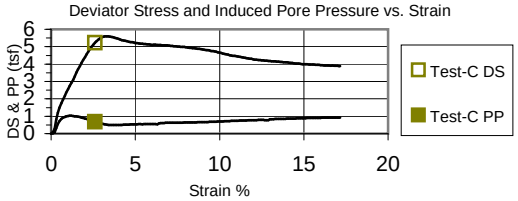
Consolidation

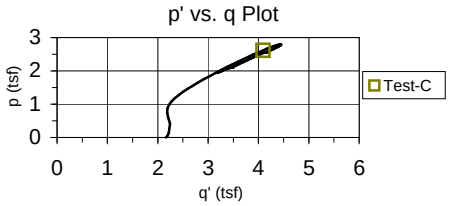
Height Readings (in.)		Back Pressure Burette Readings		Chamber Burette Readings		Pressures (psi)	
Initial	0.2536	Initial	1.23 (in.)	Initial	17.7 (in.)	Chamber	90
Final	0.2758	Final	3.61 (in.)	Final	14.05 (in.)	Back	60
Change	-0.0222 (ΔH_c)	Change	-2.38 (in.)	Change	-3.65 (in.)	Lateral	30 (σ_3)
Height (in.)	5.9636 (H_c)			Volume (in^3)	37.2256 (V_c)		
Area (in^3) Method B	6.2422 (A_c)			Volume - Water (in^3)	17.2485 (VWc)	D_{50} (min.)	0.049
Diameter (in.)	2.8192 (D_c)			Water Content (%)	37.4		
Dry Density (pcf)	77.4			Degree of Saturation (%)	100.0 (S_c)	Void Ratio	0.863

After Test

Final Measurements			Final Moisture Content		Stresses (membrane corrected) at Failure (psi)		
Maximum Diameter	3.41	(in.)	Wet Weight (g)	1038.93	Corrected Deviator	5.23	σ_d (tsf)
Wet weight (g)	1038.93	(WWf)	Dry Weight (g)	756.26	Major Principal	6.71	$\sigma_1'_f$ (tsf)
Corrected Diameter	3.386	(in.)	Tare Weight (g)	0.00	Minor Principal	1.47	$\sigma_3'_f$ (tsf)
					Rate of Strain (% / min.)	0.275	
Youngs Modulus for Membrane (psi)	200				Axial Strain at Failure (%)	2.61	
Membrane Thickness (in.)	0.012				Failure Criterion:	Maximum Effective Principal Stress Ratio	

Failure Sketch



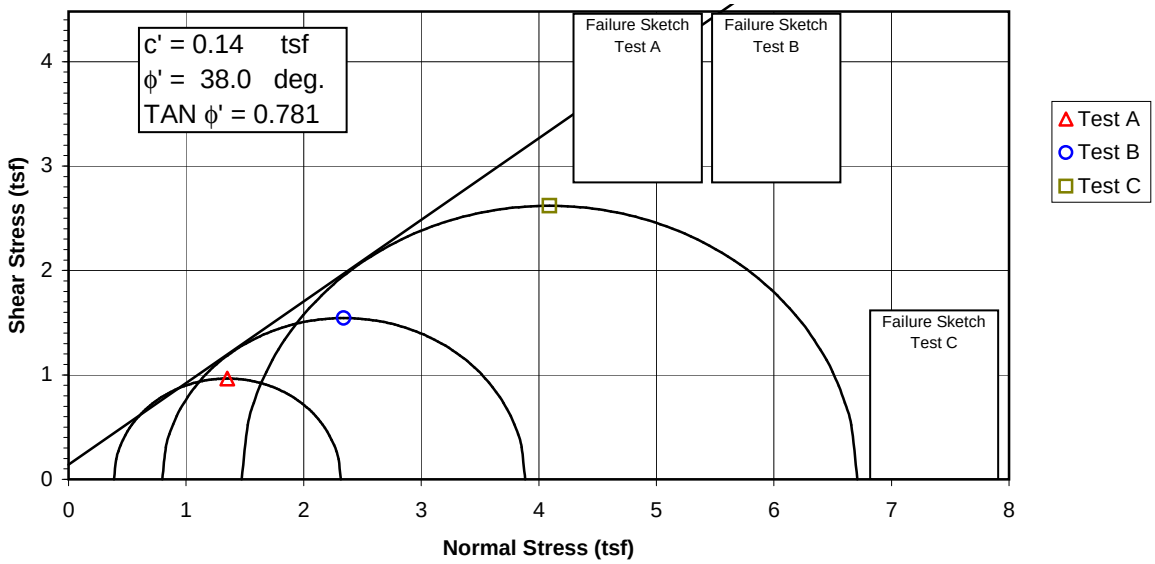


Comments:

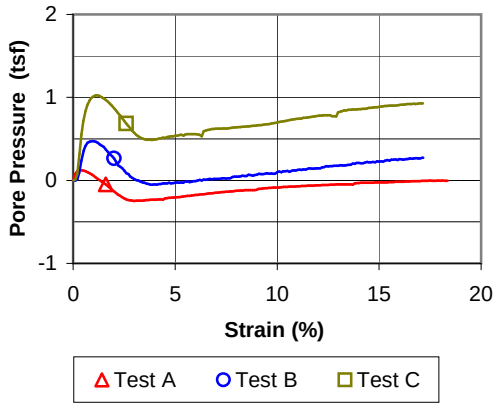
EM 1110-2-1906
Appendix X
30 Nov. 70

Failure Criterion: Maximum Effective Principal Stress Ratio

Effective Strength Envelope



Induced Pore Pressure vs. Strain



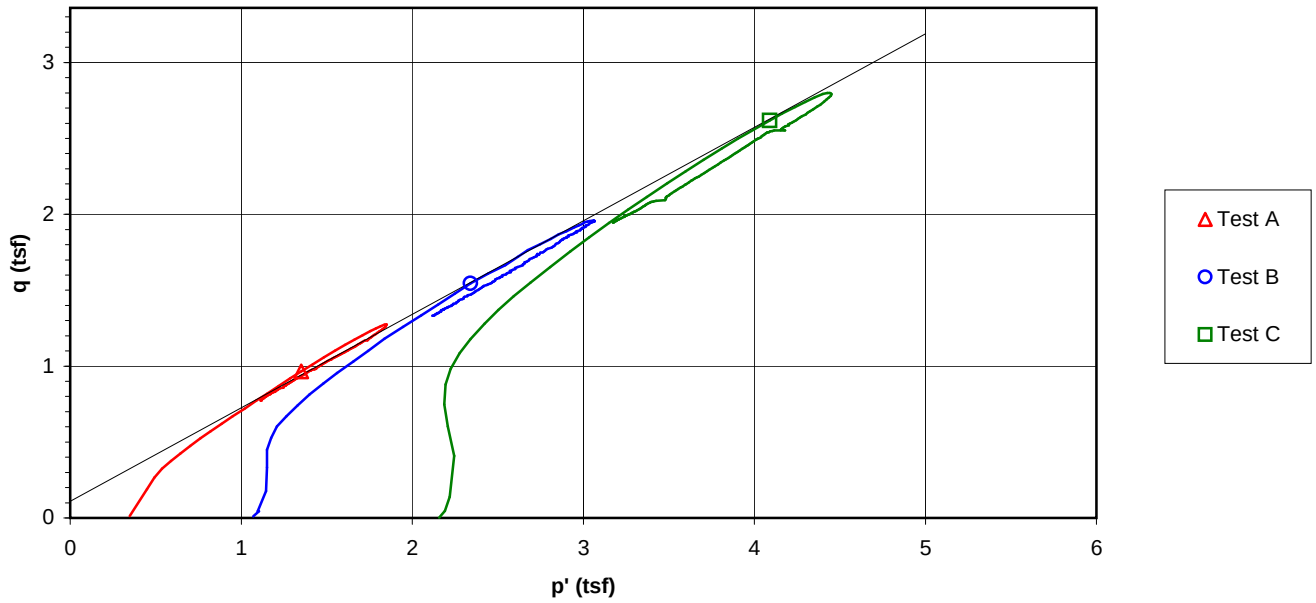
Specimen No.			A	B	C	
Initial Data	Water content %	W_o	26.1	26.6	26.8	
	Dry Density PCF	γd_o	74.0	73.9	73.9	
	Saturation %	S_o	63.5	64.5	65.0	
	Void Ratio	e_o	0.950	0.952	0.951	
After Shear	Water content %	W_f	37.1	37.7	37.4	
	Dry Density PCF	γd_f	77.7	77.1	77.4	
	Saturation %	S_f	100.0	100.0	100.0	
	Void Ratio	e_f	0.857	0.871	0.863	
	Final Back Pressure TSF	u_c	6.12	5.40	4.32	
Minor Principal Stress TSF @ failure		σ_3'f	0.39	0.80	1.47	
Maximum Deviator Stress (tsf) @ failure		$(\sigma_1'-\sigma_3')_{max}$	1.91	3.07	5.23	
Time to (σ1'-σ3')max min.		t_f	6.6	11.2	9.9	
Ultimate Deviator Stress, t/sq ft		$(\sigma1'-\sigma3')_{ult}$	1.65	2.79	3.99	
Initial Diameter, in.		D_o	2.882	2.888	2.880	
Initial Height, in.		H_o	6.031	6.022	5.981	

Controlled - Strain Test					
Description of Specimens	Silt (ML), gray				
			Type of Specimen	Compacted @ 90%	Type of test $\bar{R}$
LL NP	PL NP	PI NP	Gs 2.31	Project	TVA Kingston Plant Dike Failure
Remarks:					
			Boring No.		Sample No. 30
			Depth Elev.		
			Laboratory	Stantec	Date 4-10-09
			TRIAXIAL COMPRESSION TEST REPORT		

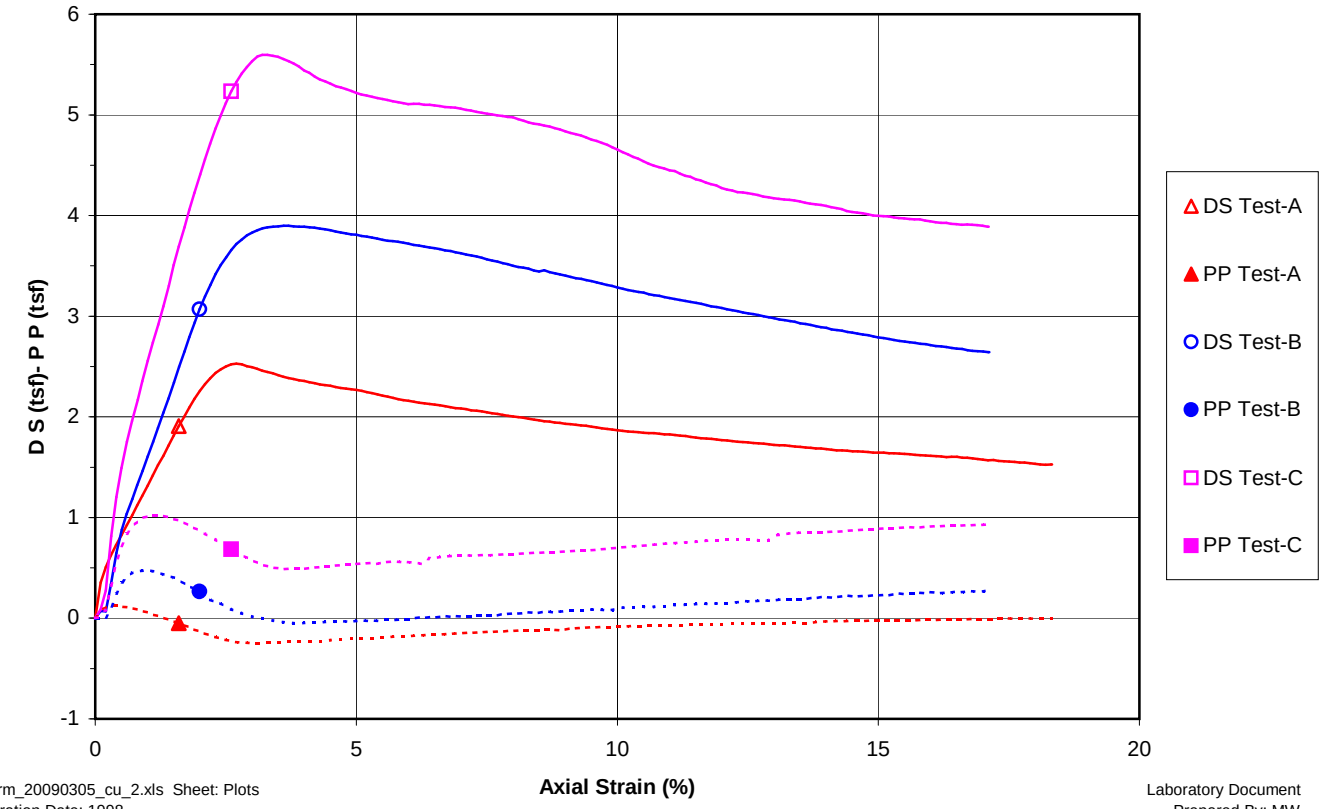
Consolidated Undrained Triaxial Test
EM 1110-2-1906 Appendix X

Project	TVA Kingston Plant Dike Failure	Project No.	171468117
Sample ID	Kingston Fly Ash @ 90%	Test Number	2
Failure Criterion:	Maximum Effective Principal Stress Ratio	ϕ' = 38.0 deg.	c' = 0.14 tsf

p' vs. q Plot



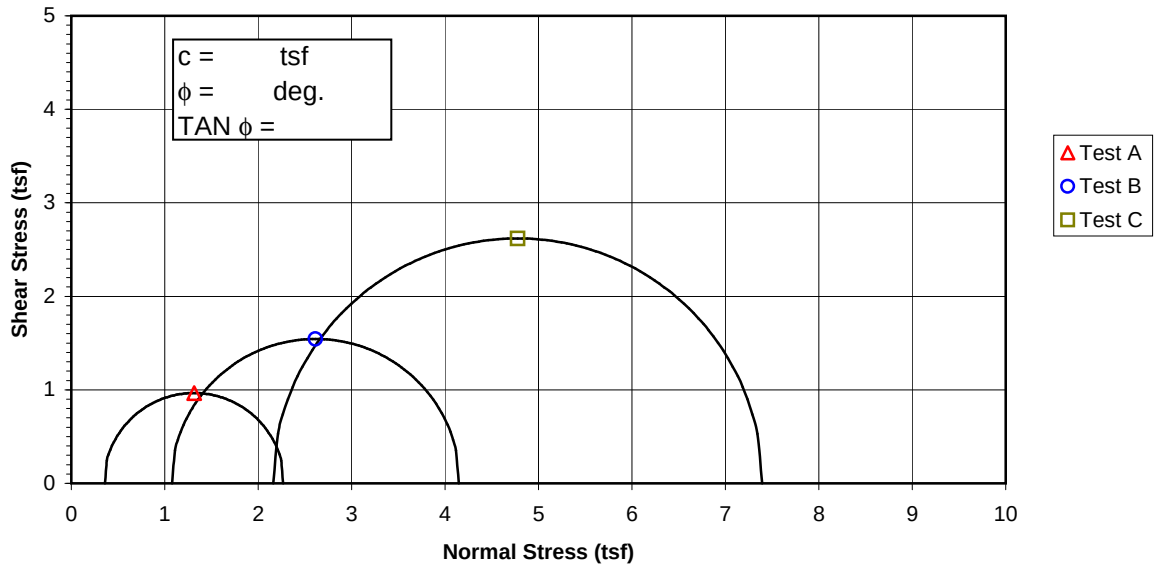
Deviator Stress and Induced Pore Pressure vs. Axial Strain



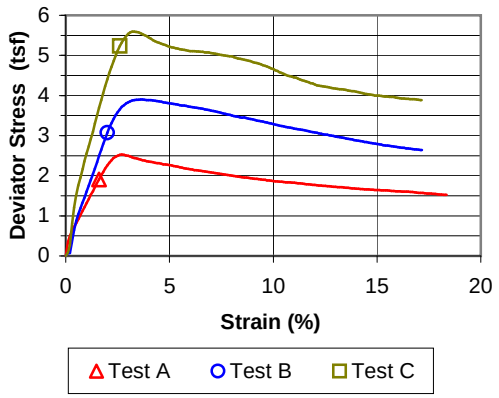
EM 1110-2-1906
Appendix X
30 Nov. 70

Failure Criterion: Maximum Effective Principal Stress Ratio

Total Strength Envelope



Deviator Stress vs. Strain



Specimen No.			A	B	C	
Initial Data	Water content %	W_o	26.1	26.6	26.8	
	Dry Density PCF	γd_o	74.0	73.9	73.9	
	Saturation %	S_o	63.5	64.5	65.0	
	Void Ratio	e_o	0.950	0.952	0.951	
After Shear	Water content %	W_f	37.1	37.7	37.4	
	Dry Density PCF	γd_f	77.7	77.1	77.4	
	Saturation %	S_f	100.0	100.0	100.0	
	Void Ratio	e_f	0.857	0.871	0.863	
	Final Back Pressure TSF	u_c	6.12	5.40	4.32	
Minor Principal Stress TSF		σ_3	0.36	1.08	2.16	
Maximum Deviator Stress (tsf) @ failure		$(\sigma_1-\sigma_3)_{max}$	1.91	3.07	5.23	
Time to $(\sigma_1-\sigma_3)_{Max.}$ min.		t_f	6.6	11.2	9.9	
Ultimate Deviator Stress, t/sq ft		$(\sigma_1-\sigma_3)_{ult}$	1.65	2.79	3.99	
Initial Diameter, in.		D_o	2.882	2.888	2.880	
Initial Height, in.		H_o	6.031	6.022	5.981	

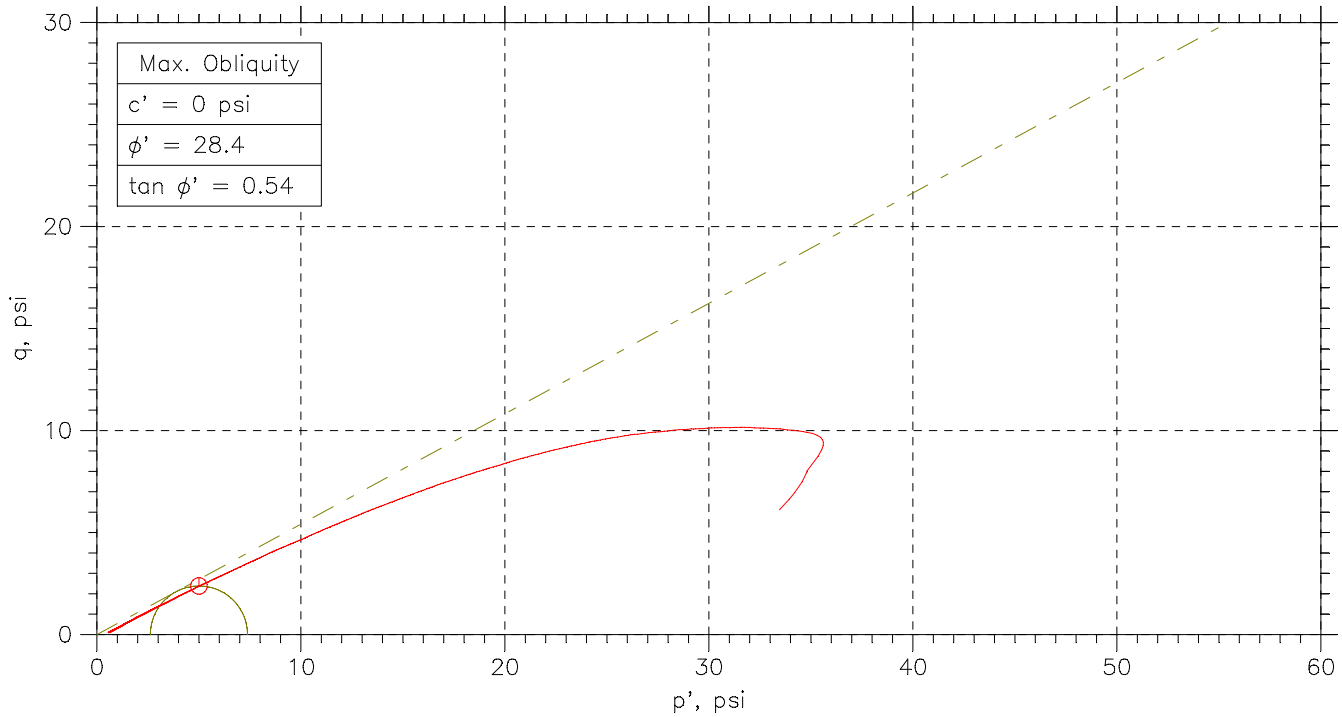
Controlled - Strain Test			
Description of Specimens	Silt (ML), gray		
		Type of Specimen Compacted @ 90%	Type of test $\bar{R}$
LL NP	PL NP	PI NP	Gs 2.31
Project	TVA Kingston Plant Dike Failure		
Remarks:			
		Boring No.	Sample No. 30
		Depth Elev.	
		Laboratory Stantec	Date 4-10-09
		TRIAXIAL COMPRESSION TEST REPORT	

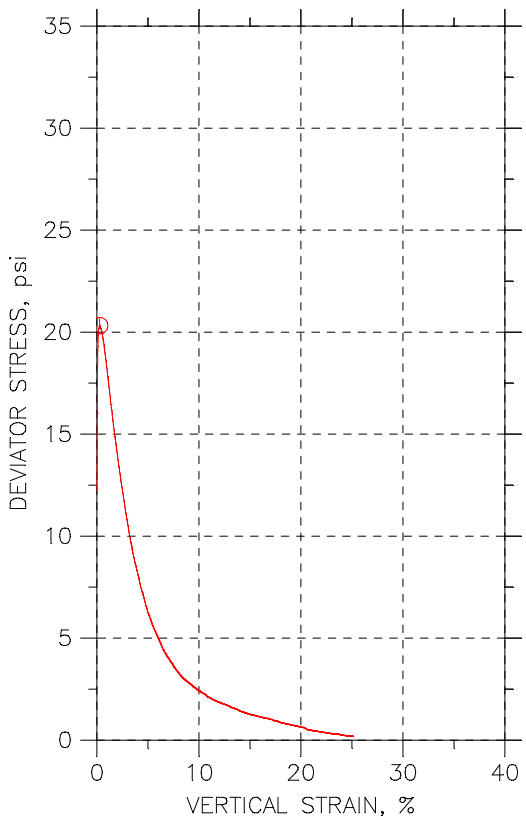
Appendix B

Test Data – Hydraulically Placed Ash

WORK IN PROGRESS

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





Symbol		⏲			
Sample No.		200s			
Test No.		MTe07s72			
Depth		----			
Initial	Diameter, mm	71.1			
	Height, mm	158.5			
	Water Content, %	40.5			
	Dry Density, pcf	74.43			
	Saturation, %	98.9			
	Void Ratio	0.954			
Before Shear	Water Content, %	38.6			
	Dry Density, pcf	76.54			
	Saturation*, %	100.0			
	Void Ratio	0.901			
	Back Press., psi	80.61			
Ver. Eff. Cons. Stress, psi		27.34			
Shear Strength, psi		10.16			
Strain at Failure, %		0.267			
Strain Rate, %/min		0.08			
B-Value		---			
Measured Specific Gravity		2.33			
Liquid Limit		NP			
Plastic Limit		NP			

Project: KINGSTON COAL

Location: TN

Project No.: 60095742

Boring No.: RECONST

Sample Type: RECONST

Description: MOIST TAMP - RECONSTITUTED ASH - DARK GRAY

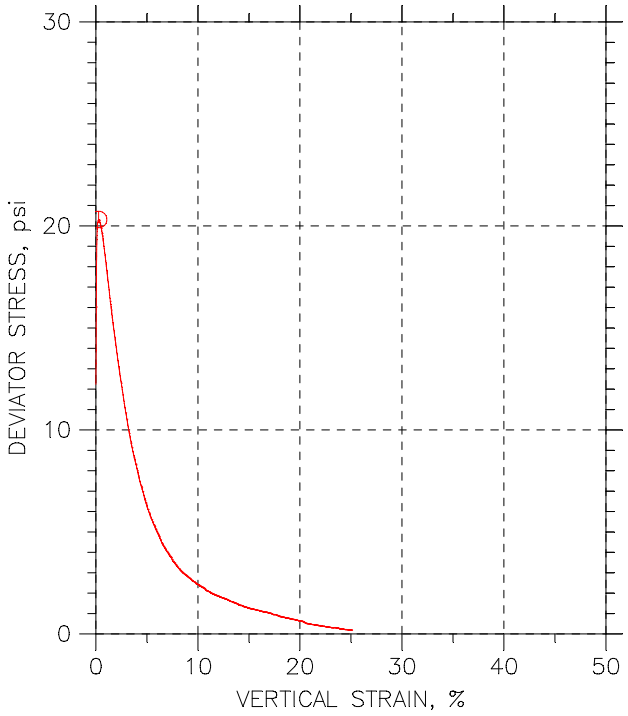
Remarks: FAILURE CRITERIA = MAX. OBLIQUITY, Ko CONSOLIDATION, B=0.95

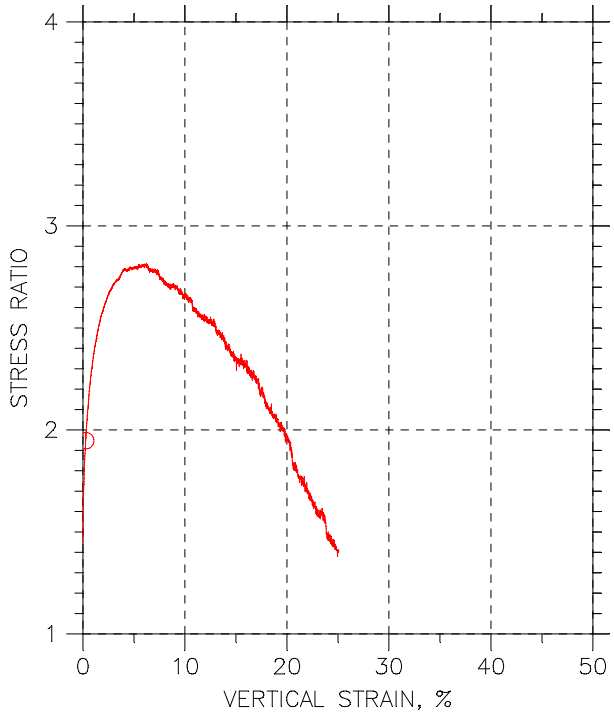
Phase calculations based on end of test.

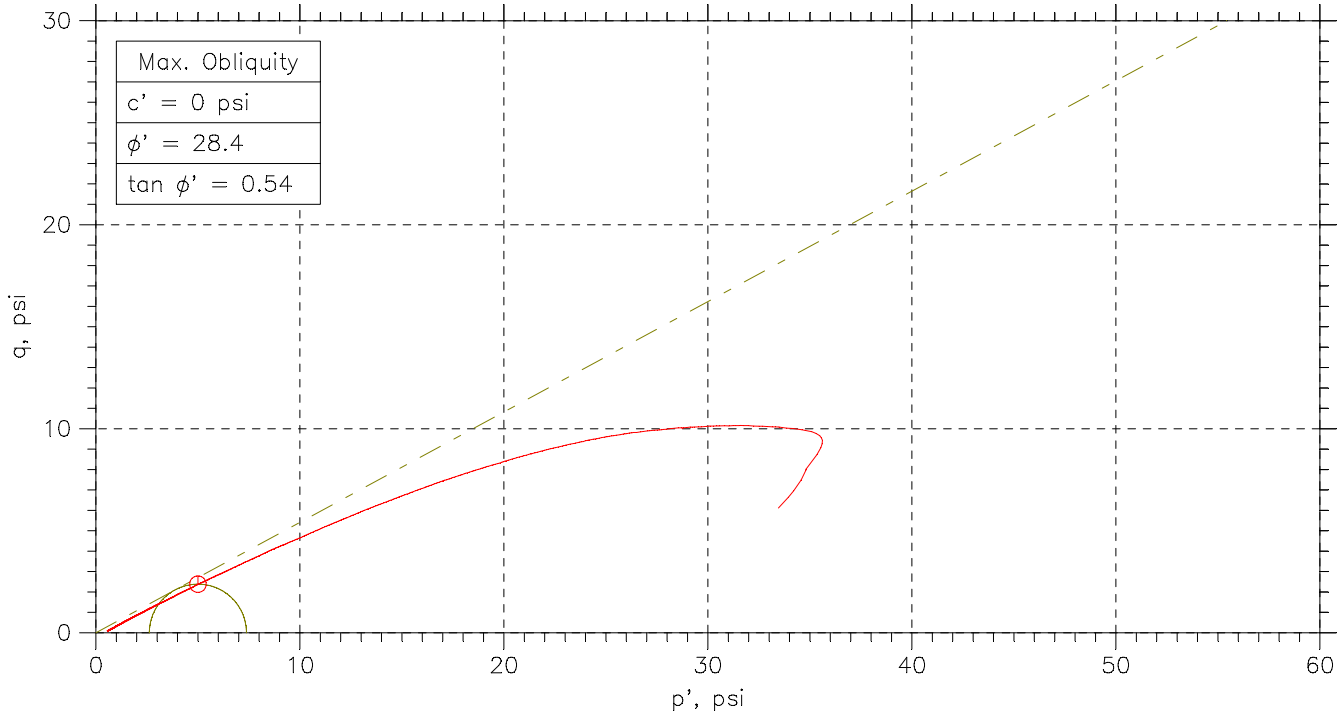
* Saturation is set to 100% for phase calculations

Fri, 01-MAY-2009 10:18:25

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





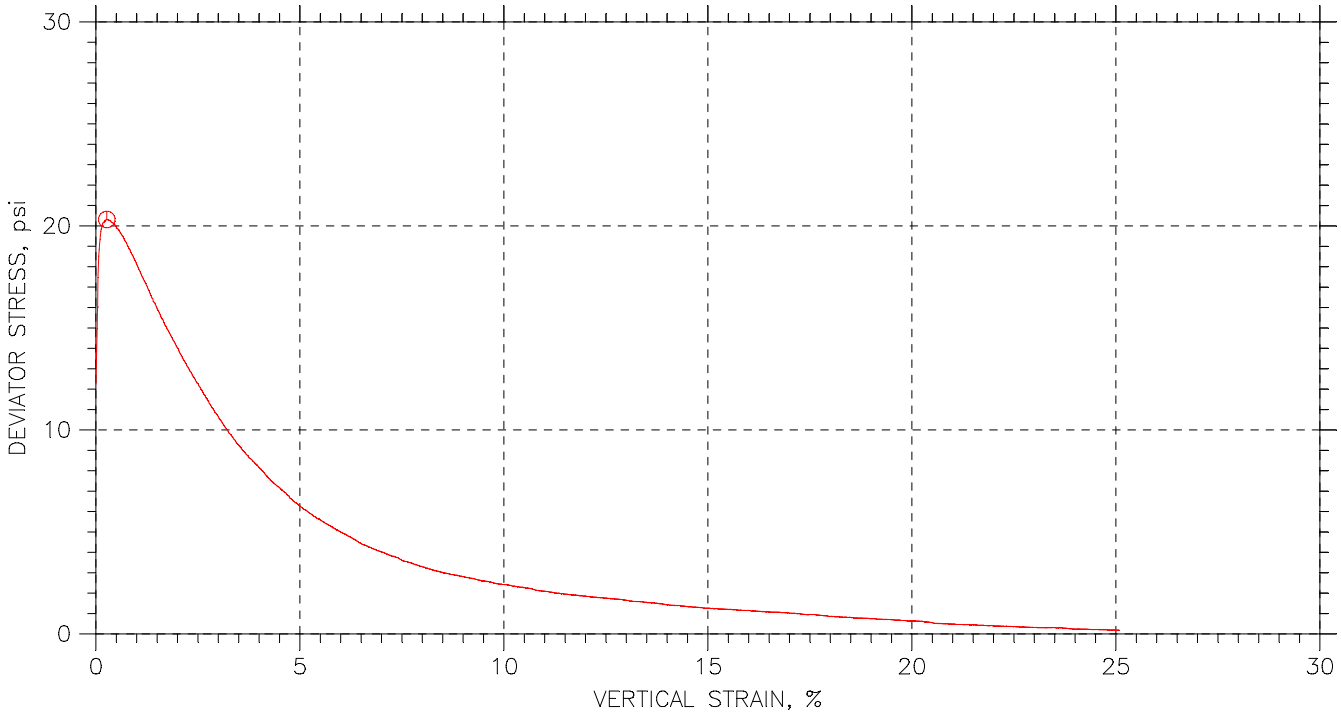


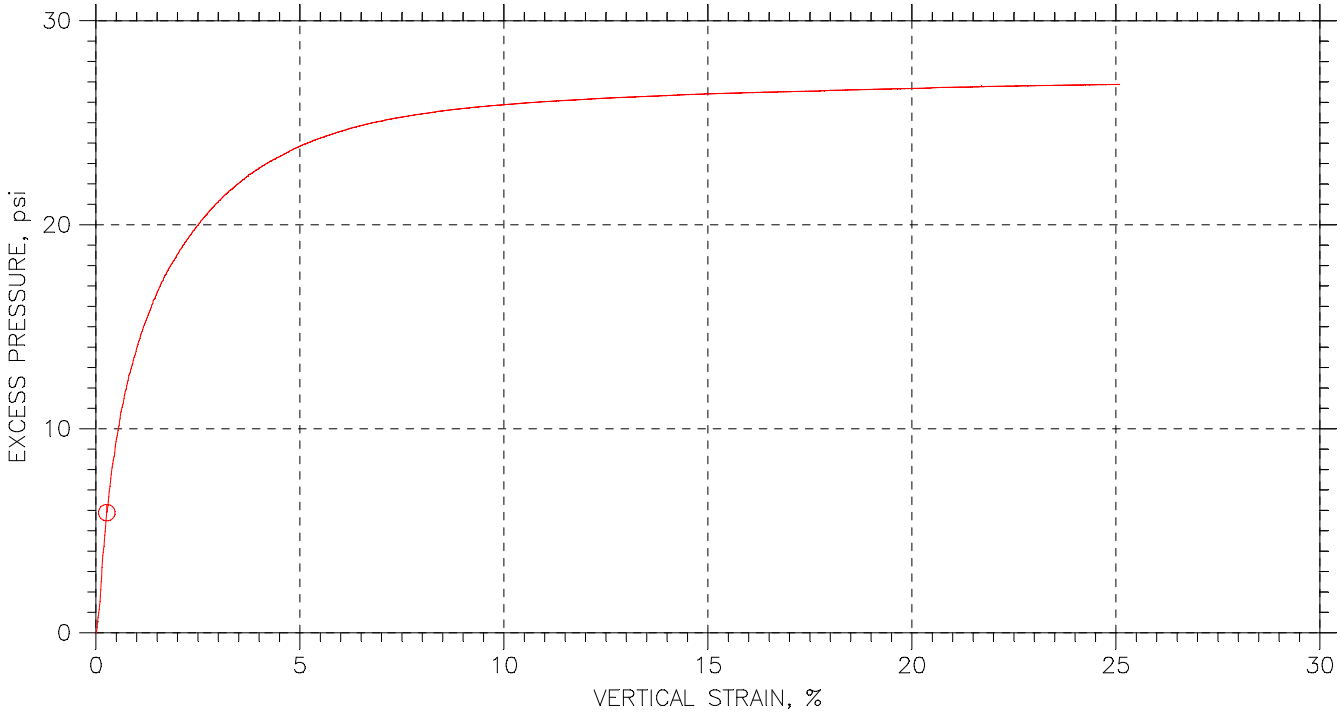
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
⏲	200s	MTe07s72	----	BCM/JMT	4-18-09	WPQ		5742200SERIES K e=0_8 96 ps

Project: KINGSTON COAL	Location: TN	Project No.: 60095742
Boring No.: RECONST	Sample Type: RECONST	
Description: MOIST TAMP - RECONSTITUTED ASH - DARK GRAY		
Remarks: FAILURE CRITERIA = MAX. OBLIQUITY, Ko CONSOLIDATION, B=0.95		

Fri, 01-MAY-2009 10:18:25

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



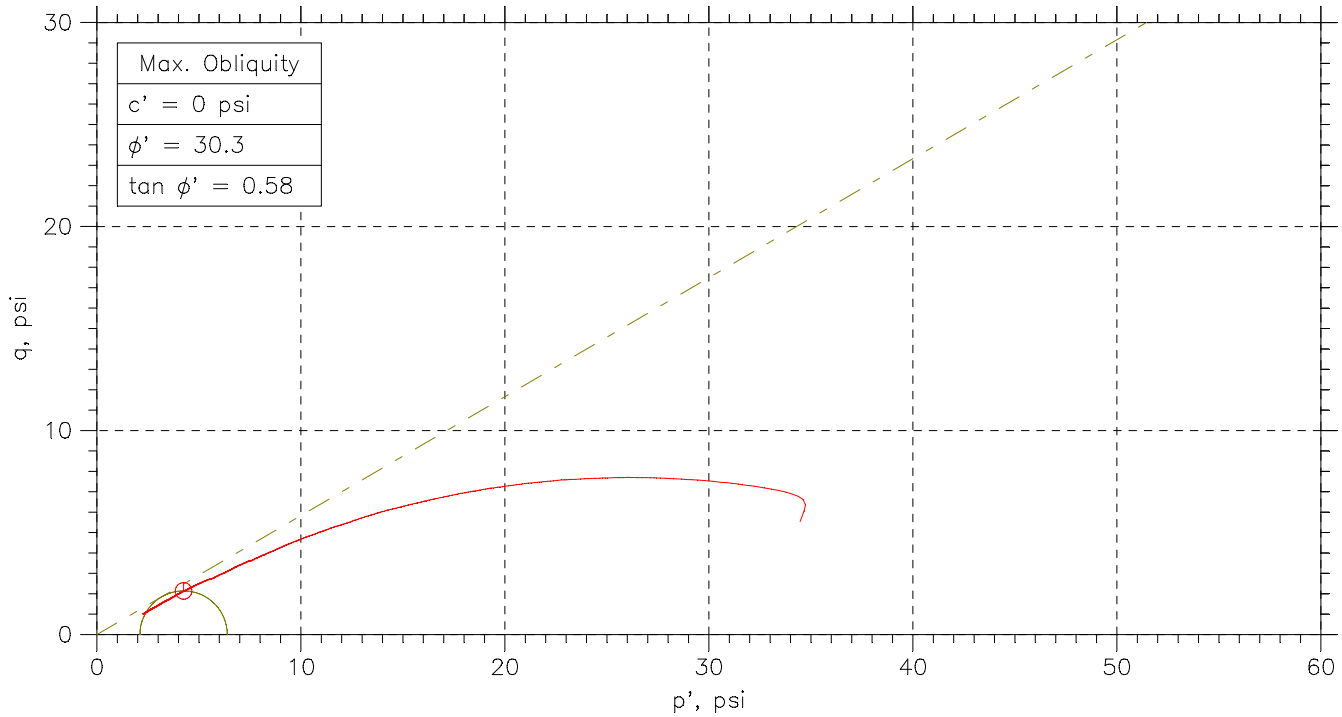


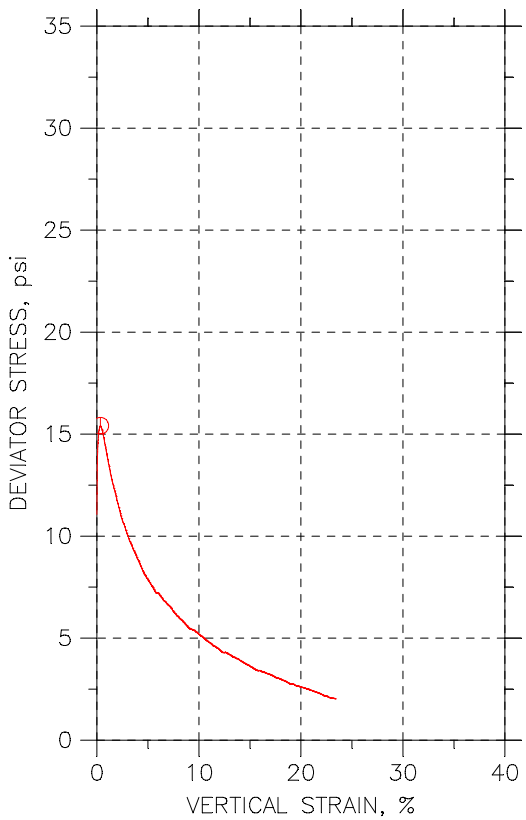
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
⏲	200s	MTe07s72	----	BCM/JMT	4-18-09	WPQ		5742200SERIES K e=0_8 96 ps

Project: KINGSTON COAL	Location: TN	Project No.: 60095742
Boring No.: RECONST	Sample Type: RECONST	
Description: MOIST TAMP - RECONSTITUTED ASH - DARK GRAY		
Remarks: FAILURE CRITERIA = MAX. OBLIQUITY, Ko CONSOLIDATION, B=0.95		

Fri, 01-MAY-2009 10:18:26

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





Symbol		⏲			
Sample No.		200s			
Test No.		MTe08s32			
Depth		----			
Initial	Diameter, mm	71.3			
	Height, mm	155			
	Water Content, %	36.2			
	Dry Density, pcf	79.45			
	Saturation, %	101.5			
	Void Ratio	0.831			
Before Shear	Water Content, %	33.5			
	Dry Density, pcf	81.68			
	Saturation*, %	100.0			
	Void Ratio	0.781			
	Back Press., psi	67.19			
Ver. Eff. Cons. Stress, psi		28.93			
Shear Strength, psi		7.706			
Strain at Failure, %		0.34			
Strain Rate, %/min		0.08			
B-Value		---			
Estimated Specific Gravity		2.33			
Liquid Limit		NP			
Plastic Limit		NP			

Project: KINGSTON COAL

Location: TN

Project No.: 60095742

Boring No.: RECONST

Sample Type: RECONST

Description: MOST TAMP - RECONSTITUTED ASH - DARK GRAY

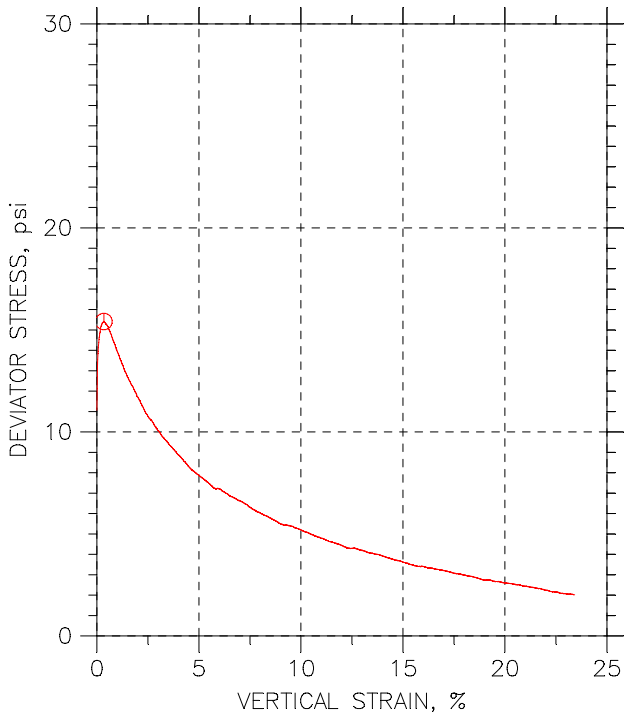
Remarks: TEST PERFORMED AS PER ASTM D 4767 (B=0.95)

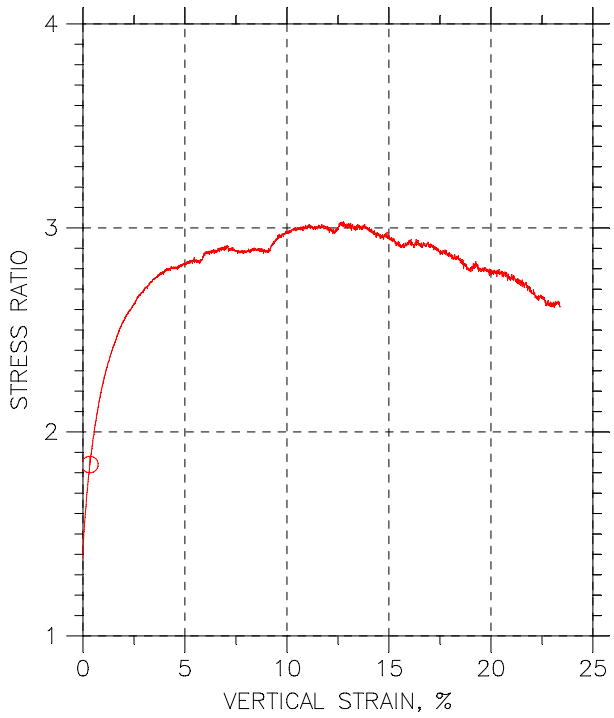
Phase calculations based on end of test.

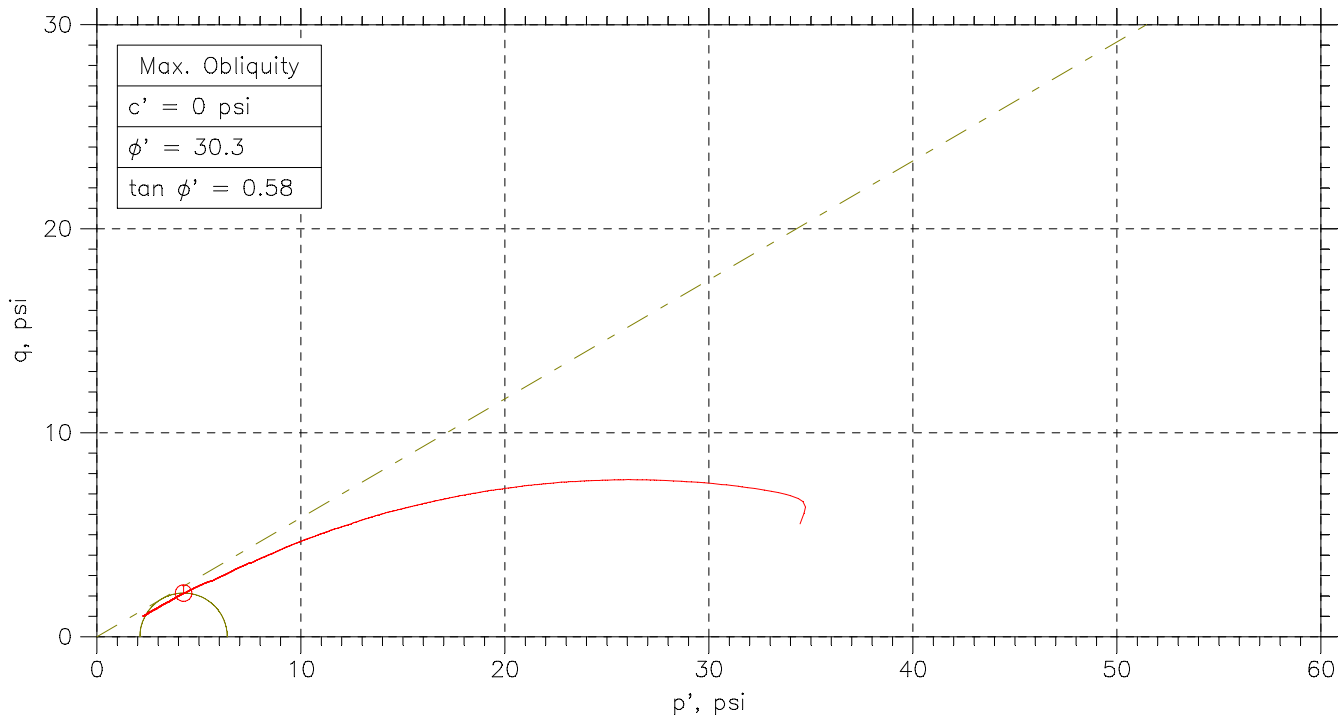
* Saturation is set to 100% for phase calculations

Wed, 22-APR-2009 18:06:02

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





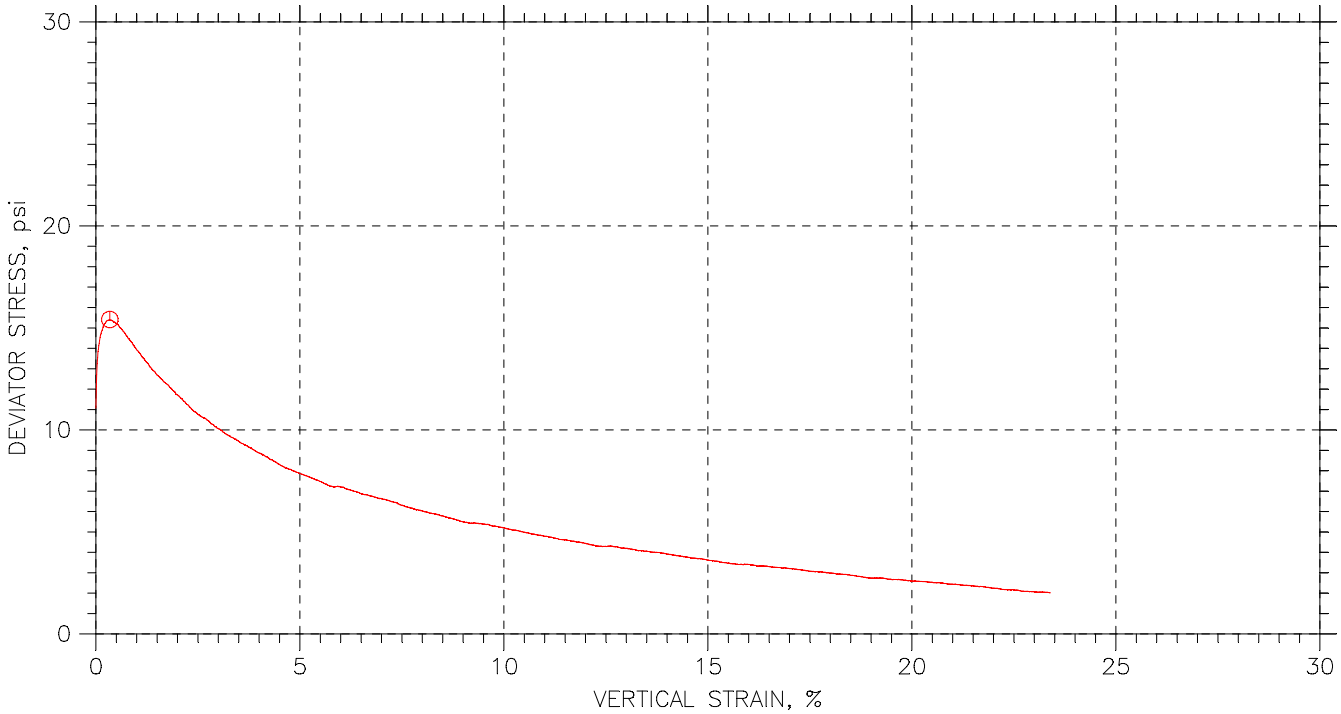


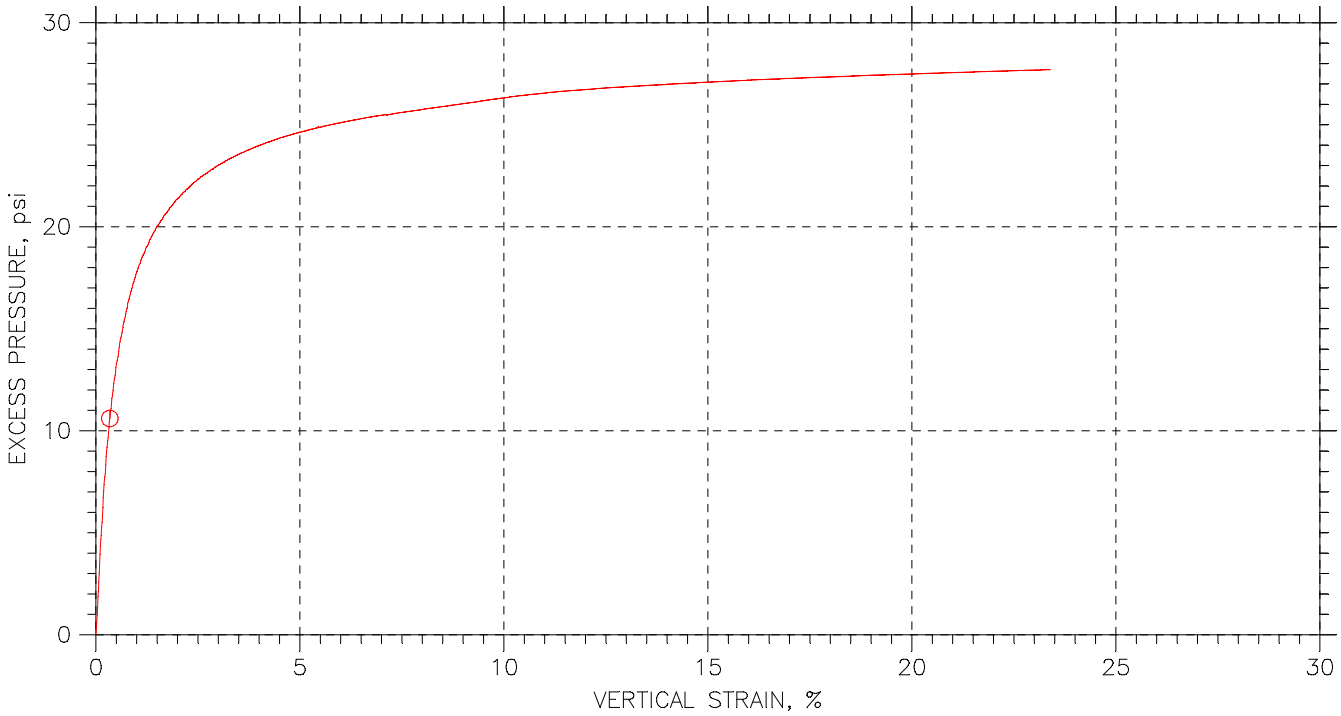
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
⏲	200s	MTe08s32	----	BCM	4/8/09	WPQ		5742200SERIES K e=1 32PSI Tr

Project: KINGSTON COAL	Location: TN	Project No.: 60095742
Boring No.: RECONST	Sample Type: RECONST	
Description: MOST TAMP - RECONSTITUTED ASH - DARK GRAY		
Remarks: TEST PERFORMED AS PER ASTM D 4767 (B=0.95)		

Wed, 22-APR-2009 18:06:03

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





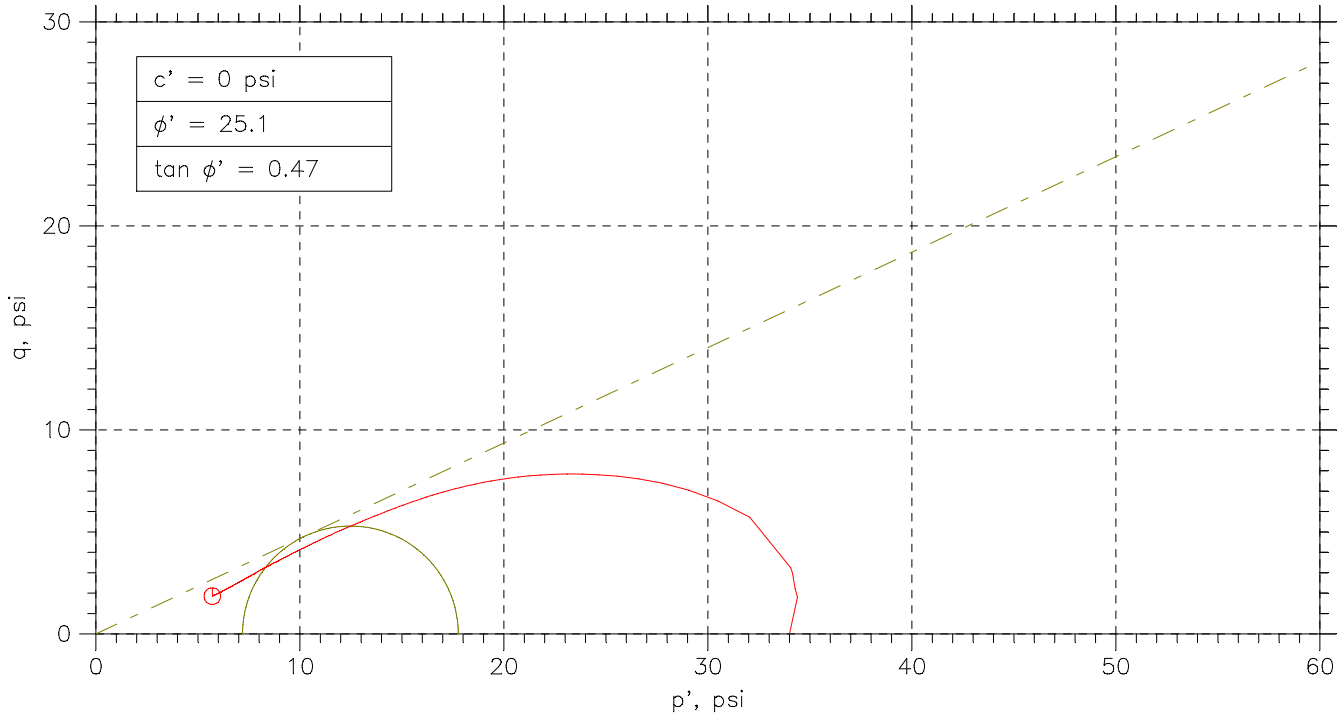
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
⊘	200s	MTe08s32	----	BCM	4/8/09	WPQ		5742200SERIES K e=1 32PSI Tr

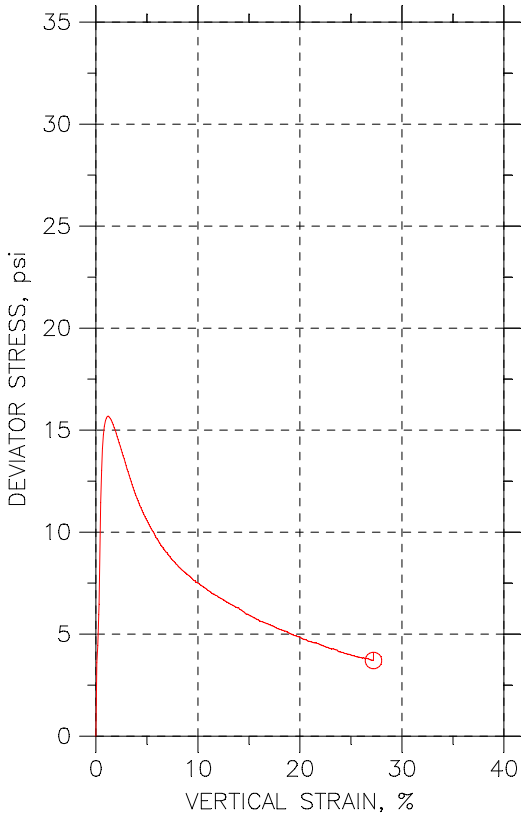
Project: KINGSTON COAL	Location: TN	Project No.: 60095742
Boring No.: RECONST	Sample Type: RECONST	
Description: MOST TAMP - RECONSTITUTED ASH - DARK GRAY		
Remarks: TEST PERFORMED AS PER ASTM D 4767 (B=0.95)		

Wed, 22-APR-2009 18:06:04

WORK IN PROGRESS

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⏲			
Test No.		MTe08s34			
Initial	Diameter, mm	67.84			
	Height, mm	155.03			
	Water Content, %	37.86			
	Dry Density, pcf	77.29			
	Saturation, %	100.01			
	Void Ratio	0.88191			
Before Shear	Water Content, %	34.72			
	Dry Density, pcf	80.41			
	Saturation, %	100.00			
	Void Ratio	0.80902			
	Back Press., psi	70			
Minor Prin. Stress, psi		34			
Max. Dev. Stress, psi		15.681			
Time to Failure, min		18.724			
Strain Rate, %/min		0.1			
B-Value		---			
Estimated Specific Gravity		2.33			
Liquid Limit		NP			
Plastic Limit		NP			
Plasticity Index		NP			

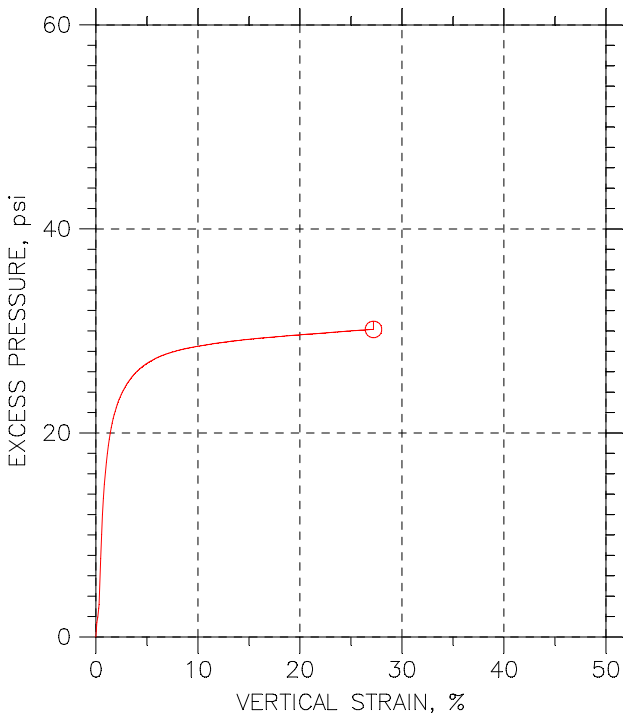
Project: KINGSTON COAL TVA	Failure Sketch			
Location: TN				
Project No.: 60095742				
Boring No.: RECONS				
Sample Type: RECONST				

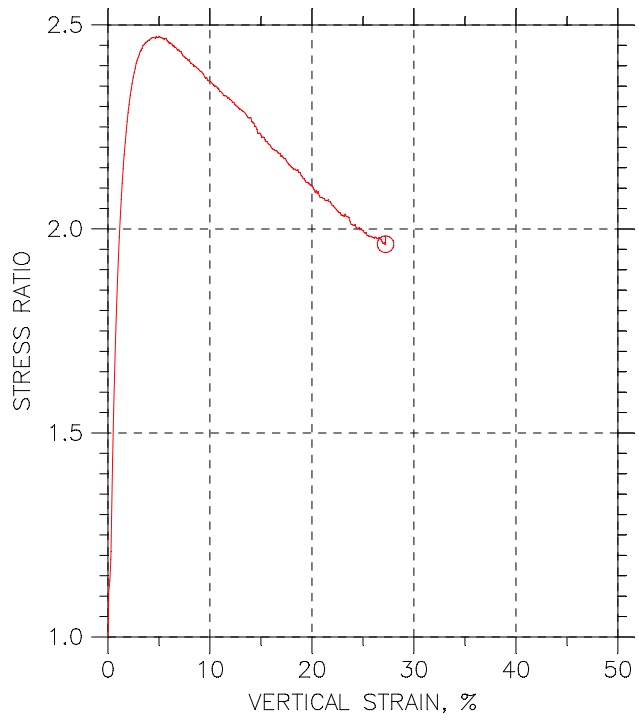
Description: MOIST TAMP RECONSTITUTED ASH - DARK GRAY

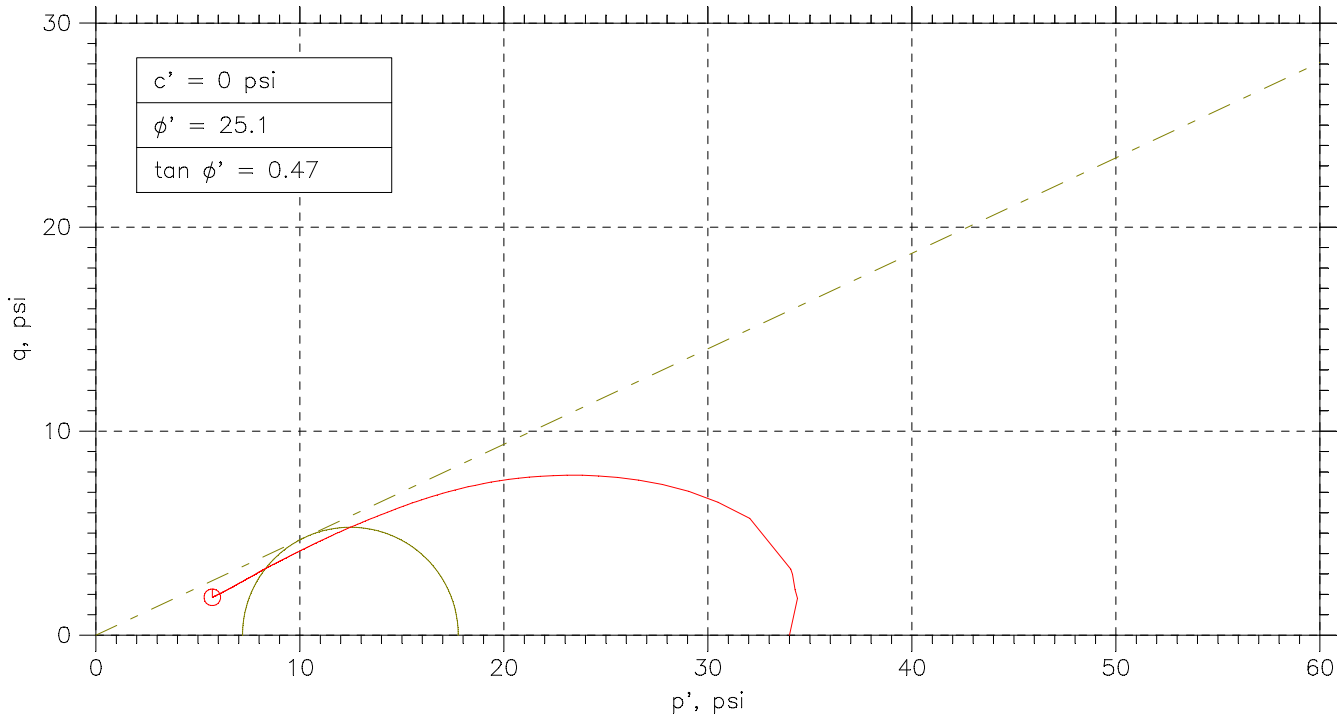
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (B=0.91)

Mon, 20-APR-2009 19:22:09

TRIAXIAL COMPRESSION TEST REPORT



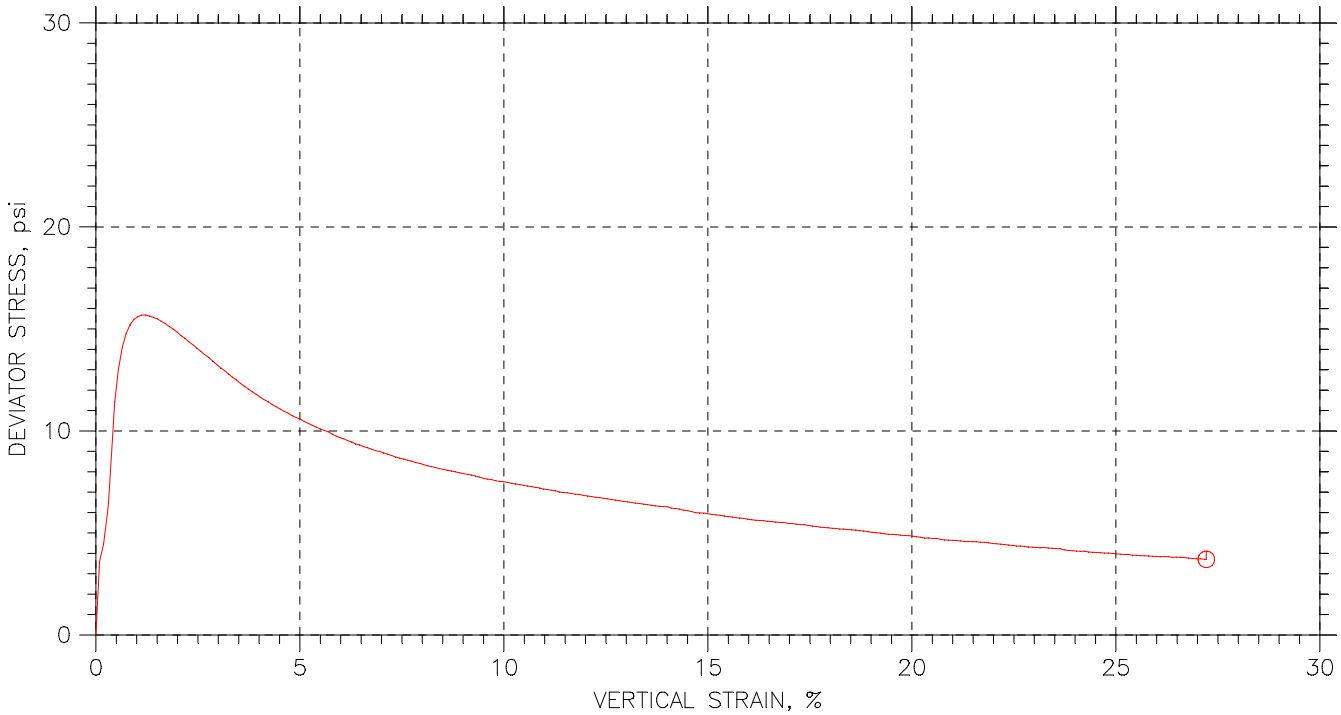


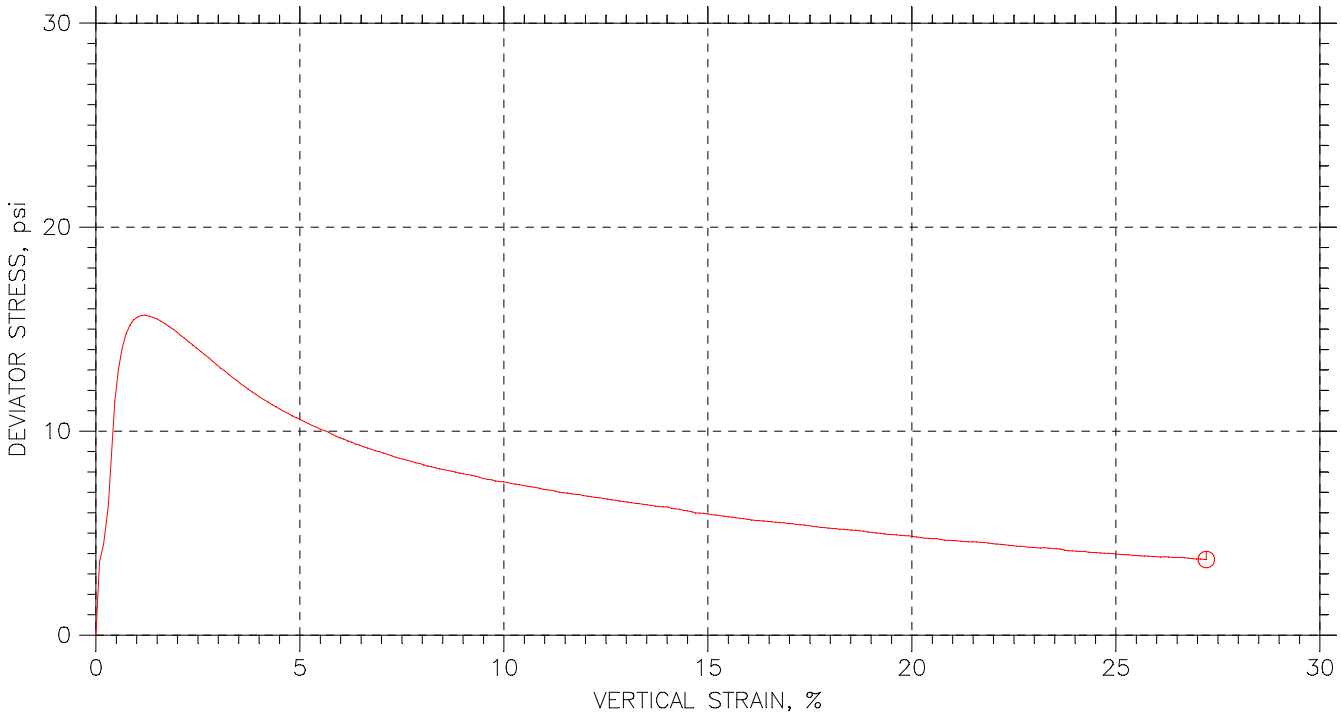


Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: RECONS	Tested By: BCM	Checked By: WPQ
Sample No.: 200s	Test Date: 4/07/09	Depth: -----
Test No.: MTe08s34	Sample Type: RECONST	Elevation: -----
Description: MOIST TAMP RECONSTITUTED ASH - DARK GRAY		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (B=0.91)		

Mon, 20-APR-2009 19:22:09

TRIAXIAL COMPRESSION TEST REPORT

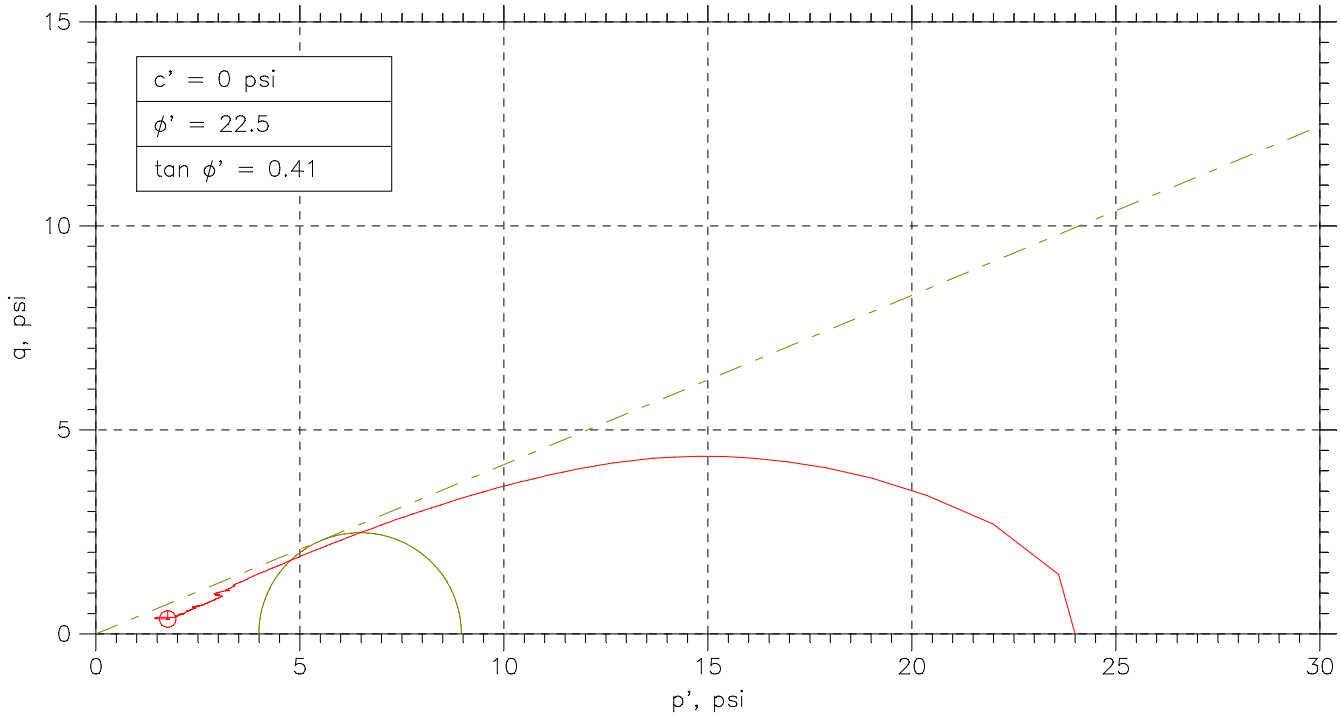


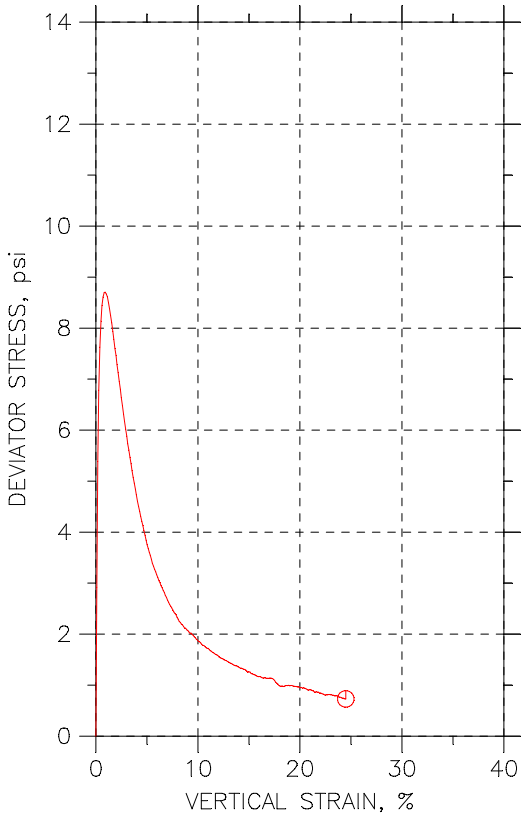


Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: RECONS	Tested By: BCM	Checked By: WPQ
Sample No.: 200s	Test Date: 4/07/09	Depth: -----
Test No.: MTe08s34	Sample Type: RECONST	Elevation: -----
Description: MOIST TAMP RECONSTITUTED ASH - DARK GRAY		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (B=0.91)		

Mon, 20-APR-2009 19:22:10

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⏲			
Test No.		MTe1ClUs32			
Initial	Diameter, mm	74.098			
	Height, mm	150.75			
	Water Content, %	46.45			
	Dry Density, pcf	67.24			
	Saturation, %	93.05			
	Void Ratio	1.1632			
Before Shear	Water Content, %	35.13			
	Dry Density, pcf	79.98			
	Saturation, %	100.00			
	Void Ratio	0.81859			
	Back Press., psi	106			
Minor Prin. Stress, psi		24			
Max. Dev. Stress, psi		8.703			
Time to Failure, min		45.004			
Strain Rate, %/min		0.06			
B-Value		---			
Estimated Specific Gravity		2.33			
Liquid Limit		NP			
Plastic Limit		NP			
Plasticity Index		NP			

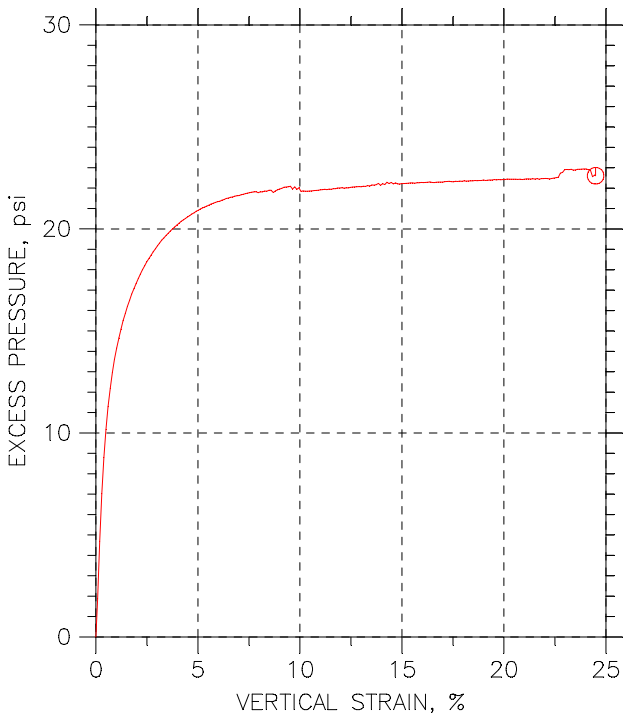
Project: KINGSTON COAL TVA	Failure Sketch			
Location: TN				
Project No.: 60095742				
Boring No.: RECONST				
Sample Type: RECONST				

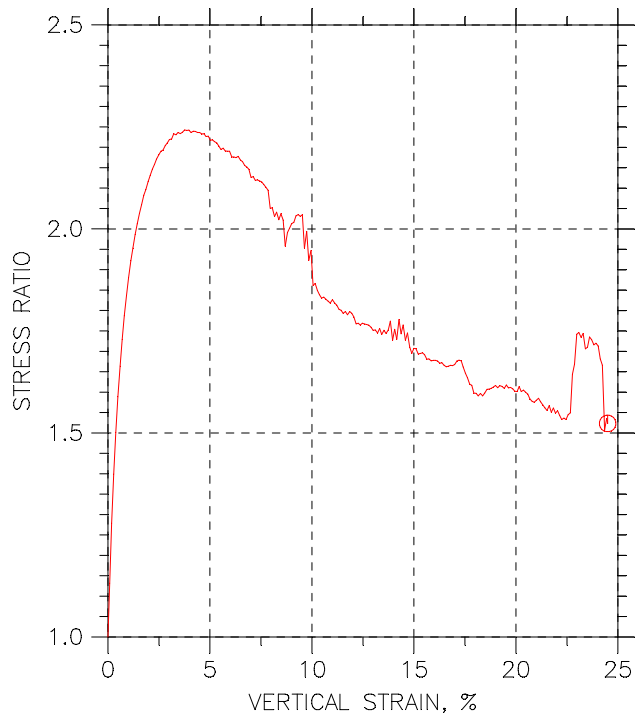
Description: MOIST TAMP RECONSTITUTED ASH - DARK GRAY

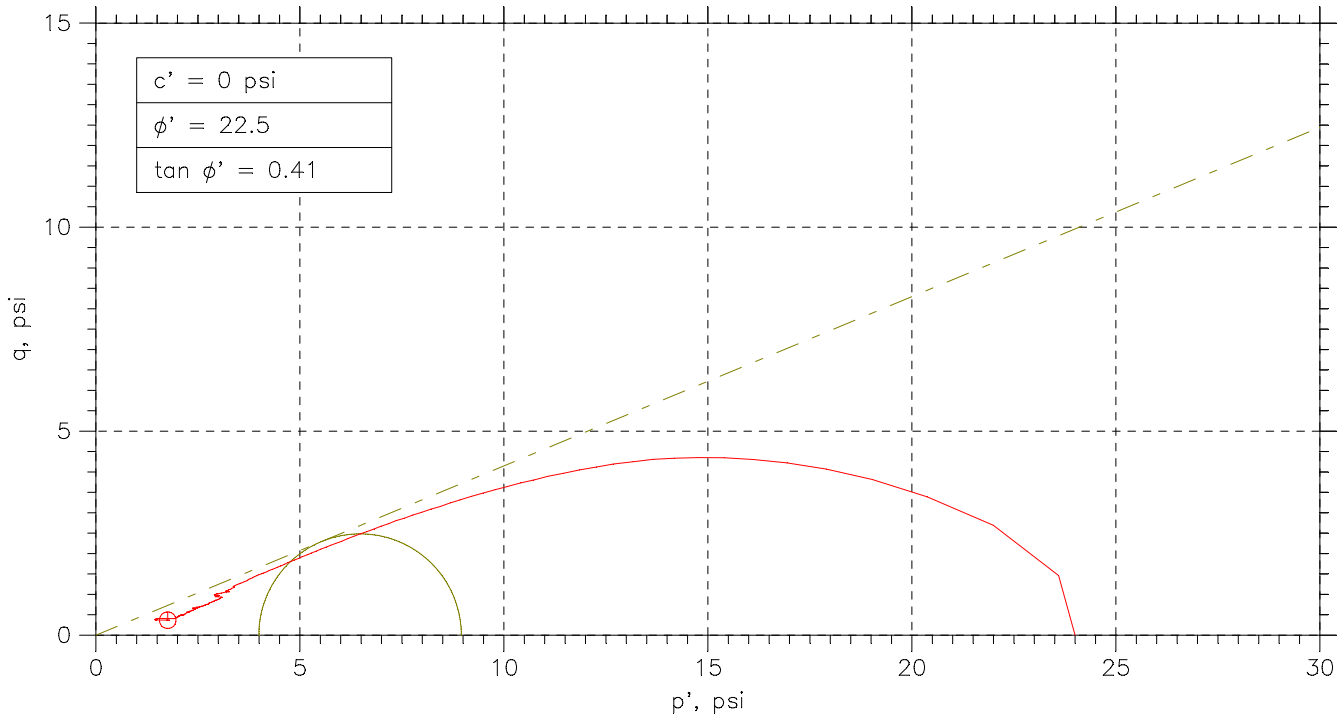
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (B=0.95)

Wed, 22-APR-2009 19:28:27

TRIAXIAL COMPRESSION TEST REPORT



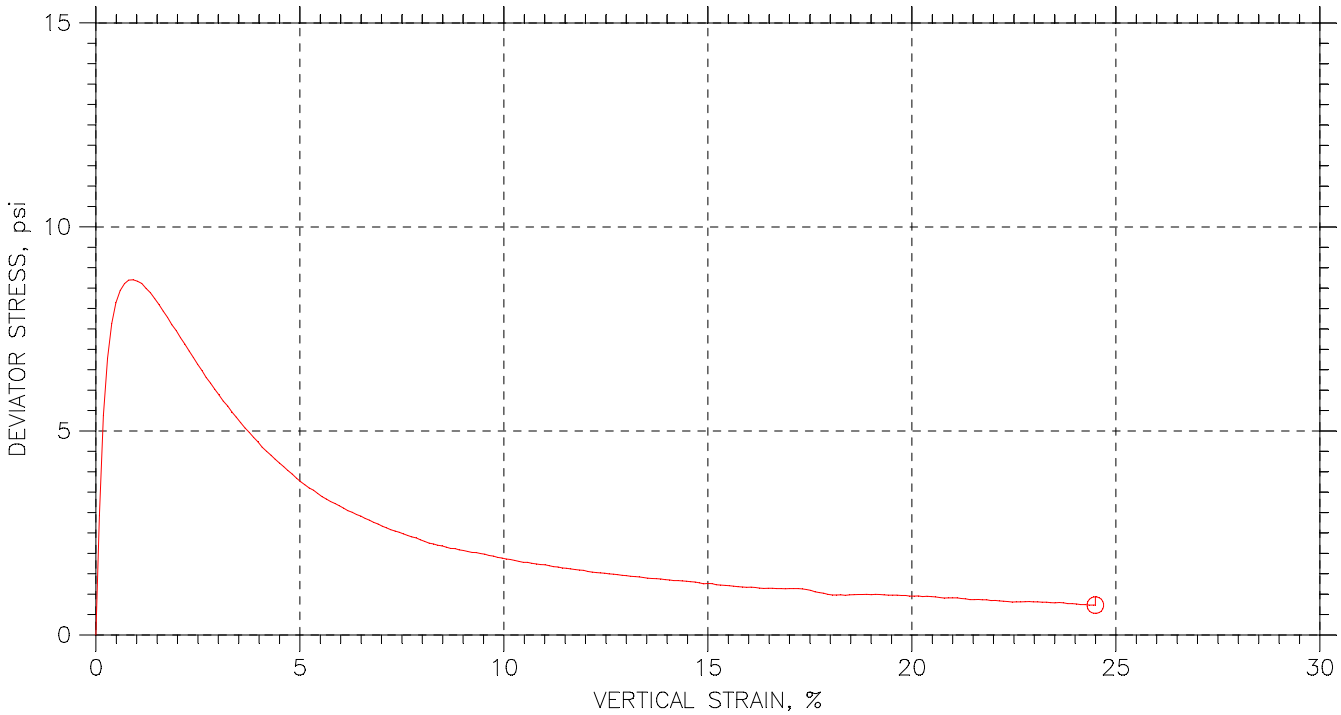


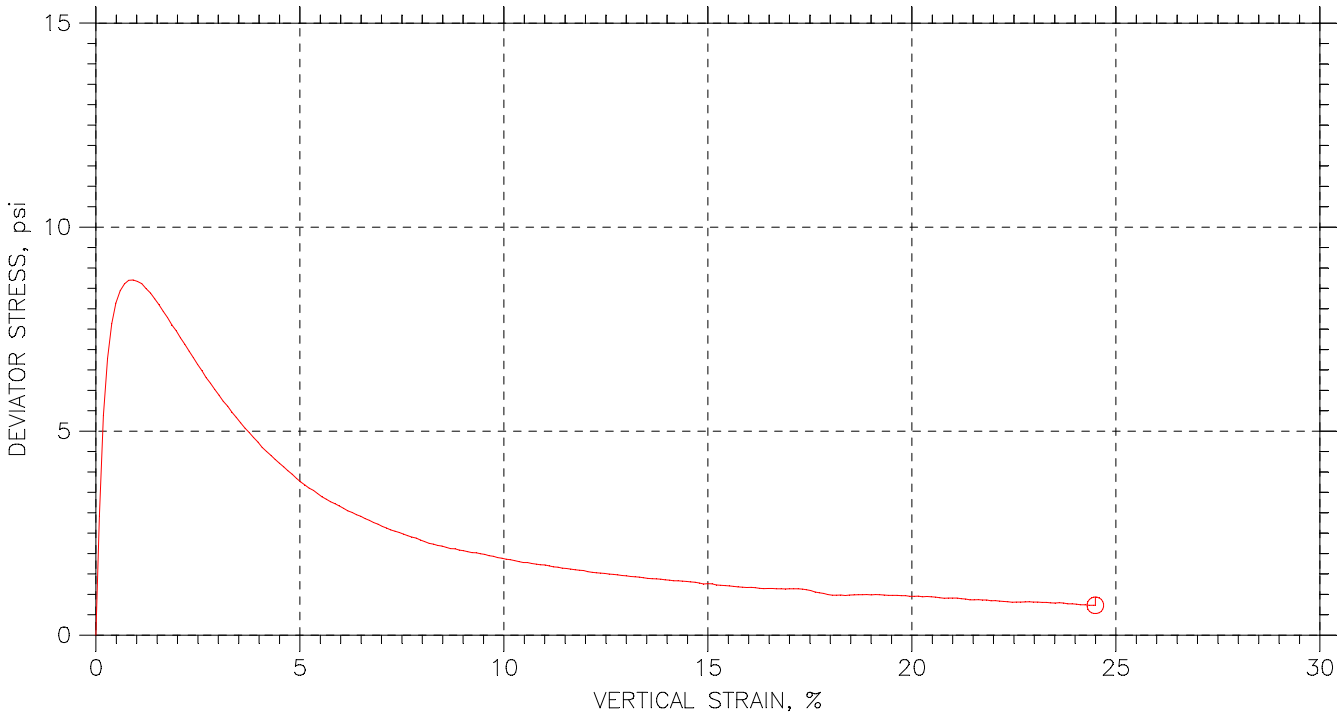


Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: RECONST	Tested By: BCM	Checked By: WPQ
Sample No.: 200S	Test Date: 4/17/09	Depth: ----
Test No.: MTe1ClUs32	Sample Type: RECONST	Elevation: ----
Description: MOIST TAMP RECONSTITUTED ASH - DARK GRAY		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (B=0.95)		

Wed, 22-APR-2009 19:28:28

TRIAXIAL COMPRESSION TEST REPORT

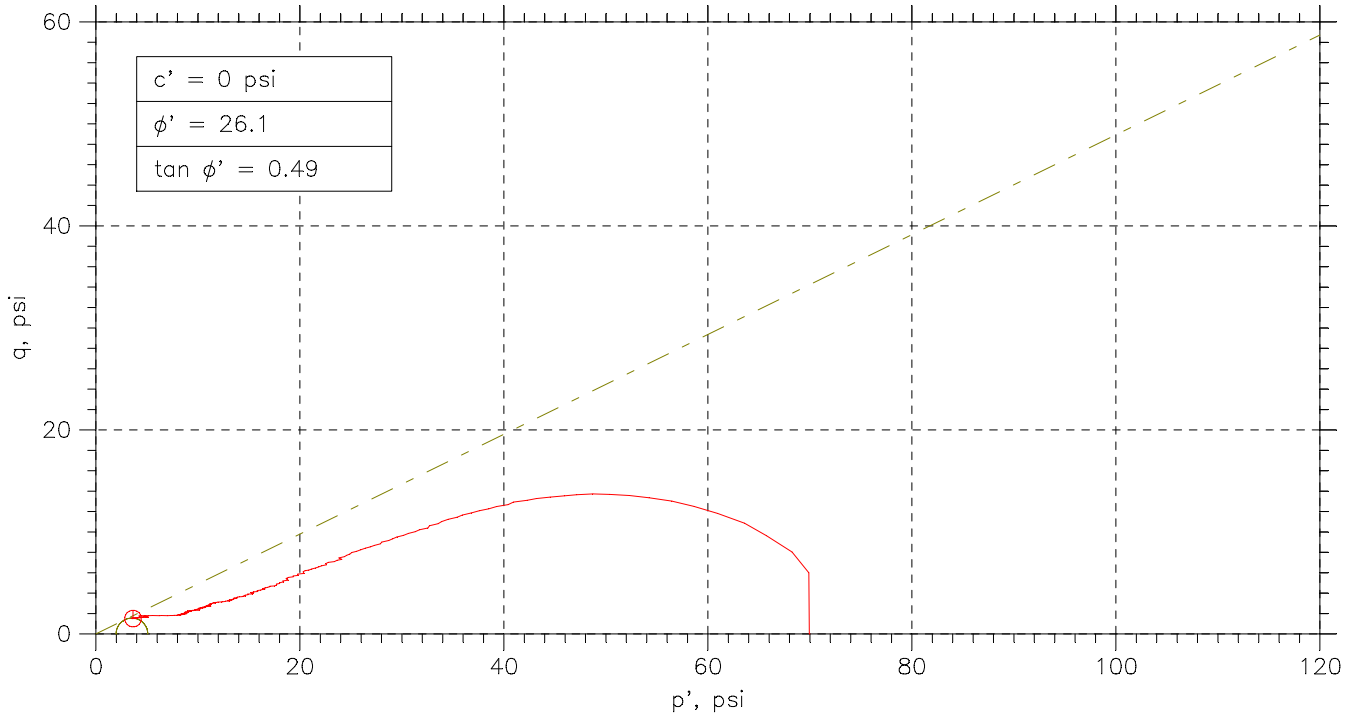


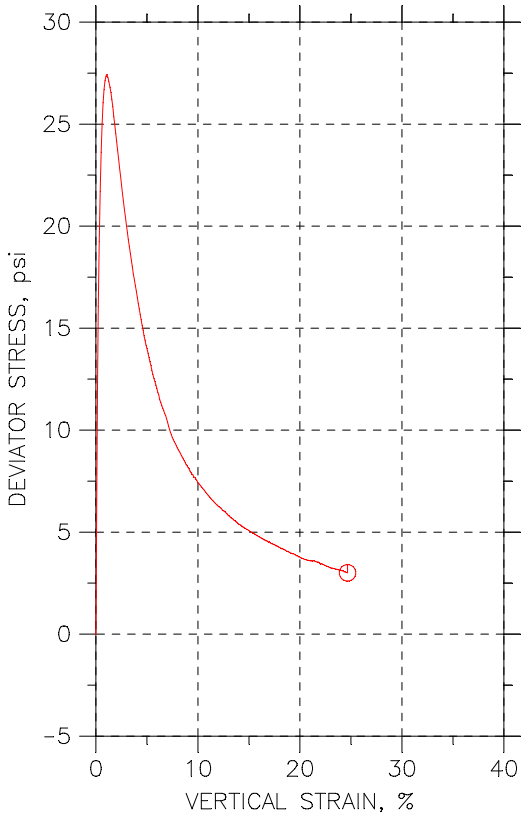


Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: RECONST	Tested By: BCM	Checked By: WPQ
Sample No.: 200S	Test Date: 4/17/09	Depth: ----
Test No.: MTe1CIUs32	Sample Type: RECONST	Elevation: ----
Description: MOIST TAMP RECONSTITUTED ASH - DARK GRAY		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (B=0.95)		

Wed, 22-APR-2009 19:28:28

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⏲			
Test No.		MTe09s70			
Initial	Diameter, mm	69.24			
	Height, mm	155.95			
	Water Content, %	51.68			
	Dry Density, pcf	66.			
	Saturation, %	100.03			
	Void Ratio	1.2039			
Before Shear	Water Content, %	37.11			
	Dry Density, pcf	78.			
	Saturation, %	100.00			
	Void Ratio	0.86474			
	Back Press., psi	71			
Minor Prin. Stress, psi		70			
Max. Dev. Stress, psi		27.439			
Time to Failure, min		60.004			
Strain Rate, %/min		0.08			
B-Value		---			
Estimated Specific Gravity		2.33			
Liquid Limit		NP			
Plastic Limit		NP			
Plasticity Index		NP			

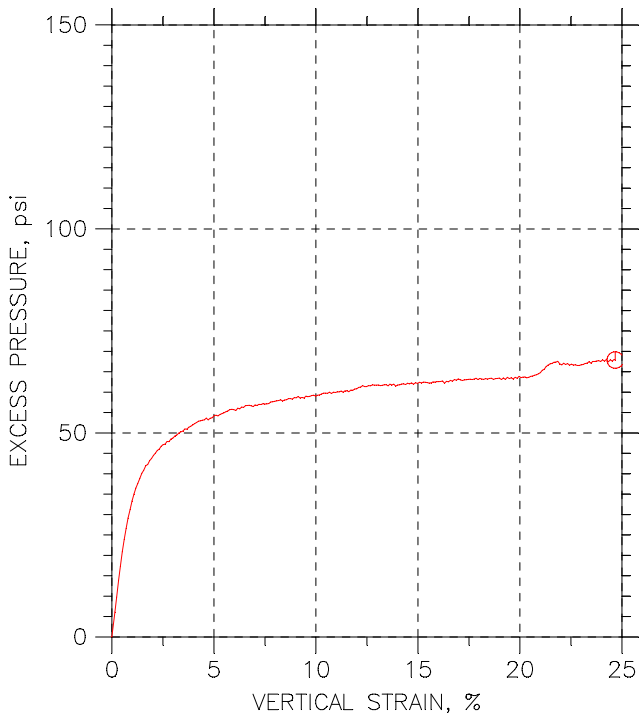
Project: KINGSTON COAL TVA	Failure Sketch			
Location: TN				
Project No.: 60095742				
Boring No.: RECONST				
Sample Type: RECONST				

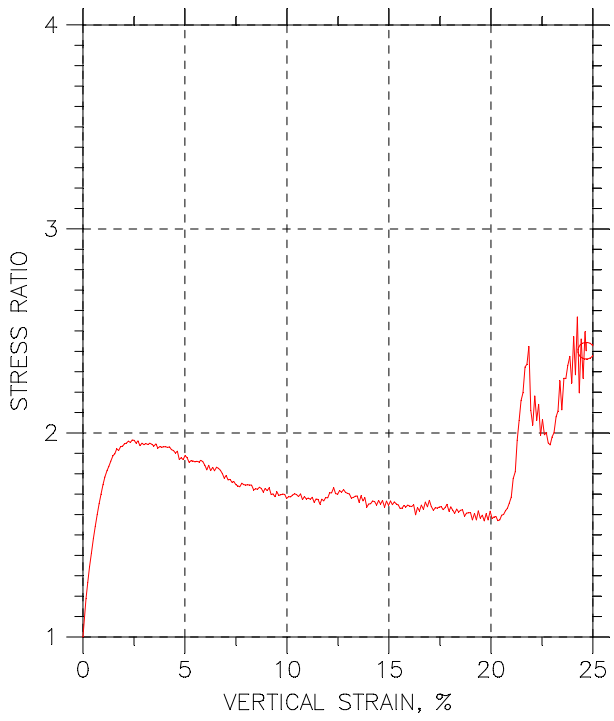
Description: MOIST TAMP RECONSTITUTED ASH - DARK GRAY

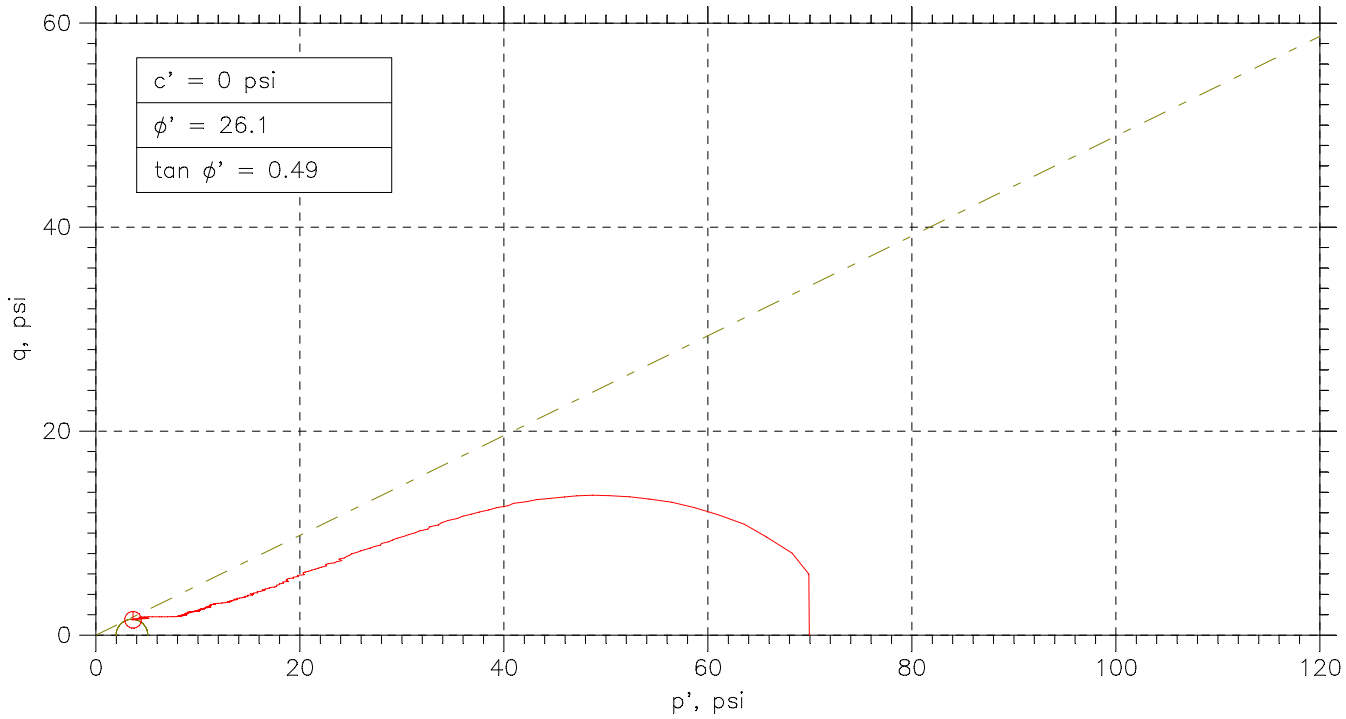
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (B=0.86)

Mon, 20-APR-2009 19:05:25

TRIAXIAL COMPRESSION TEST REPORT



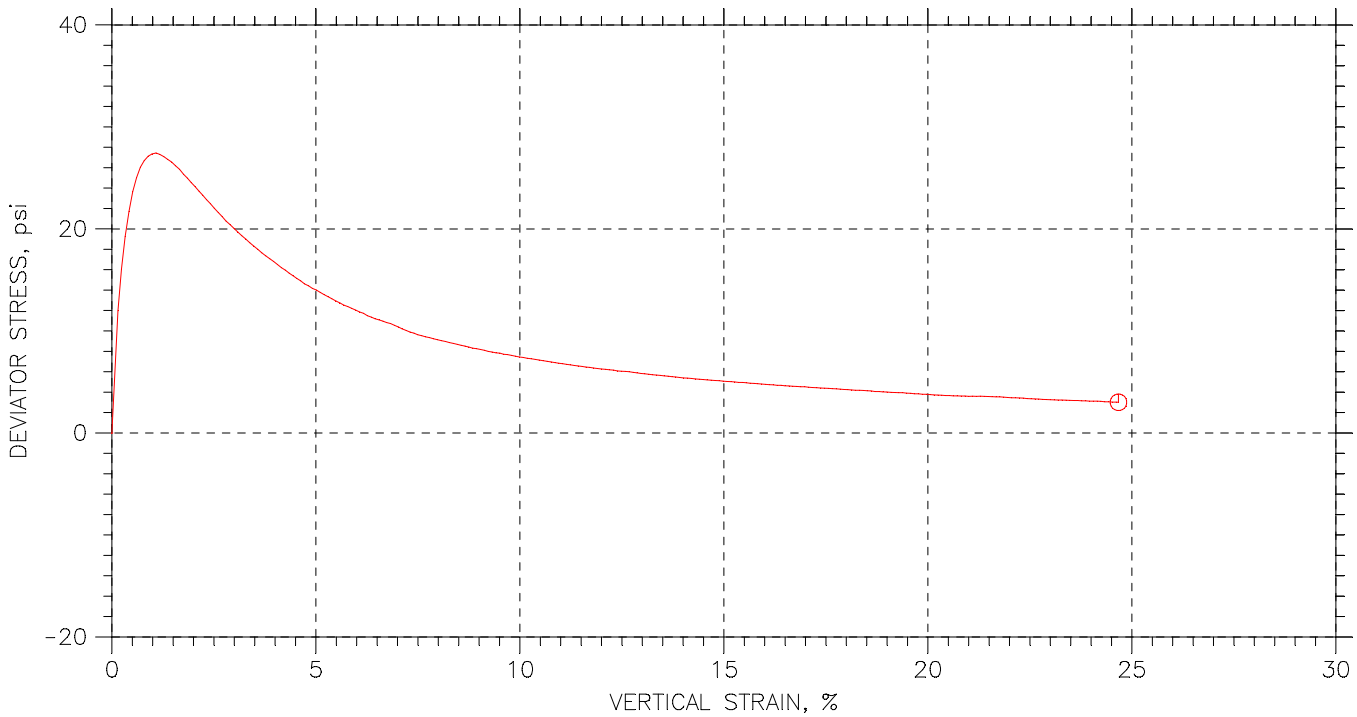


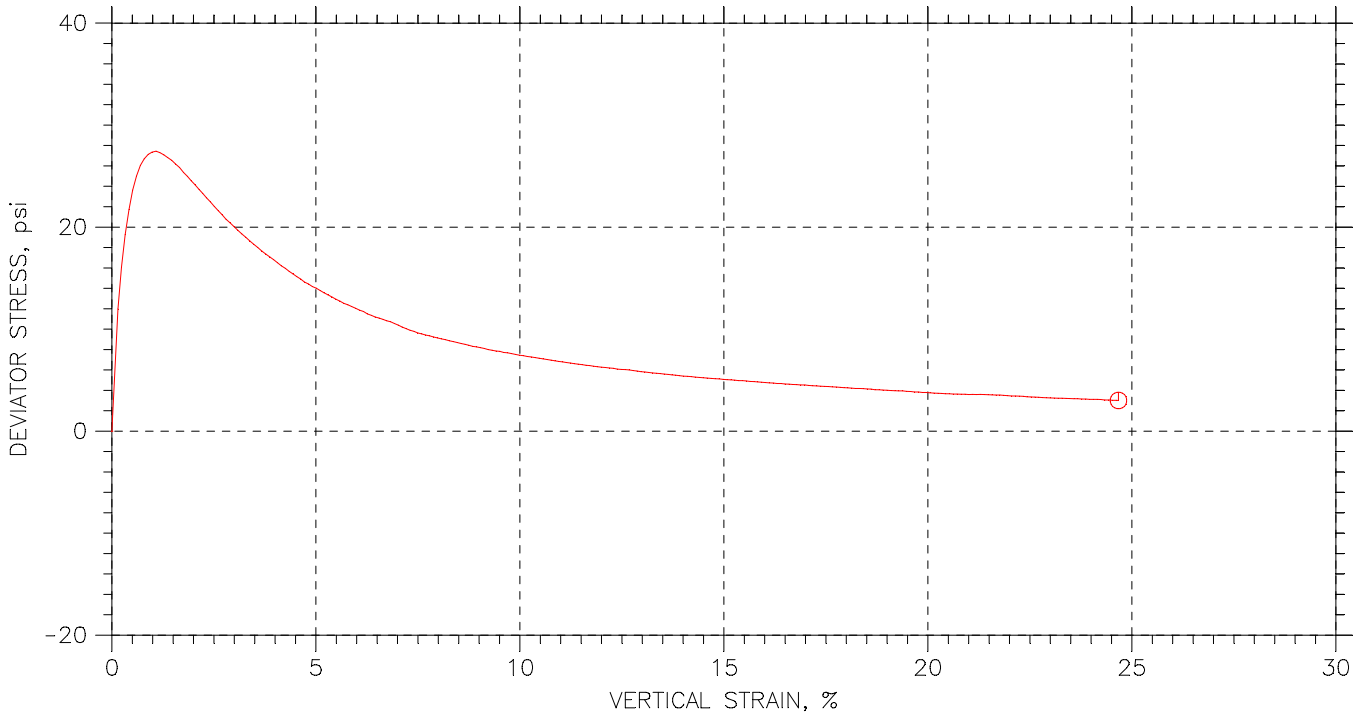


Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: RECONST	Tested By: BCM	Checked By: WPQ
Sample No.: 200s	Test Date: 4/18/09	Depth: ----
Test No.: MTe09s70	Sample Type: RECONST	Elevation: -----
Description: MOIST TAMP RECONSTITUTED ASH - DARK GRAY		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (B=0.86)		

Mon, 20-APR-2009 19:05:26

TRIAXIAL COMPRESSION TEST REPORT

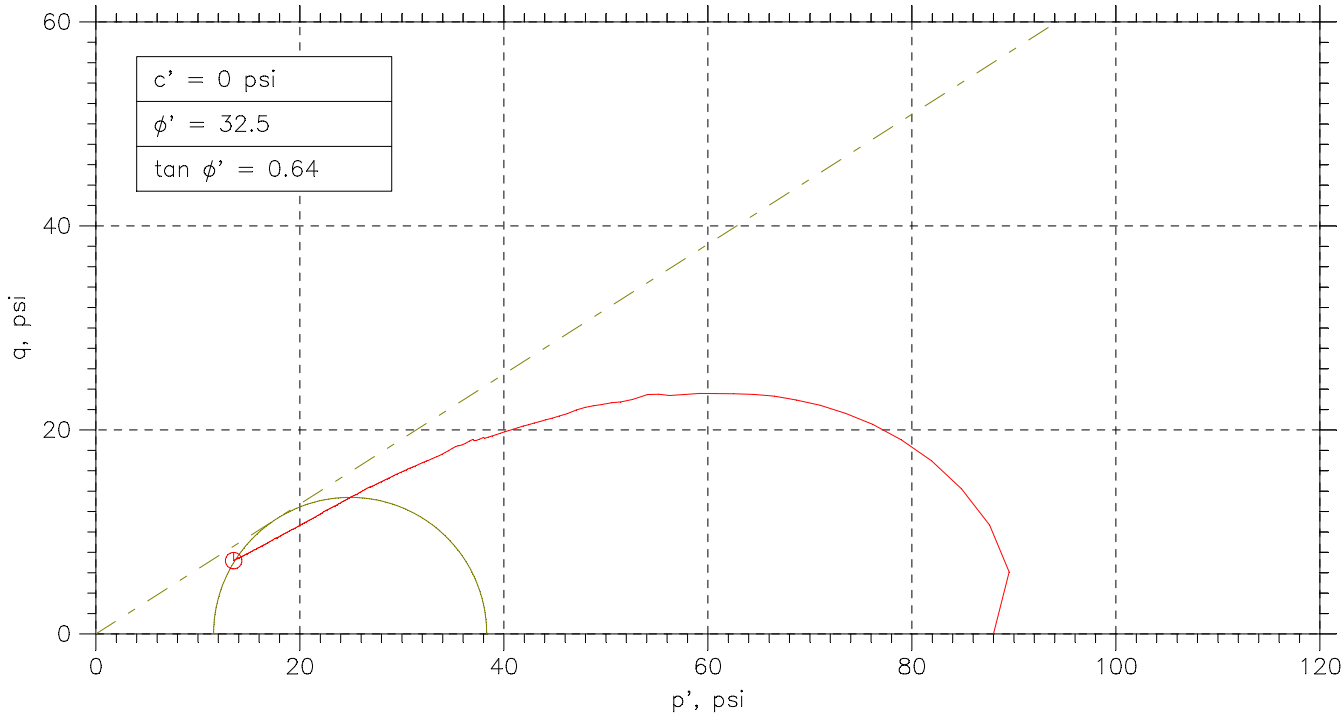


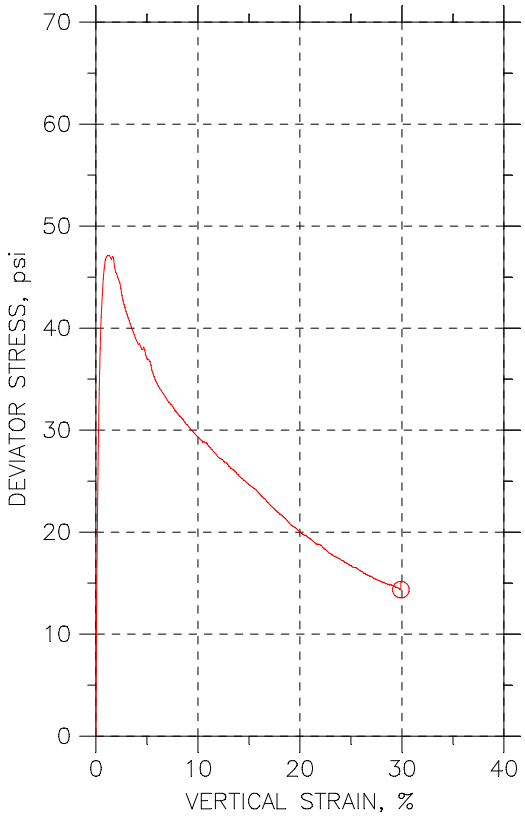


Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: RECONST	Tested By: BCM	Checked By: WPQ
Sample No.: 200s	Test Date: 4/18/09	Depth: ----
Test No.: MTe09s70	Sample Type: RECONST	Elevation: -----
Description: MOIST TAMP RECONSTITUTED ASH - DARK GRAY		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (B=0.86)		

Mon, 20-APR-2009 19:05:26

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⏲			
Test No.		MTe08s96			
Initial	Diameter, mm	70.285			
	Height, mm	155.01			
	Water Content, %	40.59			
	Dry Density, pcf	74.76			
	Saturation, %	100.01			
	Void Ratio	0.94571			
Before Shear	Water Content, %	36.06			
	Dry Density, pcf	79.04			
	Saturation, %	100.00			
	Void Ratio	0.84018			
	Back Press., psi	46			
Minor Prin. Stress, psi		88			
Max. Dev. Stress, psi		47.142			
Time to Failure, min		70.004			
Strain Rate, %/min		0.08			
B-Value		---			
Estimated Specific Gravity		2.33			
Liquid Limit		NP			
Plastic Limit		NP			
Plasticity Index		NP			

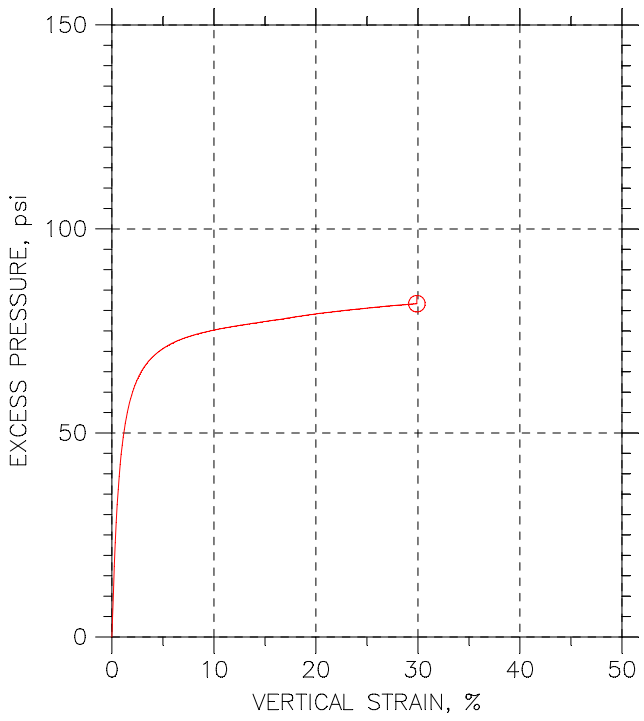
Project: KINGSTON COAL TVA	Failure Sketch			
Location: TN				
Project No.: 60095742				
Boring No.: RECONST				
Sample Type: RECONST				

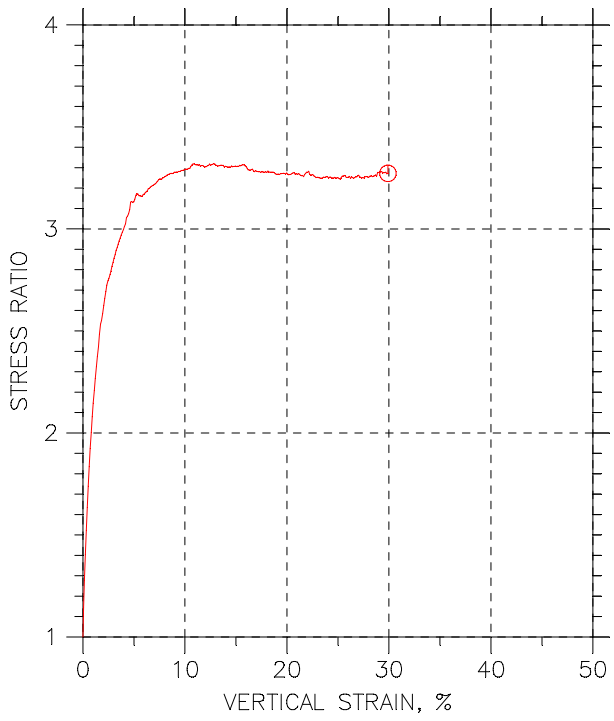
Description: MOIST TAMPED RECONSTITUTED ASH - DARK GRAY

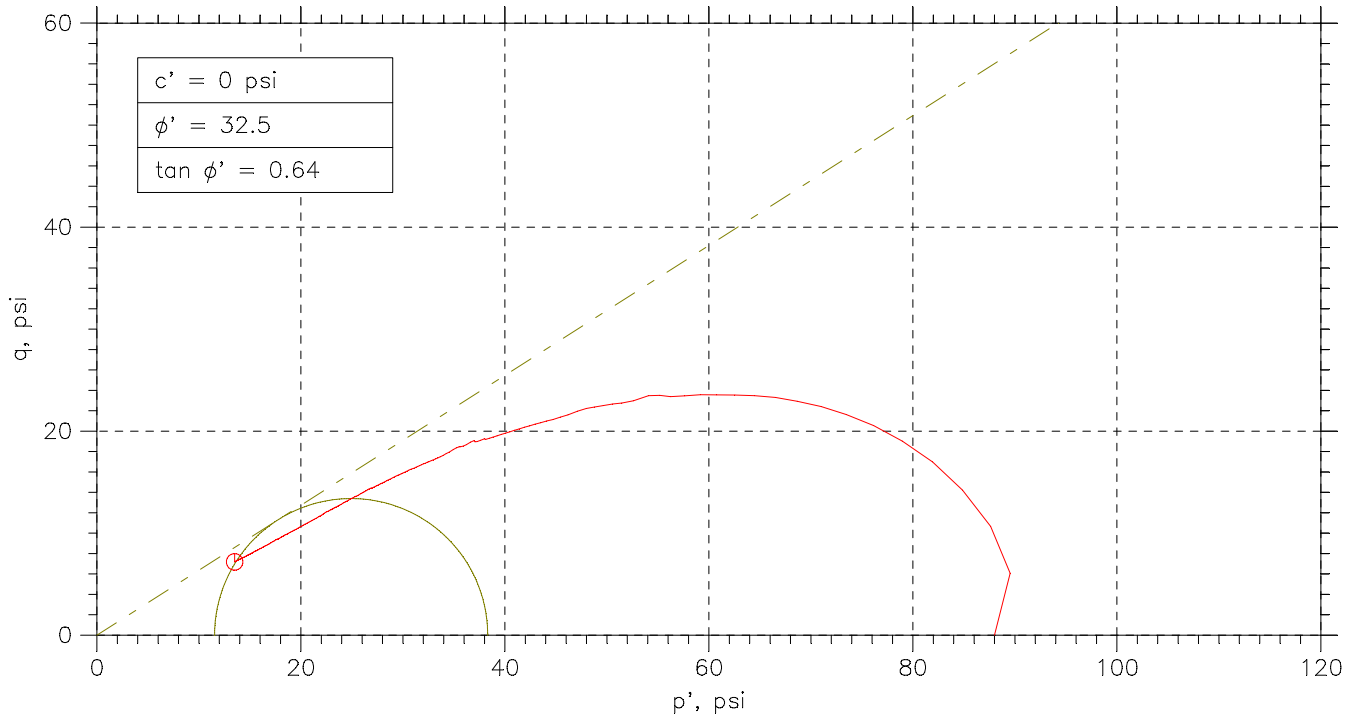
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (B=0.98)

Mon, 20-APR-2009 18:40:58

TRIAXIAL COMPRESSION TEST REPORT



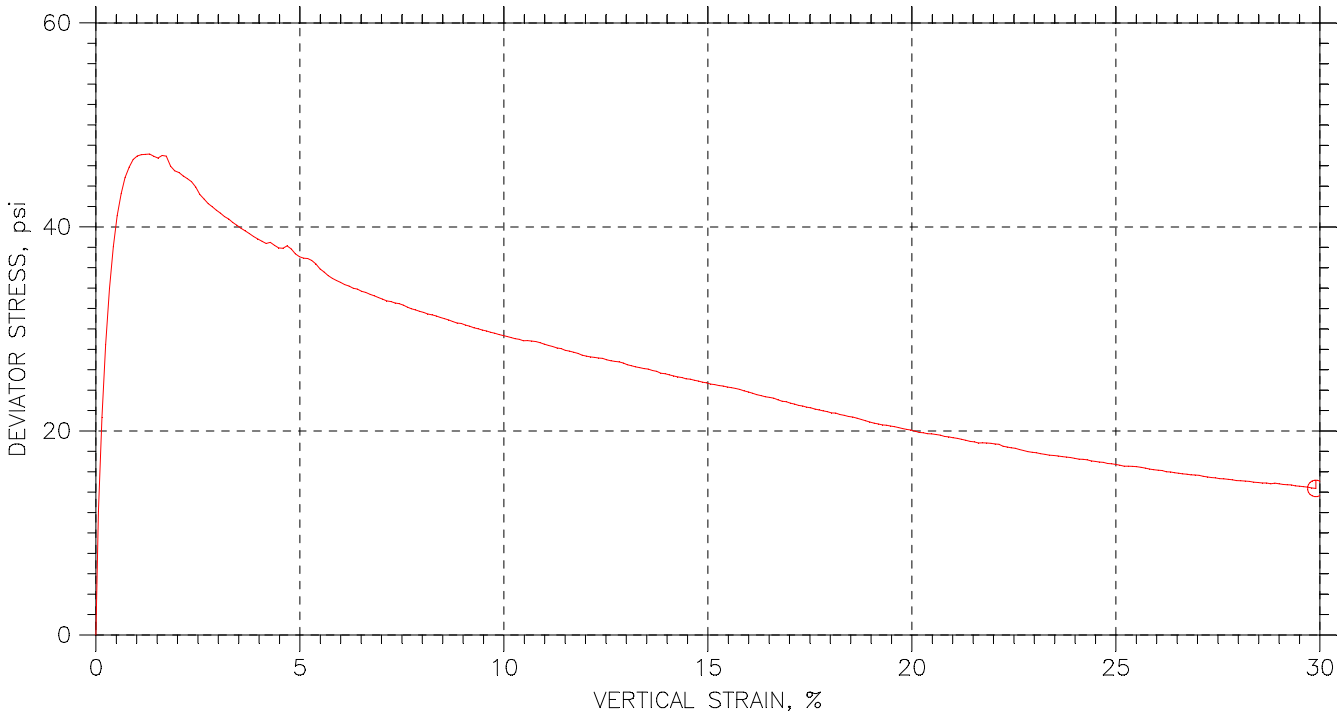


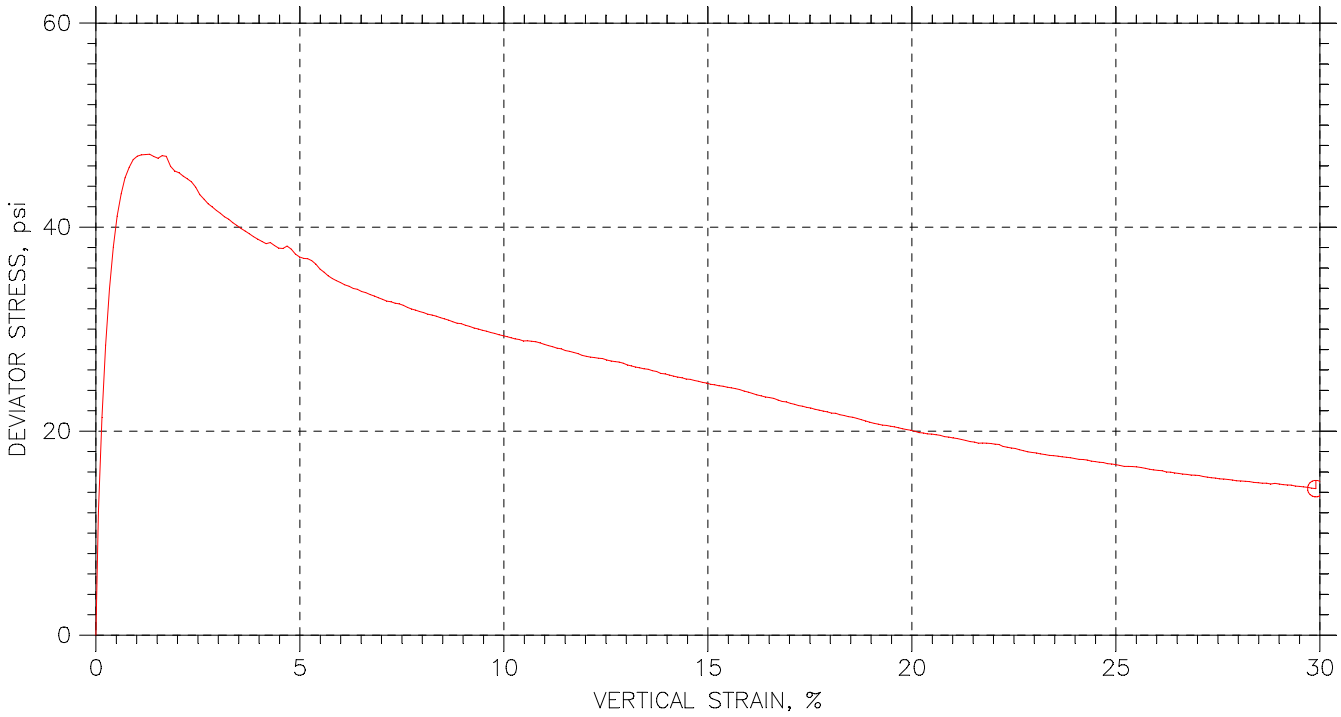


Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: RECONST	Tested By: BCM	Checked By: WPQ
Sample No.: 200s	Test Date: 04/07/09	Depth: -----
Test No.: MTe08s96	Sample Type: RECONST	Elevation: -----
Description: MOIST TAMPED RECONSTITUTED ASH - DARK GRAY		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (B=0.98)		

Mon, 20-APR-2009 18:40:58

TRIAXIAL COMPRESSION TEST REPORT

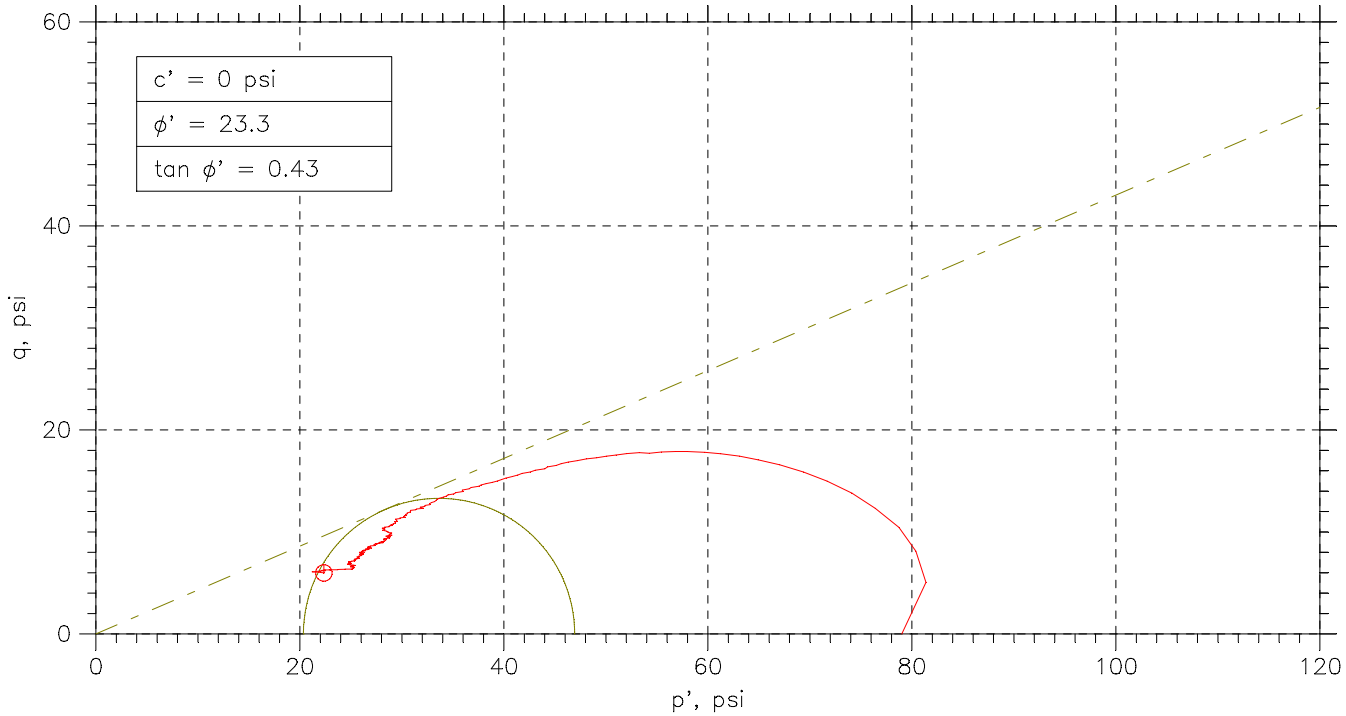


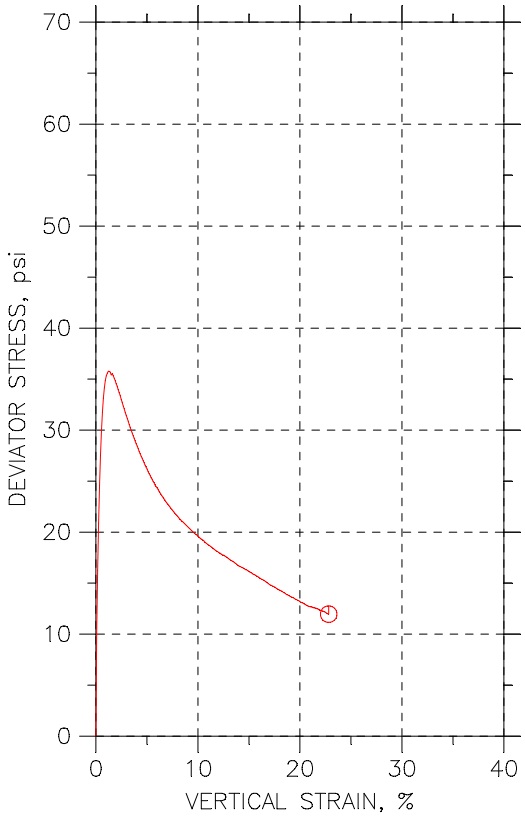


Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: RECONST	Tested By: BCM	Checked By: WPQ
Sample No.: 200s	Test Date: 04/07/09	Depth: -----
Test No.: MTe08s96	Sample Type: RECONST	Elevation: -----
Description: MOIST TAMPED RECONSTITUTED ASH - DARK GRAY		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (B=0.98)		

Mon, 20-APR-2009 18:40:59

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⏲			
Test No.		MTe0.9s80			
Initial	Diameter, mm	69.127			
	Height, mm	152.09			
	Water Content, %	49.02			
	Dry Density, pcf	67.91			
	Saturation, %	100.01			
	Void Ratio	1.142			
Before Shear	Water Content, %	37.96			
	Dry Density, pcf	77.19			
	Saturation, %	100.00			
	Void Ratio	0.88449			
	Back Press., psi	70			
Minor Prin. Stress, psi		79			
Max. Dev. Stress, psi		35.769			
Time to Failure, min		65			
Strain Rate, %/min		0.069			
B-Value		---			
Estimated Specific Gravity		2.33			
Liquid Limit		NP			
Plastic Limit		NP			
Plasticity Index		NP			

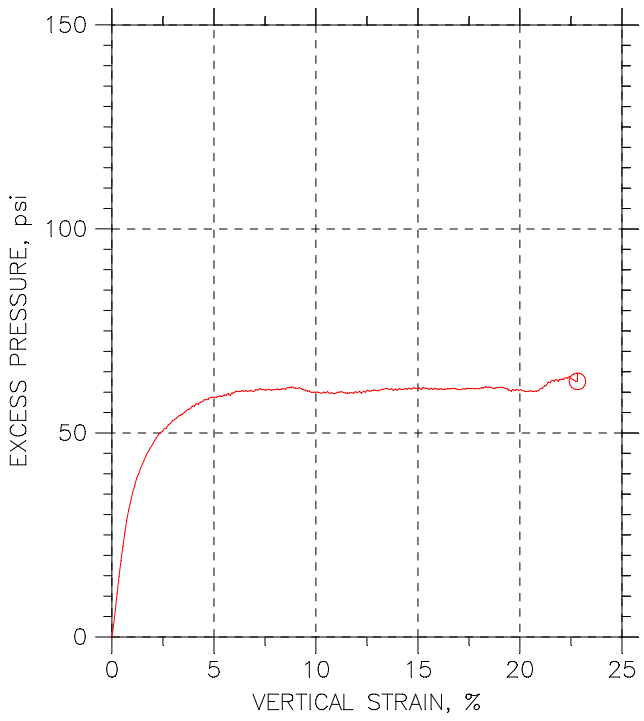
Project: KINGSTON COAL TVA	Failure Sketch			
Location: TN				
Project No.: 60095742				
Boring No.: RECONST				
Sample Type: RECONST				

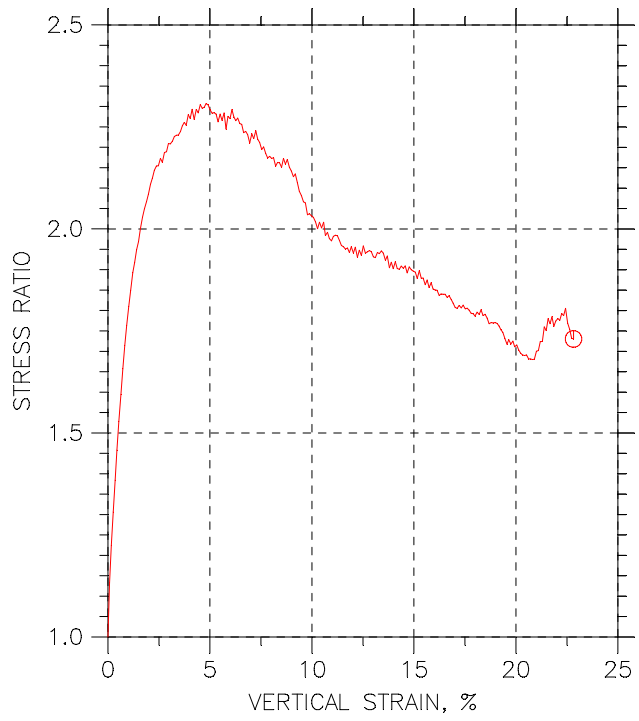
Description: MOIST TAMP RECONSTITUTED ASH - DARK GRAY

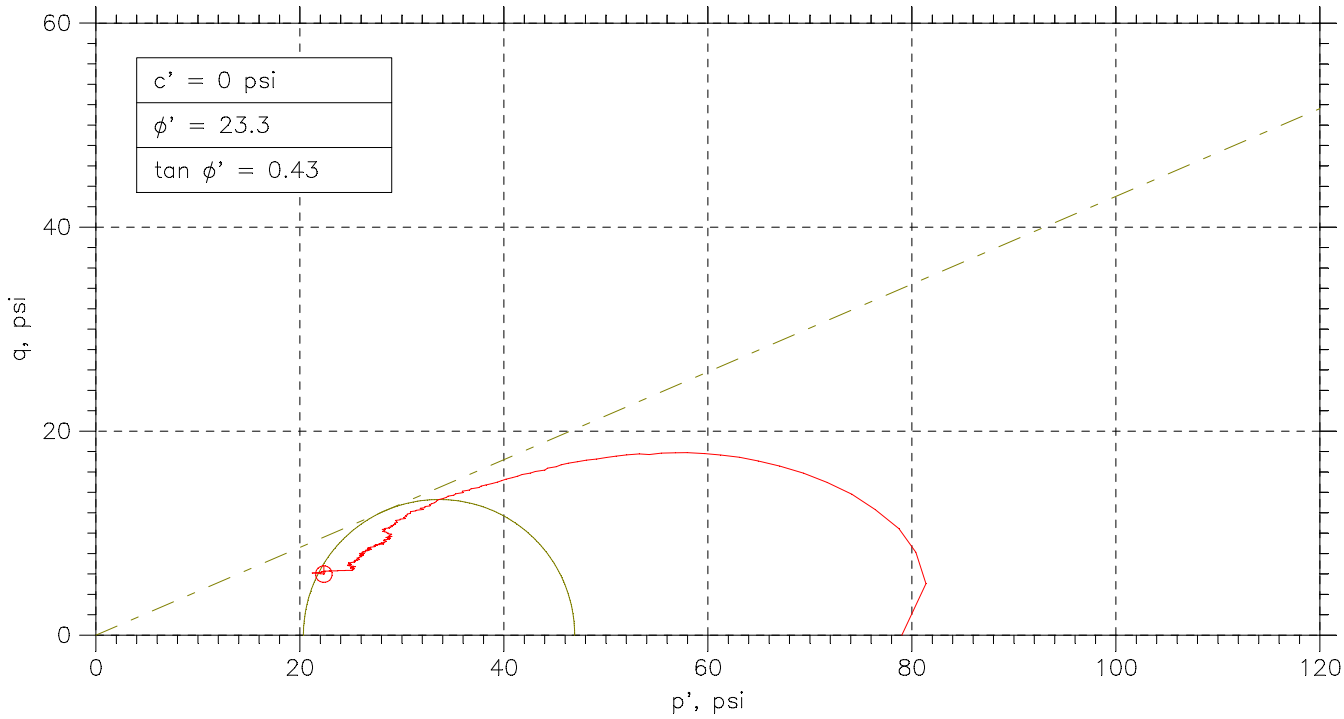
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (B=0.92)

Mon, 20-APR-2009 18:16:44

TRIAXIAL COMPRESSION TEST REPORT



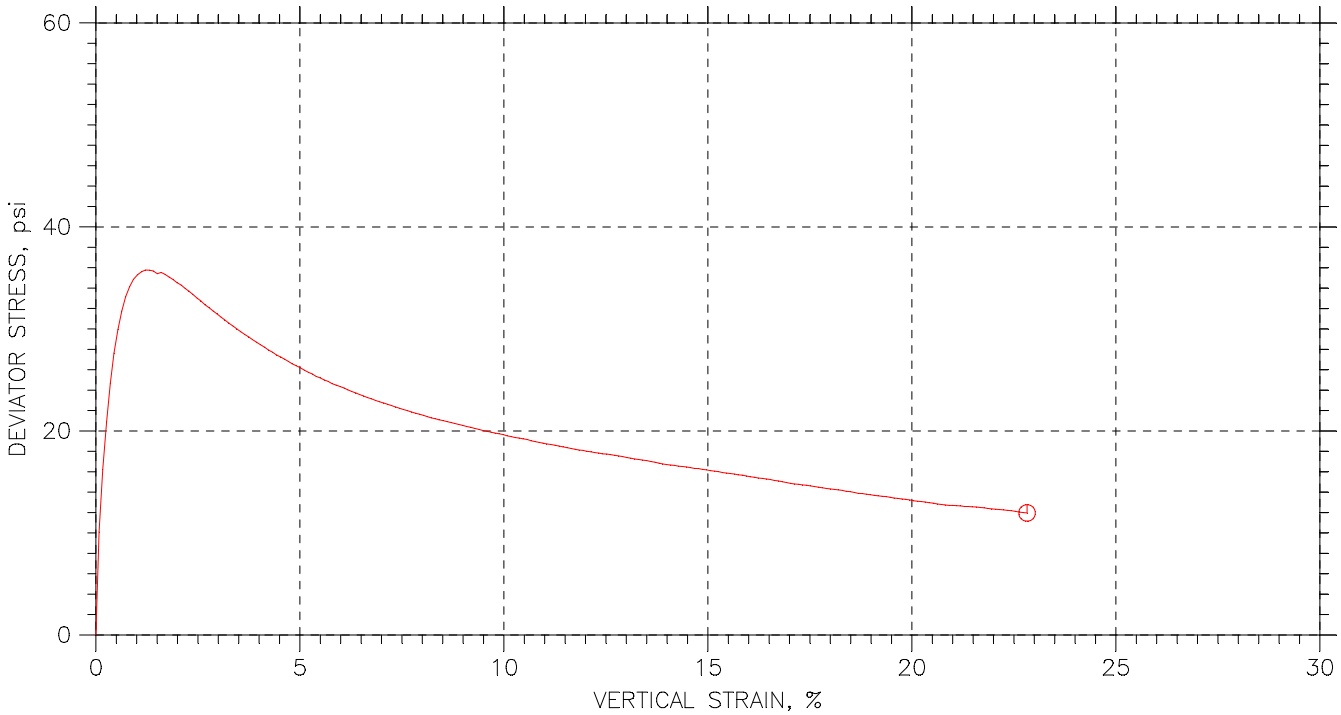


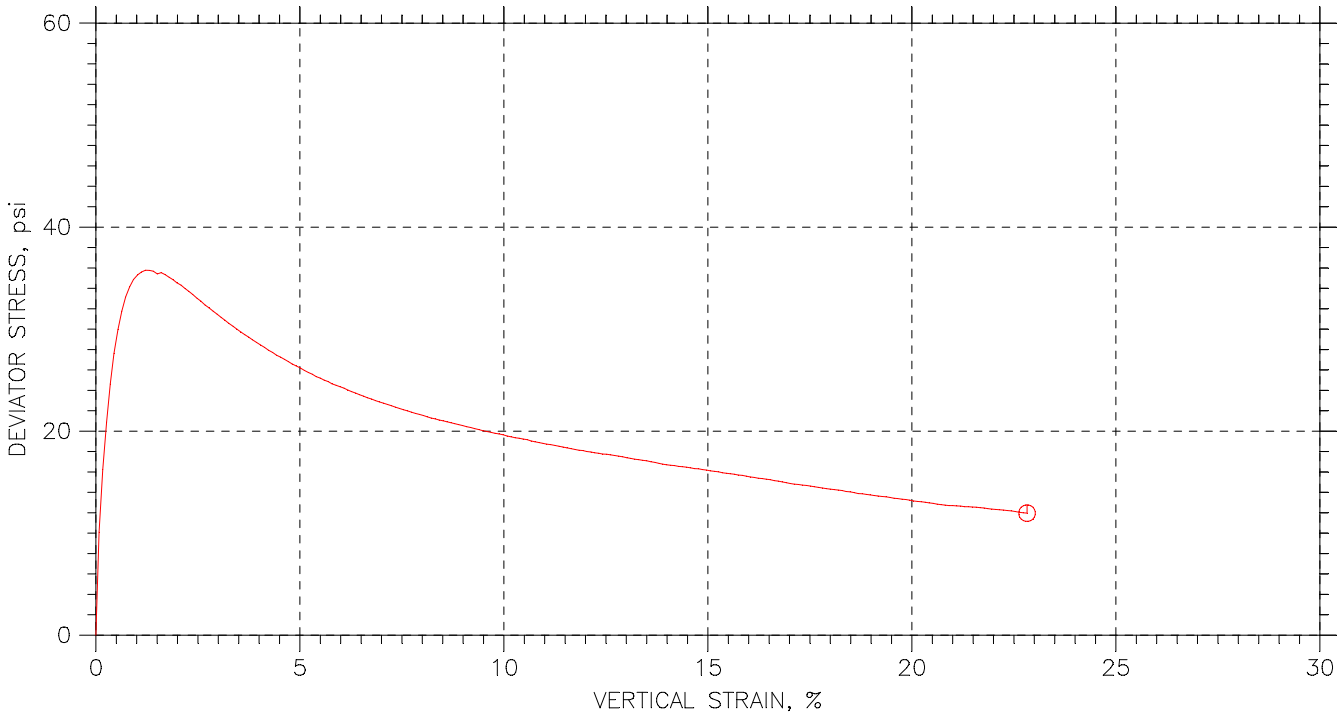


Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: RECONST	Tested By: BCM	Checked By: WPQ
Sample No.: 200S	Test Date: 4/17/09	Depth: ----
Test No.: MTe0.9s80	Sample Type: RECONST	Elevation: ----
Description: MOIST TAMP RECONSTITUTED ASH - DARK GRAY		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (B=0.92)		

Mon, 20-APR-2009 18:16:45

TRIAXIAL COMPRESSION TEST REPORT

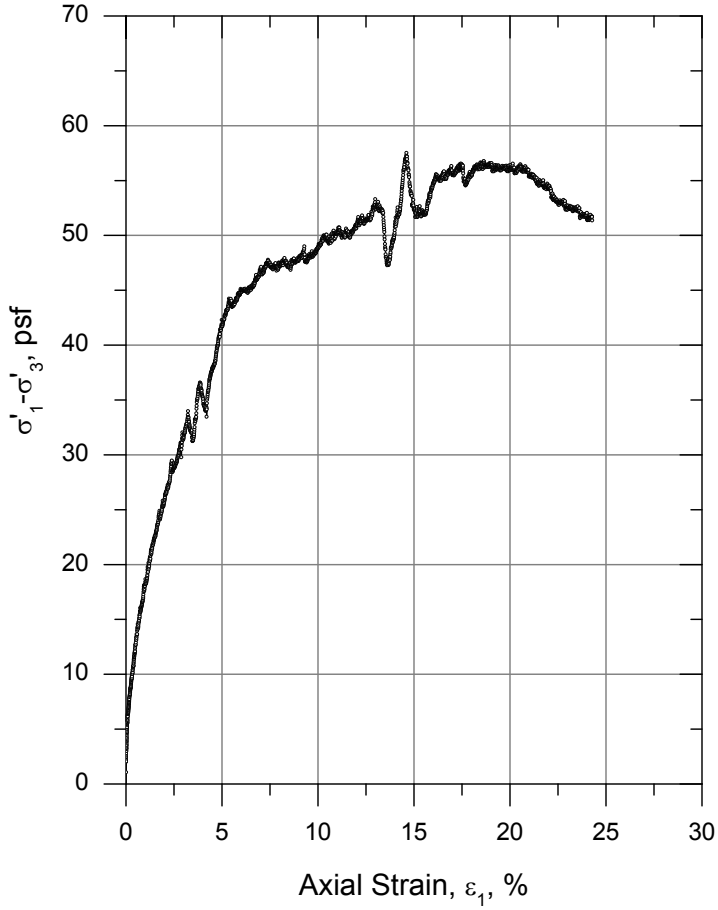




Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: RECONST	Tested By: BCM	Checked By: WPQ
Sample No.: 200S	Test Date: 4/17/09	Depth: ----
Test No.: MTe0.9s80	Sample Type: RECONST	Elevation: ----
Description: MOIST TAMP RECONSTITUTED ASH - DARK GRAY		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (B=0.92)		

Mon, 20-APR-2009 18:16:45

WORK IN PROGRESS

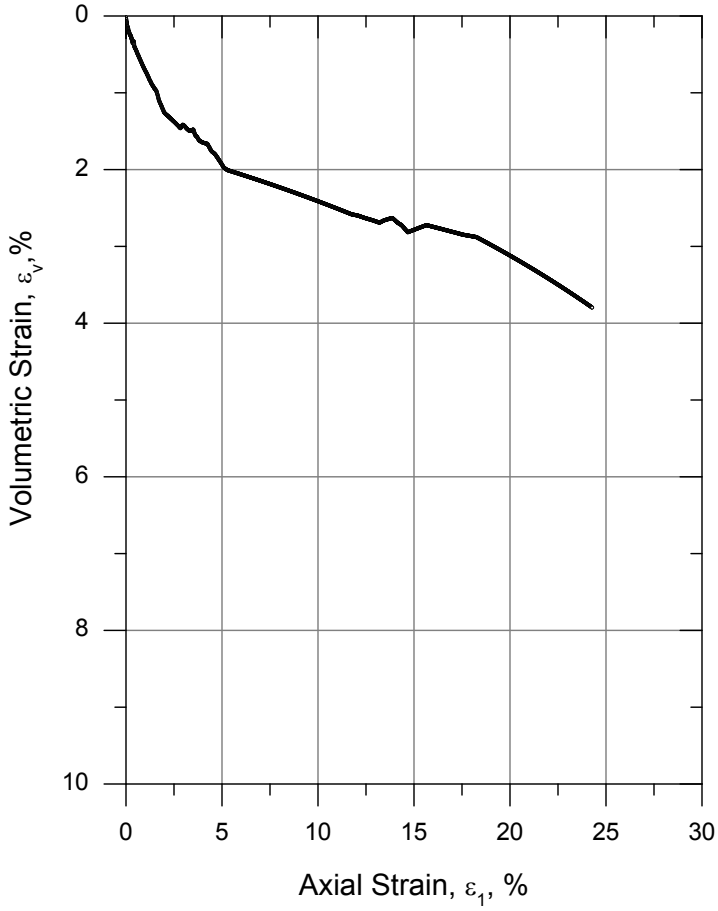


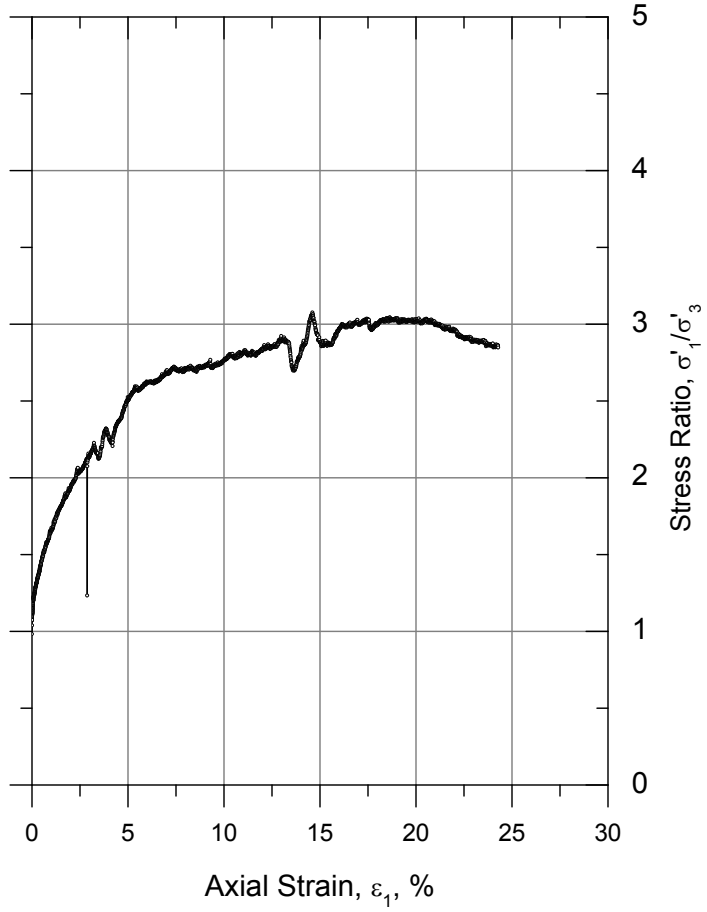
Project: TVA Kingston Dam
Project Number: 60095742

Sample: Reconstituted Fly Ash
Method: Moist Tamp

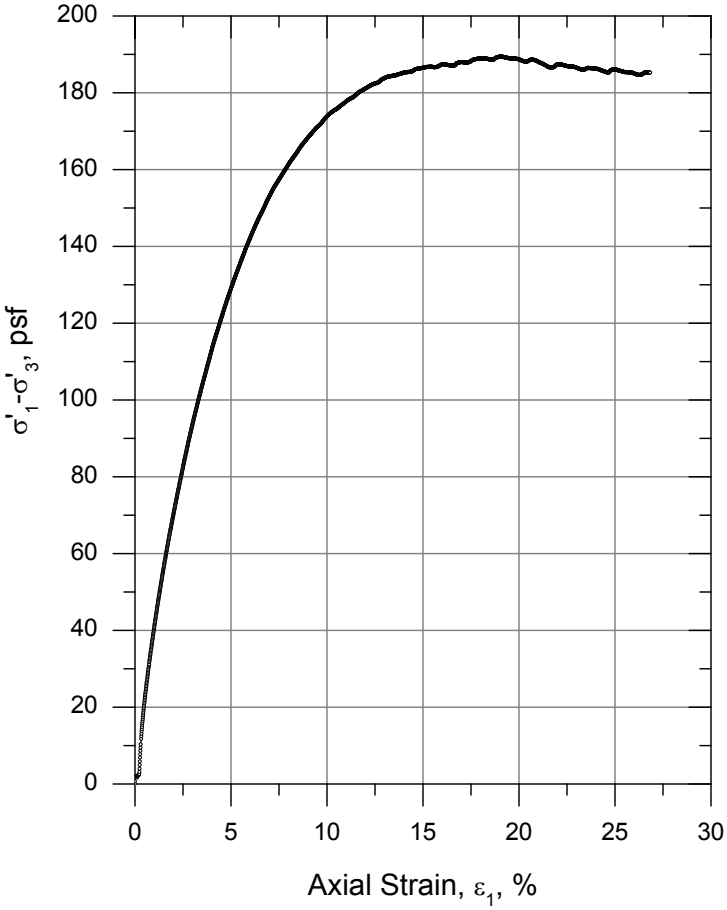
DRAINED TRIAXIAL COMPRESSION TEST RESULTS

Test ID		CID-301
Initial	Initial Void Ratio	0.90
	Specific Gravity	2.33
	Diameter, mm	71.18
	Height, mm	161.46
	Saturation, %	100.00
Shear	Effective Confining Stress, psi	28.00
	Type of Shear Test	CID-TXC
	Initial Void Ratio	0.87
	Final Void Ratio	0.83
Deviator Stress at Peak, psi		57.52
Axial Strain at Peak, psi		15
Mobilized Friction Angle, deg		31





WORK IN PROGRESS

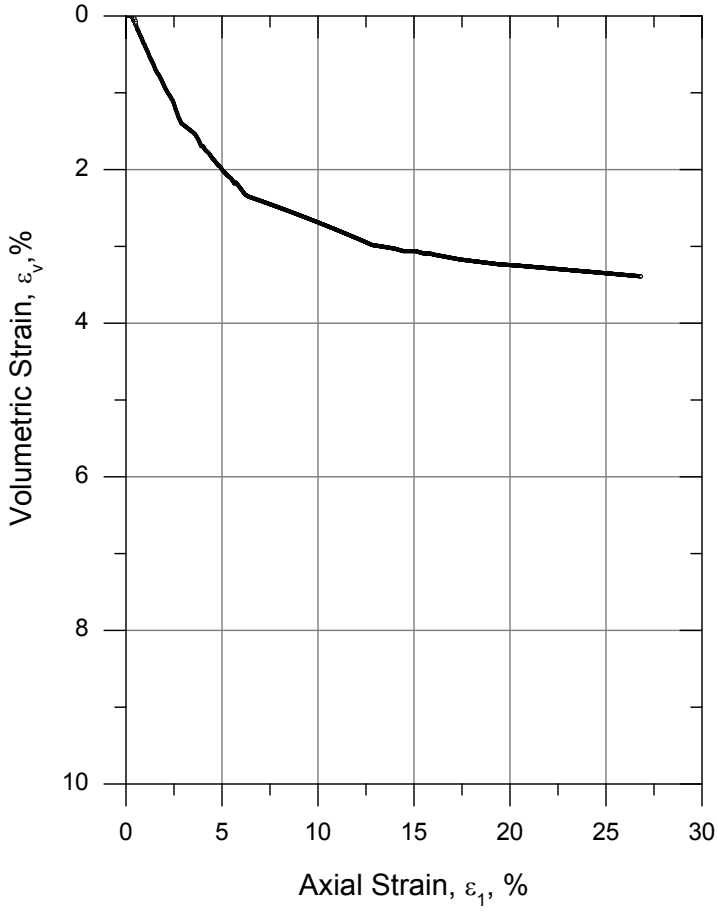


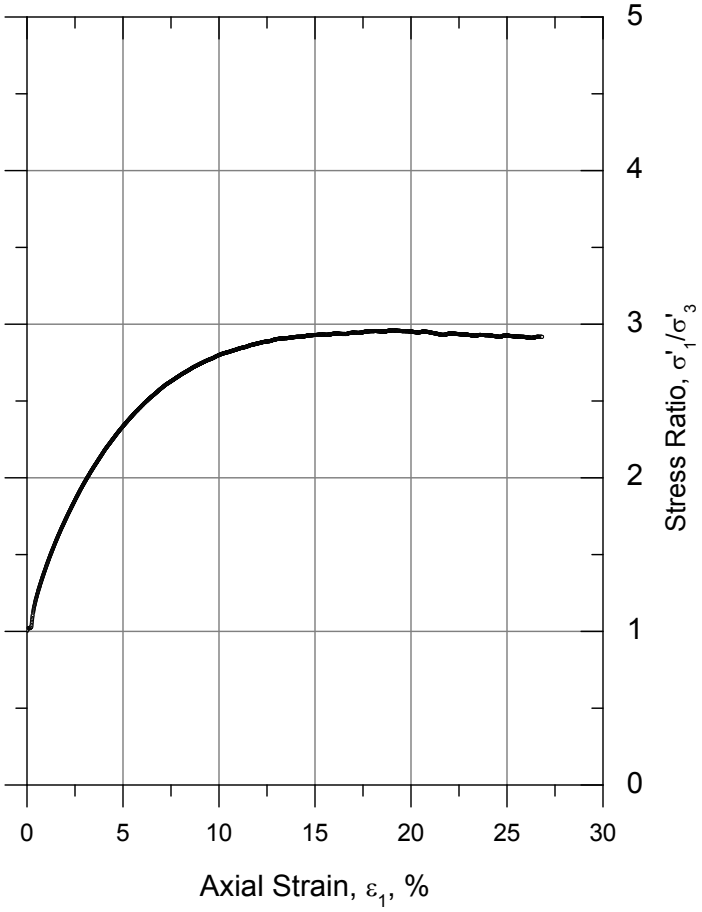
Project: TVA Kingston Dam
Project Number: 60095742

Sample: Reconstituted Fly Ash
Method: Moist Tamp

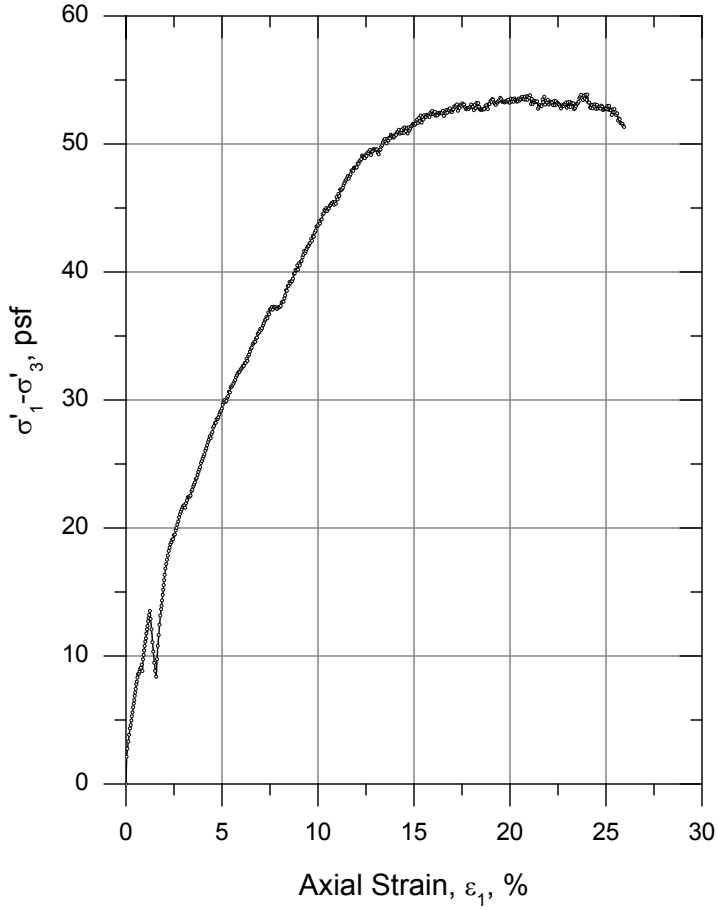
DRAINED TRIAXIAL COMPRESSION TEST RESULTS

Test ID		CID-302
Initial	Initial Void Ratio	0.86
	Specific Gravity	2.33
	Diameter, mm	70.92
	Height, mm	161.24
	Saturation, %	100.00
Shear	Effective Confining Stress, psi	96.72
	Type of Shear Test	CID-TXC
	Initial Void Ratio	0.82
	Final Void Ratio	0.79
Deviator Stress at Peak, psi		189.46
Axial Strain at Peak, psi		19.04
Mobilized Friction Angle, deg		30





WORK IN PROGRESS

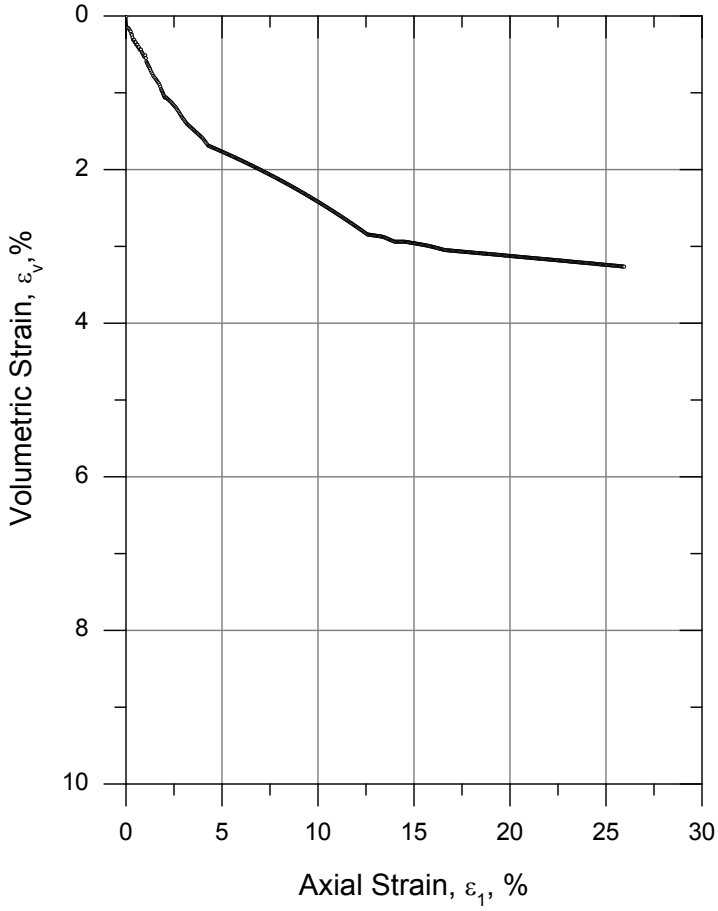


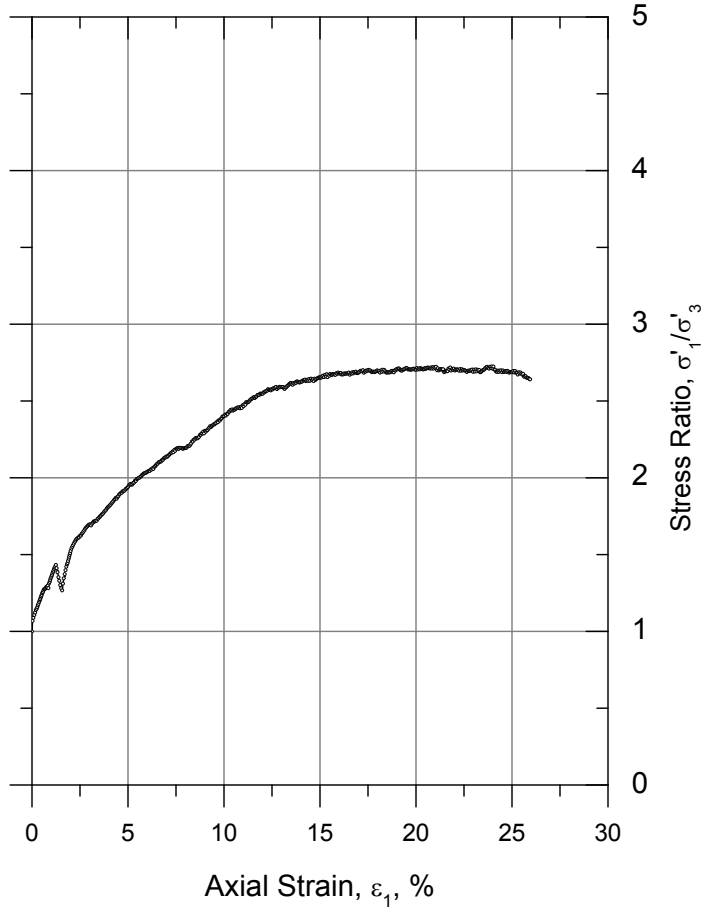
Project: TVA Kingston Dam
Project Number: 60095742

Sample: Reconstituted Fly Ash
Method: Moist Tamp

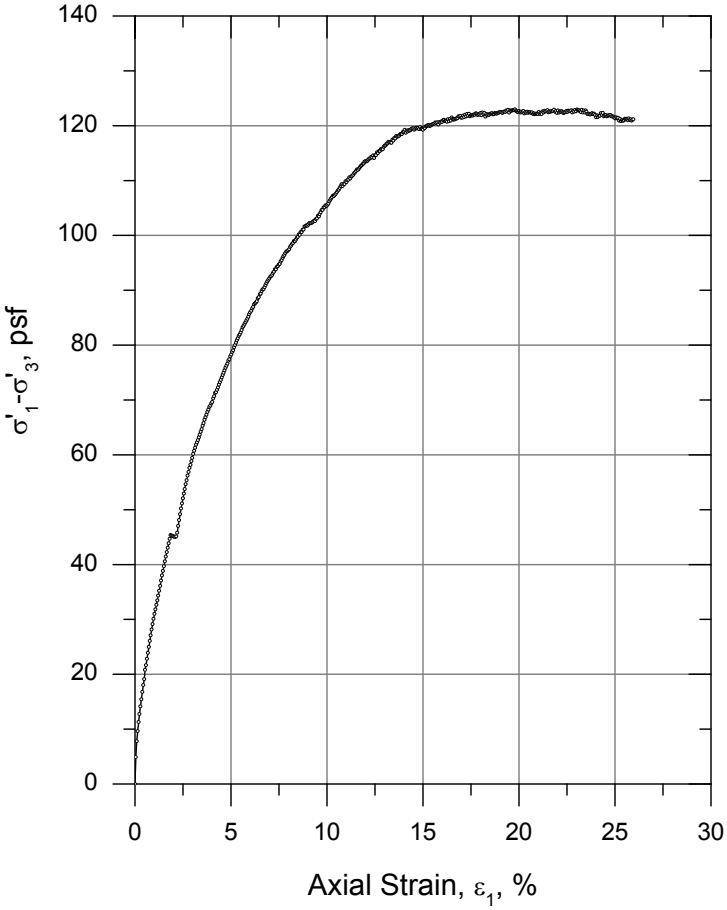
DRAINED TRIAXIAL COMPRESSION TEST RESULTS

Test ID		CID-304
Initial	Initial Void Ratio	0.92
	Specific Gravity	2.33
	Diameter, mm	70.97
	Height, mm	161.71
	Saturation, %	100.00
Shear	Effective Confining Stress, psi	31.25
	Type of Shear Test	CID-TXC
	Initial Void Ratio	0.87
	Final Void Ratio	0.84
Deviator Stress at Peak, psi		53.85
Axial Strain at Peak, psi		24.03
Mobilized Friction Angle, deg		28





WORK IN PROGRESS

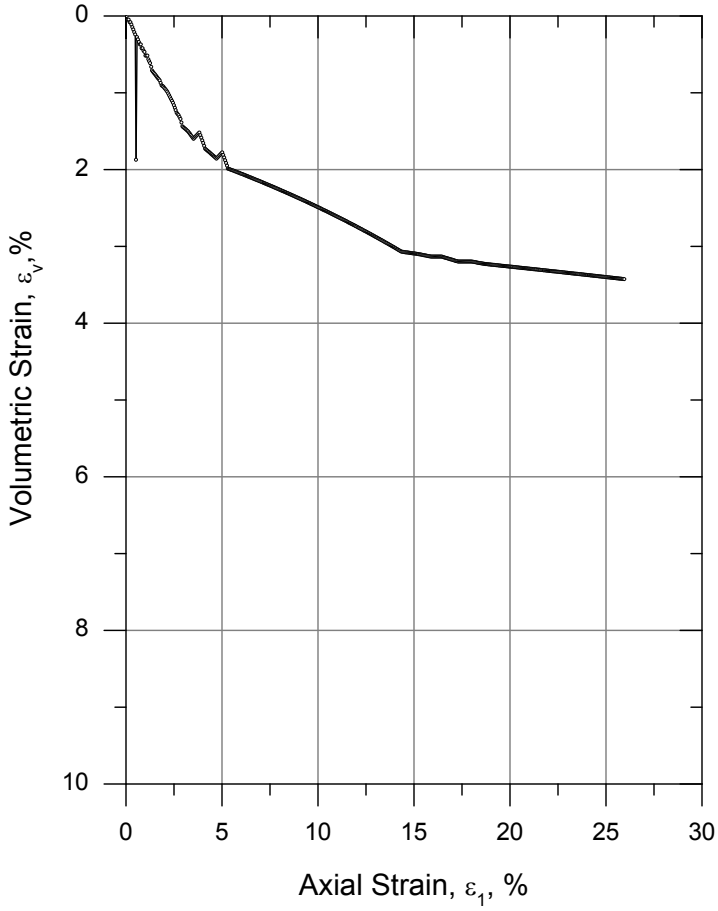


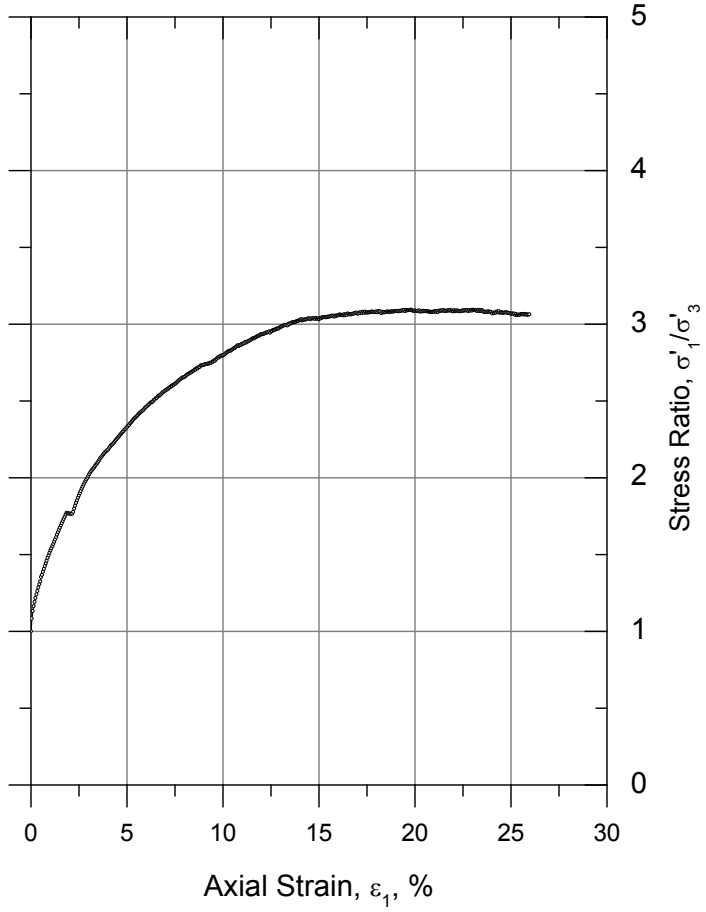
Project: TVA Kingston Dam
Project Number: 60095742

Sample: Reconstituted Fly Ash
Method: Moist Tamp

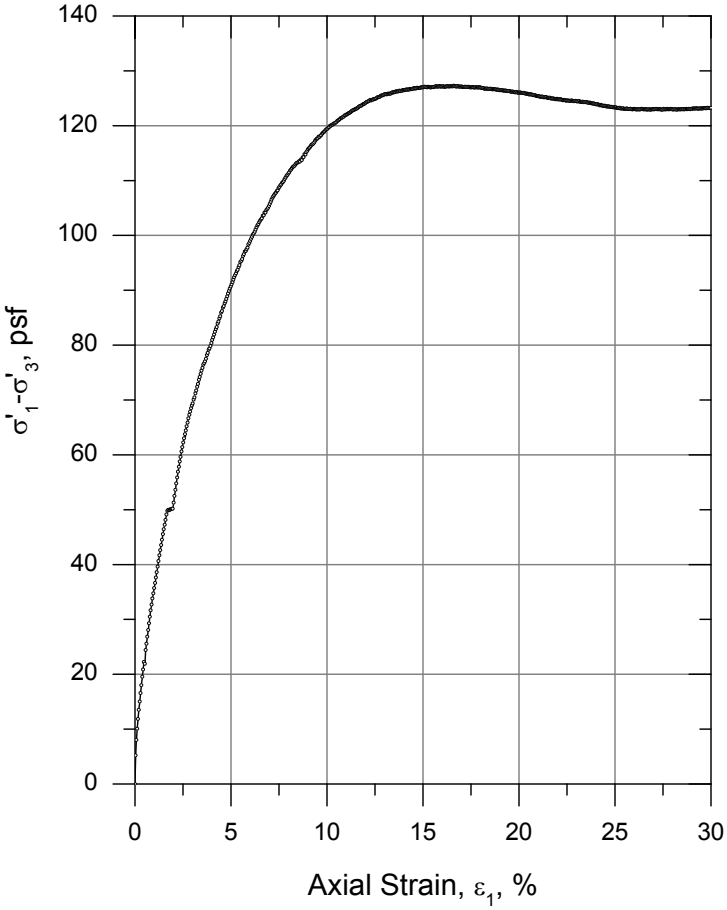
DRAINED TRIAXIAL COMPRESSION TEST RESULTS

Test ID		CID-305
Initial	Initial Void Ratio	0.93
	Specific Gravity	2.33
	Diameter, mm	70.62
	Height, mm	158.04
	Saturation, %	100.00
Shear	Effective Confining Stress, psi	58.82
	Type of Shear Test	CID-TXC
	Initial Void Ratio	0.89
	Final Void Ratio	0.85
Deviator Stress at Peak, psi		122.97
Axial Strain at Peak, psi		19.79
Mobilized Friction Angle, deg		31





WORK IN PROGRESS

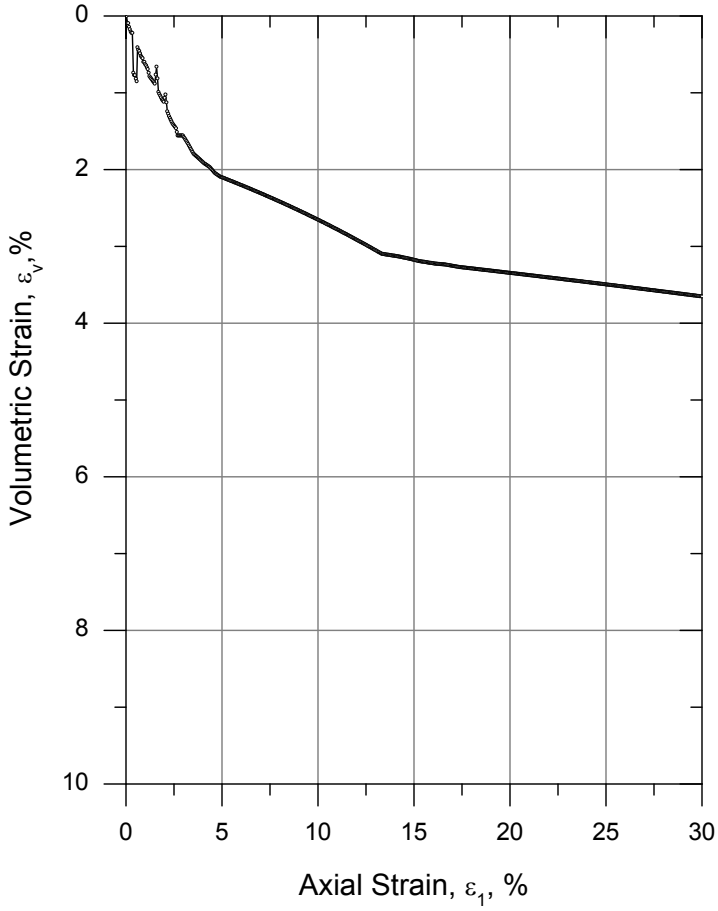


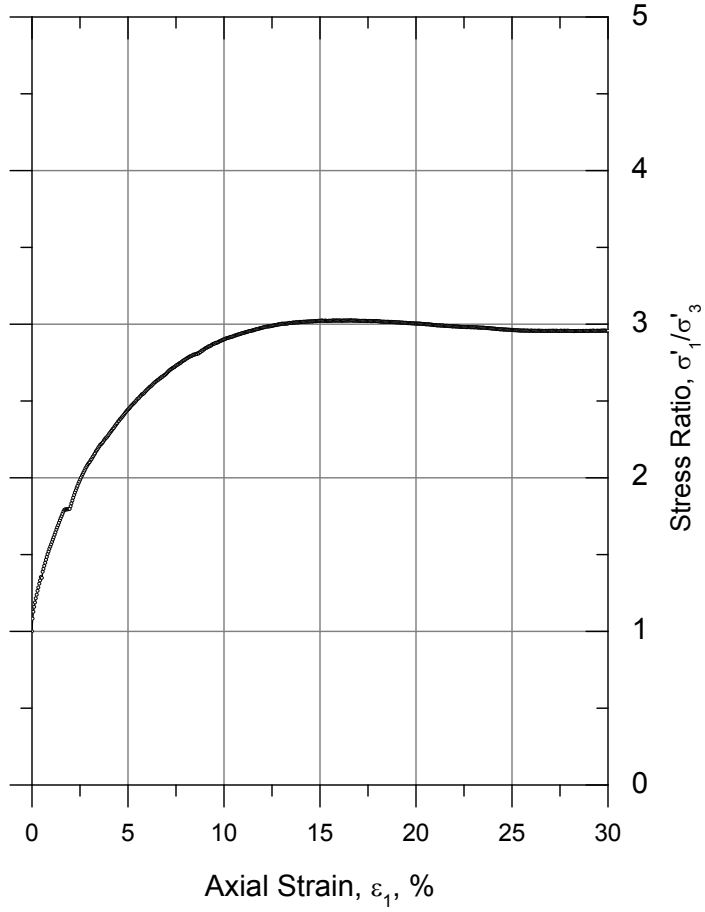
Project: TVA Kingston Dam
Project Number: 60095742

Sample: Reconstituted Fly Ash
Method: Moist Tamp

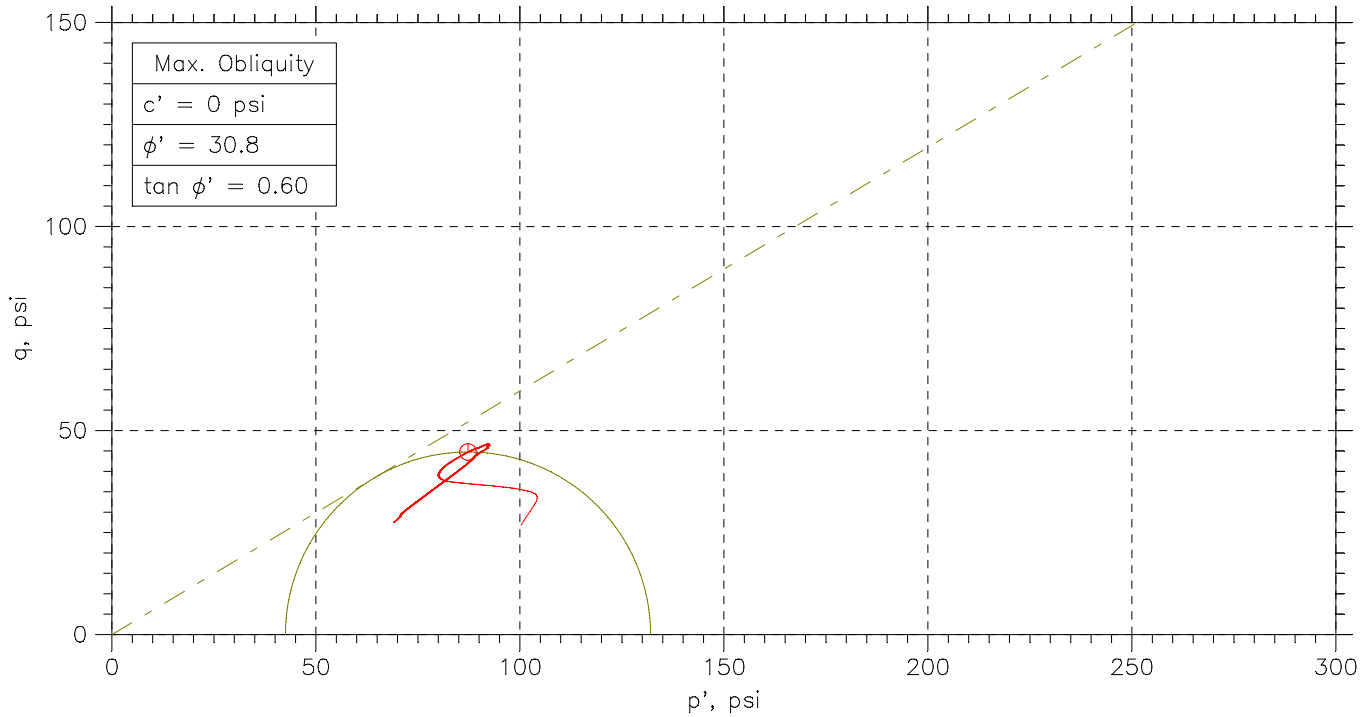
DRAINED TRIAXIAL COMPRESSION TEST RESULTS

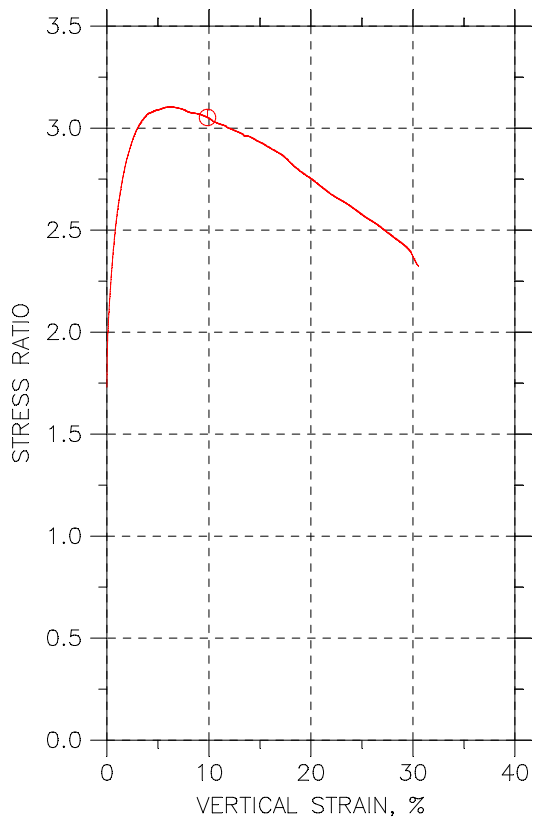
Test ID		CID-306
Initial	Initial Void Ratio	0.93
	Specific Gravity	2.33
	Diameter, mm	71.17
	Height, mm	159.89
	Saturation, %	100.00
Shear	Effective Confining Stress, psi	62.95
	Type of Shear Test	CID-TXC
	Initial Void Ratio	0.89
	Final Void Ratio	0.85
Deviator Stress at Peak, psi		127.29
Axial Strain at Peak, psi		16.61
Mobilized Friction Angle, deg		30





CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





Symbol		⏲			
Sample No.		200s			
Test No.		PLe08s128			
Depth		----			
Initial	Diameter, mm	70.68			
	Height, mm	163.5			
	Water Content, %	35.5			
	Dry Density, pcf	77.62			
	Saturation, %	94.7			
	Void Ratio	0.874			
Before Shear	Water Content, %	34.0			
	Dry Density, pcf	81.13			
	Saturation*, %	100.0			
	Void Ratio	0.793			
	Back Press., psi	56.72			
Ver. Eff. Cons. Stress, psi		73.45			
Shear Strength, psi		46.66			
Strain at Failure, %		9.86			
Strain Rate, %/min		0.08			
B-Value		0.94			
Estimated Specific Gravity		2.33			
Liquid Limit		NP			
Plastic Limit		NP			

Project: KINGSTON COAL

Location: TN

Project No.: 60095742

Boring No.: RECONST

Sample Type: RECONST

Description: PLUVIATED - RECONSTITUTED ASH - DARK GRAY

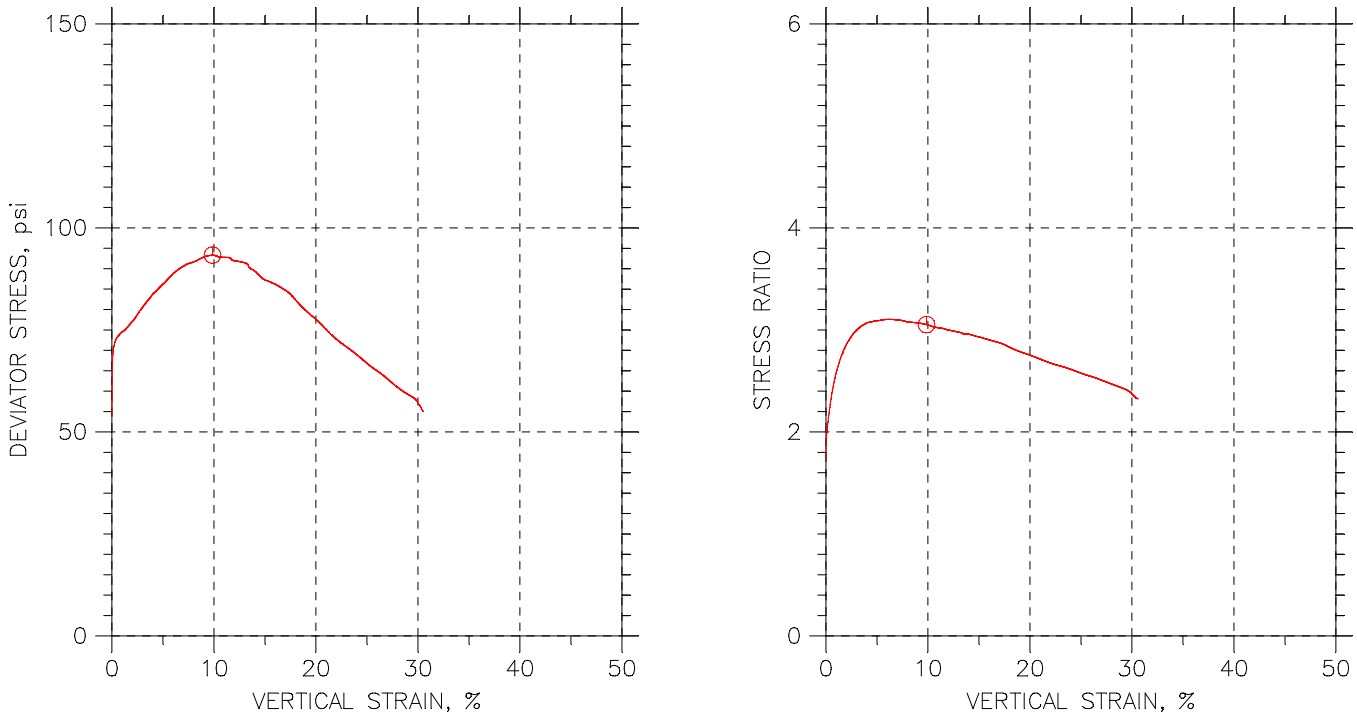
Remarks: FAILURE CRITERIA = MAX. OBLIQUITY, Ko CONSOLIDATION, B=0.95

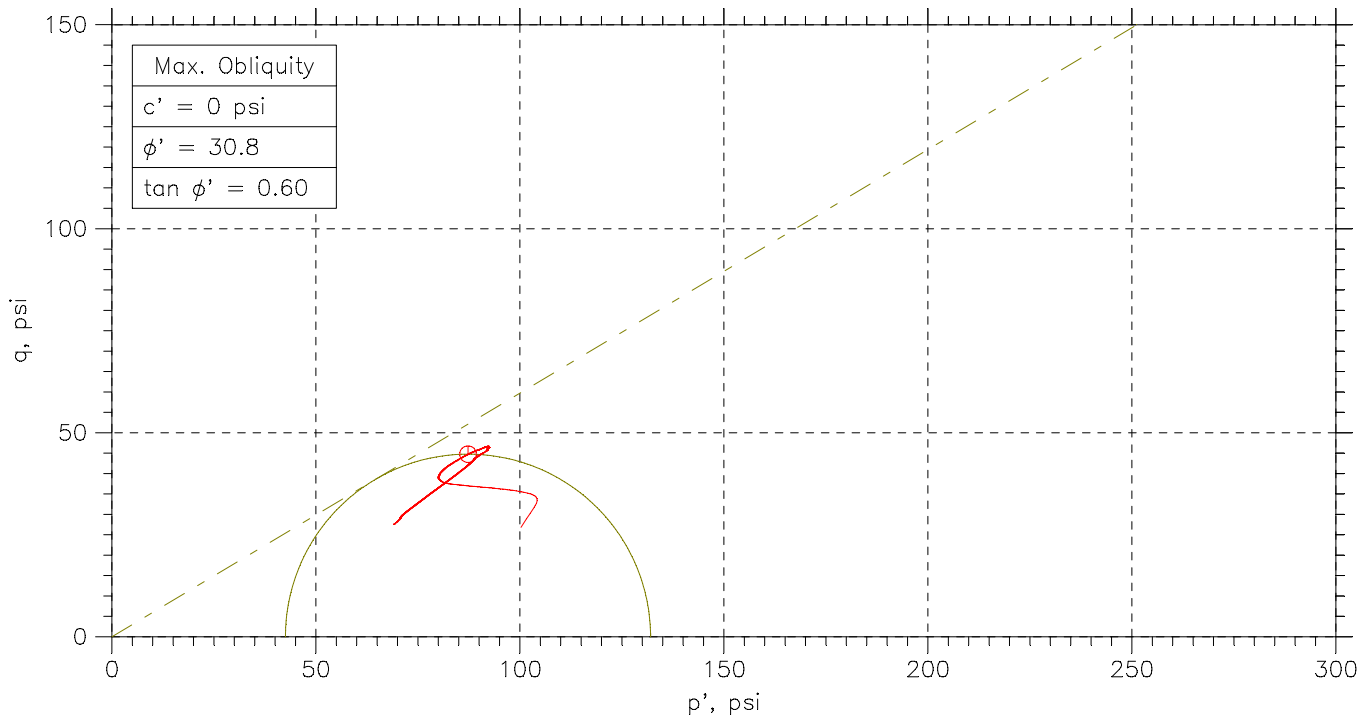
Phase calculations based on end of test.

* Saturation is set to 100% for phase calculations

Wed, 22-APR-2009 15:55:22

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



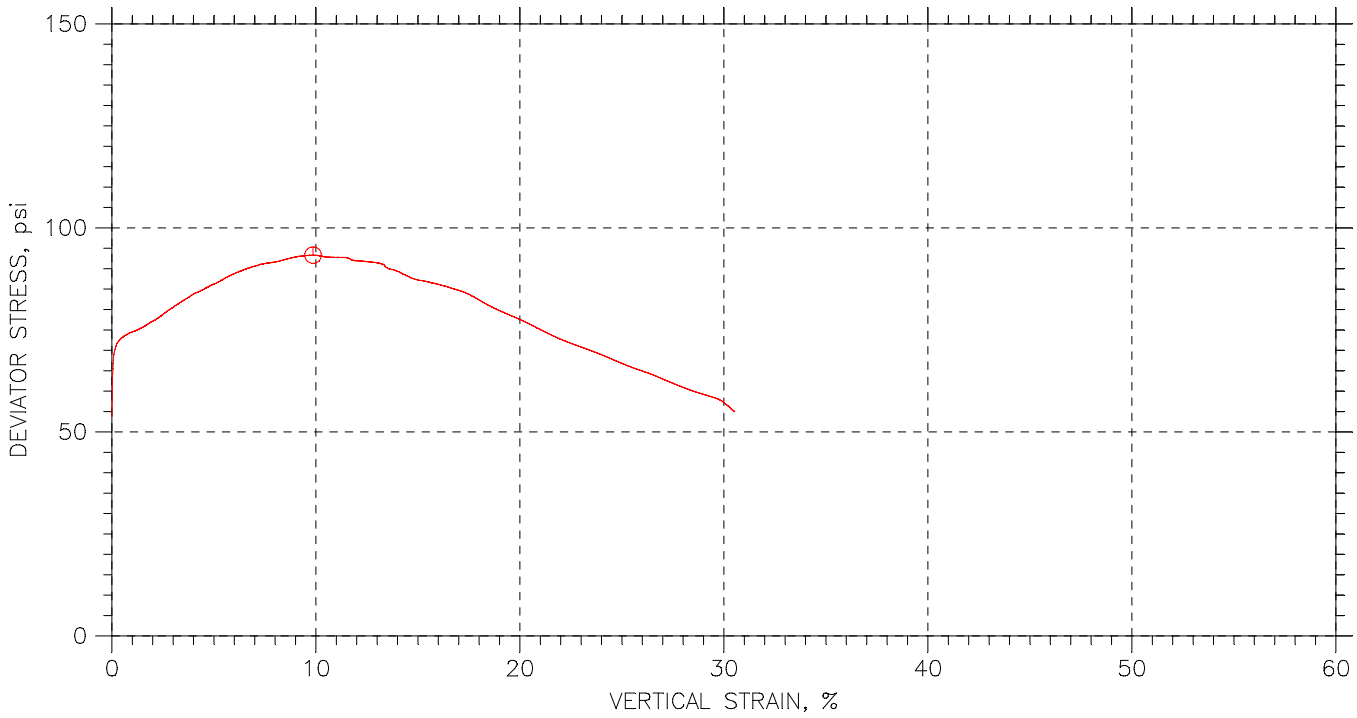


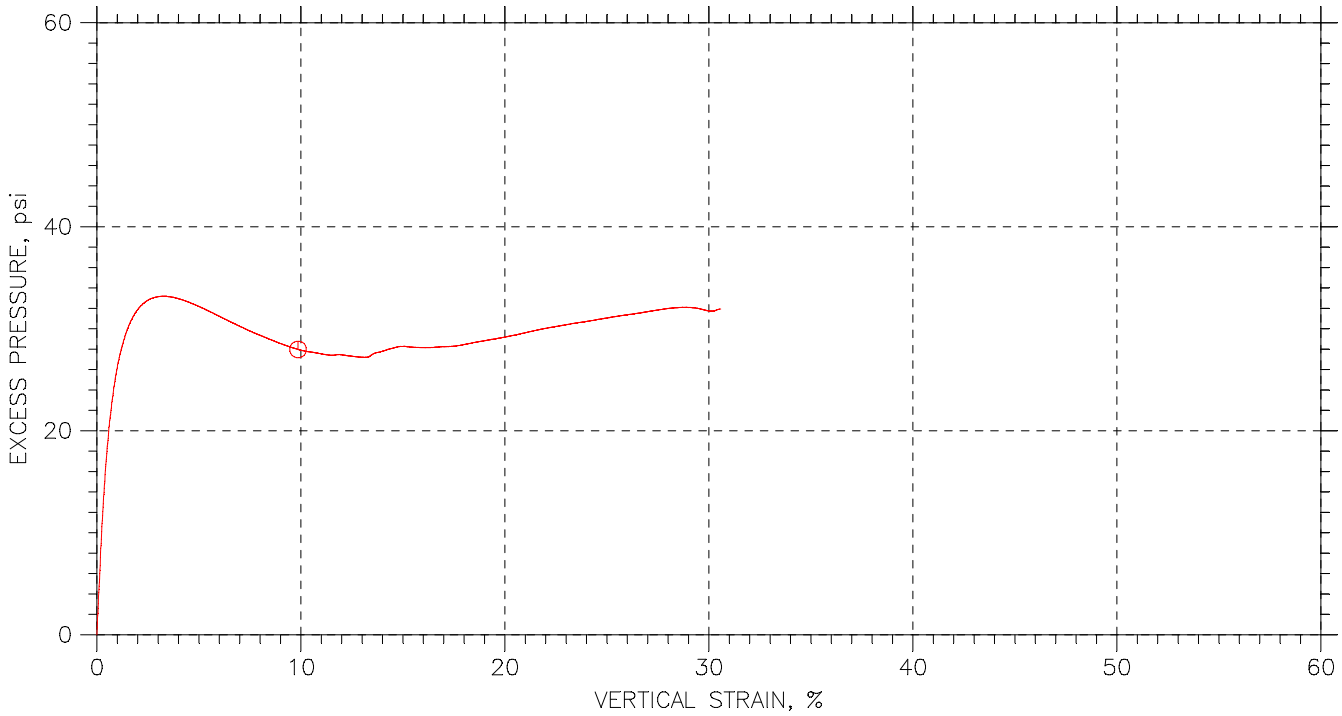
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
⏲	200s	PLe08s128	----	BCM	4/6/09	WPQ		5742200SERIES K 112 PSI Trial

Project: KINGSTON COAL	Location: TN	Project No.: 60095742
Boring No.: RECONST	Sample Type: RECONST	
Description: PLUVIATED - RECONSTITUTED ASH - DARK GRAY		
Remarks: FAILURE CRITERIA = MAX. OBLIQUITY, Ko CONSOLIDATION, B=0.95		

Wed, 22-APR-2009 15:55:23

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



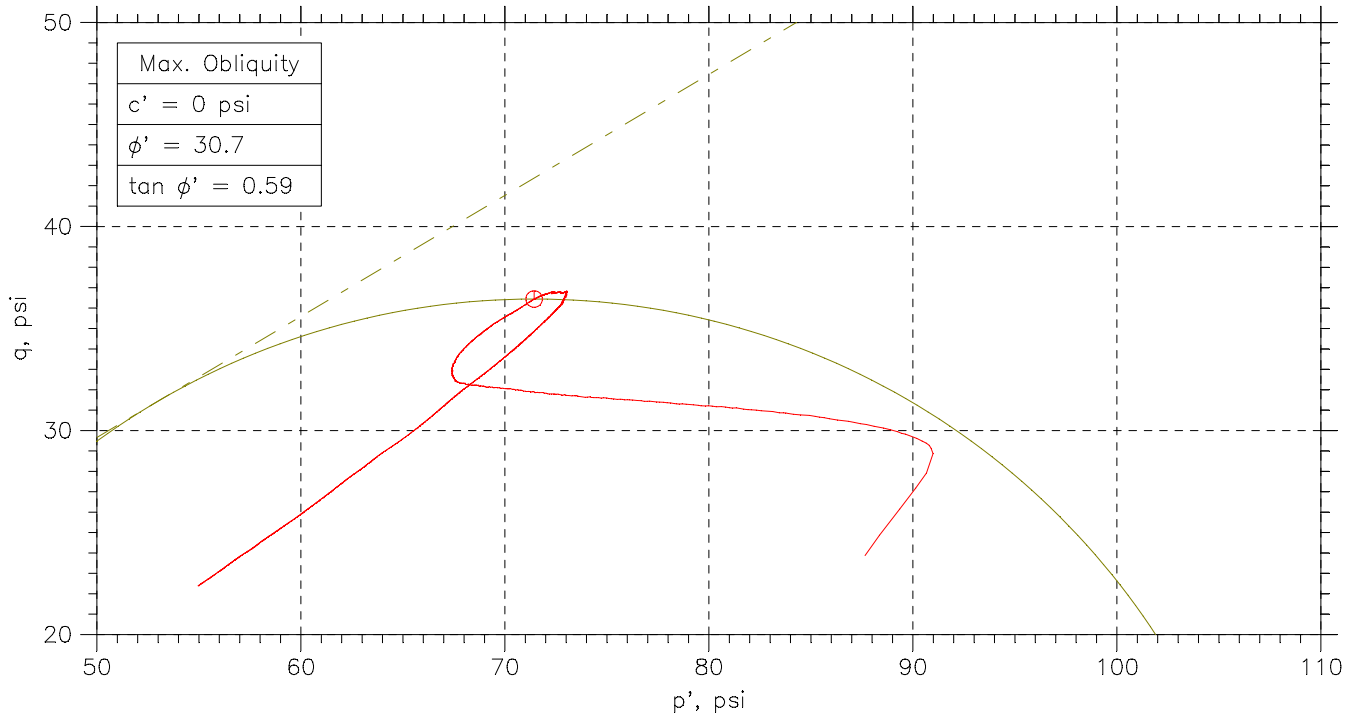


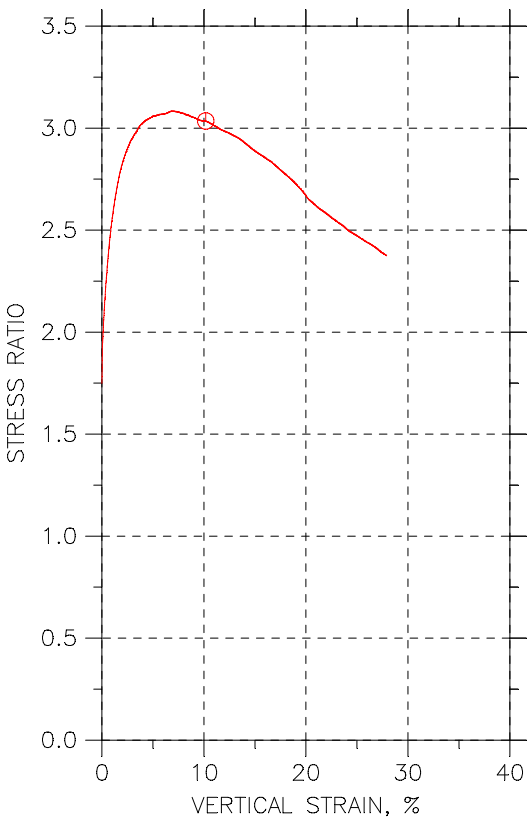
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
⊙	200s	PLe08s128	----	BCM	4/6/09	WPQ		5742200SERIES K 112 PSI Trial

Project: KINGSTON COAL	Location: TN	Project No.: 60095742
Boring No.: RECONST	Sample Type: RECONST	
Description: PLUVIATED - RECONSTITUTED ASH - DARK GRAY		
Remarks: FAILURE CRITERIA = MAX. OBLIQUITY, Ko CONSOLIDATION, B=0.95		

Wed, 22-APR-2009 15:55:24

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





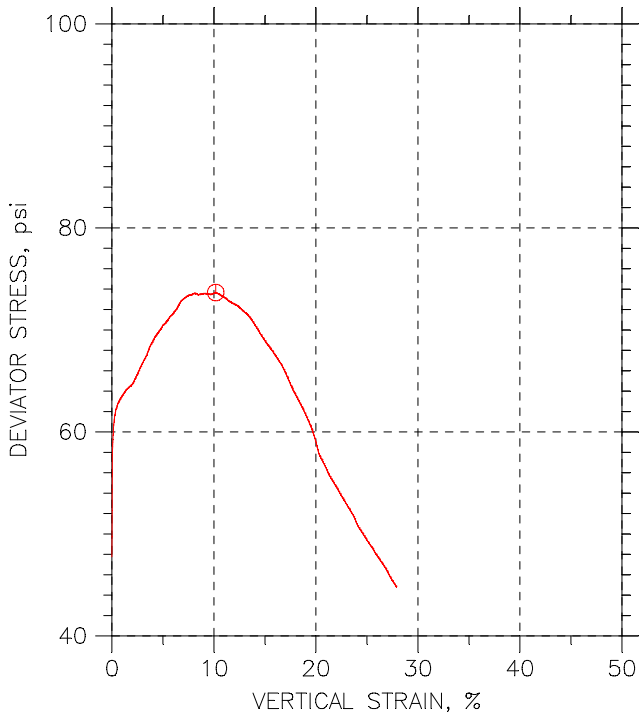
Symbol		⏲			
Sample No.		112 PSI			
Test No.		PL			
Depth		----			
Initial	Diameter, mm	70.56			
	Height, mm	165.9			
	Water Content, %	35.1			
	Dry Density, pcf	80.07			
	Saturation, %	100.0			
	Void Ratio	0.817			
Before Shear	Water Content, %	33.1			
	Dry Density, pcf	82.16			
	Saturation*, %	100.0			
	Void Ratio	0.771			
	Back Press., psi	35.85			
Ver. Eff. Cons. Stress, psi		63.78			
Shear Strength, psi		36.83			
Strain at Failure, %		10.2			
Strain Rate, %/min		0.08			
B-Value		0.92			
Estimated Specific Gravity		2.33			
Liquid Limit		NP			
Plastic Limit		NP			

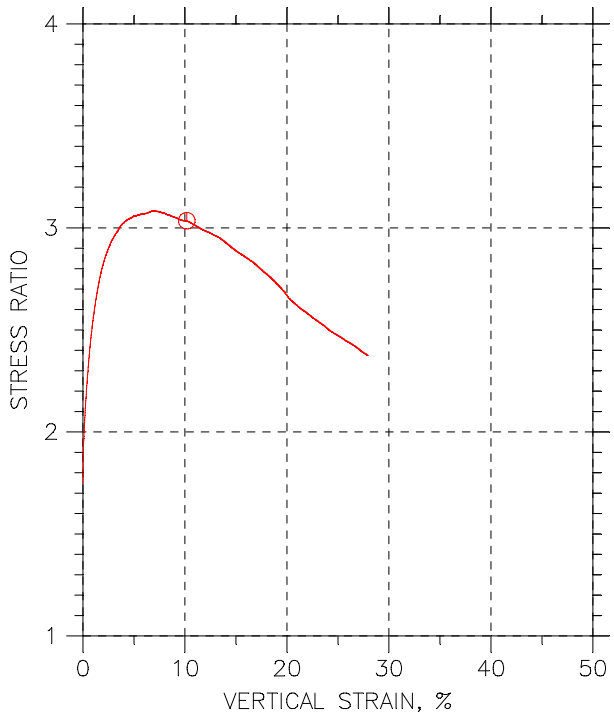
Project: KINGSTON COAL
Location: TN
Project No.: 60095742
Boring No.: RECONST
Sample Type: 3" OST
Description: PLUVIATED - RECONSTITUTED ASH -DARK GRAY
Remarks: TEST PERFORMED AS PER ASTM D 4767

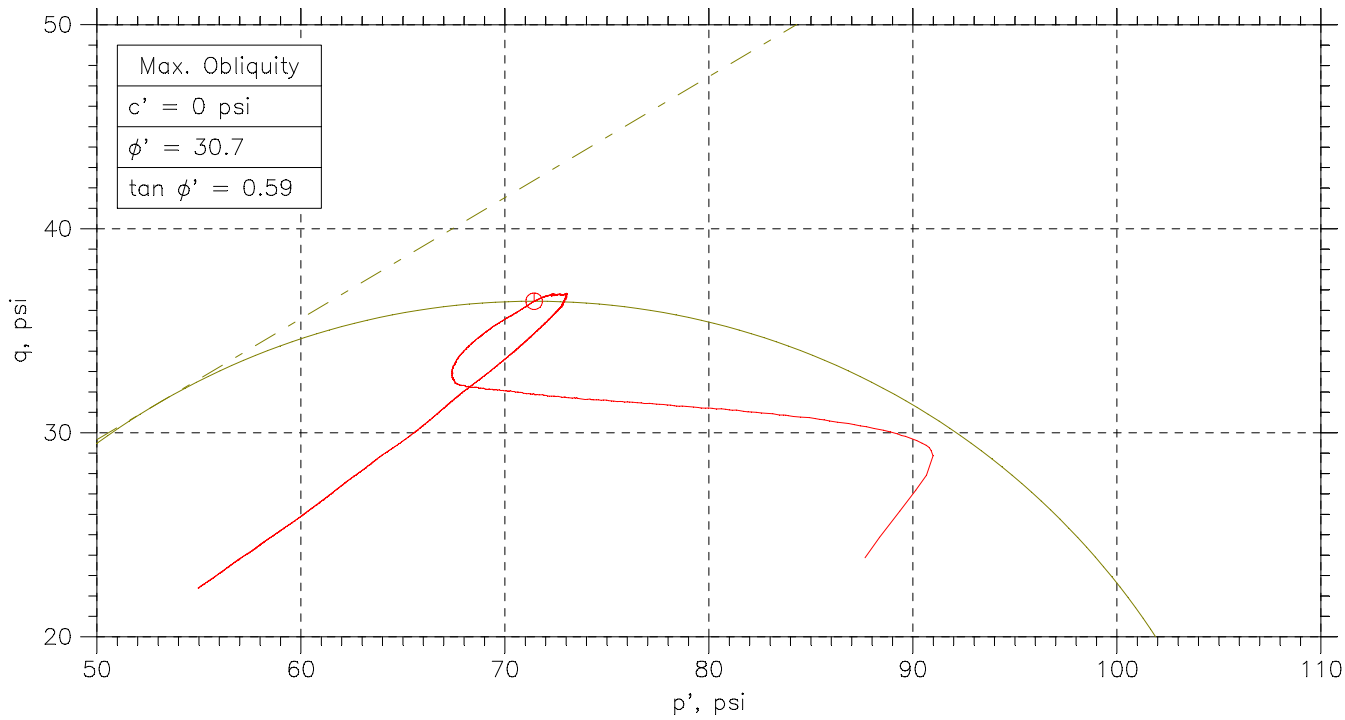
Phase calculations based on end of test.
* Saturation is set to 100% for phase calculations

Wed, 22-APR-2009 15:00:25

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





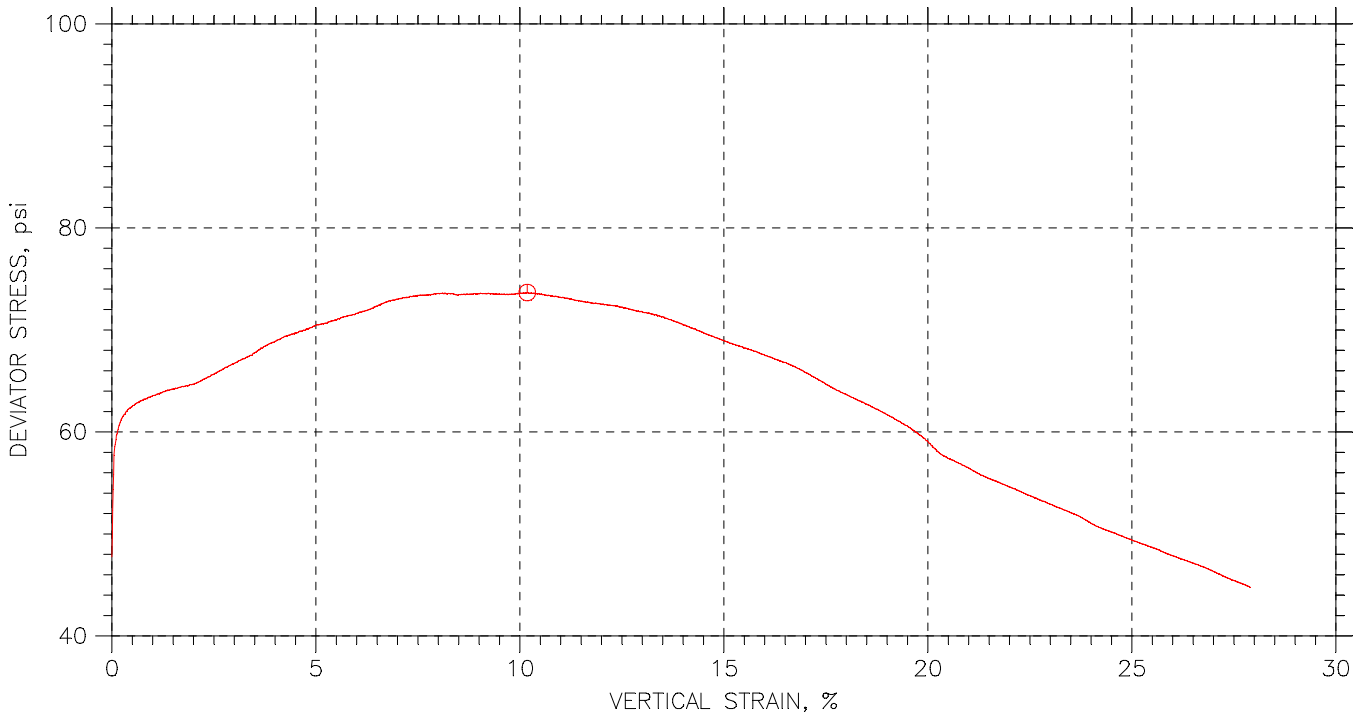


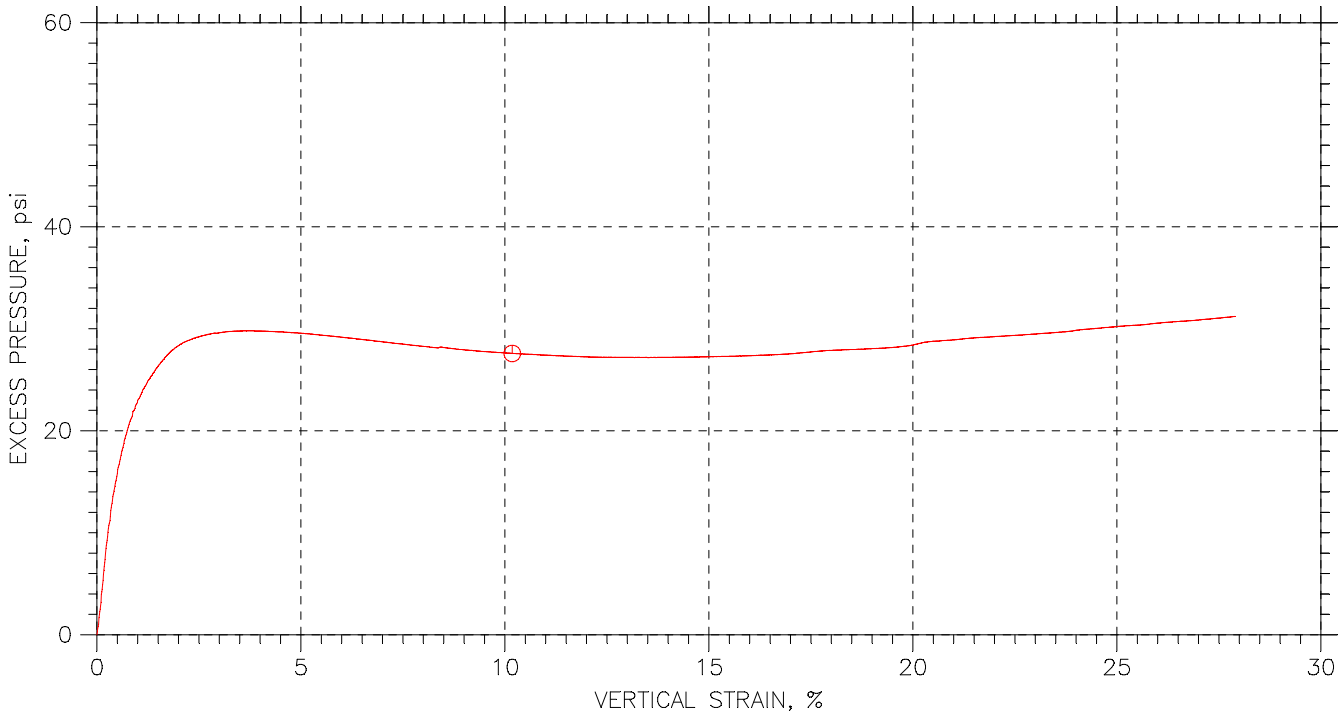
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
⏲	112 PSI	PL	----	BCM	3/27/09	WPQ		5742200SERIES K 112 PSI Trial

Project: KINGSTON COAL	Location: TN	Project No.: 60095742
Boring No.: RECONST	Sample Type: 3" OST	
Description: PLUVIATED - RECONSTITUTED ASH -DARK GRAY		
Remarks: TEST PERFORMED AS PER ASTM D 4767		

Wed, 22-APR-2009 15:00:26

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



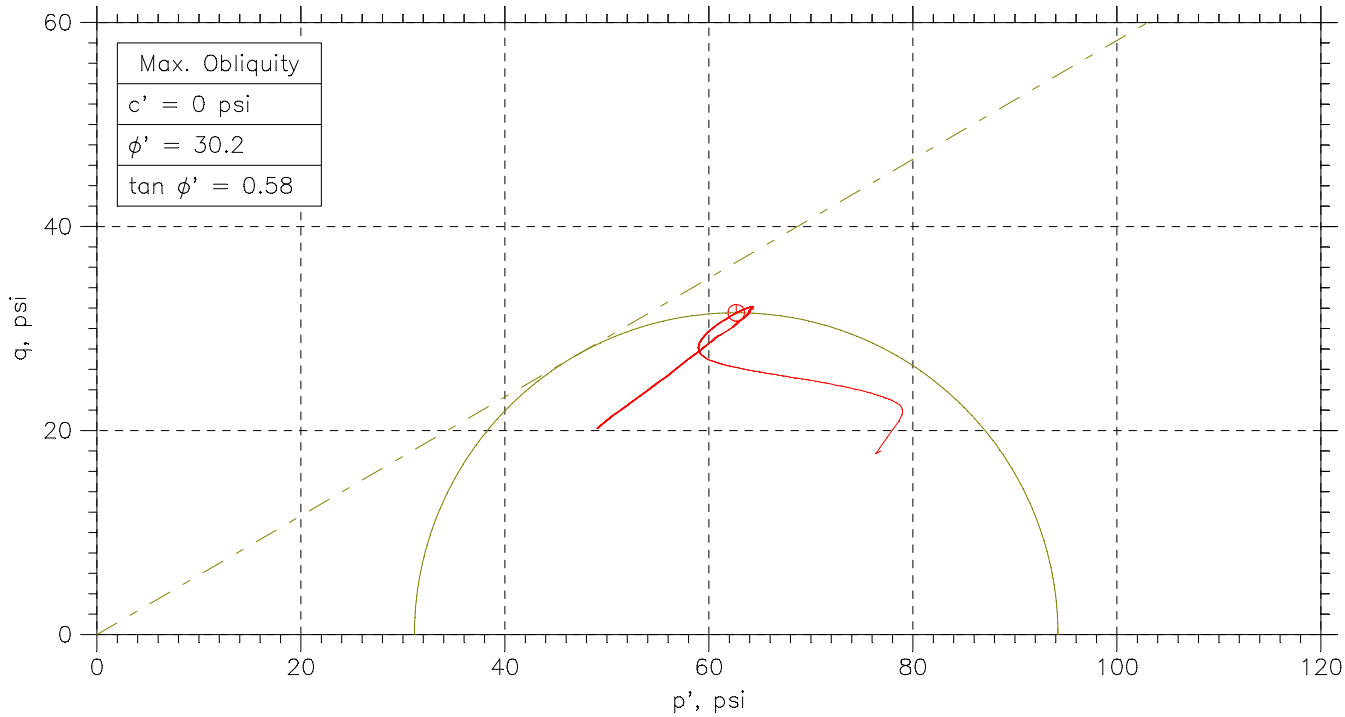


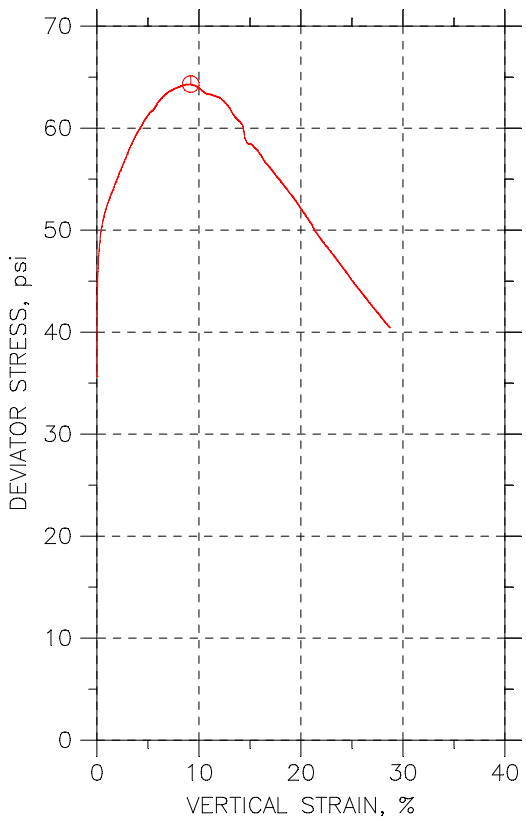
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
⊙	112 PSI	PL	----	BCM	3/27/09	WPQ		5742200SERIES K 112 PSI Trial

Project: KINGSTON COAL	Location: TN	Project No.: 60095742
Boring No.: RECONST	Sample Type: 3" OST	
Description: PLUVIATED - RECONSTITUTED ASH -DARK GRAY		
Remarks: TEST PERFORMED AS PER ASTM D 4767		

Wed, 22-APR-2009 15:00:27

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





Symbol		⏲			
Sample No.		96 PSI			
Test No.		5742200B			
Depth		----			
Initial	Diameter, mm	71.04			
	Height, mm	156.8			
	Water Content, %	32.5			
	Dry Density, pcf	83.37			
	Saturation, %	101.6			
	Void Ratio	0.745			
Before Shear	Water Content, %	32.1			
	Dry Density, pcf	83.21			
	Saturation*, %	100.0			
	Void Ratio	0.748			
	Back Press., psi	50.98			
Ver. Eff. Cons. Stress, psi		58.87			
Shear Strength, psi		32.16			
Strain at Failure, %		9.2			
Strain Rate, %/min		0.08			
B-Value		0.93			
Estimated Specific Gravity		2.33			
Liquid Limit		NP			
Plastic Limit		NP			

Project: KINGSTON COAL

Location: TN

Project No.: 60095742

Boring No.: 09-200B SET

Sample Type: 3" OST

Description: PLUVIATED - RECONSTITUTED ASH - DARK GRAY

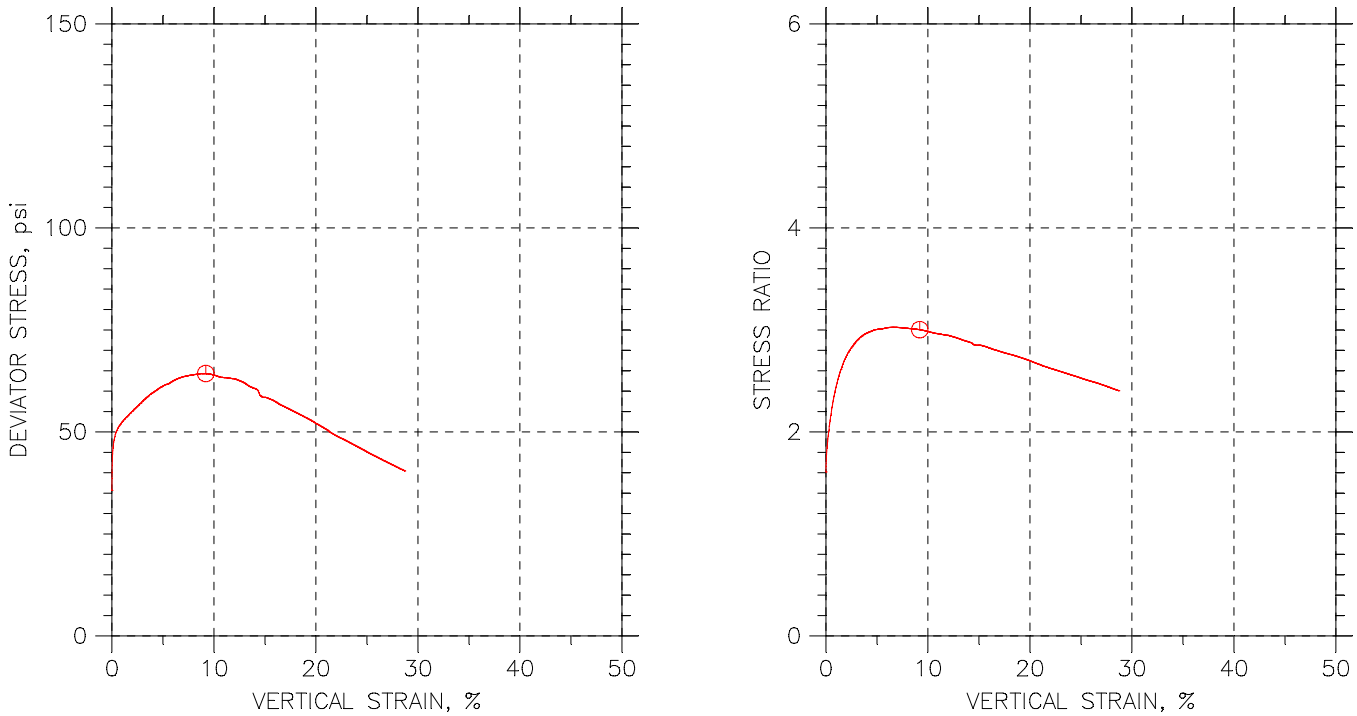
Remarks: FAILURE CRITERIA = MAX. OBLIQUITY, Ko CONSOLIDATION, B=0.95

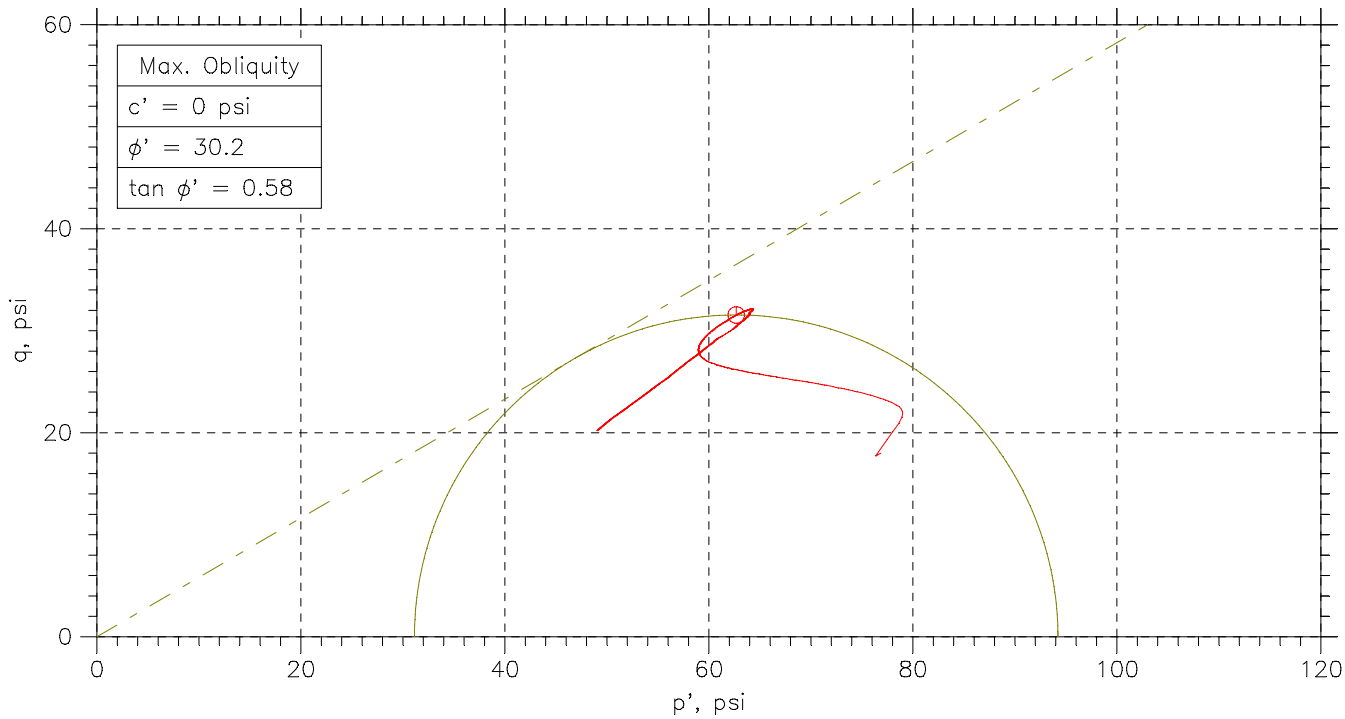
Phase calculations based on end of test.

* Saturation is set to 100% for phase calculations

Wed, 22-APR-2009 16:28:04

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



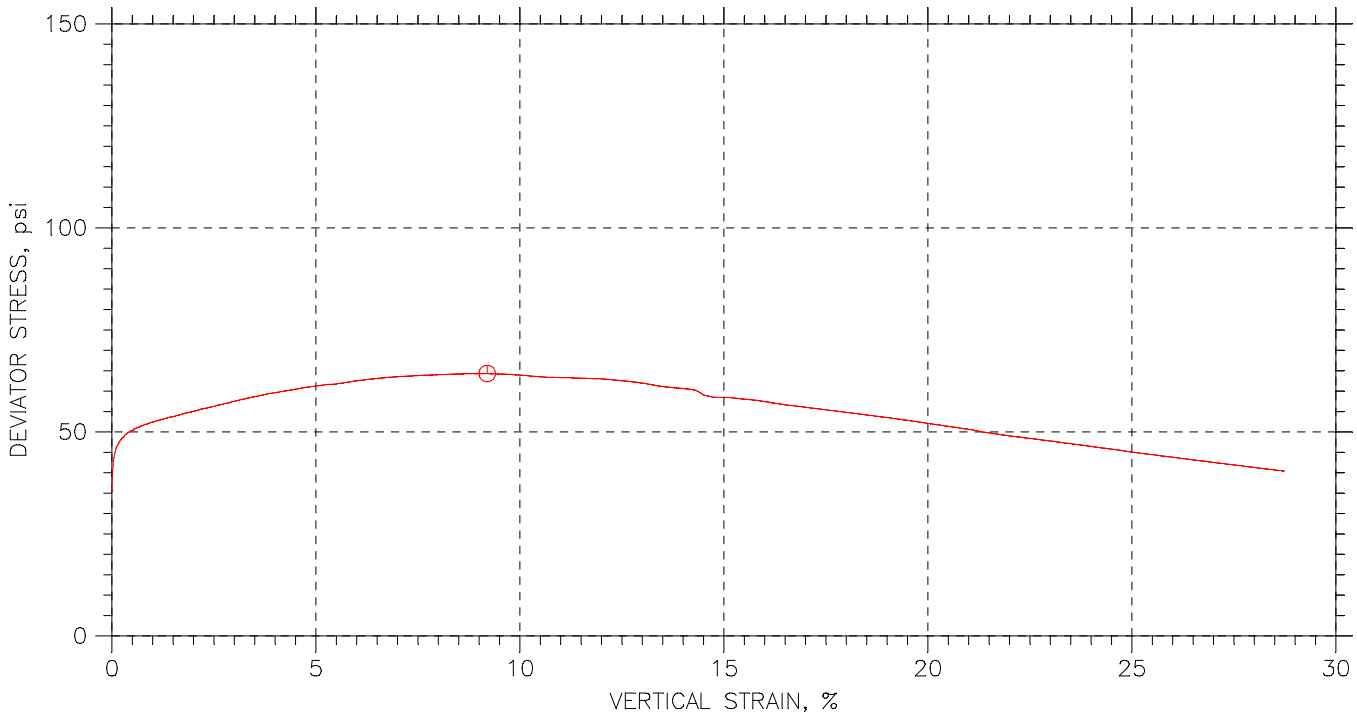


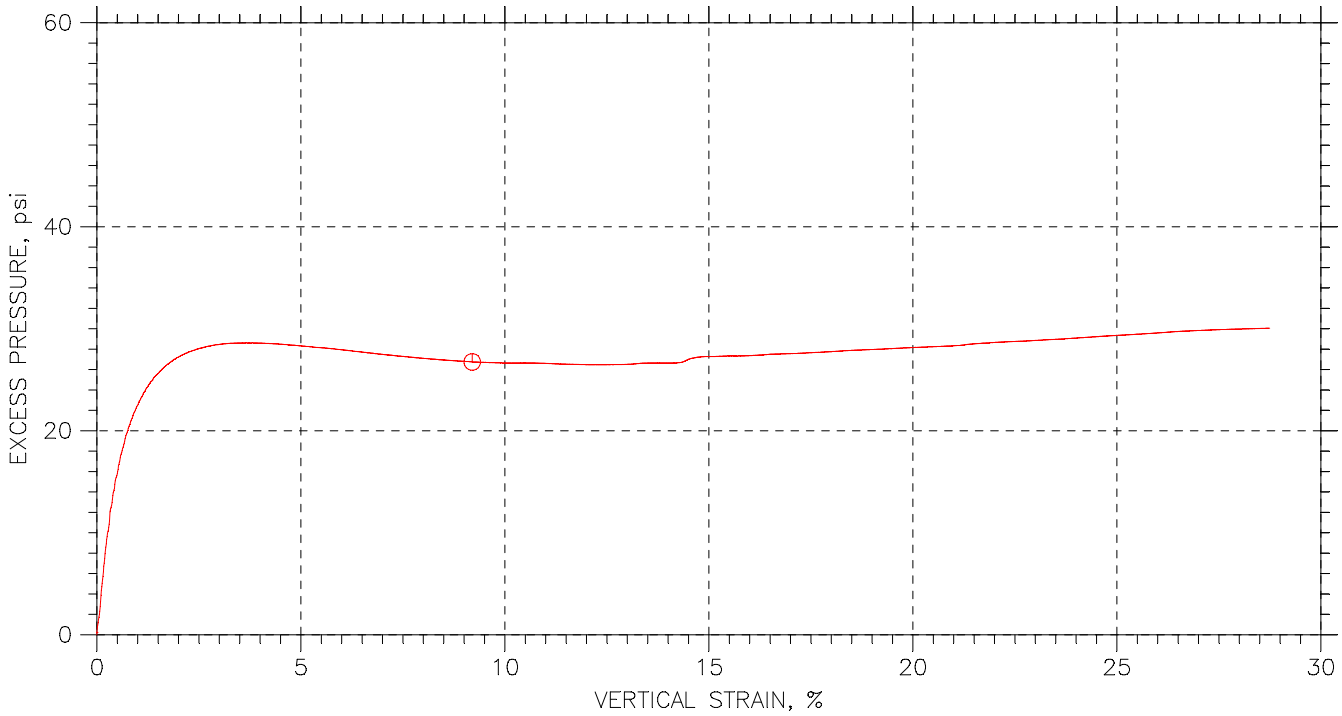
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
⊙	96 PSI	5742200B	----	BCM	3/24/09	WPQ		5742200SERIES K 96 PSI TRIAL

Project: KINGSTON COAL	Location: TN	Project No.: 60095742
Boring No.: 09-200B SET	Sample Type: 3" OST	
Description: PLUVIATED - RECONSTITUTED ASH - DARK GRAY		
Remarks: FAILURE CRITERIA = MAX. OBLIQUITY, Ko CONSOLIDATION, B=0.95		

Wed, 22-APR-2009 16:28:05

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





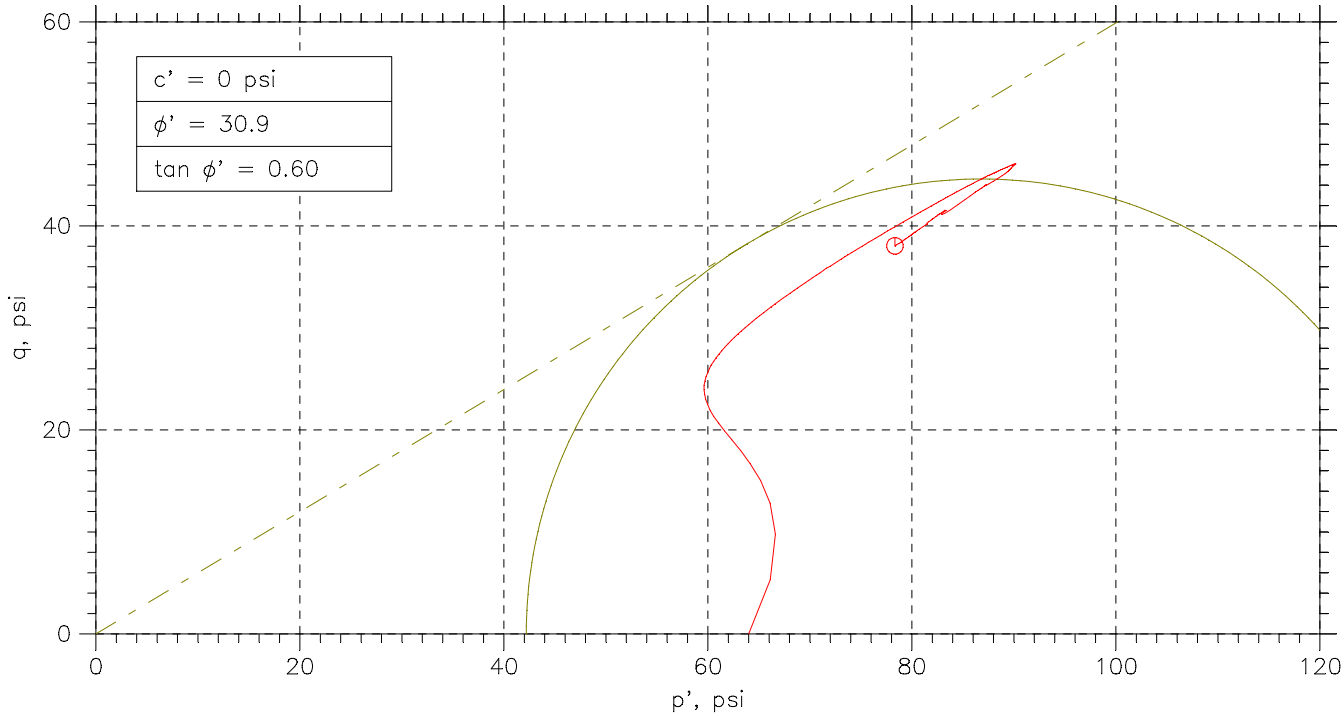
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
⊙	96 PSI	5742200B	----	BCM	3/24/09	WPQ		5742200SERIES K 96 PSI TRIAL

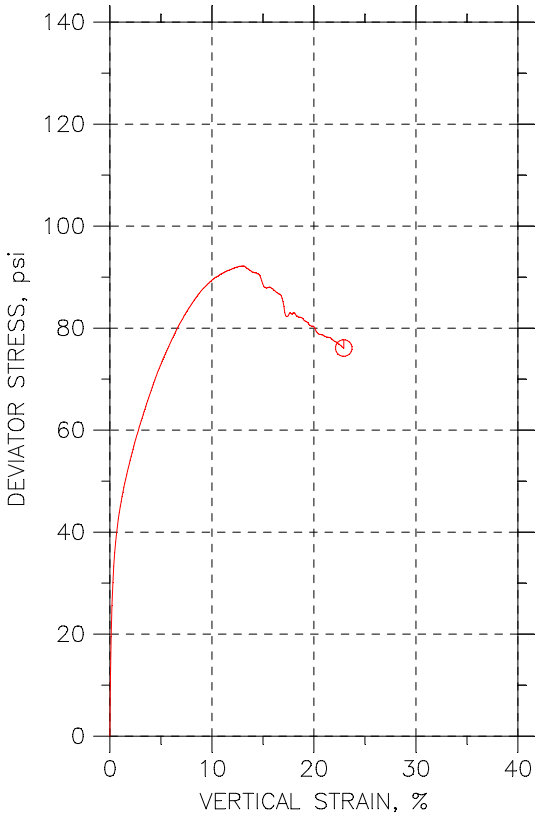
Project: KINGSTON COAL	Location: TN	Project No.: 60095742
Boring No.: 09-200B SET	Sample Type: 3" OST	
Description: PLUVIATED - RECONSTITUTED ASH - DARK GRAY □○		
Remarks: FAILURE CRITERIA = MAX. OBLIQUITY, Ko CONSOLIDATION, B=0.95□○		

Wed, 22-APR-2009 16:28:06

WORK IN PROGRESS

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⏲			
Test No.		PLe08s64			
Initial	Diameter, mm	71.04			
	Height, mm	158.69			
	Water Content, %	25.57			
	Dry Density, pcf	89.47			
	Saturation, %	95.21			
	Void Ratio	0.62568			
Before Shear	Water Content, %	26.57			
	Dry Density, pcf	89.84			
	Saturation, %	100.00			
	Void Ratio	0.61901			
	Back Press., psi	69.999			
Minor Prin. Stress, psi		64.001			
Max. Dev. Stress, psi		92.187			
Time to Failure, min		211.68			
Strain Rate, %/min		0.0069			
B-Value		---			
Estimated Specific Gravity		2.33			
Liquid Limit		NP			
Plastic Limit		NP			
Plasticity Index		NP			

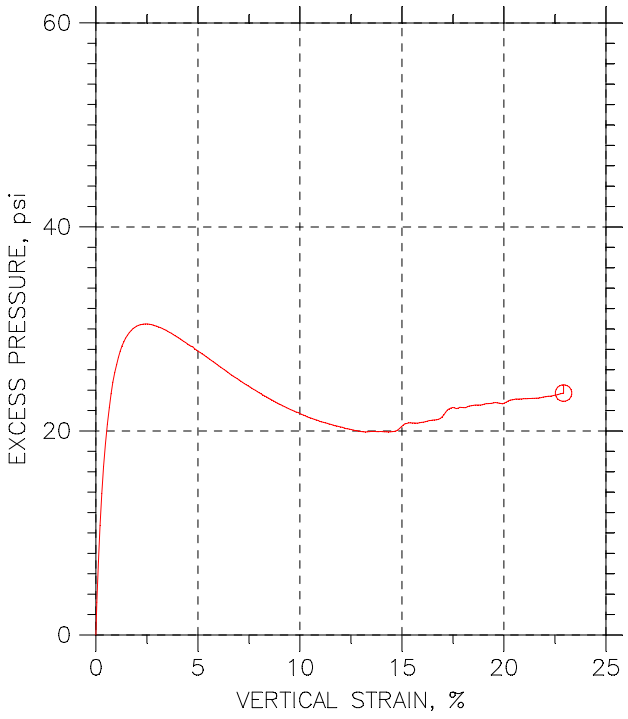
Project: KINGSTON COAL TVA	Failure Sketch			
Location: TN				
Project No.: 60095742				
Boring No.: RECONST				
Sample Type: RECONST				

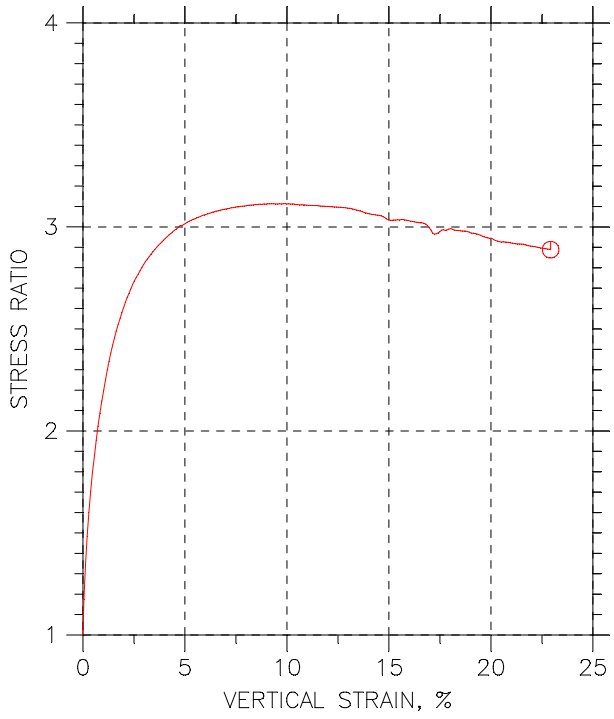
Description: PLUVIATION - RECONSTITUTED ASH - DARK GRAY

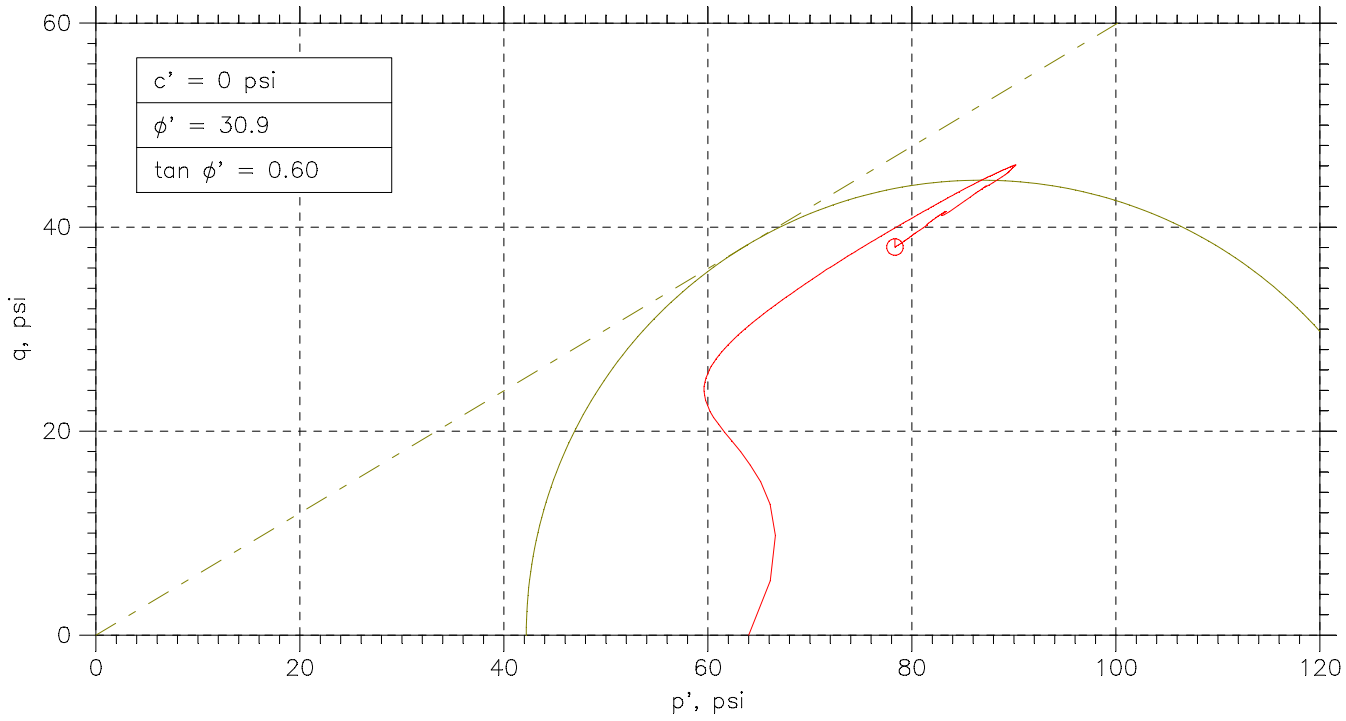
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (b=0.99)

Wed, 22-APR-2009 09:13:55

TRIAXIAL COMPRESSION TEST REPORT



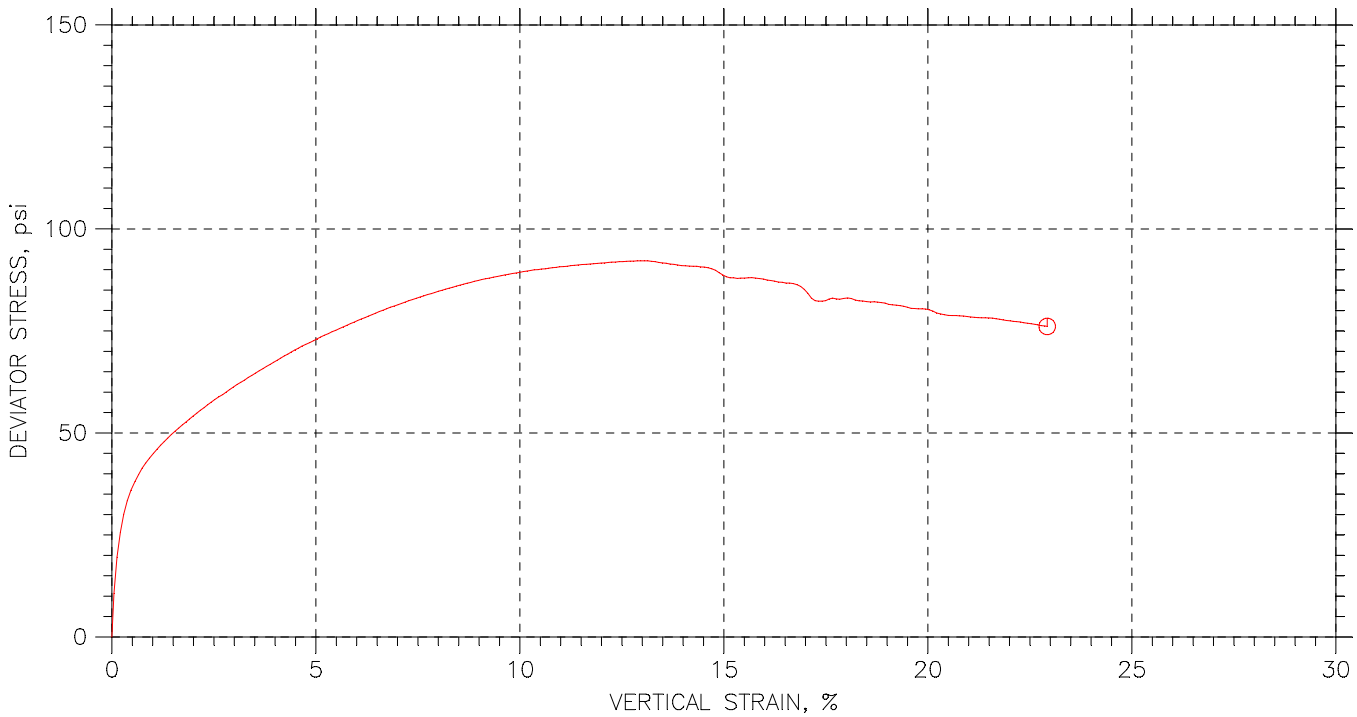


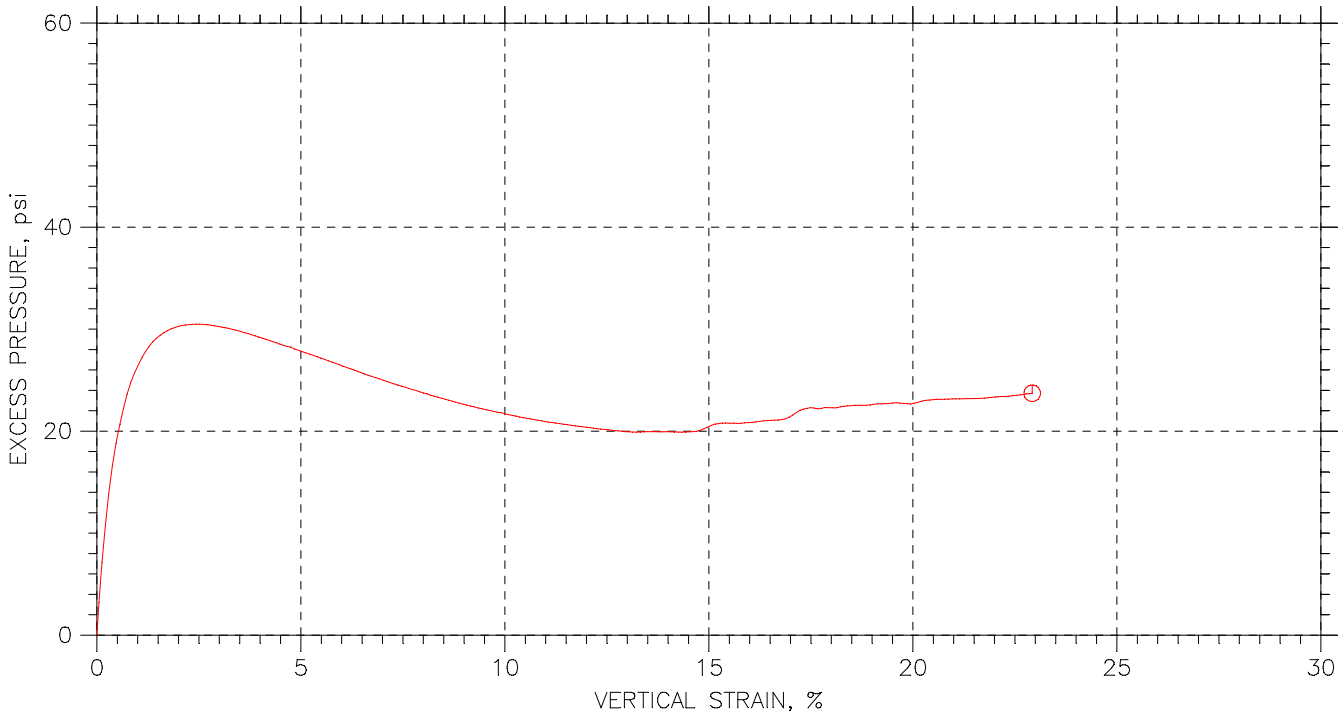


Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: RECONST	Tested By: BCM	Checked By: WPQ
Sample No.: 200s	Test Date: 3/20/09	Depth: ----
Test No.: PLe08s64	Sample Type: RECONST	Elevation: ----
Description: PLUVIATION - RECONSTITUTED ASH - DARK GRAY		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (b=0.99)		

Wed, 22-APR-2009 09:13:55

TRIAXIAL COMPRESSION TEST REPORT

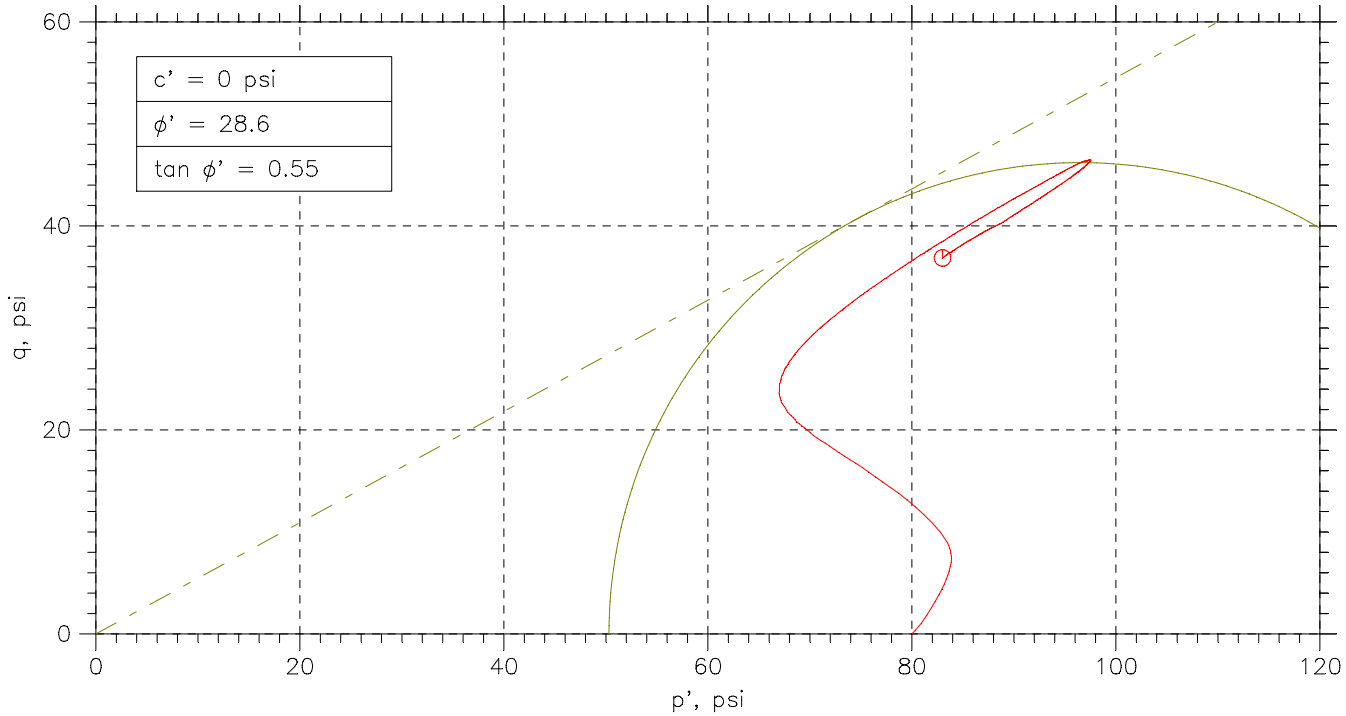


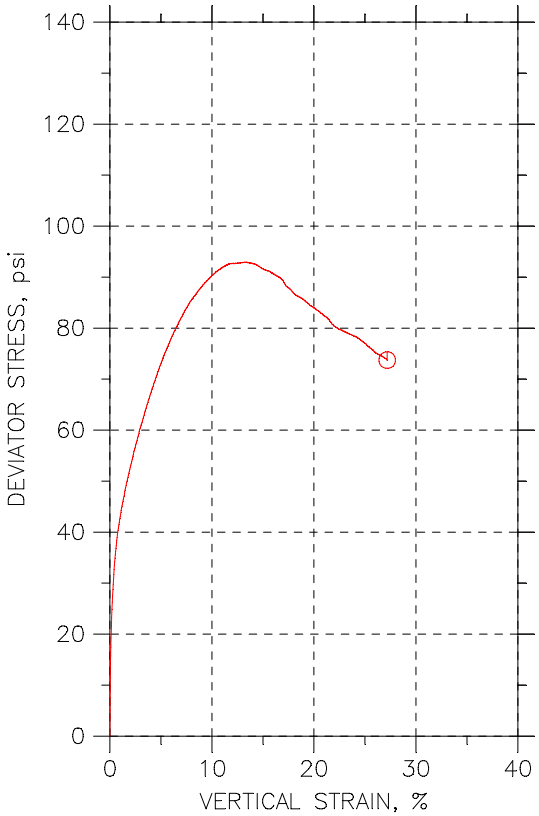


Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: RECONST	Tested By: BCM	Checked By: WPQ
Sample No.: 200s	Test Date: 3/20/09	Depth: ----
Test No.: PLe08s64	Sample Type: RECONST	Elevation: ----
Description: PLUVIATION - RECONSTITUTED ASH - DARK GRAY		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (b=0.99)		

Wed, 22-APR-2009 09:13:56

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⏲			
Test No.		PLe08s80			
Initial	Diameter, mm	70.56			
	Height, mm	159.8			
	Water Content, %	31.08			
	Dry Density, pcf	81.38			
	Saturation, %	91.98			
	Void Ratio	0.78729			
Before Shear	Water Content, %	32.32			
	Dry Density, pcf	82.97			
	Saturation, %	100.00			
	Void Ratio	0.75312			
	Back Press., psi	52			
Minor Prin. Stress, psi		80			
Max. Dev. Stress, psi		92.952			
Time to Failure, min		217			
Strain Rate, %/min		0.0625			
B-Value		---			
Estimated Specific Gravity		2.33			
Liquid Limit		NP			
Plastic Limit		NP			
Plasticity Index		NP			

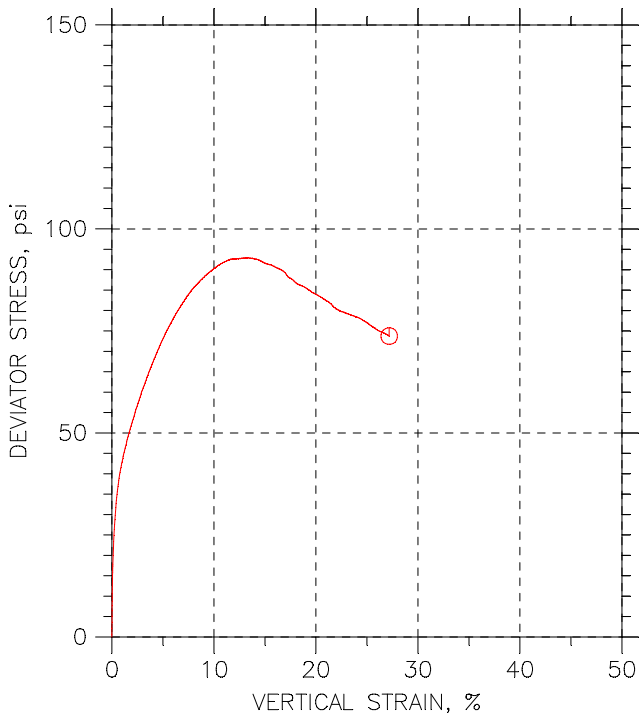
Project: KINGSTON COAL TVA	Failure Sketch			
Location: TN				
Project No.: 60095742				
Boring No.: RECONST				
Sample Type: RECONST				

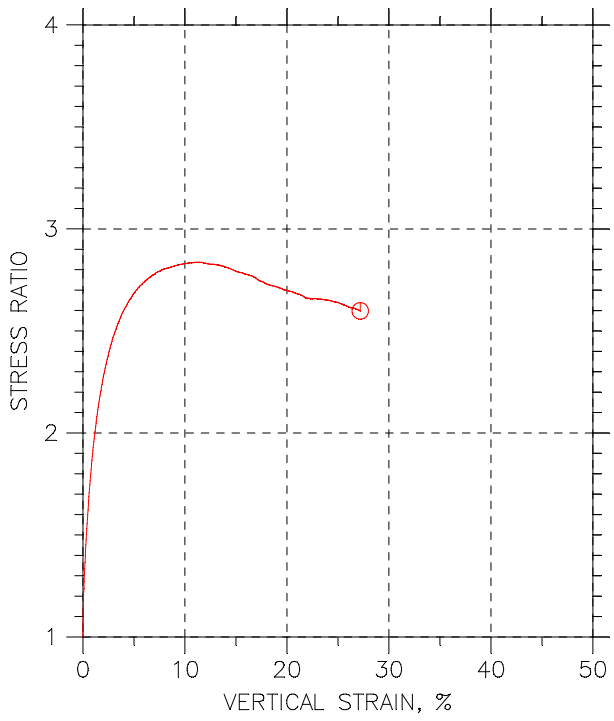
Description: PLUVIATED - RECONSTITUTED ASH - DARK GRAY

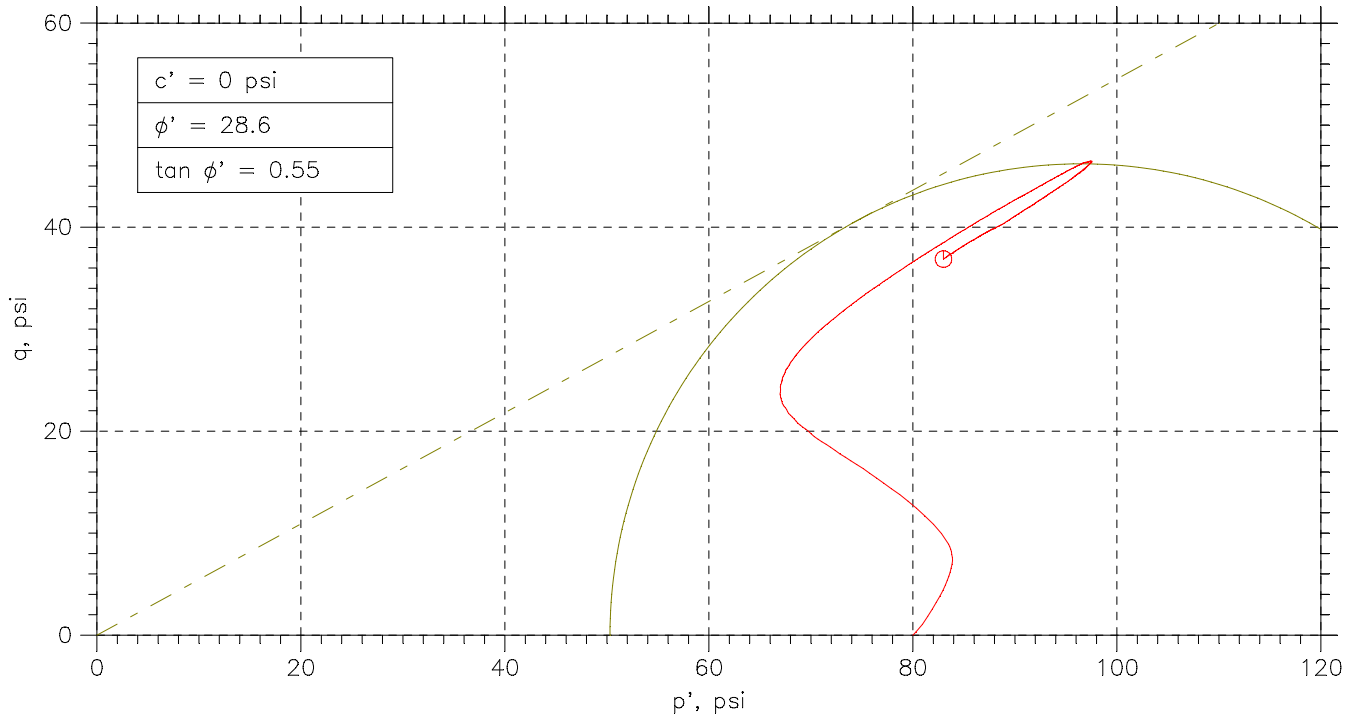
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (0.99)

Fri, 01-MAY-2009 10:11:07

TRIAXIAL COMPRESSION TEST REPORT



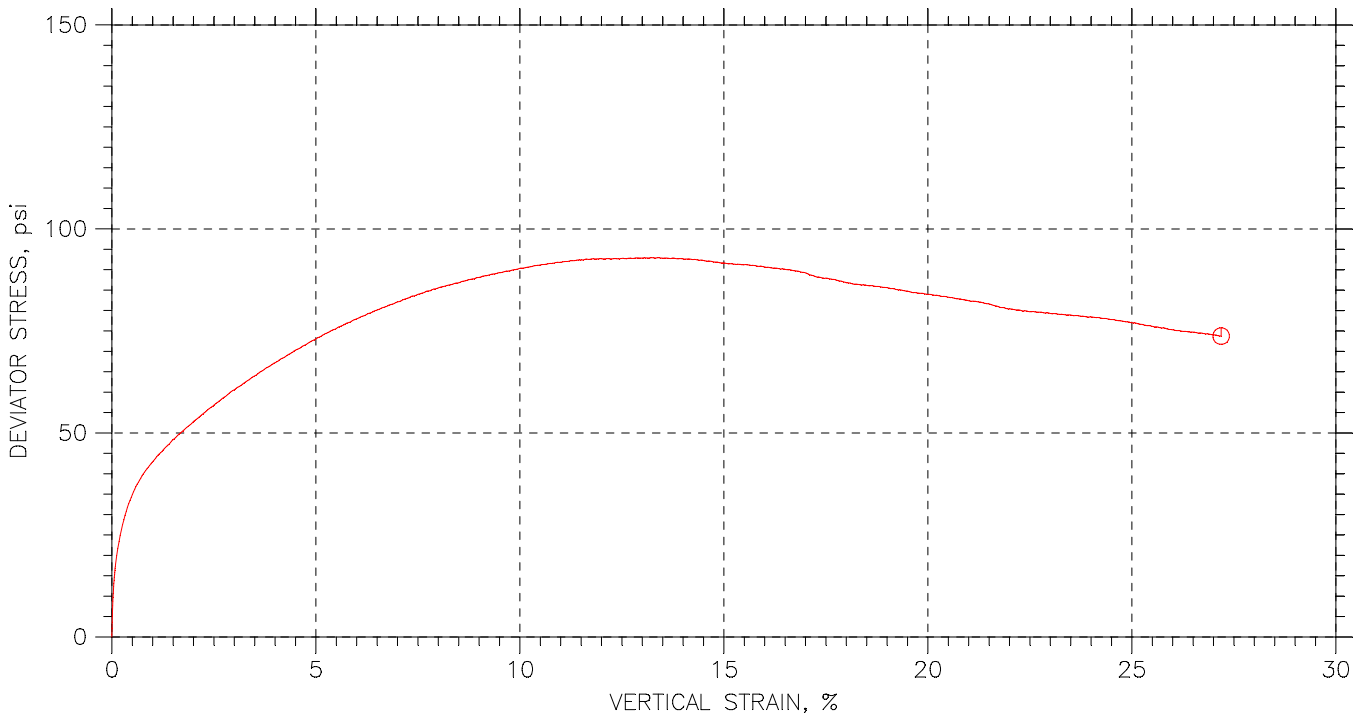


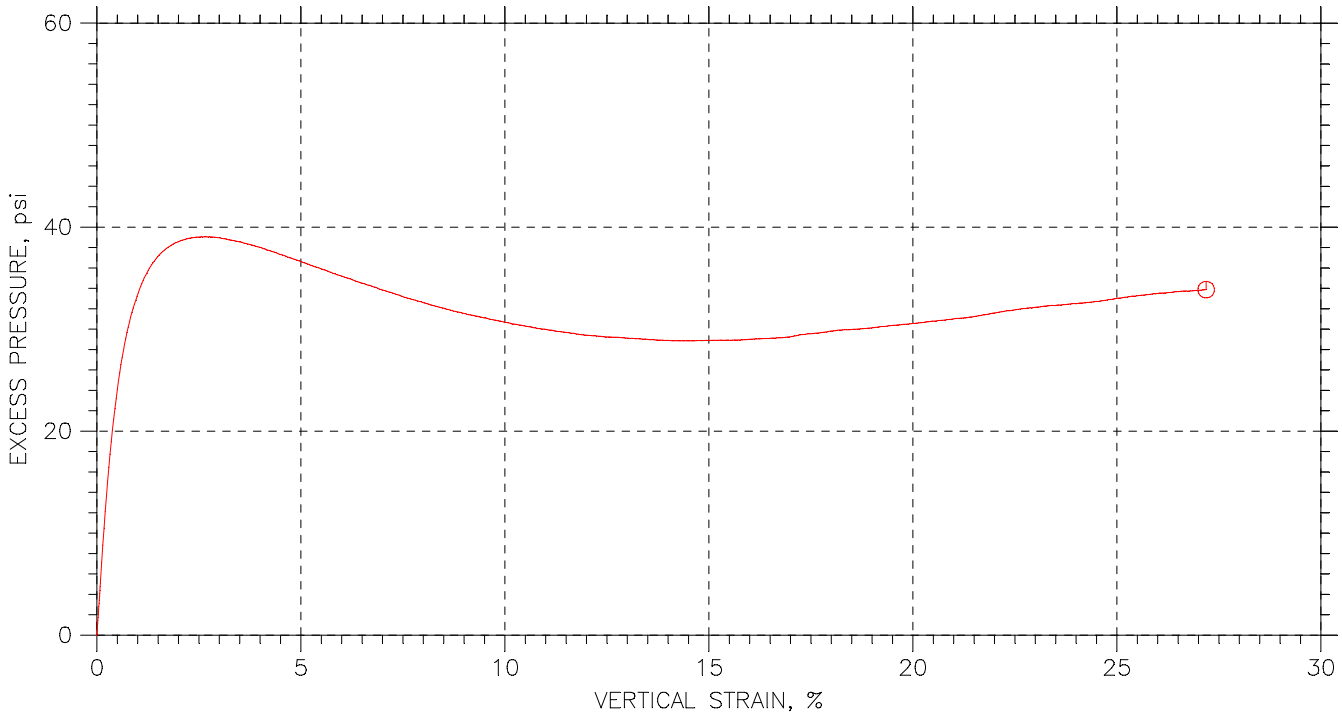


Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: RECONST	Tested By: BCM	Checked By: WPQ
Sample No.: 200s	Test Date: 3/25/09	Depth: ----
Test No.: PLe08s80	Sample Type: RECONST	Elevation: ----
Description: PLUVIATED - RECONSTITUTED ASH - DARK GRAY		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (0.99)		

Fri, 01-MAY-2009 10:11:08

TRIAXIAL COMPRESSION TEST REPORT

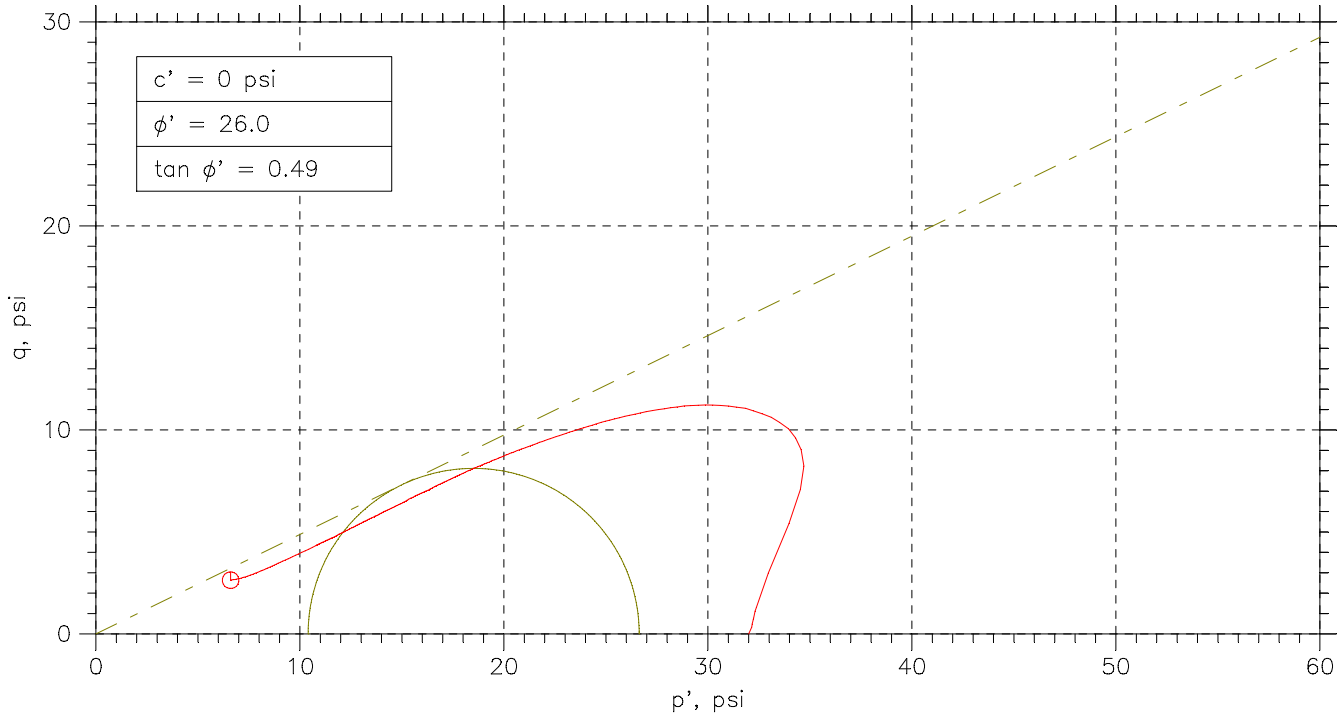


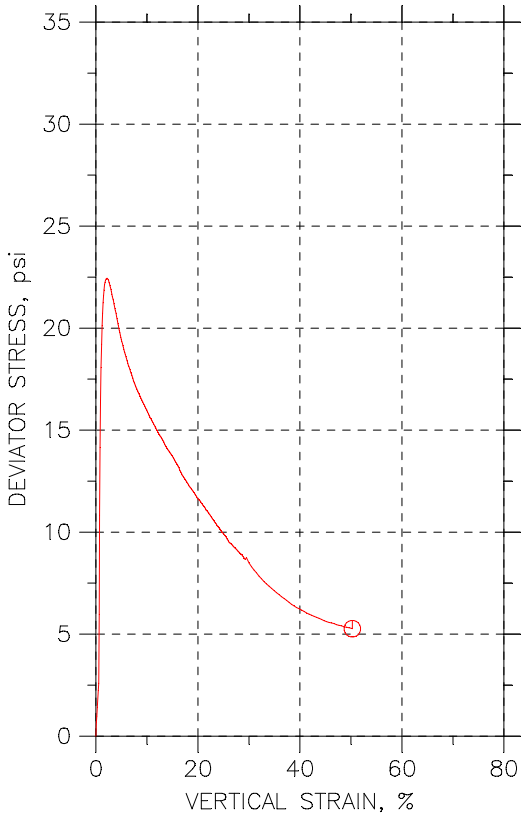


Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: RECONST	Tested By: BCM	Checked By: WPQ
Sample No.: 200s	Test Date: 3/25/09	Depth: ----
Test No.: PLe08s80	Sample Type: RECONST	Elevation: ----
Description: PLUVIATED - RECONSTITUTED ASH - DARK GRAY		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (0.99)		

Fri, 01-MAY-2009 10:11:09

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⏲			
Test No.		PLe14s32			
Initial	Diameter, mm	71.4			
	Height, mm	155			
	Water Content, %	35.18			
	Dry Density, pcf	79.94			
	Saturation, %	100.00			
	Void Ratio	0.81958			
Before Shear	Water Content, %	34.69			
	Dry Density, pcf	80.44			
	Saturation, %	100.00			
	Void Ratio	0.80833			
	Back Press., psi	70			
Minor Prin. Stress, psi		32			
Max. Dev. Stress, psi		22.446			
Time to Failure, min		33.121			
Strain Rate, %/min		0.065			
B-Value		---			
Estimated Specific Gravity		2.33			
Liquid Limit		NP			
Plastic Limit		NP			
Plasticity Index		NP			
Failure Sketch					

Project: KINGSTON COAL TVA

Location: TN

Project No.: 60095742

Boring No.: RECONST

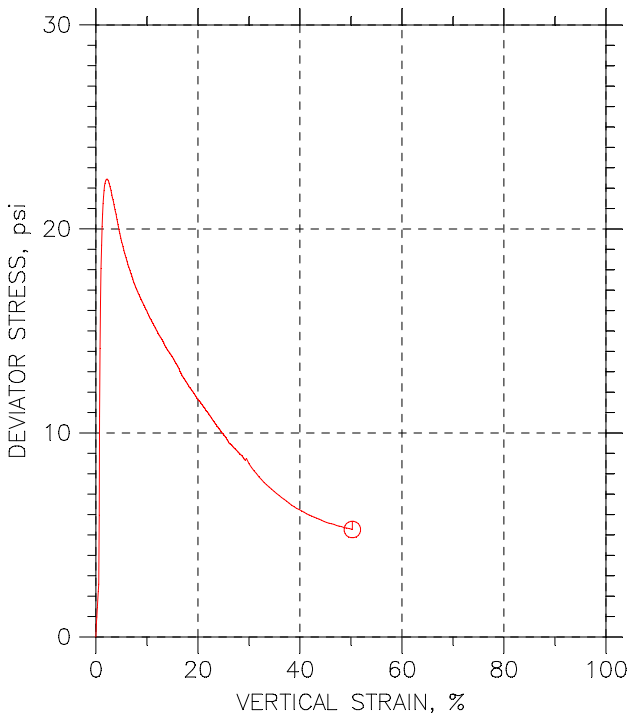
Sample Type: RECONST

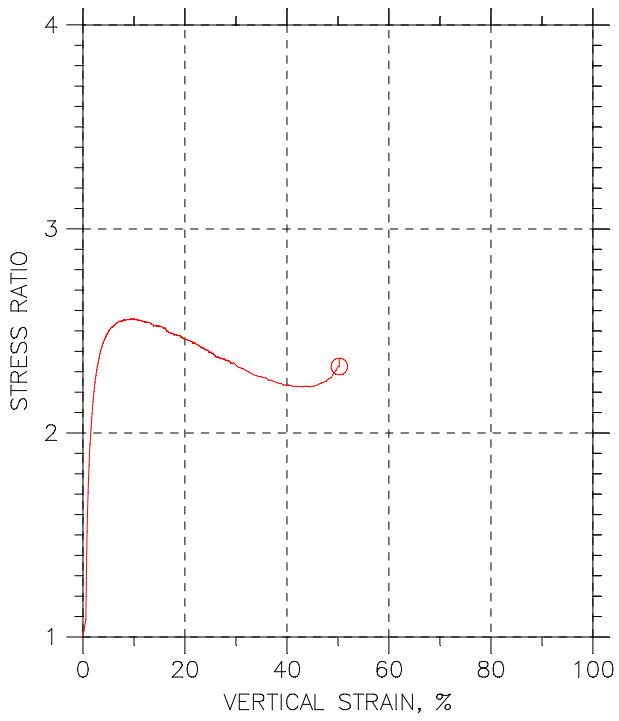
Description: PLUVIATION - RECONSTITUTED ASH - DARK GRAY

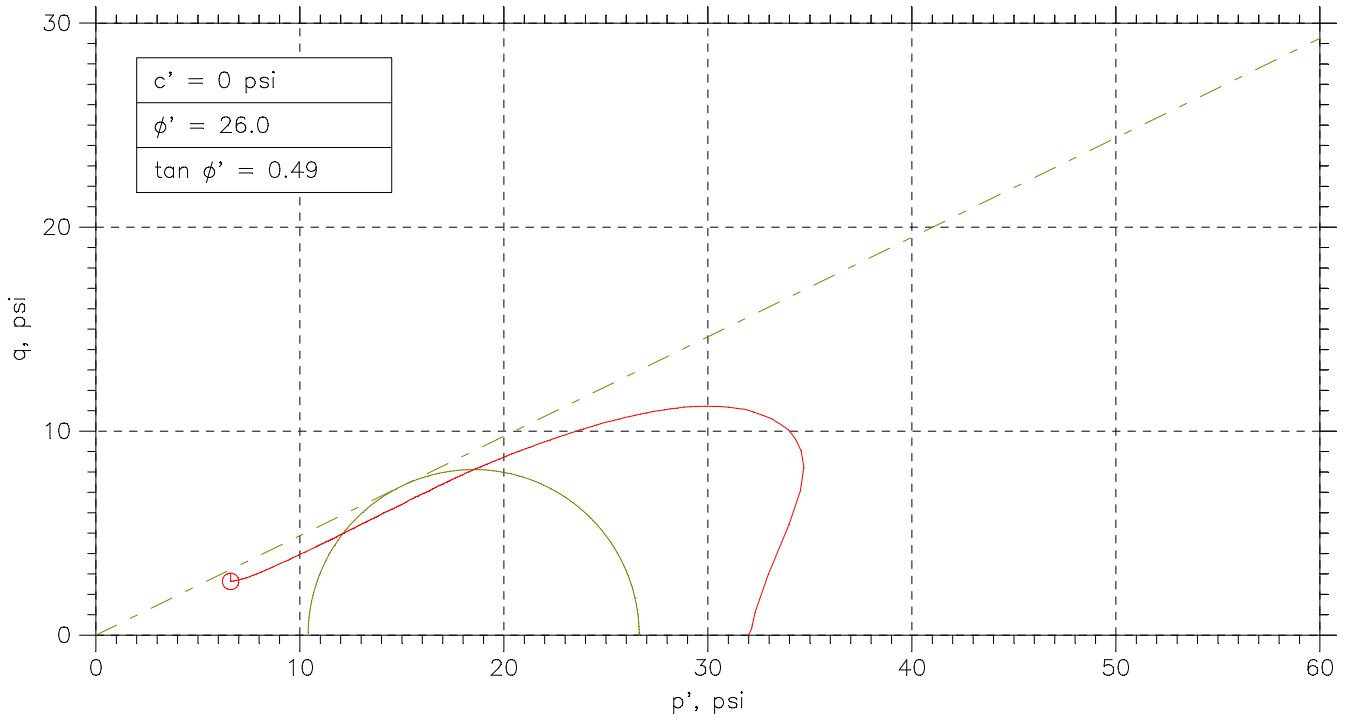
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (B=0.95)

Wed, 22-APR-2009 19:00:57

TRIAXIAL COMPRESSION TEST REPORT



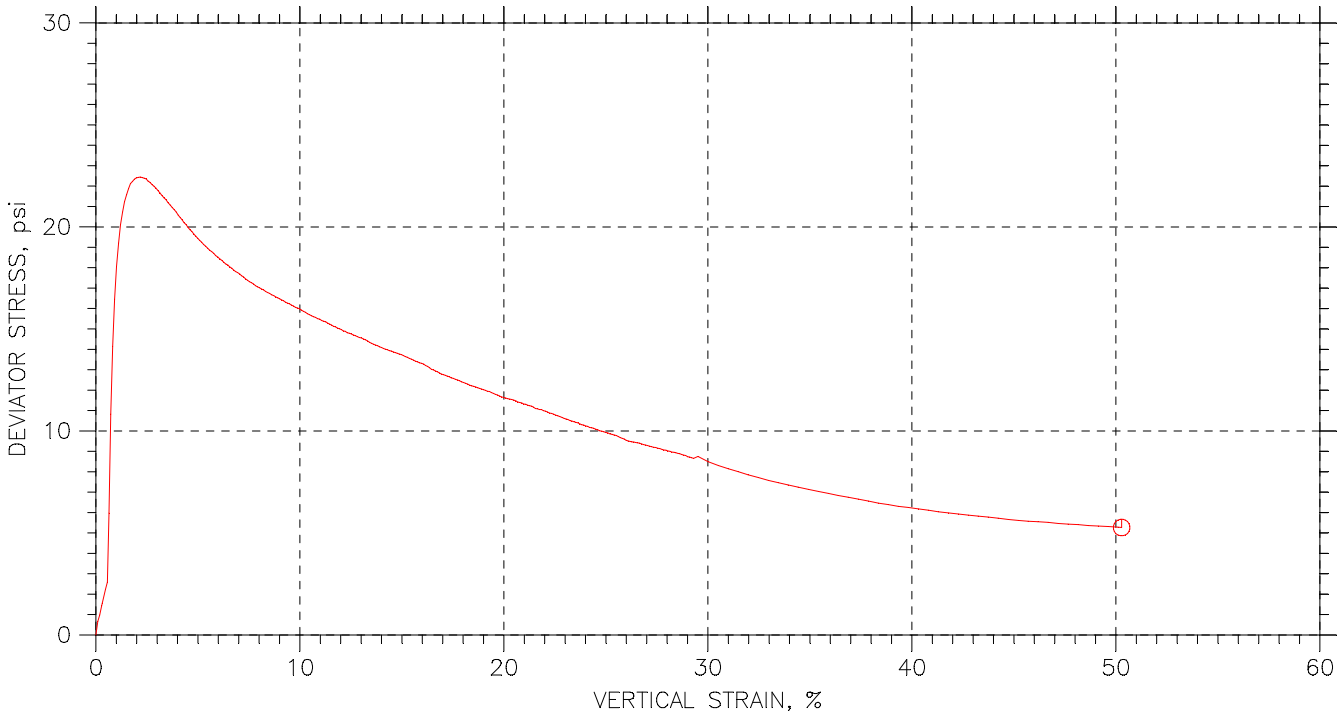


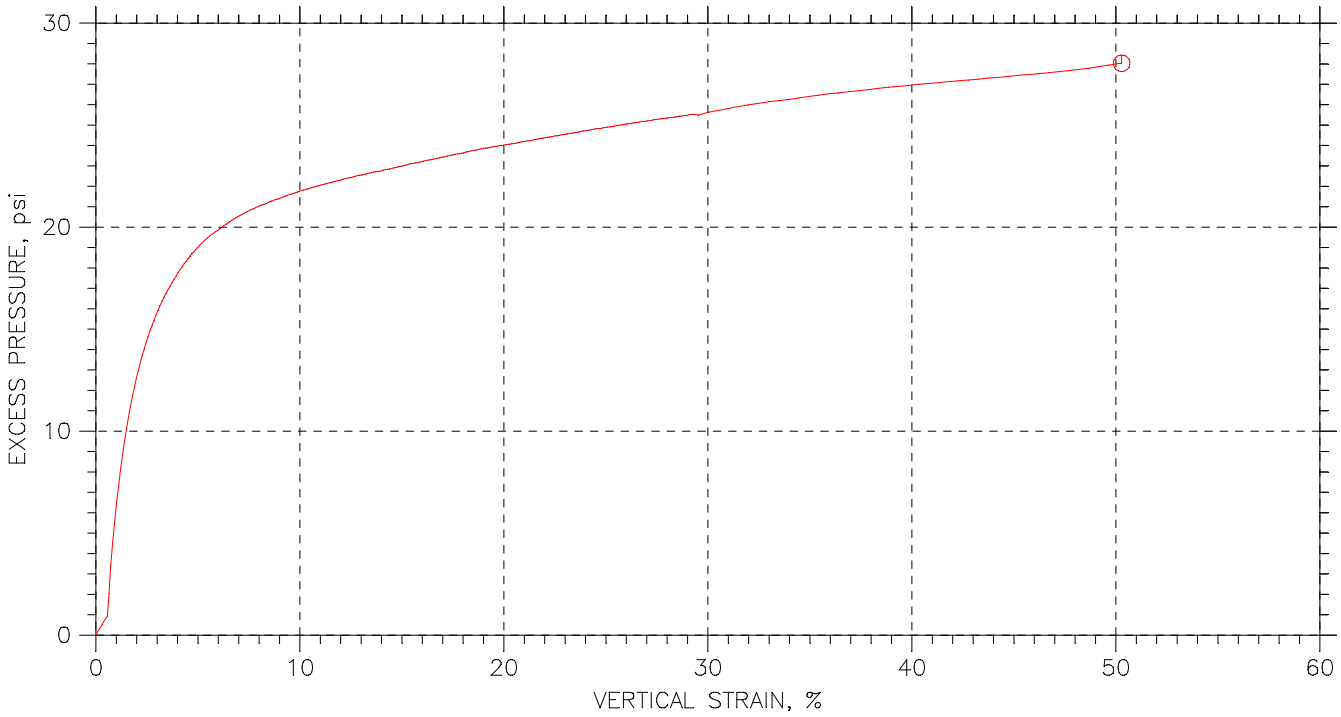


Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: RECONST	Tested By: BCM	Checked By: WPQ
Sample No.: 200s	Test Date: 3/26/09	Depth: -----
Test No.: PLe14s32	Sample Type: RECONST	Elevation: -----
Description: PLUVIATION - RECONSTITUTED ASH - DARK GRAY		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (B=0.95)		

Wed, 22-APR-2009 19:00:58

TRIAXIAL COMPRESSION TEST REPORT

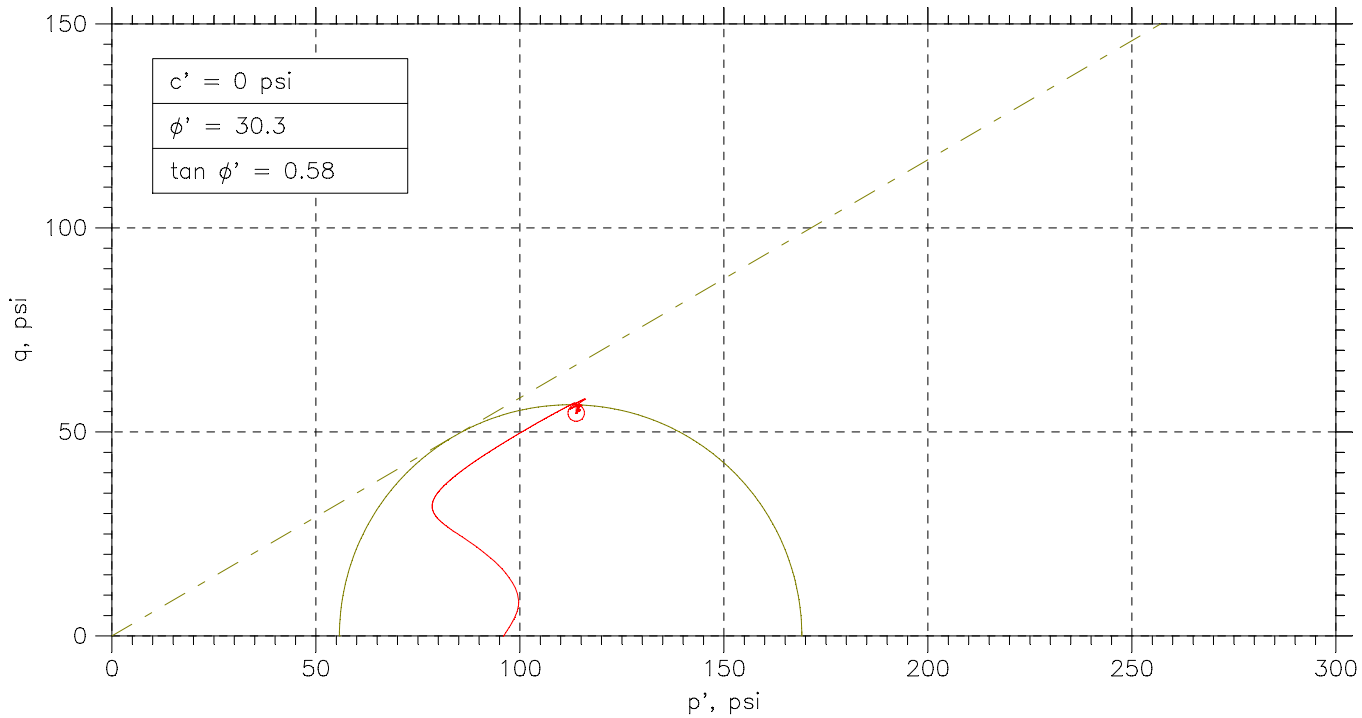


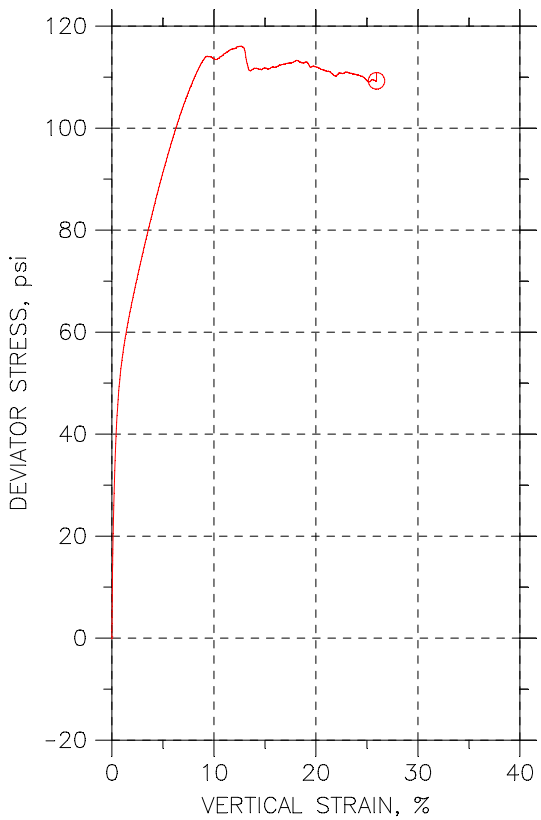


Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: RECONST	Tested By: BCM	Checked By: WPQ
Sample No.: 200s	Test Date: 3/26/09	Depth: -----
Test No.: PLe14s32	Sample Type: RECONST	Elevation: -----
Description: PLUVIATION - RECONSTITUTED ASH - DARK GRAY		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (B=0.95)		

Wed, 22-APR-2009 19:00:58

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⏲			
Test No.		PLe07s96			
Initial	Diameter, mm	70.57			
	Height, mm	147.36			
	Water Content, %	33.38			
	Dry Density, pcf	81.22			
	Saturation, %	98.35			
	Void Ratio	0.79086			
Before Shear	Water Content, %	31.55			
	Dry Density, pcf	83.83			
	Saturation, %	100.00			
	Void Ratio	0.73514			
	Back Press., psi	40			
Minor Prin. Stress, psi		96			
Max. Dev. Stress, psi		116.07			
Time to Failure, min		180			
Strain Rate, %/min		0.081			
B-Value		---			
Estimated Specific Gravity		2.33			
Liquid Limit		NP			
Plastic Limit		NP			
Plasticity Index		NP			

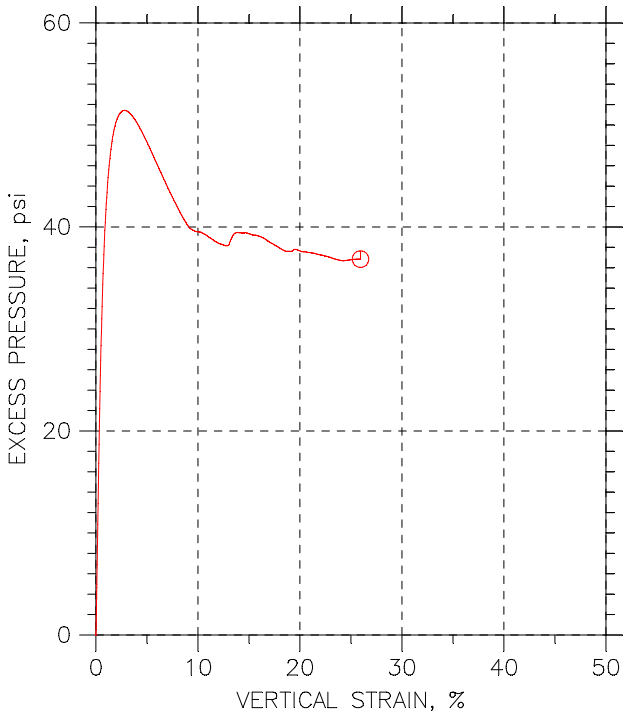
Project: KINGSTON COAL TVA	Failure Sketch			
Location: TN				
Project No.: 60095742				
Boring No.: RECONST				
Sample Type: RECONST				

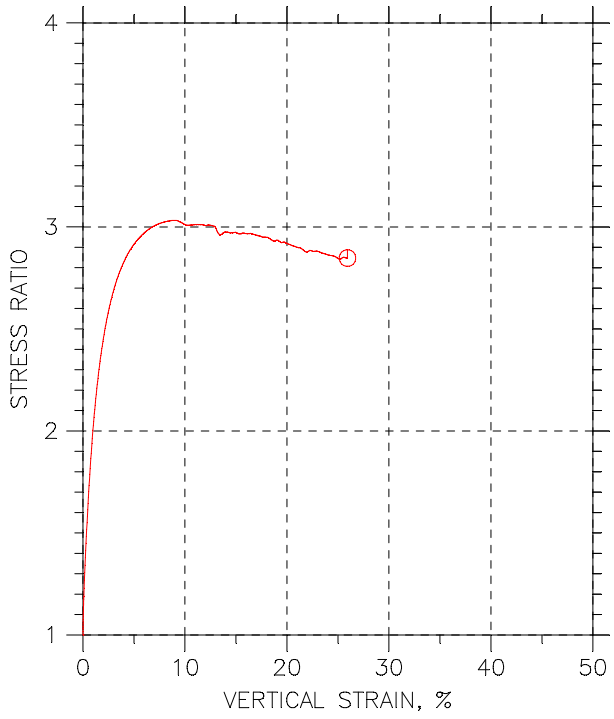
Description: PLUVIATION - RECONSTITUTED ASH - DARK GRAY (B=0.98)

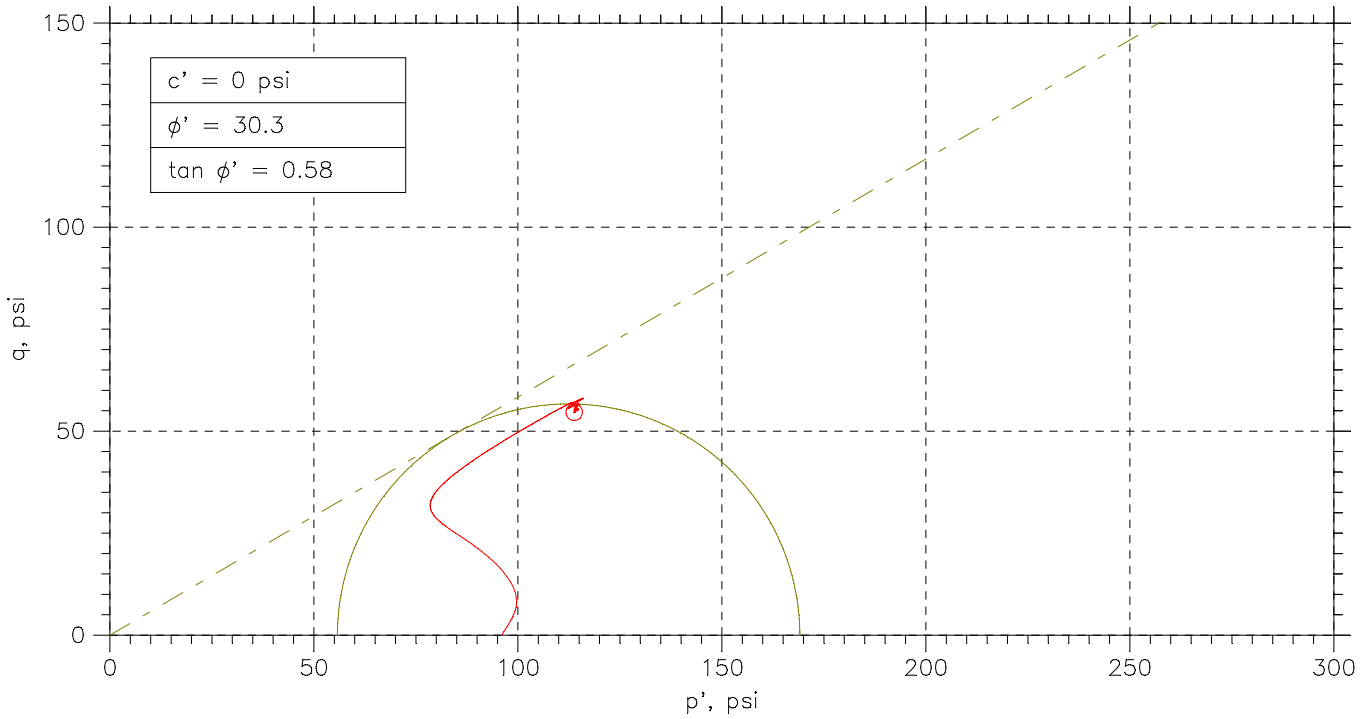
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (B=0.98)

Mon, 20-APR-2009 20:23:18

TRIAXIAL COMPRESSION TEST REPORT



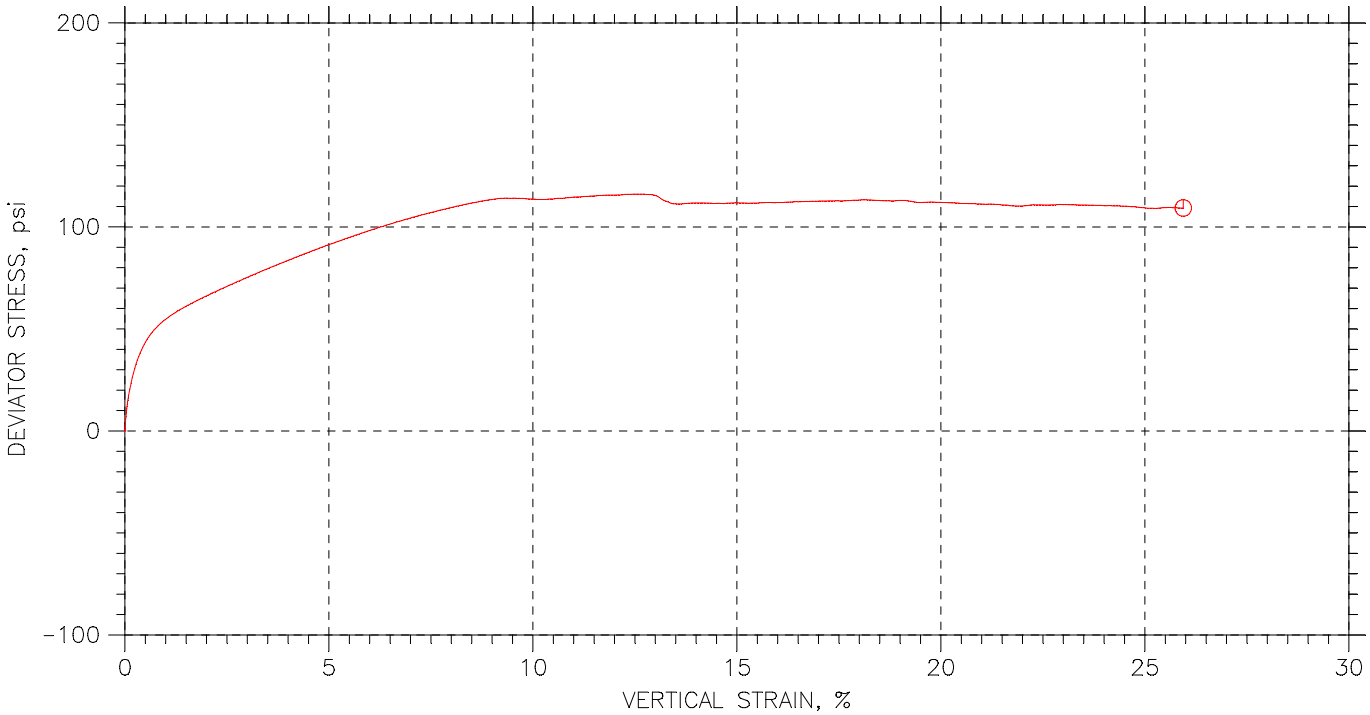


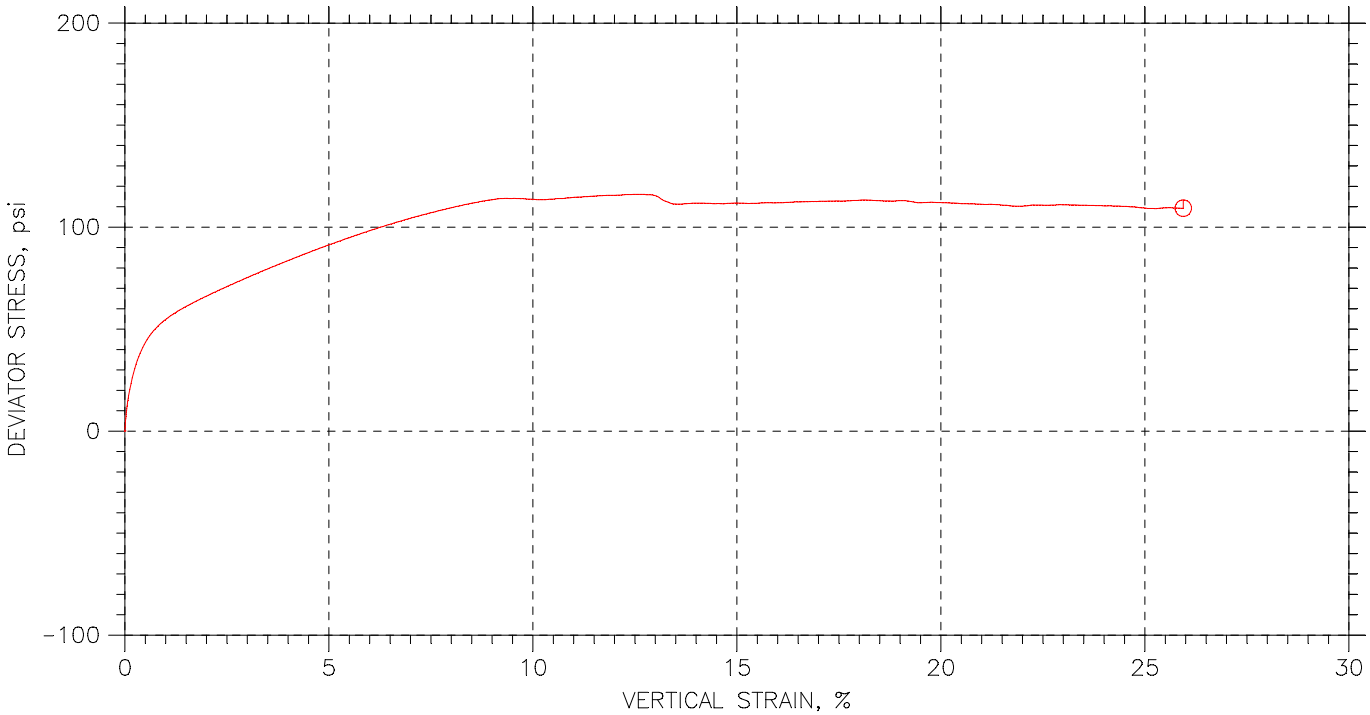


Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: RECONST	Tested By: BCM	Checked By: WPQ
Sample No.: 200s	Test Date: 3/26/09	Depth: -----
Test No.: PLe07s96	Sample Type: RECONST	Elevation: -----
Description: PLUVIATION - RECONSTITUTED ASH - DARK GRAY (B=0.98)		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (B=0.98)		

Mon, 20-APR-2009 20:23:19

TRIAXIAL COMPRESSION TEST REPORT

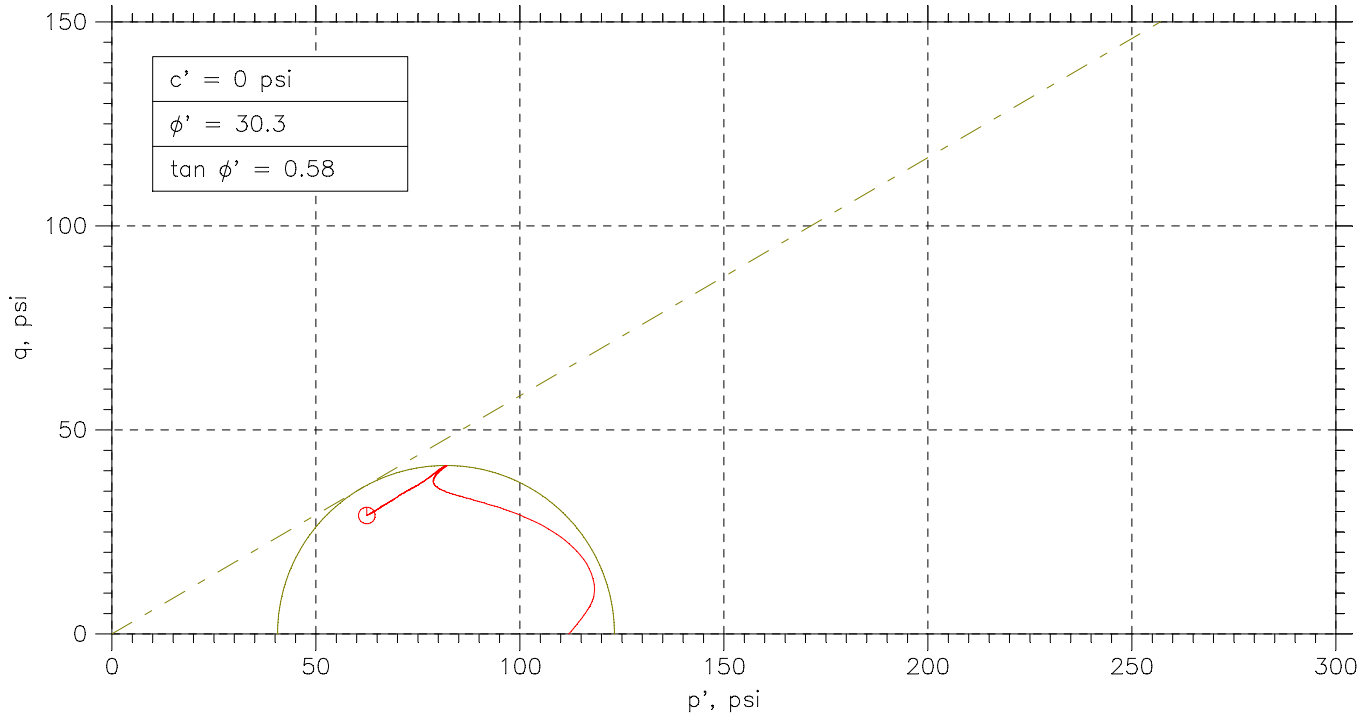


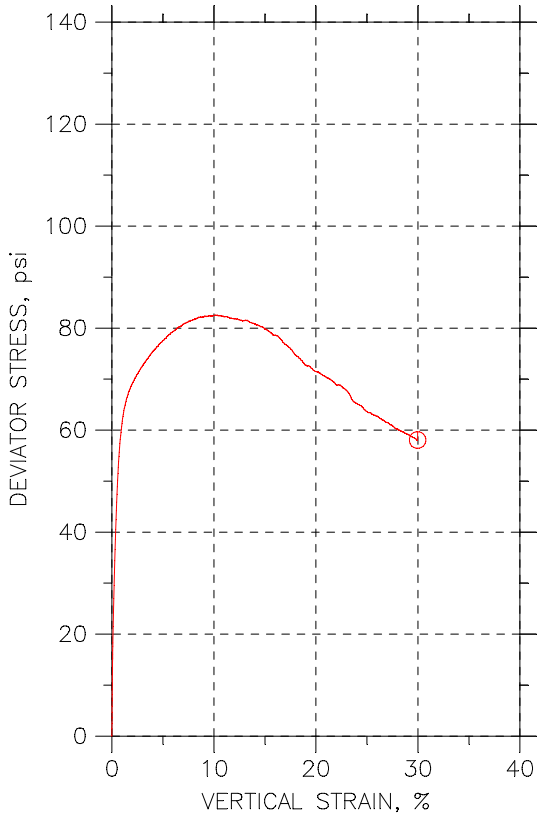


Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: RECONST	Tested By: BCM	Checked By: WPQ
Sample No.: 200s	Test Date: 3/26/09	Depth: -----
Test No.: PLe07s96	Sample Type: RECONST	Elevation: -----
Description: PLUVIATION - RECONSTITUTED ASH - DARK GRAY (B=0.98)		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (B=0.98)		

Mon, 20-APR-2009 20:23:19

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⏲			
Test No.		PLe06s112			
Initial	Diameter, mm	70.62			
	Height, mm	146.24			
	Water Content, %	26.63			
	Dry Density, pcf	89.05			
	Saturation, %	97.95			
	Void Ratio	0.6334			
Before Shear	Water Content, %	24.73			
	Dry Density, pcf	92.28			
	Saturation, %	100.00			
	Void Ratio	0.5763			
	Back Press., psi	20			
Minor Prin. Stress, psi		112			
Max. Dev. Stress, psi		82.572			
Time to Failure, min		151.5			
Strain Rate, %/min		0.069			
B-Value		---			
Estimated Specific Gravity		2.33			
Liquid Limit		NP			
Plastic Limit		NP			
Plasticity Index		NP			

Project: KINGSTON COAL TVA

Location: TN

Project No.: 60095742

Boring No.: RECONST

Sample Type: RECONST

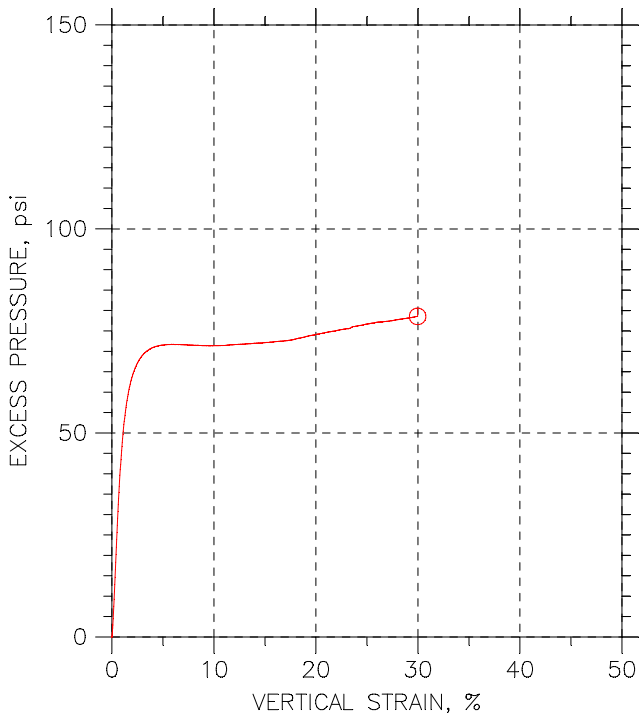
Failure Sketch

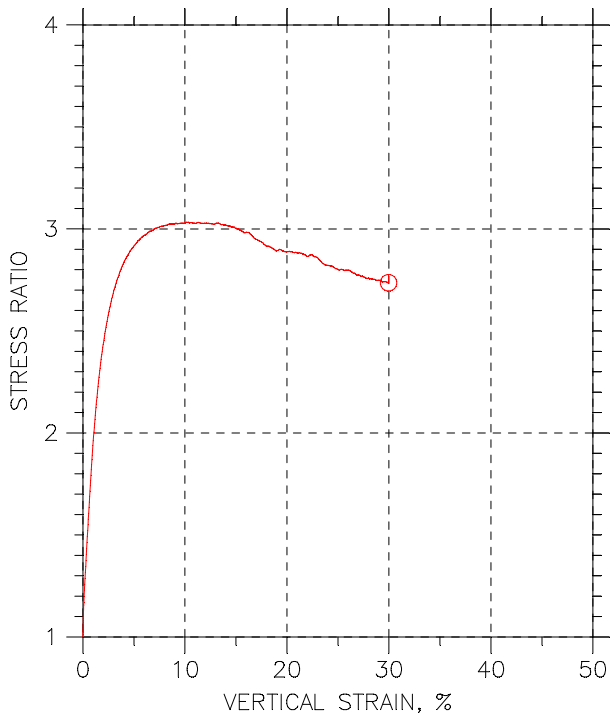
Description: PLUVIATION - RECONSTITUTED ASH - DARK GRAY

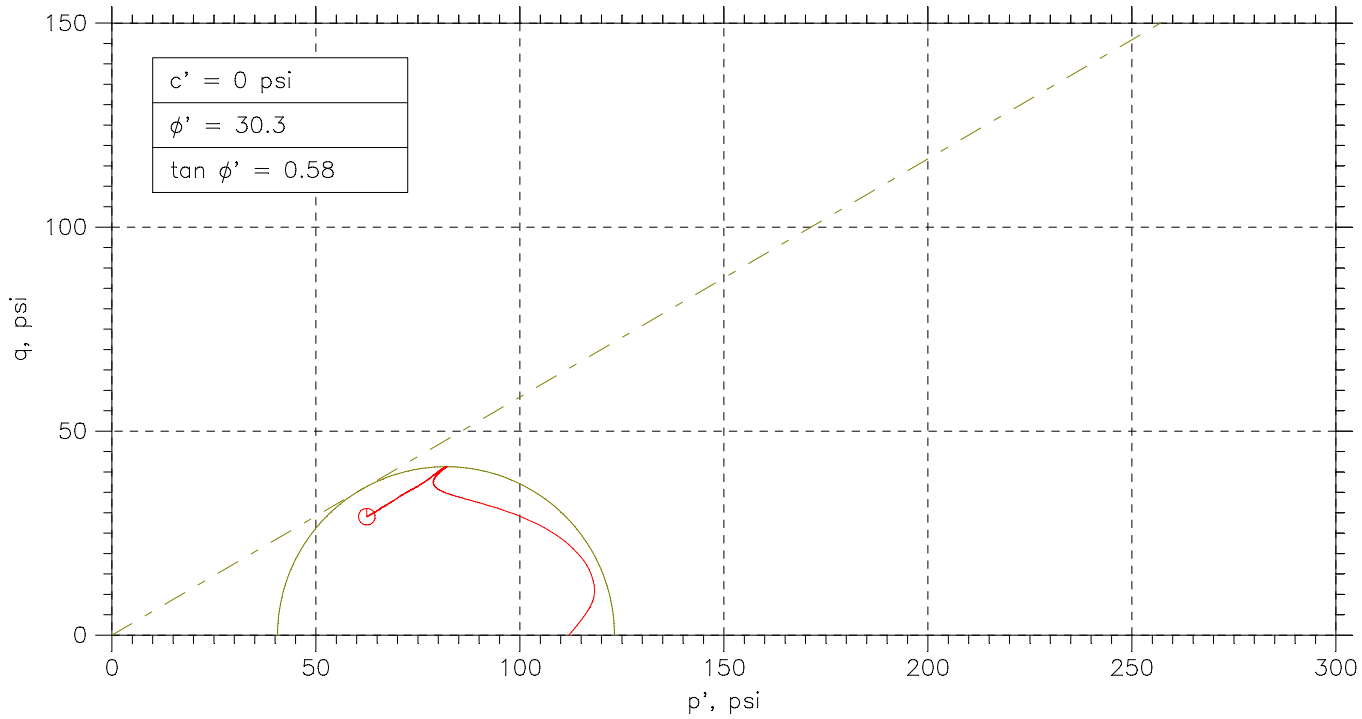
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (B=1.00)

Mon, 20-APR-2009 20:48:01

TRIAXIAL COMPRESSION TEST REPORT



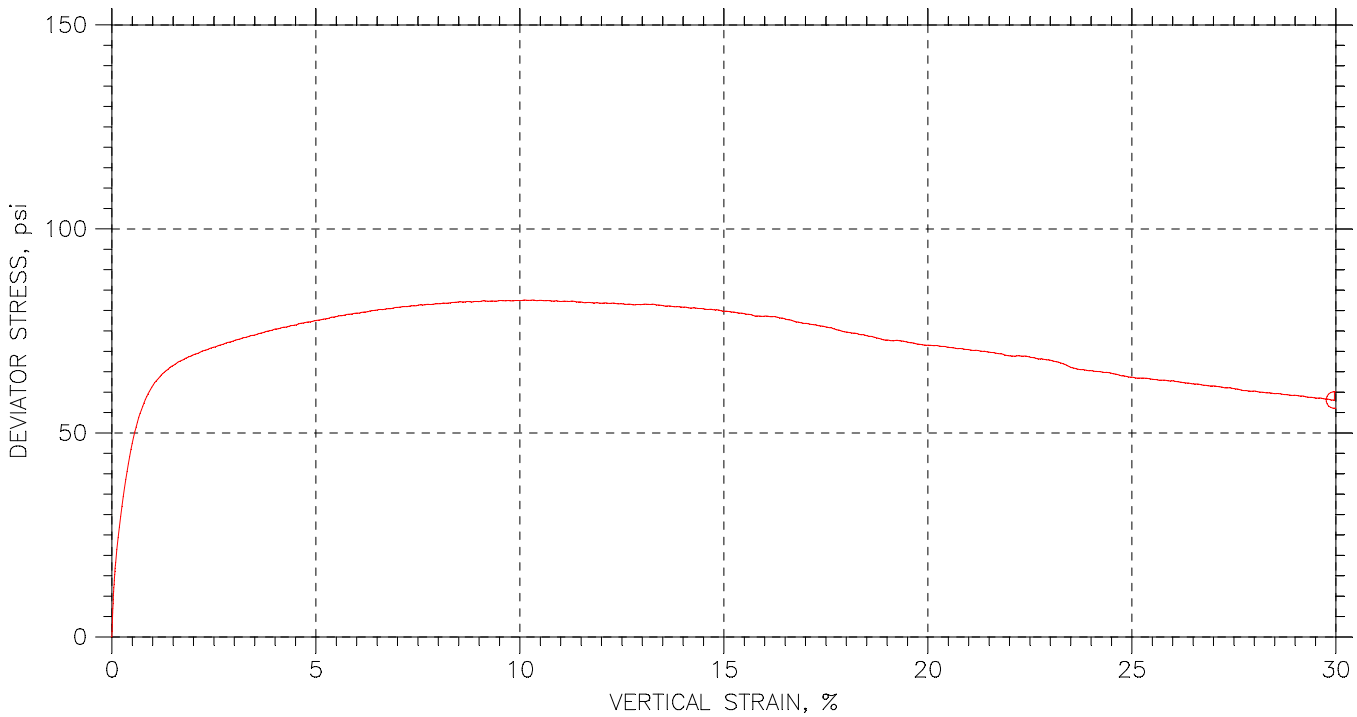


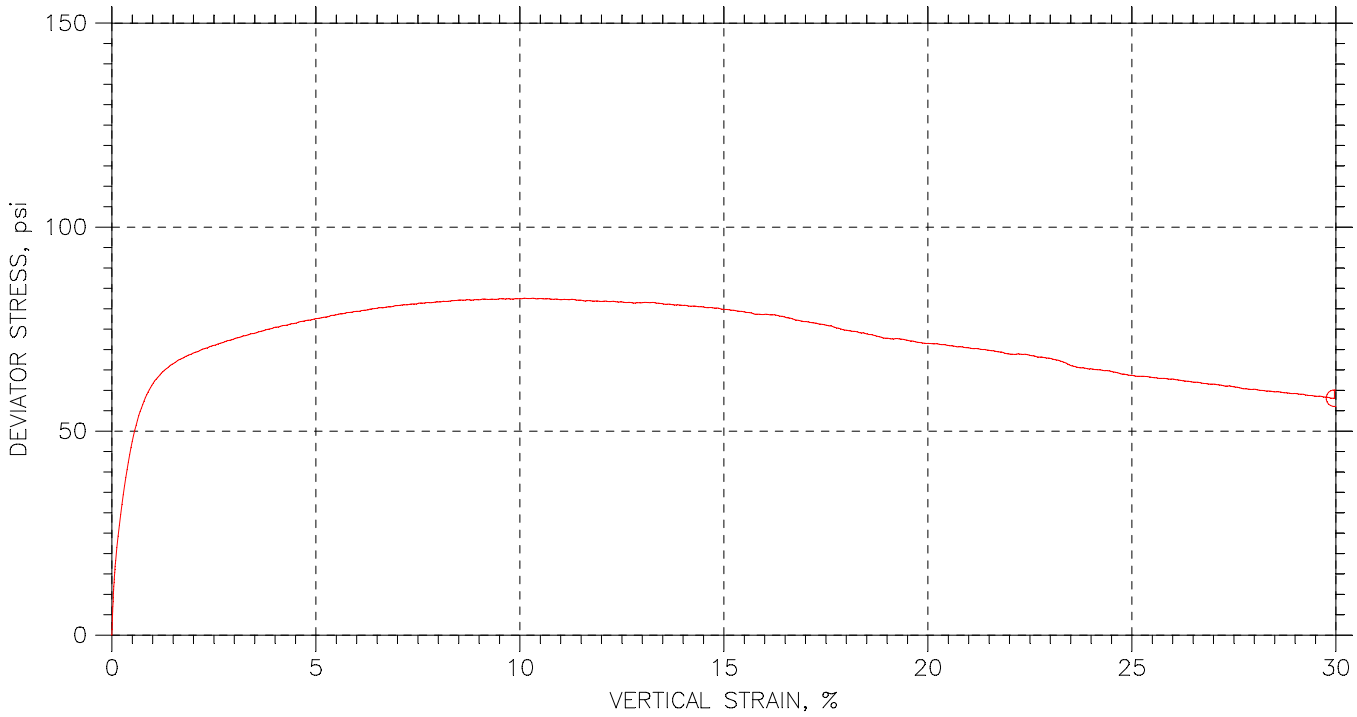


Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: RECONST	Tested By: BCM	Checked By: WPQ
Sample No.: 200s	Test Date: 3/26/09	Depth: ----
Test No.: PLe06s112	Sample Type: RECONST	Elevation: -----
Description: PLUVIATION - RECONSTITUTED ASH - DARK GRAY		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (B=1.00)		

Mon, 20-APR-2009 20:48:01

TRIAXIAL COMPRESSION TEST REPORT

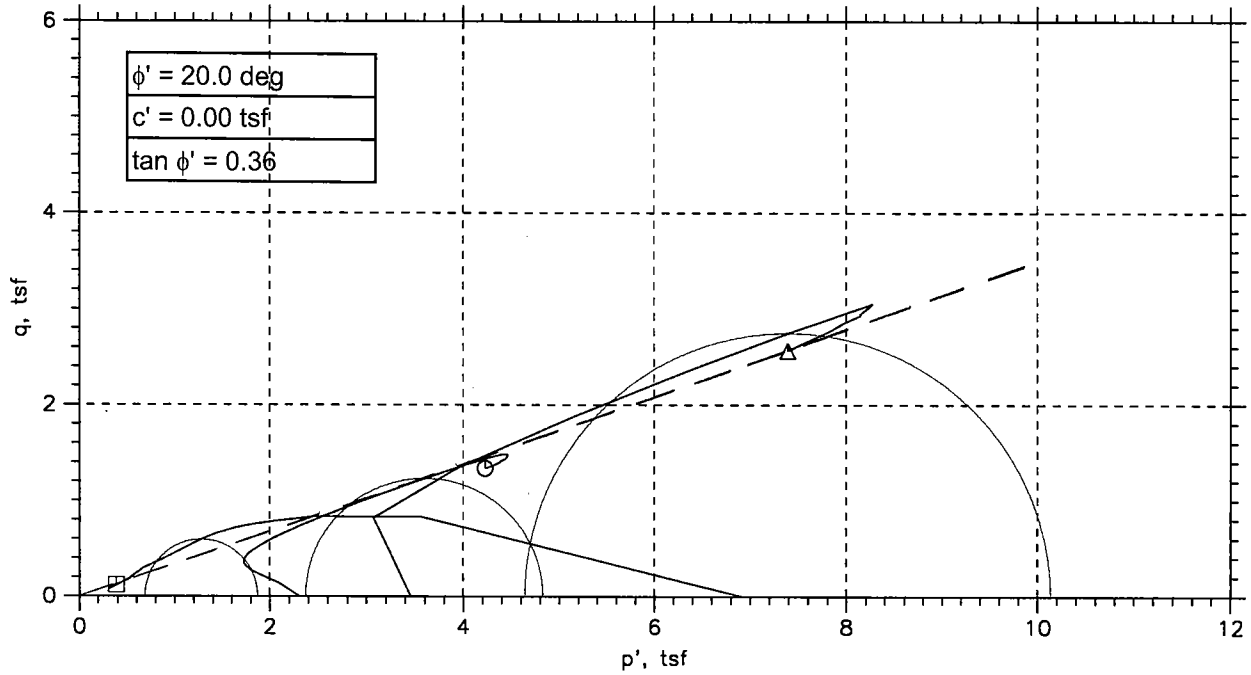


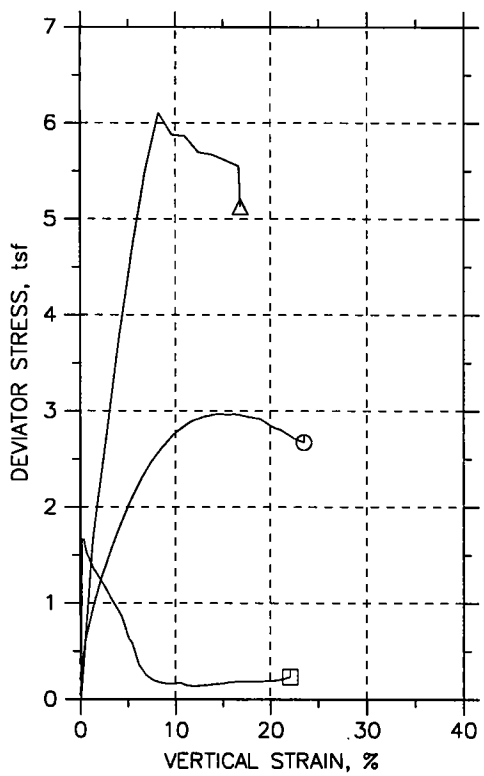


Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: RECONST	Tested By: BCM	Checked By: WPQ
Sample No.: 200s	Test Date: 3/26/09	Depth: ----
Test No.: PLe06s112	Sample Type: RECONST	Elevation: -----
Description: PLUVIATION - RECONSTITUTED ASH - DARK GRAY		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 (B=1.00)		

Mon, 20-APR-2009 20:48:02

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⊙	△	□	
Test No.		B 200 SET	9-200 SET	742200B T2	
Initial	Diameter, in	2.7933	2.8524	2.8098	
	Height, in	5.5744	6.1378	6.5102	
	Water Content, %	56.99	49.07	45.99	
	Dry Density, pcf	78.35	72.59	75.69	
	Saturation, %	135.89	101.66	102.80	
	Void Ratio	1.1114	1.279	1.1856	
Before Shear	Water Content, %	35.86	50.69	43.30	
	Dry Density, pcf	86.3	73.13	79.24	
	Saturation, %	103.63	106.42	105.49	
	Void Ratio	0.91704	1.2623	1.0878	
	Back Press., tsf	5.0402	5.0402	5.0405	
Minor Prin. Stress, tsf		2.3038	3.4558	6.9115	
Max. Dev. Stress, tsf		2.9666	6.1004	1.6633	
Time to Failure, min		234	12.004	4.0005	
Strain Rate, %/min		0.1	0.73	0.15	
B-Value		---	---	---	
Estimated Specific Gravity		2.65	2.65	2.65	
Liquid Limit		NP	NP	NP	
Plastic Limit		NP	NP	NP	
Plasticity Index		NP	NP	NP	
Failure Sketch					

Project: KINGSTON COAL TVA

Location: TN

Project No.: 6005742

Boring No.: B200 SET

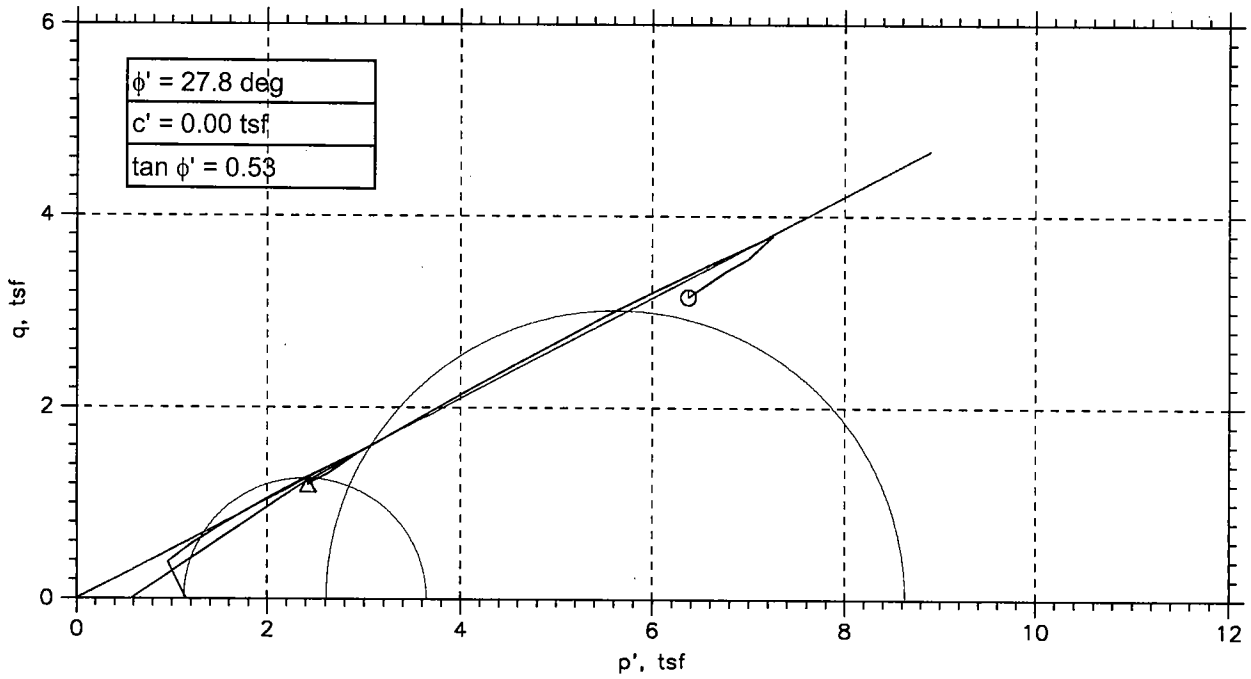
Sample Type: REMOLDED

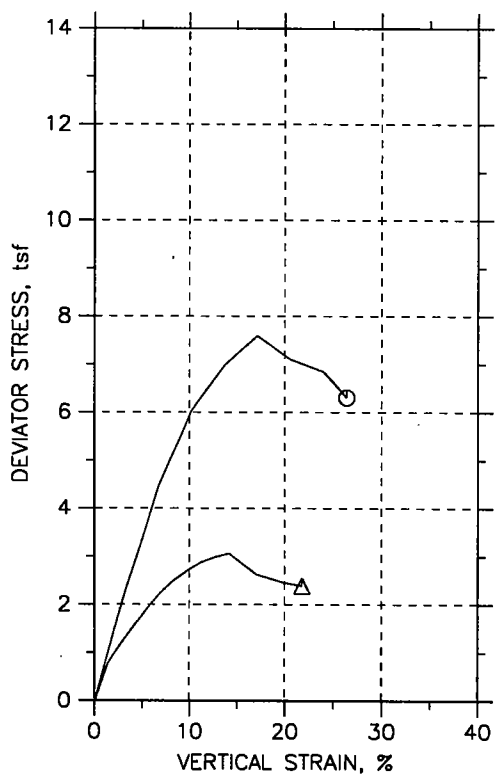
Description: COAL ASH - DARK GRAY SPECIMEN RECONSTITUTED BY DRAINED SLURRY TECHNIQUE

Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767

Mon, 16-MAR-2009 17:35:55

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⏲	△		
Test No.		5742200S09-200 SET			
Initial	Diameter, in	2.8327	2.8524		
	Height, in	6.2661	6.1075		
	Water Content, %	56.99	49.07		
	Dry Density, pcf	66.92	73.01		
	Saturation, %	102.59	102.73		
	Void Ratio	1.4722	1.2658		
Before Shear	Water Content, %	57.45	48.60		
	Dry Density, pcf	67.28	74.27		
	Saturation, %	104.35	104.92		
	Void Ratio	1.4589	1.2274		
	Back Press., tsf	5.0404	5.0399		
Minor Prin. Stress, tsf		0.57559	1.1521		
Max. Dev. Stress, tsf		7.5897	3.0543		
Time to Failure, min		25	20		
Strain Rate, %/min		0.9	0.77		
B-Value		---	---		
Estimated Specific Gravity		2.65	2.65		
Liquid Limit		NP	NP		
Plastic Limit		NP	NP		
Plasticity Index		NP	NP		
Failure Sketch					

Project: KINGSTON COAL TVA

Location: TN

Project No.: 60095742

Boring No.: 09-200 SET

Sample Type: REMOLDED

Description: COAL ASH - DARK GRAY SPECIMEN RECONSTITUTED BY DRAINED SLURRY TECHNIQUE

Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767

Mon, 16-MAR-2009 17:26:59

Appendix C

Test Data – Sensitive Silt/Clay

Results of Laboratory Tests Conducted for the AECOM TVA Kingston RCA Project

Prepared for

Bill Walton, P.E., S.E., F.ASCE
Senior Principal Engineer
Vice President
AECOM
750 Corporate Woods Parkway
Vernon Hills, IL 60061

Prepared by

Don J. DeGroot, Sc.D., P.E.
218 Grantwood Drive
Amherst, MA

3 June 2009

TABLE OF CONTENTS

List of Tables 3

List of Figures 3

1 INTRODUCTION **10**

2 TEST SAMPLES **10**

3 SCOPE OF SERVICES **11**

4 METHODS **11**
4.1 Sample Extraction 11
4.2 Water Content 11
4.3 Total and Dry Unit Weight 11
4.4 Atterberg Limits 11
4.5 Specific Gravity 12
4.6 Organic Content 12
4.7 Grain Size Distribution 12
4.8 Constant Rate of Strain (CRS) Consolidation 12
4.9 Direct Simple Shear (DSS) 13

5 RESULTS **14**
5.1 Tests Conducted, Photographs and Sample Logs 14
5.2 Index and Classification Data 14
5.3 Constant Rate of Strain Tests 14
5.4 Direct Simple Shear Tests 14
5.5 Test Specimen Photographs 15

6 REFERENCES **15**

7 NOTATION AND UNITS **16**

List of Tables

Table 1 Summary of Tests Performed.....17
Table 2 Summary of Water Content Data.....19
Table 3 Summary of Classification Test Results 21
Table 4 Summary of Constant Rate of Strain (CRS) specimen properties and test results 23
Table 5 Summary of Direct Simple Shear (DSS) specimen properties and test results.....25

List of Figures

Figure 4.8.1 Schematic of the Geonor Direct Simple Shear Apparatus 26

Figure 5.1.1 Photograph of Sample 09-100B S4 30.5 to 33.0 ft.....27
Figure 5.1.2 Photograph 1 of Sample 09-100B S5 33.0 to 35.5 ft.....28
Figure 5.1.3 Photograph 2 of Sample 09-100B S5 33.0 to 35.5 ft.....29
Figure 5.1.4 Photograph of Sample 09-100B S6 35.5 to 38.0 ft. 30
Figure 5.1.5 Photographs of Sample 09-101B S5 38.0 to 40.5 ft. 31
Figure 5.1.6 Photograph of Sample 09-103B S2 33.5 to 36.0 ft–as received 2 six inch sections.32
Figure 5.1.7 Photograph of Sample 09-103B S3 36.0 to 38.5 ft.....33
Figure 5.1.8 Photograph of Sample 09-103B S4 38.5 to 41.0 ft – as received 10inch section.....34
Figure 5.1.9 Photograph of Sample 09-103B S5 41.0 to 42.0 ft – as received section. 35
Figure 5.1.10 Photograph of Sample 09-104B S4 22.5 to 25.0 ft.....36
Figure 5.1.11 Photograph 1 of Sample 09-108B S2 23.5 to 26.0 ft.....37
Figure 5.1.12 Photograph 2 of Sample 09-108B S2 23.5 to 26.0 ft – section from 12 to 18 inches (right to left).....38
Figure 5.1.13 Photograph of Sample 09-109B S2 25.5 to 28.0 ft.....39
Figure 5.1.14 Photograph of Sample 09-200B S4 32.5 to 35.0 ft.....40
Figure 5.1.15 Photograph 1 of Sample 09-201B S2 25.5 to 28.0 ft.....41
Figure 5.1.16 Photograph 2 of Sample 09-201B S2 25.5 to 28.0 ft.....42
Figure 5.1.17 Photographs of Sample 09-202B S2 28.5 to 31.0 ft. 43
Figure 5.1.18 Photograph of Sample 09-207B S3 13.0 to 15.5 ft.....44
Figure 5.1.19 Photograph of Sample 09-210B S2 – as received 10.5 to 11.0 ft section. 45
Figure 5.1.20 Photograph of Sample 09-301B S6 87.5 to 90.0 ft.....46
Figure 5.1.21 Photograph of Sample 09-303B S9 87.5 to 90.0 ft.....47
Figure 5.1.22 Photograph of Sample 09-408B S6 33.0 to 35.5 ft.....48
Figure 5.1.23 Photograph of Sample 09-408B S7 35.5 to 38.0 ft.....49
Figure 5.1.24 Photograph of Sample 09-500B S4 26.5 to 29.0 ft.....50
Figure 5.1.25 Photograph of Sample 09-500B S5 29.0 to 31.5 ft.....51
Figure 5.1.26 Photograph of Sample 09-502B S4 22.0 to 24.5 ft.....52
Figure 5.1.27 Photograph of Sample 09-503B S2 31.0 to 33.5 ft.....53
Figure 5.1.28 Photograph of Sample 09-503B S3 33.5 to 38.0 ft.....54
Figure 5.1.29 Photograph of Sample 09-600B S2 44.0 to 46.5 ft.....55
Figure 5.1.30 Photograph 1of Sample 09-605B S7 47.0 to 49.5 ft – 12 to 29 inch section (right to left).....56

Figure 5.1.31 Photograph 2 of Sample 09-605B S7 47.0 to 49.5 ft – 1 to 12 inch section (left to right).....57
Figure 5.1.32 Photograph of Sample 09-700B S3 22.5 to 25.0 ft.....58
Figure 5.1.33 Photograph of Sample 09-800B S4 26.5 to 29.0 ft.....59
Figure 5.1.34 Photograph of Sample 09-801B S5 37.5 to 40.0 ft.....60

Figure 5.1.35 Sample log and test locations for Sample 09-100B S4 30.5 to 33.0 ft.....61
Figure 5.1.36 Sample log and test locations for Sample 09-100B S5 33.0 to 35.5 ft.....62
Figure 5.1.37 Sample log and test locations for Sample 09-100B S6 35.5 to 38.0 ft.....63
Figure 5.1.38 Sample log and test locations for Sample 09-101B S5 38.0 to 40.5 ft.....64
Figure 5.1.39 Sample log and test locations for Sample 09-103B S2 33.5 to 36.0 ft.....65
Figure 5.1.40 Sample log and test locations for Sample 09-103B S3 36.0 to 38.5 ft.....66
Figure 5.1.41 Sample log and test locations for Sample 09-103B S4 38.5-39.0 ft.....67
Figure 5.1.42 Sample log and test locations for Sample 09-103B S5 41.0-42.0 ft.....68
Figure 5.1.43 Sample log and test locations for Sample 09-104B S4 22.5-25.0 ft.....69
Figure 5.1.44 Sample log and test locations for Sample 09-108B S2 23.5-26.0 ft.....70
Figure 5.1.45 Sample log and test locations for Sample 09-109B S3 25.5-28.0 ft.....71
Figure 5.1.46 Sample log and test locations for Sample 09-200B S4 32.5-35.0 ft.....72
Figure 5.1.47 Sample log and test locations for Sample 09-201B S2 25.5-28.0 ft.....73
Figure 5.1.48 Sample log and test locations for Sample 09-202B S2 28.5-31.0 ft.....74
Figure 5.1.49 Sample log and test locations for Sample 09-206B S5 20.0-22.5 ft.....75
Figure 5.1.50 Sample log and test locations for Sample 09-207B S3 13.0-15.5 ft.....76
Figure 5.1.51 Sample log and test locations for Sample 09-210B S2 10.5-11.0 ft.....77
Figure 5.1.52 Sample log and test locations for Sample 09-301B S6 87.5-90.0 ft.....78
Figure 5.1.53 Sample log and test locations for Sample 09-303B S9 87.5-90.0 ft.....79
Figure 5.1.54 Sample log and test locations for Sample 09-408B S6 33.0-35.5 ft.....80
Figure 5.1.55 Sample log and test locations for Sample 09-408B S7 35.5-38.0 ft.....81
Figure 5.1.56 Sample log and test locations for Sample 09-500B S4 26.5-29.0 ft.....82
Figure 5.1.57 Sample log and test locations for Sample 09-500B S5 29.0-31.5 ft.....83
Figure 5.1.58 Sample log and test locations for Sample 09-502B S4 22.0-24.5 ft.....84
Figure 5.1.59 Sample log and test locations for Sample 09-503B S2 31.0-33.5 ft.....85
Figure 5.1.60 Sample log and test locations for Sample 09-503B S3 35.5-37.5 ft.....86
Figure 5.1.61 Sample log and test locations for Sample 09-600B S2 44.0-46.5 ft.....87
Figure 5.1.62 Sample log and test locations for Sample 09-601B S5 49.5-52.0 ft.....88
Figure 5.1.63 Sample log and test locations for Sample 09-602B S4 54.0-56.5 ft.....89
Figure 5.1.64 Sample log and test locations for Sample 09-605B S7 47.0-49.5 ft.....90
Figure 5.1.65 Sample log and test locations for Sample 09-605B S8 49.5-52.0 ft.....91
Figure 5.1.66 Sample log and test locations for Sample 09-700B S3 22.5-25.0 ft.....92
Figure 5.1.67 Sample log and test locations for Sample 09-800B S4 26.5-29.0 ft.....93
Figure 5.1.68 Sample log and test locations for Sample 09-801B S5 37.5-40.0 ft.....94

Figure 5.2.1 Plasticity Chart with Atterberg Limits results for selected samples (see Table 3 and sample logs for specific test specimen location) 95
Figure 5.2.2 Percent passing versus grain diameter from grain size distribution test results (see sample logs for specific test specimen location).....96

Figure 5.3.1 Vertical strain and coefficient of consolidation versus vertical effective stress for test CRS159 on sample 09-202B S2 28.5-31.0 ft. 97
Figure 5.3.2 Vertical strain and normalized base pore pressure versus vertical effective stress for test CRS159 on sample 09-202B S2 28.5-31.0 ft.....98
Figure 5.3.3 Vertical strain and coefficient of consolidation versus vertical effective stress for test CRS160 on sample 09-201B S2 25.5-28.0 ft. 99
Figure 5.3.4 Vertical strain and normalized base pore pressure versus vertical effective stress for test CRS160 on sample 09-201B S2 25.5-28.0 ft.....100
Figure 5.3.5 Vertical strain and coefficient of consolidation versus vertical effective stress for test CRS161 on sample 09-104B S4 22.5-25.0 ft. 101
Figure 5.3.6 Vertical strain and normalized base pore pressure versus vertical effective stress for test CRS161 on sample 09-104B S4 22.5-25.0 ft.....102
Figure 5.3.7 Vertical strain and coefficient of consolidation versus vertical effective stress for test CRS162 on sample 09-301B S6 87.5-90.0 ft. 103
Figure 5.3.8 Vertical strain and normalized base pore pressure versus vertical effective stress for test CRS162 on sample 09-301B S6 87.5-90.0 ft.....104
Figure 5.3.9 Vertical strain and coefficient of consolidation versus vertical effective stress for test CRS163 on sample 09-101B S5 38.0-40.5 ft. 105
Figure 5.3.10 Vertical strain and normalized base pore pressure versus vertical effective stress for test CRS163 on sample 09-101B S5 38.0-40.5 ft.....106
Figure 5.3.11 Vertical strain and coefficient of consolidation versus vertical effective stress for test CRS164 on sample 09-108B S2 23.5-26.0 ft. 107
Figure 5.3.12 Vertical strain and normalized base pore pressure versus vertical effective stress for test CRS164 on sample 09-108B S2 23.5-26.0 ft.....108
Figure 5.3.13 Vertical strain and coefficient of consolidation versus vertical effective stress for test CRS165 on sample 09-605B S7 47.0-49.5 ft. 109
Figure 5.3.14 Vertical strain and normalized base pore pressure versus vertical effective stress for test CRS165 on sample 09-605B S7 47.0-49.5 ft.....110
Figure 5.3.15 Vertical strain and coefficient of consolidation versus vertical effective stress for test CRS166 on sample 09-103B S2 33.5-36.0 ft. 111
Figure 5.3.16 Vertical strain and normalized base pore pressure versus vertical effective stress for test CRS166 on sample 09-103B S2 33.5-36.0 ft.....112
Figure 5.3.17 Vertical strain and coefficient of consolidation versus vertical effective stress for test CRS167 on sample 09-100B S5 33.0-35.5 ft. 113
Figure 5.3.18 Vertical strain and normalized base pore pressure versus vertical effective stress for test CRS167 on sample 09-100B S5 33.0-35.5 ft.....114
Figure 5.3.19 Vertical strain and coefficient of consolidation versus vertical effective stress for test CRS168 on sample 09-103B S5 41.0-43.5 ft. 115
Figure 5.3.20 Vertical strain and normalized base pore pressure versus vertical effective stress for test CRS168 on sample 09-103B S5 41.0-43.5 ft.....116
Figure 5.3.21 Vertical strain and coefficient of consolidation versus vertical effective stress for test CRS169 on sample 09-408B S7 35.5-38.0 ft. 117
Figure 5.3.22 Vertical strain and normalized base pore pressure versus vertical effective stress for test CRS169 on sample 09-408B S7 35.5-38.0 ft.....118
Figure 5.3.23 Vertical strain and coefficient of consolidation versus vertical effective stress for test CRS170 on sample 09-503B S3 35.5-38.0 ft. 119

Figure 5.3.24 Vertical strain and normalized base pore pressure versus vertical effective stress for test CRS170 on sample 09-503B S3 35.5-38.0 ft....120

Figure 5.4.1 Vertical strain versus vertical effective stress for test DSS G349 on sample 09-301B S6 87.5-90.0 ft....121
Figure 5.4.2 Horizontal shear stress and normalized shear stress versus shear strain for test DSS G349 on sample 09-301B S6 87.5-90.0 ft....122
Figure 5.4.3 Pore pressure and normalized pore pressure versus shear strain for test DSS G349 on sample 09-301B S6 87.5-90.0 ft....123
Figure 5.4.4 Horizontal shear stress versus vertical effective stress and normalized shear stress versus normalized vertical effective stress for test DSS G349 on sample 09-301B S6 87.5-90.0 ft....124
Figure 5.4.5 Shear modulus and normalized shear modulus versus shear strain for test DSS G349 on sample 09-301B S6 87.5-90.0 ft....125
Figure 5.4.6 Vertical strain versus vertical effective stress for test DSS G350 on sample 09-101B S5 38.0-40.5 ft....126
Figure 5.4.7 Horizontal shear stress and normalized shear stress versus shear strain for test DSS G350 on sample 09-101B S5 38.0-40.5 ft....127
Figure 5.4.8 Pore pressure and normalized pore pressure versus shear strain for test DSS G350 on sample 09-101B S5 38.0-40.5 ft....128
Figure 5.4.9 Horizontal shear stress versus vertical effective stress and normalized shear stress versus normalized vertical effective stress for test DSS G350 on sample 09-101B S5 38.0-40.5 ft....129
Figure 5.4.10 Shear modulus and normalized shear modulus versus shear strain for test DSS G350 on sample 09-101B S5 38.0-40.5 ft....130
Figure 5.4.11 Vertical strain versus vertical effective stress for test DSS G351 on sample 09-108B S2 23.5-26.0 ft....131
Figure 5.4.12 Horizontal shear stress and normalized shear stress versus shear strain for test DSS G351 on sample 09-108B S2 23.5-26.0 ft....132
Figure 5.4.13 Pore pressure and normalized pore pressure versus shear strain for test DSS G351 on sample 09-108B S2 23.5-26.0 ft....133
Figure 5.4.14 Horizontal shear stress versus vertical effective stress and normalized shear stress versus normalized vertical effective stress for test DSS G351 on sample 09-108B S2 23.5-26.0 ft....134
Figure 5.4.15 Shear modulus and normalized shear modulus versus shear strain for test DSS G351 on sample 09-108B S2 23.5-26.0 ft....135
Figure 5.4.16 Vertical strain versus vertical effective stress for test DSS G352 on sample 09-109B S3 25.5-28.0 ft....136
Figure 5.4.17 Horizontal shear stress and normalized shear stress versus shear strain for test DSS G352 on sample 09-109B S3 25.5-28.0 ft....137
Figure 5.4.18 Pore pressure and normalized pore pressure versus shear strain for test DSS G352 on sample 09-109B S3 25.5-28.0 ft....138
Figure 5.4.19 Horizontal shear stress versus vertical effective stress and normalized shear stress versus normalized vertical effective stress for test DSS G352 on sample 09-109B S3 25.5-28.0 ft....139
Figure 5.4.20 Shear modulus and normalized shear modulus versus shear strain

for test DSS G352 on sample 09-109B S3 25.5-28.0 ft.....140
Figure 5.4.21 Vertical strain versus vertical effective stress for test DSS G353 on sample 09-605B S7 47.5-49.5 ft 141
Figure 5.4.22 Horizontal shear stress and normalized shear stress versus shear strain for test DSS G353 on sample 09-605B S7 47.5-49.5 ft.....142
Figure 5.4.23 Pore pressure and normalized pore pressure versus shear strain for test DSS G353 on sample 09-605B S7 47.5-49.5 ft 143
Figure 5.4.24 Horizontal shear stress versus vertical effective stress and normalized shear stress versus normalized vertical effective stress for test DSS G353 on sample 09-605B S7 47.5-49.5 ft 144
Figure 5.4.25 Shear modulus and normalized shear modulus versus shear strain for test DSS G353 on sample 09-605B S7 47.5-49.5 ft.....145
Figure 5.4.26 Vertical strain versus vertical effective stress for test DSS G354 on sample 09-103B S2 33.5-36.0 ft 146
Figure 5.4.27 Horizontal shear stress and normalized shear stress versus shear strain for test DSS G354 on sample 09-103B S2 33.5-36.0 ft.....147
Figure 5.4.28 Pore pressure and normalized pore pressure versus shear strain for test DSS G354 on sample 09-103B S2 33.5-36.0 ft 148
Figure 5.4.29 Horizontal shear stress versus vertical effective stress and normalized shear stress versus normalized vertical effective stress for test DSS G354 on sample 09-103B S2 33.5-36.0 ft 149
Figure 5.4.30 Shear modulus and normalized shear modulus versus shear strain for test DSS G354 on sample 09-103B S2 33.5-36.0 ft.....150
Figure 5.4.31 Vertical strain versus vertical effective stress for test DSS G355 on sample 09-103B S2 33.5-36.0 ft 151
Figure 5.4.32 Horizontal shear stress and normalized shear stress versus shear strain for test DSS G355 on sample 09-103B S2 33.5-36.0 ft.....152
Figure 5.4.33 Pore pressure and normalized pore pressure versus shear strain for test DSS G355 on sample 09-103B S2 33.5-36.0 ft 153
Figure 5.4.34 Horizontal shear stress versus vertical effective stress and normalized shear stress versus normalized vertical effective stress for test DSS G355 on sample 09-103B S2 33.5-36.0 ft 154
Figure 5.4.35 Shear modulus and normalized shear modulus versus shear strain for test DSS G355 on sample 09-103B S2 33.5-36.0 ft.....155
Figure 5.4.36 Vertical strain versus vertical effective stress for test DSS G356 on sample 09-100B S5 33.0-35.5 ft 156
Figure 5.4.37 Horizontal shear stress and normalized shear stress versus shear strain for test DSS G356 on sample 09-100B S5 33.0-35.5 ft.....157
Figure 5.4.38 Pore pressure and normalized pore pressure versus shear strain for test DSS G356 on sample 09-100B S5 33.0-35.5 ft 158
Figure 5.4.39 Horizontal shear stress versus vertical effective stress and normalized shear stress versus normalized vertical effective stress for test DSS G356 on sample 09-100B S5 33.0-35.5 ft 159
Figure 5.4.40 Shear modulus and normalized shear modulus versus shear strain for test DSS G356 on sample 09-100B S5 33.0-35.5 ft.....160
Figure 5.4.41 Vertical strain versus vertical effective stress for test DSS G357 on

sample 09-100B S6 35.5-38.0 ft 161
Figure 5.4.42 Horizontal shear stress and normalized shear stress versus shear strain for test DSS G357 on sample 09-100B S6 35.5-38.0 ft.....162
Figure 5.4.43 Pore pressure and normalized pore pressure versus shear strain for test DSS G357 on sample 09-100B S6 35.5-38.0 ft 163
Figure 5.4.44 Horizontal shear stress versus vertical effective stress and normalized shear stress versus normalized vertical effective stress for test DSS G357 on sample 09-100B S6 35.5-38.0 ft 164
Figure 5.4.45 Shear modulus and normalized shear modulus versus shear strain for test DSS G357 on sample 09-100B S6 35.5-38.0 ft.....165
Figure 5.4.46 Vertical strain versus vertical effective stress for test DSS G358 on sample 09-108B S2 23.5-26.0 ft 166
Figure 5.4.47 Horizontal shear stress and normalized shear stress versus shear strain for test DSS G358 on sample 09-108B S2 23.5-26.0 ft.....167
Figure 5.4.48 Pore pressure and normalized pore pressure versus shear strain for test DSS G358 on sample 09-108B S2 23.5-26.0 ft 168
Figure 5.4.49 Horizontal shear stress versus vertical effective stress and normalized shear stress versus normalized vertical effective stress for test DSS G358 on sample 09-108B S2 23.5-26.0 ft 169
Figure 5.4.50 Shear modulus and normalized shear modulus versus shear strain for test DSS G358 on sample 09-108B S2 23.5-26.0 ft.....170
Figure 5.4.51 Vertical strain versus vertical effective stress for test DSS G359 on sample 09-408B S7 35.5-38.0 ft 171
Figure 5.4.52 Horizontal shear stress and normalized shear stress versus shear strain for test DSS G359 on sample 09-408B S7 35.5-38.0 ft.....172
Figure 5.4.53 Pore pressure and normalized pore pressure versus shear strain for test DSS G359 on sample 09-408B S7 35.5-38.0 ft 173
Figure 5.4.54 Horizontal shear stress versus vertical effective stress and normalized shear stress versus normalized vertical effective stress for test DSS G359 on sample 09-408B S7 35.5-38.0 ft 174
Figure 5.4.55 Shear modulus and normalized shear modulus versus shear strain for test DSS G359 on sample 09-408B S7 35.5-38.0 ft.....175
Figure 5.4.56 Vertical strain versus vertical effective stress for test DSS G360 on sample 09-503B S3 35.5-38.0 ft 176
Figure 5.4.57 Horizontal shear stress and normalized shear stress versus shear strain for test DSS G360 on sample 09-503B S3 35.5-38.0 ft.....177
Figure 5.4.58 Pore pressure and normalized pore pressure versus shear strain for test DSS G360 on sample 09-503B S3 35.5-38.0 ft 178
Figure 5.4.59 Horizontal shear stress versus vertical effective stress and normalized shear stress versus normalized vertical effective stress for test DSS G360 on sample 09-503B S3 35.5-38.0 ft 179
Figure 5.4.60 Shear modulus and normalized shear modulus versus shear strain for test DSS G360 on sample 09-503B S3 35.5-38.0 ft.....180
Figure 5.4.61 Vertical strain versus vertical effective stress for test DSS G357 and creep test G361 on sample 09-100B S6 35.5-38.0 ft 181
Figure 5.4.62 Horizontal shear stress and normalized shear stress versus shear strain

for test DSS G357 and creep test G361 on sample 09-100B S6 35.5-38.0 ft 182
Figure 5.4.63 Pore pressure and normalized pore pressure versus shear strain for test DSS G357 and creep test G361 on sample 09-100B S6 35.5-38.0 ft 183
Figure 5.4.64 Horizontal shear stress versus vertical effective stress and normalized shear stress versus normalized vertical effective stress for test DSS G357 and creep test G361 on sample 09-100B S6 35.5-38.0 ft 184
Figure 5.4.65 Horizontal shear strain versus time for DSS creep test G361 on sample 09-100B S6 35.5-38.0 ft 185

Figure 5.5.1 Photograph through a microscope of split open, oven dried, test specimen CRS167 on 09-100B S5. 186
Figure 5.5.2 Photograph through a microscope of split open, oven dried, test specimen DSS G357 on 09-100B S6. 187
Figure 5.5.3 Photograph through a microscope of split open, oven dried, test specimen DSS G350 on 09-101B S5. 188
Figure 5.5.4 Photograph through a microscope of split open, oven dried, test specimen DSS G355 on 09-103B S2. 189
Figure 5.5.5 Photograph through a microscope of split open, oven dried, test specimen DSS G349 on 09-301B S6. 190
Figure 5.5.6 Photograph through a microscope of split open, oven dried, test specimen DSS G360 on 09-503B S3. 191

1 INTRODUCTION

This report contains results of laboratory tests, including index and classification, constant rate of strain (CRS) consolidation and direct simple shear (DSS), conducted for AECOM, Vernon Hills, IL. Tube samples were provided by AECOM for testing. All tests described herein were conducted at the University of Massachusetts (UMass) Amherst during March to June 2009 under the supervision of Dr. Don J. DeGroot. The report includes information on the scope of services, test methods, and presentation of test results.

2 TEST SAMPLES

The test samples consisted of 3" diameter by 36" long tubes unless otherwise noted. All samples were delivered to UMass Amherst by AECOM.

Received 3/11/09
09-108B S2 23.5-26.0 ft
09-201B S2 25.5-28.0 ft
09-202B S2 28.5-28.0 ft
09-303B S9 87.5-90.0 ft

Received 3/25/09
09-101B S5 38.0-40.5 ft
09-104B S4 22.5-25.0 ft
09-109B S3 25.5-28.0 ft
09-206B S5 20.0-22.5 ft
09-207B S3 13.0-15.5 ft
09-301B S6 87.5-90.0 ft
09-600B S2 44.0-46.5 ft
09-601B S5 49.5-52.0 ft
09-602B S4 54.0-56.5 ft

Received 4/8/09
09-103B S3 36.0-38.5 ft
09-103B S4 38.5-39.0 ft (partial tube = 6")
09-103B S5 41.0-42.0 ft (partial tube = 12")
09-500B S4 26.5-29.0 ft
09-500B S5 29.0-31.5 ft
09-503B S3 35.5-37.5 ft (partial tube = 24")
09-605B S7 47.0-49.5 ft
09-605B S8 49.5-52.0 ft

Received 4/18/09
09-100B S4 30.5-33.0 ft
09-100B S5 33.0-35.5 ft
09-100B S6 35.5-38.0 ft
09-103B S2 partial tube: 34.9-35.4 (6") and 35.4-35.9 (6") ft

09-200B S4 32.5-35.0 ft
09-408B S6 33.0-35.5 ft
09-408B S7 35.5-38.0 ft
09-502B S4 22.0-24.5 ft
09-503B S2 31.0-33.5 ft
09-700B S3 22.5-25.0 ft
09-800B S4 26.5-29.0 ft
09-801B S5 37.5-40.0 ft

Received 4/29/09
09-210B S2 partial section 10.5 to 11.0 ft.

3 SCOPE OF SERVICES

The scope of services was discussed by telephone, electronic correspondence and in person between Dr. DeGroot and Dr. Bill Walton, of AECOM. The final test program consisted of the following tests on selected sample tubes: full extraction, photographs and sample log, Atterberg Limits, specific gravity, organic content, grain size distribution, CRS and DSS.

4 METHODS

4.1 Sample Extraction

Selected sample tubes were cut into several sub-sections using a pipe cutter or horizontal band saw. Each sub-section was extruded, examined and logged. Selected sections were split and photographed. Water content samples were collected from selected sub-sections.

4.2 Water Content

Water contents were determined in general accordance with American Society for Testing and Materials (ASTM) Standard D2216 *Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock.*

4.3 Total and Dry Unit Weight

The total and dry unit weights of the soil were determined by using the trimmed CRS and DSS specimens. The volume of the CRS or DSS specimen/trimming ring was first measured and after trimming the wet mass of the specimen was weighed for calculation of the total unit weight. The dry unit weight was determined using the oven dry weight of the test specimen.

4.4 Atterberg Limits

Atterberg Limits were determined in general accordance with ASTM Standard D4318 *Standard Test Method for Liquid Limit, Plastic Limit and Plasticity Index of Soils.* Soil was prepared using the wet method. The as received soil was mixed with distilled water to a 15 blow count consistency using a Casagrande Cup and allowed to temper in a humid room for approximately 24 hours prior to testing. Liquid and Plastic Limit data points were determined by allowing the soil to dry at room temperature from the initial wet state to lower water contents.

4.5 Specific Gravity

Specific gravity tests were performed in general accordance with ASTM D854 *Standard Test Method for Specific Gravity of Soils*. Soil was prepared using Method A - Procedure for Moist Specimens. Approximately 50 g equivalent dry weight of test soil was dispersed in distilled water using a malt mixer and thereafter placed in a 500 mL calibrated pycnometer (in some cases less than 50 g was used because of limited sample availability). The pycnometer and soil mixture were gently boiled to remove entrapped air and then allowed to cool to room temperature. At the end of testing, the specimen's oven dry weight was measured. The specific gravity was calculated based on water at 20°C as the reference.

4.6 Organic Content

Organic content tests were conducted in general accordance with ASTM 2974 *Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils*. Specimens were prepared and tested using Test Method A with a 105°C oven to determine the moisture content and Test Method C with a muffle furnace at 440°C to determine the ash content. Data from these two methods were used to compute the organic content. In some cases less than the recommended specimen dry weight was used because of limited sample availability.

4.7 Grain Size Distribution

The grain size distribution test was conducted in general accordance with ASTM D422 *Standard Test Method for Particle-Size Analysis of Soils* using a hydrometer. The specimen was prepared for testing by allowing it to soak overnight in 125 mL of a 40 g/L sodium hexametaphosphate solution. After soaking, the specimen was dispersed using *Method A* after which the soil was transferred to a 1000 mL cylinder and mixed with distilled water. Once the test started, hydrometer readings of the soil slurry were taken for a period of approximately 24 hrs. At the conclusion of the hydrometer test, the material retained on the #200 sieve was oven dried, weighed and sieved.

4.8 Constant Rate of Strain (CRS) Consolidation

The constant rate of strain (CRS) consolidation tests were performed in general accordance with ASTM D4186 *Standard Test Method for One-Dimensional Consolidation Properties of Soils Using Controlled-Strain Loading* and Sandbeakken et al. (1986). The test was conducted using a GeoTac personal computer based test control and data acquisition system, which includes a load frame, flow pump, CRS consolidometer cell and Sigma-1 CRS consolidation software.

The general CRS test sequence consisted of the following stages:

1. Preparation of the specimen for testing first consisted of removing a test sample from a sample tube sub-section. The test specimen was hand trimmed using a soil lathe together with a sharp trimming ring and sharp trimming tools. The top and bottom surfaces of the specimen were trimmed flat with a wire saw and a long sharp edged knife with the final trimmed dimensions equaling a diameter of 2.5 in and a height of 0.75 in.
2. The specimen was placed in the CRS cell with moist top and bottom porous stones. After application of the seating load, one to three incremental loads were applied. Thereafter the cell chamber was filled with deaired water and the specimen was back pressure saturated to with a typical target back pressure equal to 4200 psf and left to sit overnight.

3. Constant rate of strain loading was typically conducted using a selected nominal strain rate between 1 to 4 %/hr (2.8×10^{-6} s^{-1} or 1.1×10^{-5} s^{-1}). An unload-reload loop was conducted during most tests and included a constant stress period prior to starting the unload phase and again prior to starting the reload phase. The target unload stress was set equal to approximately 20% of the vertical stress acting on the specimen prior to start of the unload-reload loop.
4. After the unload-reload loop was completed, CRS loading continued until a maximum stress of approximately 60,000 psf or 35% strain at which point the test was either stopped or a final unload sequence was conducted.

All measurements during testing were made using load, displacement and pressure transducers. The measured data were reduced using the methods of Wissa et al. (1971; and also described in ASTM D4186 and Sandbeakken et al. 1986). All vertical strains were computed taking into account the apparatus compliance that was determined using a steel disk. The preconsolidation stress was estimated using the Casagrande and strain energy methods (Becker et al. 1987).

4.9 Direct Simple Shear (DSS)

The Direct Simple Shear (DSS) tests were conducted using a Geonor DSS device in general accordance to the procedures described by Bjerrum and Landva (1966), DeGroot et al. (1992) and ASTM D6528 *Standard Test Method for Consolidated Undrained Direct Simple Shear Testing of Cohesive Soils.* The Geonor DSS device (Figure 4.8.1) consists of a specimen chamber, lever arm for application of consolidation weights and a gear driven thrust shaft for applying the horizontal shear stress to the specimen. Load cells and linear variable differential transformers, all connected to a dedicated data acquisition system, are used for measurement of load and displacement. Specimens are prepared for testing by trimming the soil into a 5.43 in^2 (35 cm^2) set of thin stainless steel stacked rings with an internal membrane. Carborundum porous stones with imbedded pins are placed on the top and bottom of the specimen. The membrane allows for one-dimensional consolidation during the consolidation phase of a test and direct simple shear strain mode of deformation during the shear phase of a test.

The general test sequence consisted of the following stages:

1. Preparation of the specimen for testing first consisted of removing a test sample from a sample tube sub-section followed by use of Geonor trimming equipment that allows for setting up a nominal 2.6" diameter by 0.8" height specimen inside a stacked ring membrane assembly.
2. Incremental, one-dimensional consolidation to the preshear vertical effective stress using a lever arm and dead weight system with the final preshear laboratory vertical effective stress (σ'_{vc}) set approximately equal to the estimated in situ vertical effective stress.
3. Maintaining the maximum consolidation stress acting on the specimen for a period of approximately 24 hours.
4. Undrained shearing using the constant volume procedure at a nominal shear strain rate of 5%/hour during application of the horizontal shear stress.
5. Undrained shear is continued to an approximate maximum shear strain of 20%.

All vertical consolidation strains are computed taking into account the apparatus compliance which was determined using a steel disk. The measured horizontal force during undrained shear

is corrected for the calibrated resistance of the stacked ring membrane assembly. Reduced data from the undrained shear phase of the test consists of shear strain (γ), horizontal shear stress (τ_h), equivalent change in pore water pressure (Δu), vertical effective stress (σ'_v), shear modulus ($G = \tau_h/\gamma$), and undrained shear strength (s_u) which is typically assumed to be equal to the maximum measured horizontal shear stress $(\tau_h)_{max}$ (Ladd 1991, DeGroot et al. 1992).

An undrained creep test (DSS G361) was conducted on sample 09-100B S6 (35.5 – 38.0 ft). The specimen was first consolidated to the target final vertical effective stress state. Thereafter undrained shear was performed as per the standard DSS test until the horizontal shear stress reached $0.8s_u$, with s_u = 985 psf from companion test DSS G357 on the sample, at which point the loading was switched from strain control to stress control with $\tau_h = 0.8s_u$. This shear stress was left acting on the specimen for about 3 days at which point it was increased to $0.85s_u$ and this stress was maintained until the specimen failed.

5 RESULTS

5.1 Tests Conducted, Photographs and Sample Logs

Table 1 presents a summary of the tests conducted on the sample tubes.

Figures 5.1.1 to 5.1.34 presents photographs of selected samples.

Figures 5.1.35 to 5.1.68 present the samples logs and show the location of the Atterberg Limits, grain size distribution, CRS and DSS test specimens.

5.2 Index and Classification Data

Table 2 presents water content data for the sample tubes.

Table 3 presents a summary of the Atterberg Limits results and Figure 5.2.1 plots them in a Casagrande Plasticity Chart.

Figure 5.2.2 plots results of the grain size distribution tests conducted on selected samples.

5.3 Constant Rate of Strain Tests

Table 4 presents a summary of the CRS test specimen properties and results. Figures 5.3.1 to 5.3.24 present plots of the CRS test results including the compression curve (ε_v versus σ'_v), coefficient of consolidation versus vertical effective stress, void ratio versus hydraulic conductivity and constrained modulus versus vertical effective stress.

5.4 Direct Simple Shear Tests

Table 5 presents a summary of the DSS test specimen properties and results. Figures 5.4.1 to 5.4.65 present plots of the DSS tests results including the compression curve from the consolidation phase of the test and plots from the undrained shear phase including shear stress versus shear strain, shear induced pore pressure versus shear strain, shear stress versus vertical effective stress, and shear modulus versus shear strain.

5.5 Test Specimen Photographs

Figures 5.5.1 to 5.5.6 present photographs of several DSS and CRS test specimens. The photographs are of post-test, oven dried, specimens that were split open and placed under a microscope.

6 REFERENCES

ASTM (2007). Annual Book of Standards, *Volume 04.08, Soil and Rock (1): D420 - D4914*. ASTM, Philadelphia, PA.

Becker, D.E., Crooks, J.H.A., Been, K., and Jeffries, M.G. (1987). "Work as a criterion for determining in situ and yield stresses in clays." *Canadian Geotechnical Journal*, 24(4), pp. 549-564.

Bjerrum, L. and Landva, A. (1966). "Direct Simple Shear Testing on Norwegian Quick Clays." *Geotechnique*, Vol. 16, No. 1, 1-20.

DeGroot, D.J., Ladd, C.C. and Germaine, J.T. (1992) "Direct Simple Shear Testing of Cohesive Soils." *Research Report R92-18*, Center for Scientific Excellence in Offshore Engineering, Department of Civil Engineering, MIT, Cambridge, MA, 153pp.

Ladd, C.C. (1991). "Stability evaluation during stage construction." *Journal of Geotechnical Engineering*, 117(4), 540-615.

Lunne, T., Berre, T., Andersen, K.H., Strandvik, S. and Sjursen, M. (2006). "Effects of sample disturbance and consolidation procedures on measured shear strength of soft marine Norwegian clays." *Canadian Geotechnical Journal*. 43(7), pp. 726-750.

Sandbeakken, G., Berre, T., and Lacasse, S. (1986). "Oedometer Testing at the Norwegian Geotechnical Institute." *Consolidation of Soils: Testing and Evaluation*, ASTM STP 892, pp. 329-353.

Terzaghi, K., Peck, R.B., and Mesri, G. (1996). *Soil Mechanics in Engineering Practice*. Third Edition, John Wiley & Sons, Inc., New York.

Wissa, A.E.Z., Christian, J.T., Davis, E.H. and Heiberg, S. (1971). "Consolidation at Constant Rate of Strain." *Journal of the Soil Mechanics and Foundations Division*, 97(SM10), 1393-1413.

7 NOTATION AND UNITS

Notation

c_v = vertical coefficient of consolidation (ft^2/day)
e = void ratio (-)
e_0 = initial void ratio (-)
G_s = specific gravity of solids (-)
k_v = vertical hydraulic conductivity (ft/day)
LL = liquid limit (%)
LI = liquidity index = (w – PL)/PI (-)
PI = plasticity index (%)
PL = plastic limit (%)
s_u = undrained shear strength (psf)
t = time (day, hr, min, s)
w = water content (%)

% = percentage
Δu = equivalent DSS shear induced pore pressure (psf)
ε_v = vertical strain (%)
ε_{vc} = vertical consolidation strain (%)
ε_{vf} = final vertical consolidation strain (%)
ε_{vmax} = maximum vertical strain during consolidation phase of DSS test (%)
γ = shear strain for DSS test (%)
γ_t = total unit weight (pcf)
γ_d = dry unit weight (pcf)
σ'_p = preconsolidation stress (psf)
σ'_v = vertical effective stress (psf)
σ'_{vc} = vertical consolidation effective stress (psf)
σ'_{vmax} = maximum vertical stress during consolidation phase of DSS test (psf)
σ'_{v0} = in situ vertical effective stress (psf)
τ_h = horizontal shear stress (psf)

Units

cm = centimeter
ft = feet
in = inches
min = minute
pcf = pounds per cubic foot
psf = pounds per square foot
s = seconds

Table 1 Summary of tests performed

Boring, Sample, Depth	Inspection and Sample Log	Water Content	Total and Dry Unit Weights	Atterberg Limits	Specific Gravity	Grain Size	Organic Content	Constant Rate of Strain Consolidation	Direct Simple Shear
09-100B S4 30.5-33.0 ft	complete tube	multiple							
09-100B S5 33.0-35.5 ft	complete tube	multiple	2			2 (ash)		CRS167	G356
09-100B S6 35.5-38.0 ft	complete tube	multiple	2	1	1		2		G357, G361
09-101B S5 38.0-40.5 ft	complete tube	multiple	2	1				CRS163	G350
09-103B S2 33.5-36.0 ft	partial tube (as received)	multiple	3	2 (natural and oven dry				CRS166	G354, G355
09-103B S3 36.0-38.5 ft	complete tube	multiple		1					
09-103B S4 38.5-39.0 ft	partial tube (as received)	multiple							
09-103B S5 41.0-42.0 ft	partial tube (as received)	multiple	1	1				CRS168	
09-104B S4 22.5-25.0 ft	complete tube	multiple	1	2 (natural and oven dry				CRS161	
09-108B S2 23.5-26.0 ft	complete tube	multiple	3			1 (ash)		CRS164	G351, G358
09-109B S3 25.5-28.0 ft	complete tube	multiple	1			1 (ash)			G352
09-200B S4 32.5-35.0 ft	complete tube	multiple							
09-201B S2 25.5-28.0 ft	complete tube	multiple	1	1				CRS160	
09-202B S2 28.5-31.0 ft	complete tube	multiple	1	1				CRS159	
09-206B S5 20.0-22.5 ft	tube cut in sections but not extruded	multiple							
09-207B S3 13.0-15.5 ft	complete tube	multiple							
09-210B S2 10.5-11.0 ft	partial tube (as received)	multiple		1	1				
09-301B S6 87.5-90.0 ft	complete tube	multiple	2	1		1 (ash)		CRS162	G349
09-303B S9 87.5-90.0 ft	complete tube	multiple				1 (ash)			
09-408B S6 33.0-35.5 ft	complete tube	-							
09-408B S7 35.5-38.0 ft	complete tube	multiple	2	1	1			CRS169	G359
09-500B S4 26.5-29.0 ft	complete tube	multiple		1					
09-500B S5 29.0-31.5 ft	complete tube	multiple		1	1	1 (ash)			
09-502B S4 22.0-24.5 ft	complete tube	multiple							
09-503B S2 31.0-33.5 ft	complete tube	multiple							

Boring, Sample, Depth	Inspection and Sample Log	Water Content	Total and Dry Unit Weights	Atterberg Limits	Specific Gravity	Grain Size	Organic Content	Constant Rate of Strain Consolidation	Direct Simple Shear
09-503B S3 35.5-37.5 ft	complete tube	multiple	2			1 (ash)		CRS170	G360
09-600B S2 44.0-46.5 ft	complete tube	multiple		2					
09-601B S5 49.5-52.0 ft	tube cut in sections but not extruded	multiple							
09-602B S4 54.0-56.5 ft	tube cut in sections but not extruded	multiple							
09-605B S7 47.0-49.5 ft	complete tube	multiple	2					CRS165	G353
09-605B S8 49.5-52.0 ft	tube cut in sections but not extruded	multiple							
09-700B S3 22.5-25.0 ft	complete tube	multiple							
09-800B S4 26.5-29.0 ft	complete tube	multiple							
09-801B S5 37.5-40.0 ft	complete tube	multiple							

Note: see sample logs for specific test specimen locations

Table 2 Summary of water content data

Boring, Sample, Depth	Distance from bottom of sample tube (inches)/Water Content (%)										
Distance Interval (ft)	**0-3"**	**3-6"**	**6-9"**	**9-12"**	**12-15"**	**15-18"**	**18-21"**	**21-24"**	**24-27"**	**27-30"**	
09-100B S4 30.5-33.0 ft	1"/28%		7"/31%		13"/47%		19"/42%				
09-100B S5 33.0-35.5 ft	1"/32%		7"/31%		13"/41%		19"/35%		25"/33%		
09-100B S6 35.5-38.0 ft	2"/20%		8"/18%		14"/25%		20"/21"		26"/85%	27"/96%	
09-101B S5 38.0-40.5 ft						15"/25%	21"/29%	21"/35%			
09-103B S2 33.5-36.0 ft			7"/51%		13"/29%						
09-103B S3 36.0-38.5 ft	1"/21%			10"/25%		16"/27%					
09-103B S4 38.5-39.0 ft							19"/26%			29"/19%	
09-103B S5 41.0-42.0 ft					14"/27%				26"/25%	29.5"/20%	
09-104B S4 22.5-25.0 ft	1"/23%		8"/41%	9"/118% 10"/34%	14"/78%						
09-108B S2 23.5-26.0 ft	1"/18%				12"/32%	18"/32%	19"/38% 20"/29%				
09-109B S3 25.5-28.0 ft	2"/107%		6"/32%		12"/51%		18"/34%				
09-200B S4 32.5-35.0 ft			8"/19%		14"/19%		20"/25%			28"/29%	
09-201B S2 25.5-28.0 ft						15"/20%		21"/18%			
09-202B S2 28.5-31.0 ft							20"/16%	23"/20%			
09-206B S5 20.0-22.5 ft		6"/21%				18"/21%				28.5"/23%	
09-207B S3 13.0-15.5 ft		4"/29%	8"/17%		14"/19%		20"/28%				
09-210B S2 10.5-11.0 ft	10.9'/23%	10.5'/38%									
09-301B S6 87.5-90.0 ft	1"/18%		8"/22%	10"/23%			20"/33%				
09-303B S9 87.5-90.0 ft	2"/21%			9"/23%		17"/36% 17.5"/34%		22"/30%	27"/29%		
09-408B S6 33.0-35.5 ft											
09-408B S7 35.5-38.0 ft	1"/31%		7"/31%		13"/44%		19"/36%		25"/31%		
09-500B S4 26.5-29.0 ft							19"/31%	22"/45%	24"/38%		
09-500B S5 29.0-31.5 ft					12"/34%	16.5"/48%	18"/31%				

Boring, Sample, Depth	Distance from bottom of sample tube (inches)/Water Content (%)										
Distance Interval (ft)	**0-3"**	**3-6"**	**6-9"**	**9-12"**	**12-15"**	**15-18"**	**18-21"**	**21-24"**	**24-27"**	**27-30"**	
09-502B S4 22.0-24.5 ft			6"/18%		12"/18%		18"/16%				
09-503B S2 31.0-33.5 ft	1"/31%		7"/31%		13"/31%						
09-503B S3 35.5-37.5 ft					12"/40%		18'/30%				
09-600B S2 44.0-46.5 ft			6"/22%			18"/20%		24"/23%		28"/26% 29.5"/48%	
09-601B S5 49.5-52.0 ft			8"/21%		14"/22%		20"/22%				
09-602B S4 54.0-56.5 ft		4"/24%		10"/22%				22"/22%			
09-605B S7 47.0-49.5 ft	1"/23%	6"/25%		12"/31%		17.5"/41%	18"/28%				
09-605B S8 49.5-52.0 ft	1.5"/20%		8"/21%		14"/21%		20"/21%				
09-700B S3 22.5-25.0 ft			6"/33%		12"/34%		18"/34%		24"/26%		
09-800B S4 26.5-29.0 ft			6"/22%		12"/21%		18"/21%	24"/33%			
09-801B S5 37.5-40.0 ft			6"/30%		12"/29%		18"/28%				

Notes: see also sample logs for test specimen locations

Table 3 Summary of classification test results

Boring, Sample, Depth	CRS/DSS Test	LL	PL	PI	w	LI	Void Ratio (e) DSS Specimen		Organic Matter	G_s
-	-	(%)	(%)	(%)	(%)	(-)	G_s = 2.70	G_s = 2.30	(%)	(-)
1	2	3	4	5	6	7	8	9	10	11
09-100B S4 30.5-33.0 ft										
09-100B S5 33.0-35.5 ft	G356 & CRS167				58		1.885	1.458		
09-100B S6 35.5-38.0 ft	G357	57	40	17	119	4.65	3.422	2.767	3.5 & 6.0	2.20†
09-101B S5 38.0-40.5 ft	G350 & CRS163	34	22	12	23	0.08	0.680			
09-103B S2 33.5-36.0 ft	G354 & CRS166, G355	45	35	10	105	7.00	3.122	2.511		
	oven dry sample	34	30	4						
09-103B S3 36.0-38.5 ft		27	21	6	27	1.00				
09-103B S4 38.5-39.0 ft										
09-103B S5 41.0-42.0 ft	CRS168	24	20	4	26	1.50				
09-104B S4 22.5-25.0 ft	CRS161	47	39	8	88	6.13	2.686	2.140		
	oven dry sample	43	36	6						
09-108B S2 23.5-26.0 ft	G351 & CRS164, G358				76		2.168	1.698		
09-109B S3 25.5-28.0 ft	G352				87		2.645	2.105		
09-200B S4 32.5-35.0 ft										
09-201B S2 25.5-28.0 ft	CRS160	29	20	9	20	0.00				
09-202B S2 28.5-31.0 ft	CRS159	25	17	8	19	0.25				
09-206B S5 20.0-22.5 ft										
09-207B S3 13.0-15.5 ft										
09-210B S2 10.5-11.0 ft		34	21	13	23	0.15				2.66
09-301B S6 87.5-90.0 ft	G349 & CRS162	32	19	13	22	0.23	0.638			
09-303B S9 87.5-90.0 ft										
09-408B S6 33.0-35.5 ft										
09-408B S7 35.5-38.0 ft	G359 & CRS169	30	NP		68		2.028	1.580		2.28†

Boring, Sample, Depth	CRS/DSS Test	LL	PL	PI	w	LI	Void Ratio (e) DSS Specimen		Organic Matter	G_s
-	-	(%)	(%)	(%)	(%)	(-)	G_s = 2.70	G_s = 2.30	(%)	(-)
1	2	3	4	5	6	7	8	9	10	11
09-500B S4 26.5-29.0 ft		27	NP							
09-500B S5 29.0-31.5 ft		58	31	27	34	0.11				2.58
09-502B S4 22.0-24.5 ft										
09-503B S2 31.0-33.5 ft										
09-503B S3 35.5-37.5 ft	G360 & CRS170				83		2.674	2.129		
09-600B S2 44.0-46.5 ft	at 1.5'	22	16	6	20	0.67				
	at 2.3'	24	20	4	26	1.50				
09-601B S5 49.5-52.0 ft										
09-602B S4 54.0-56.5 ft										
09-605B S7 47.0-49.5 ft	G353 & CRS165				49		1.437	1.076		
09-605B S8 49.5-52.0 ft										
09-700B S3 22.5-25.0 ft										
09-800B S4 26.5-29.0 ft										
09-801B S5 37.5-40.0 ft										

Notes:
1. See sample logs for specific test specimen locations.
2. Water content and void ratio values from adjacent DSS test specimen or CRS test specimen or water content sample if adjacent DSS or CRS test specimen not available (see Tables 4 and 5)
3.† Specific gravity test specimen contained some floating (ash) particles.

Table 4 Summary of Constant Rate of Strain (CRS) specimen properties and test results

Test #	Boring Sample	Depth	w	e_0	γ_t	γ_d	σ'_{v0}	Sample Quality (at σ'_{v0})		σ'_p		OCR		Figure Numbers
								$\Delta e/e_0$	ε_v	Casa.	S.E.	Casa.	S.E.	
-	-	ft	%	-	pcf	pcf	psf	-	%	psf	psf	-	-	-
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
CRS159	09-202B S2	28.5-31.0	19	0.579	128	107	1276	0.000	0.0	9525†	9815	7.33	7.55	5.3.1-5.3.2
CRS160	09-201B S2	25.5-28.0	20	0.605	127	105	1359	0.022	0.8	6935†	7100	5.10	5.22	5.3.3-5.3.4
CRS161	09-104B S4	22.5-25.0	88	2.713	86	46	1584	0.019	1.4	4760	5160	3.00	3.26	5.3.5-5.3.6
CRS162	09-301B S6	87.5-90.0	25	0.714	123	99	4236	0.092	3.8	4805†	7310	1.13	1.73	5.3.7-5.3.8
CRS163	09-101B S5	38.0-40.5	24	0.703	123	99	2136	0.022	0.9	10445†	12530	4.89	5.87	5.3.9-5.3.10
CRS164	09-108B S2	23.5-26.0	39	1.150	110	79	1577	0.032	1.7	2860	2990	1.81	1.90	5.3.11-5.3.12
CRS165	09-605B S7	47.0-49.5	31	0.942	114	87	2356	0.003	0.1	4760	4990	2.02	2.12	5.3.13-5.3.14
CRS166	09-103B S2	33.5-36.0	108	3.080	86	42	1935	0.012	0.9	3615	3780	1.87	1.95	5.3.15-5.3.16
CRS167	09-100B S5	33.0-35.5	67	2.189	88	53	1958	0.022	1.5	4929	5305	2.52	2.71	5.3.17-5.3.18
CRS168	09-103 S5	41.0-43.5	23	0.650	126	103	2221	0.034	1.3	-†	-†	-	-	5.3.19-5.3.20
CRS169	09-408B S7	35.5-38.0	46	1.434	101	70	1707	0.000	0.0	7205	6892	4.22	4.04	5.3.21-5.3.22
CRS170	09-503B S3	35.5-38.0	131	3.781	82	35	3923	0.014	1.1	7519	7519	1.92	1.92	5.3.23-5.3.24

Notes:

†no distinct break evident in compression curve

1. Notation given in Section 7
2. See sample logs for specific test specimen locations.
3. e_0 based on assumed specific gravity = 2.72
4. σ'_{v0} values from AECOM
5. For estimates of σ'_p: Casa. = Casagrande construction; S.E. = strain energy method of Becker et al. (1987)

6. Sample quality evaluation methods:

Lunne et al. (2006) quality ratings 1 = very good to excellent, 2 = fair to good, 3 = poor, 4 = very poor					Terzaghi et al. (1996) Specimen Quality Designation (SQD) A (best) to E (worst)					
OCR	$\Delta e/e_0$ at σ'_{v0}									
1 to 2	< 0.04	0.04 – 0.07	0.07 – 0.14	> 0.14	ε_v at σ'_{v0}	< 1	1 - 2	2 - 4	4 - 8	> 8
2 to 4	< 0.03	0.03 – 0.05	0.05 – 0.10	> 0.10	SQD	A	B	C	D	E
Quality	1	2	3	4						

Table 5 Summary of Direct Simple Shear (DSS) specimen properties and test results

Test No.	Boring Sample	Depth	w_n	γ_t	γ_d	σ'_{vo}	Laboratory Consolidation					Shear Data at Peak $(\tau_h)_{max} = s_u$				Figure Numbers
							σ'_{vc}	σ'_{vmax}	OCR	ε_{vmax}	ε_{vf}	γ	τ_h	τ_h/σ'_{vc}	σ'_v/σ'_{vc}	
-	-	ft	%	pcf	pcf	psf	psf	psf	-	%	%	%	psf	-	-	-
1	2	3	4	5	5	6	7	8	9	10	11	12	13	14	15	16
G349	09-301B S6	87.5-90.0	22	126	103	4236	4174	4174	-	5.0	5.0	10.8	1190	0.285	0.484	5.4.1-5.4.5
G350	09-101B S5	38.0-40.5	23	123	100	2136	2103	4220	-	3.0	2.9	13.0	1840	0.874	1.409	5.4.6-5.4.10
G351	09-108B S2	23.5-26.0	76	93	53	1577	1591	2053	-	3.2	3.1	5.6	690	0.434	0.698	5.4.11-5.4.15
G352	09-109B S3	25.5-28.0	87	86	46	1483	1505	1891	-	3.9	3.9	4.8	599	0.398	0.625	5.4.16-5.4.20
G353	09-605B S7	47.0-49.5	49	102	69	2356	2370	2904	-	3.8	3.8	5.2	805	0.340	0.624	5.4.21-5.4.25
G354	09-103B S2	33.5-36.0	105	84	41	1935	1927	2580	-	3.6	3.4	5.1	779	0.404	0.672	5.4.26-5.4.30
G355	09-103B S2	33.5-36.0	67	92	55	1935	1939	2592	-	4.2	4.1	5.7	684	0.353	0.652	5.4.31-5.4.35
G356	09-100B S5	33.0-35.5	58	92	58	1958	1971	3436	-	3.2	3.1	12.2	1060	0.538	0.872	5.4.36-5.4.40
G357	09-100B S6	35.5-38.0	119	83	38	2020	2024	3398	-	4.6	4.4	4.0	985	0.487	0.864	5.4.41-5.4.45
G358	09-108B S2	23.5-26.0	104	84	41	1577	8288	8281	1.0†	15.1	15.1	4.0	1967	0.237	0.520	5.4.46-5.4.50
G359	09-408B S7	35.5-38.0	68	93	56	1707	1692	4193	-	2.4	2.3	5.1	1106	0.654	1.065	5.4.51-5.4.55
G360	09-503B S3	35.5-38.0	83	83	46	3923	3937	4177	-	3.5	3.5	4.9	1249	0.317	0.563	5.4.56-5.4.60
G361	09-100B S6	35.5-38.0		95		2020	2022	3442	-	4.0	3.8	creep test				5.4.61-5.4.65

Notes: 1. Notation given in Section 7. 2. See sample logs for specific test specimen locations. 3. σ'_{v0} values provided by AECOM
4. Undrained shear strength $s_u(DSS) = (\tau_h)_{max}$ 5. †SHANSEP OCR = 1 test.

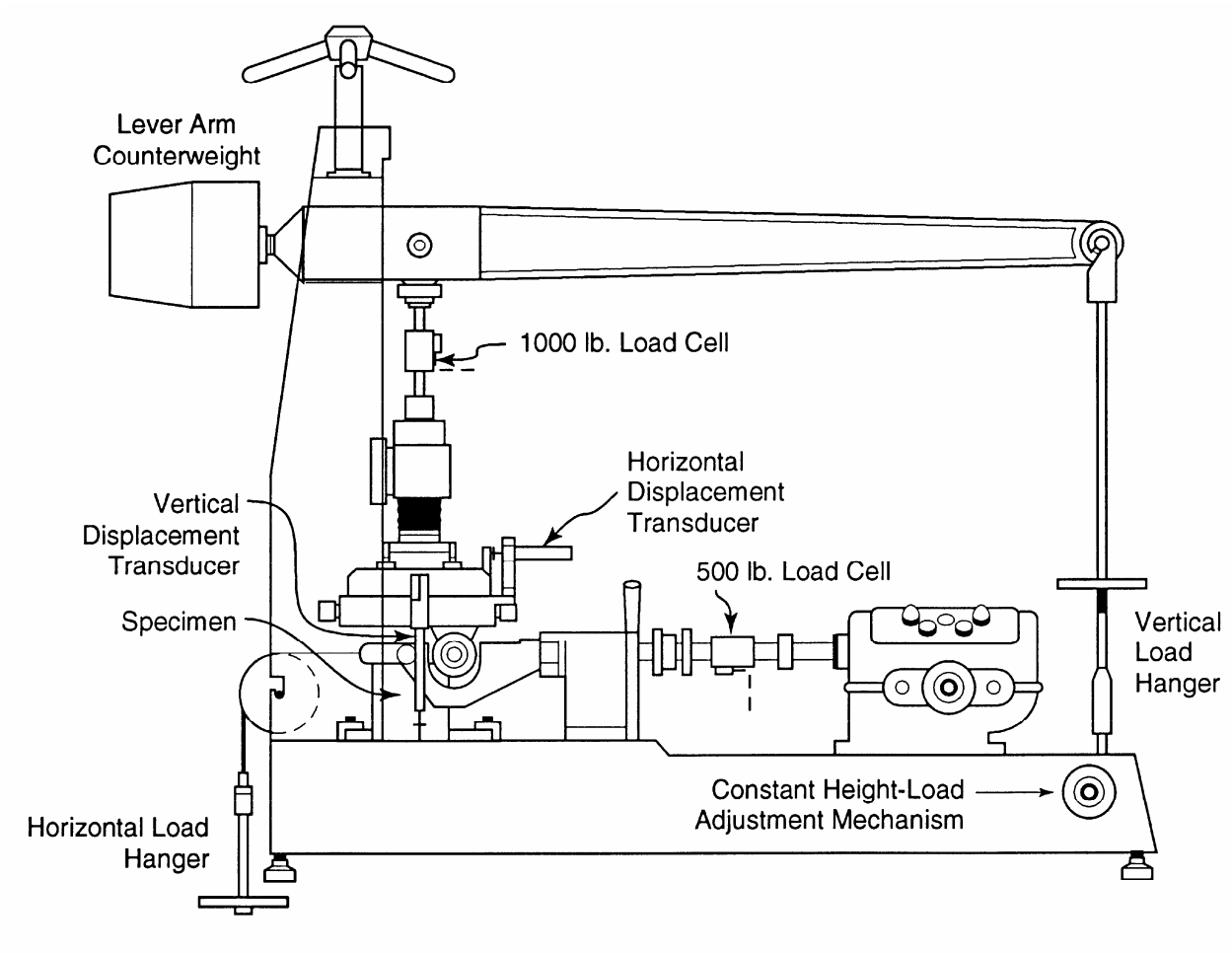


Figure 4.8.1 Schematic of the Geonor Direct Simple Shear Apparatus.

Figure 5.1.1 Photograph of Sample 09-100B S4 30.5 to 33.0 ft.

Figure 5.1.2 Photograph 1 of Sample 09-100B S5 33.0 to 35.5 ft.

Figure 5.1.3 Photograph 2 of Sample 09-100B S5 33.0 to 35.5 ft.

Figure 5.1.4 Photograph of Sample 09-100B S6 35.5 to 38.0 ft.

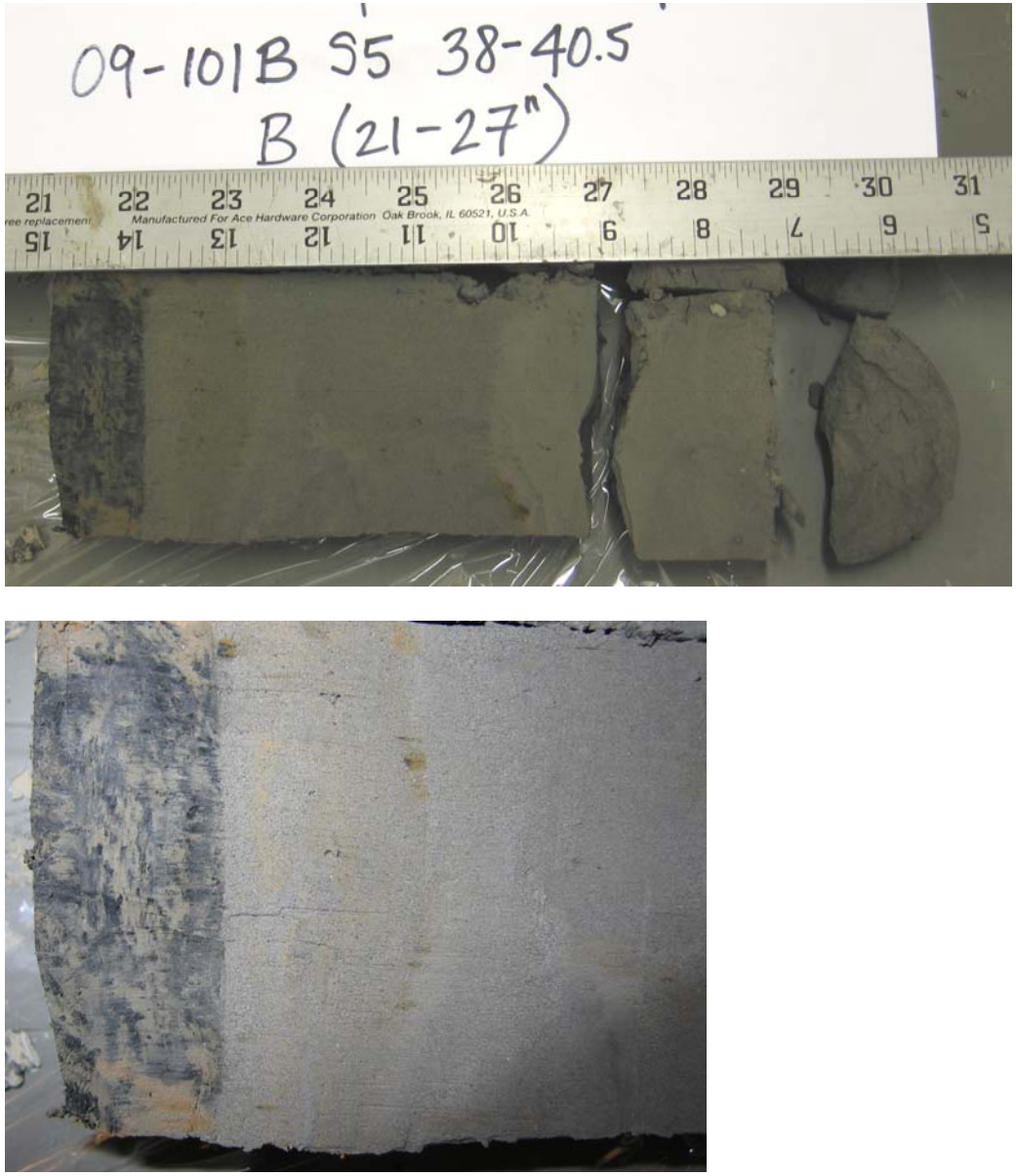


Figure 5.1.5 Photographs of Sample 09-101B S5 38.0 to 40.5 ft.

Figure 5.1.6 Photograph of Sample 09-103B S2 33.5 to 36.0 ft – as received 2 six inch sections.

Figure 5.1.7 Photograph of Sample 09-103B S3 36.0 to 38.5 ft.

Figure 5.1.8 Photograph of Sample 09-103B S4 38.5 to 41.0 ft – as received 10inch section.

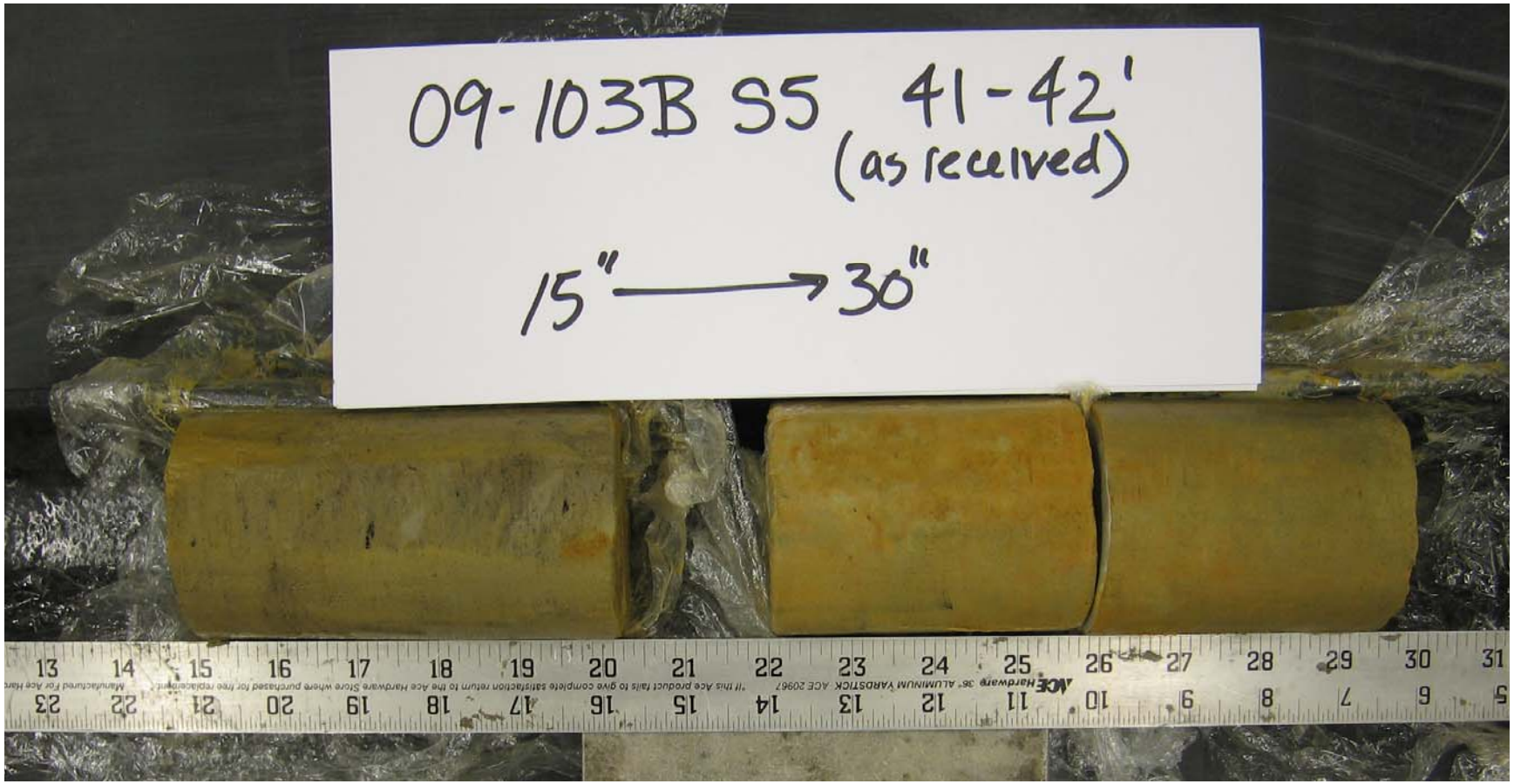


Figure 5.1.9 Photograph of Sample 09-103B S5 41.0 to 42.0 ft – as received section.

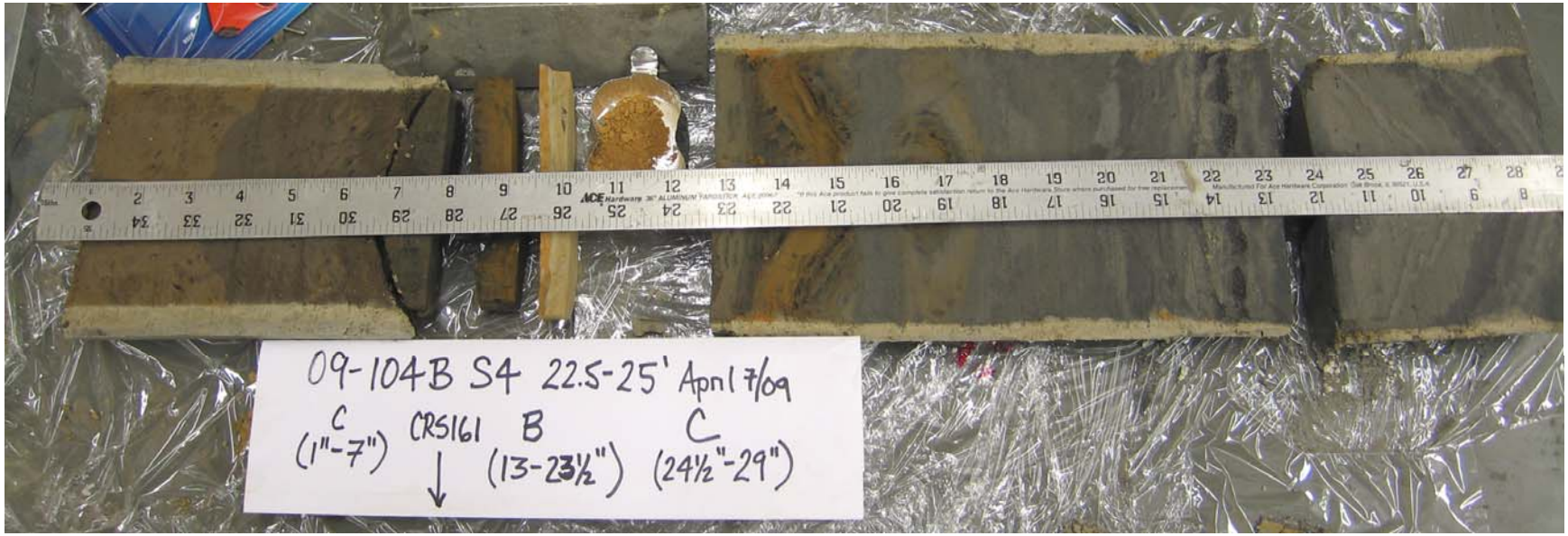


Figure 5.1.10 Photograph of Sample 09-104B S4 22.5 to 25.0 ft.

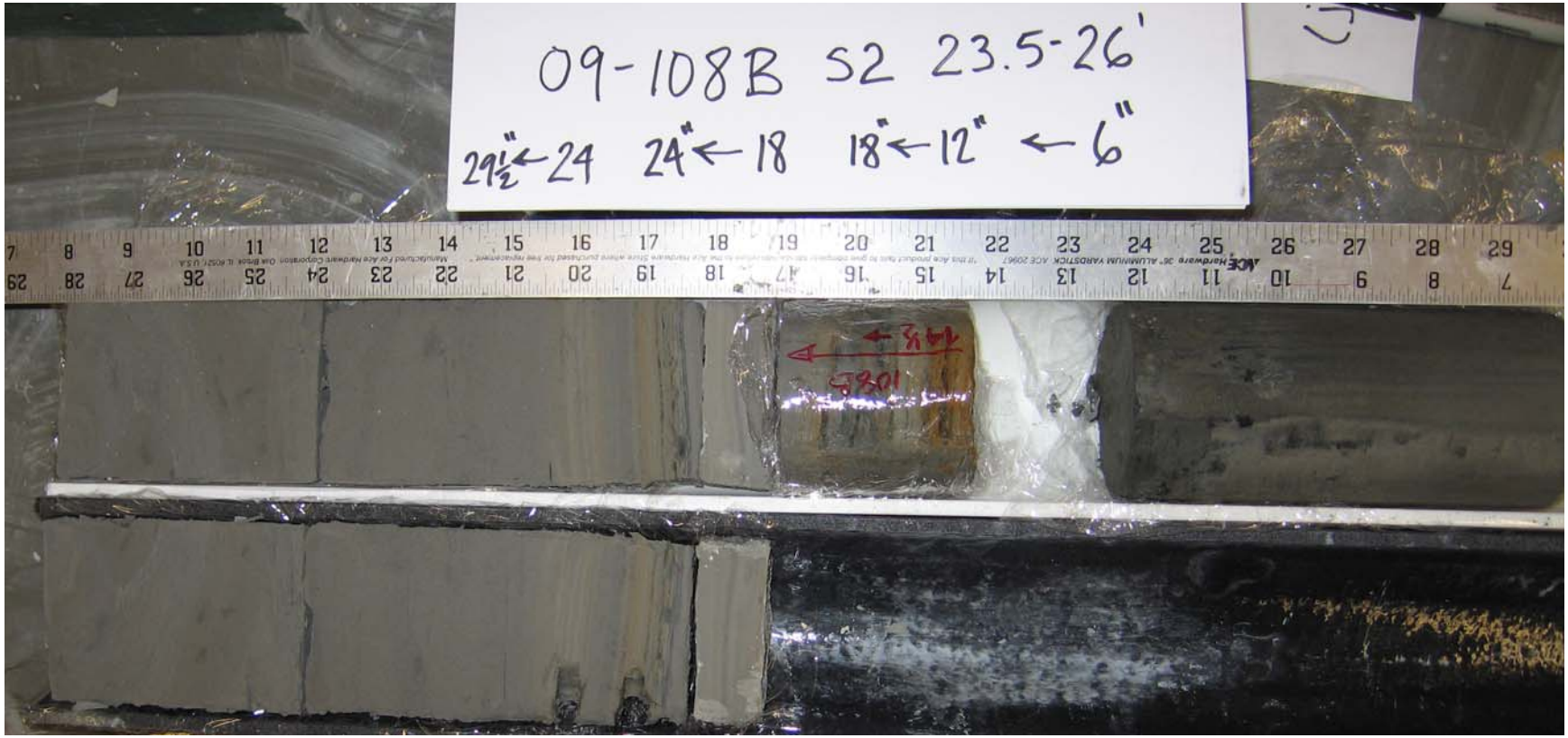


Figure 5.1.11 Photograph 1 of Sample 09-108B S2 23.5 to 26.0 ft.

Figure 5.1.12 Photograph 2 of Sample 09-108B S2 23.5 to 26.0 ft – section from 12 to 18 inches (right to left).

Figure 5.1.13 Photograph of Sample 09-109B S2 25.5 to 28.0 ft.

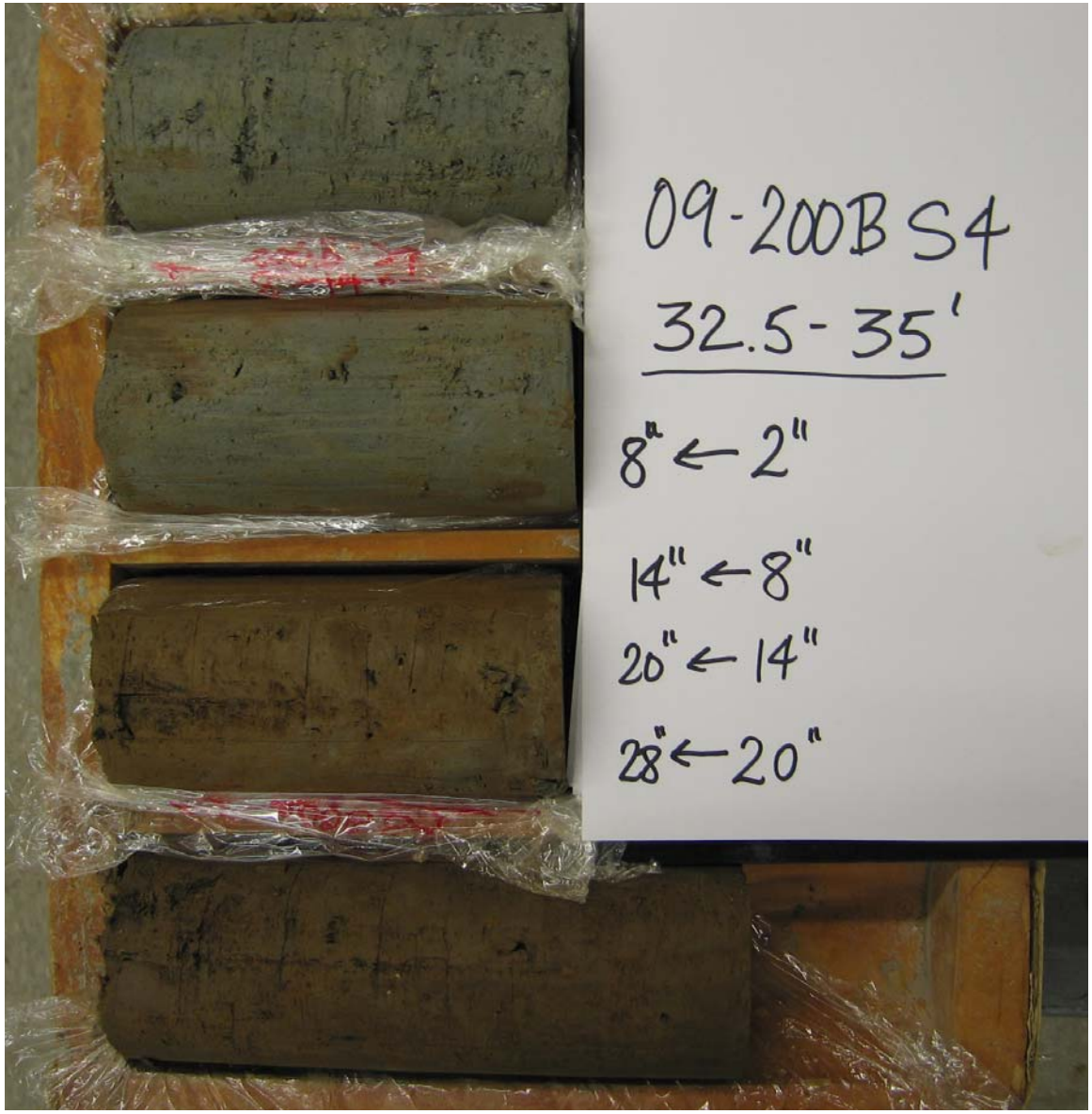


Figure 5.1.14 Photograph of Sample 09-200B S4 32.5 to 35.0 ft.

Figure 5.1.15 Photograph 1 of Sample 09-201B S2 25.5 to 28.0 ft.

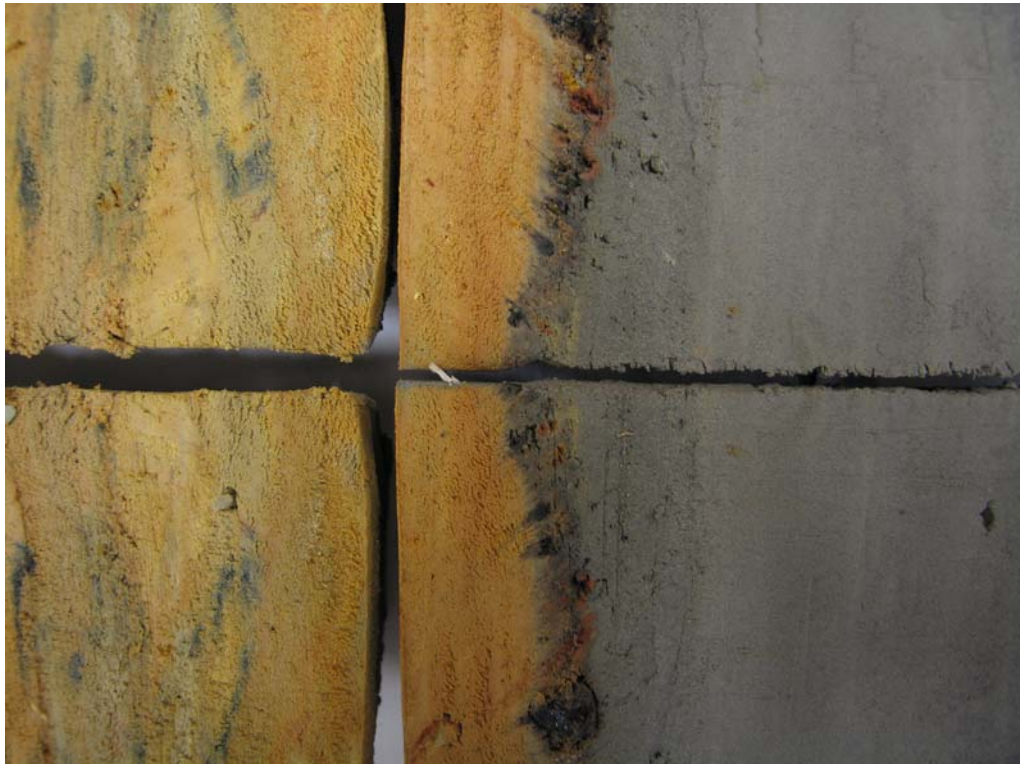


Figure 5.1.16 Photograph 2 of Sample 09-201B S2 25.5 to 28.0 ft.

Figure 5.1.17 Photographs of Sample 09-202B S2 28.5 to 31.0 ft.

Figure 5.1.18 Photograph of Sample 09-207B S3 13.0 to 15.5 ft.

Figure 5.1.19 Photograph of Sample 09-210B S2 – as received 10.5 to 11.0 ft section.

Figure 5.1.20 Photograph of Sample 09-301B S6 87.5 to 90.0 ft.

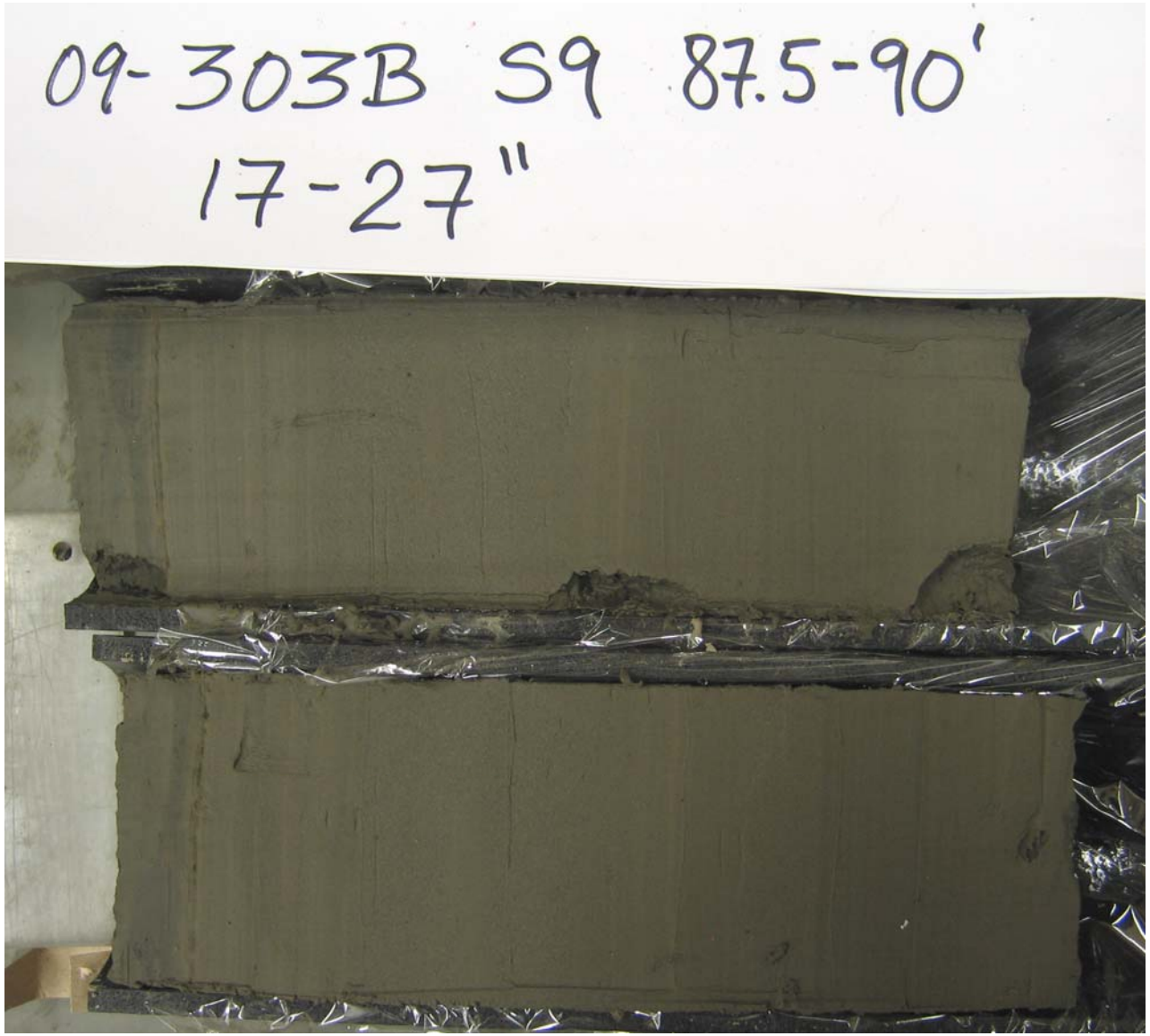


Figure 5.1.21 Photograph of Sample 09-303B S9 87.5 to 90.0 ft.

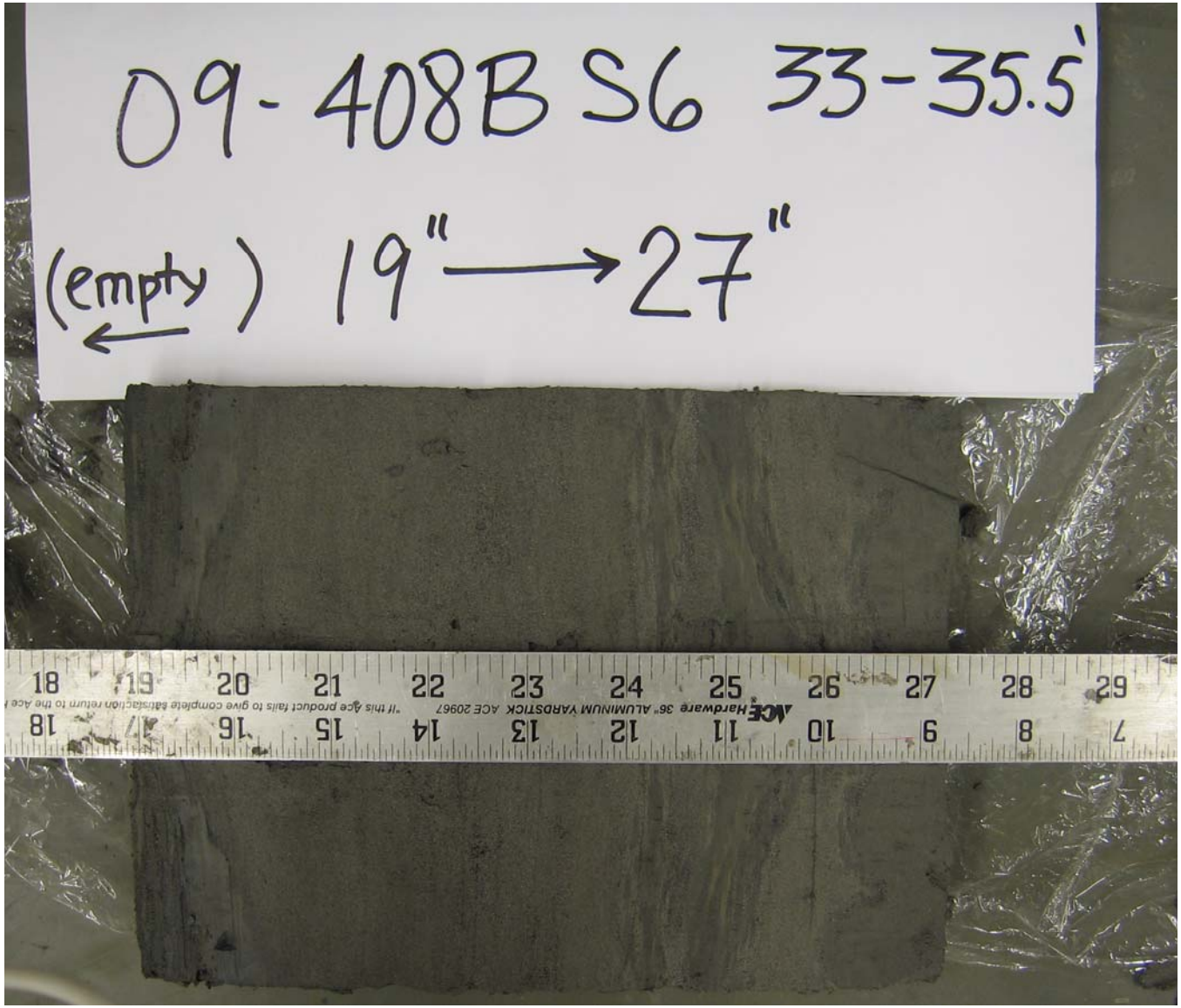


Figure 5.1.22 Photograph of Sample 09-408B S6 33.0 to 35.5 ft.

Figure 5.1.23 Photograph of Sample 09-408B S7 35.5 to 38.0 ft.

Figure 5.1.24 Photograph of Sample 09-500B S4 26.5 to 29.0 ft.

Figure 5.1.25 Photograph of Sample 09-500B S5 29.0 to 31.5 ft.

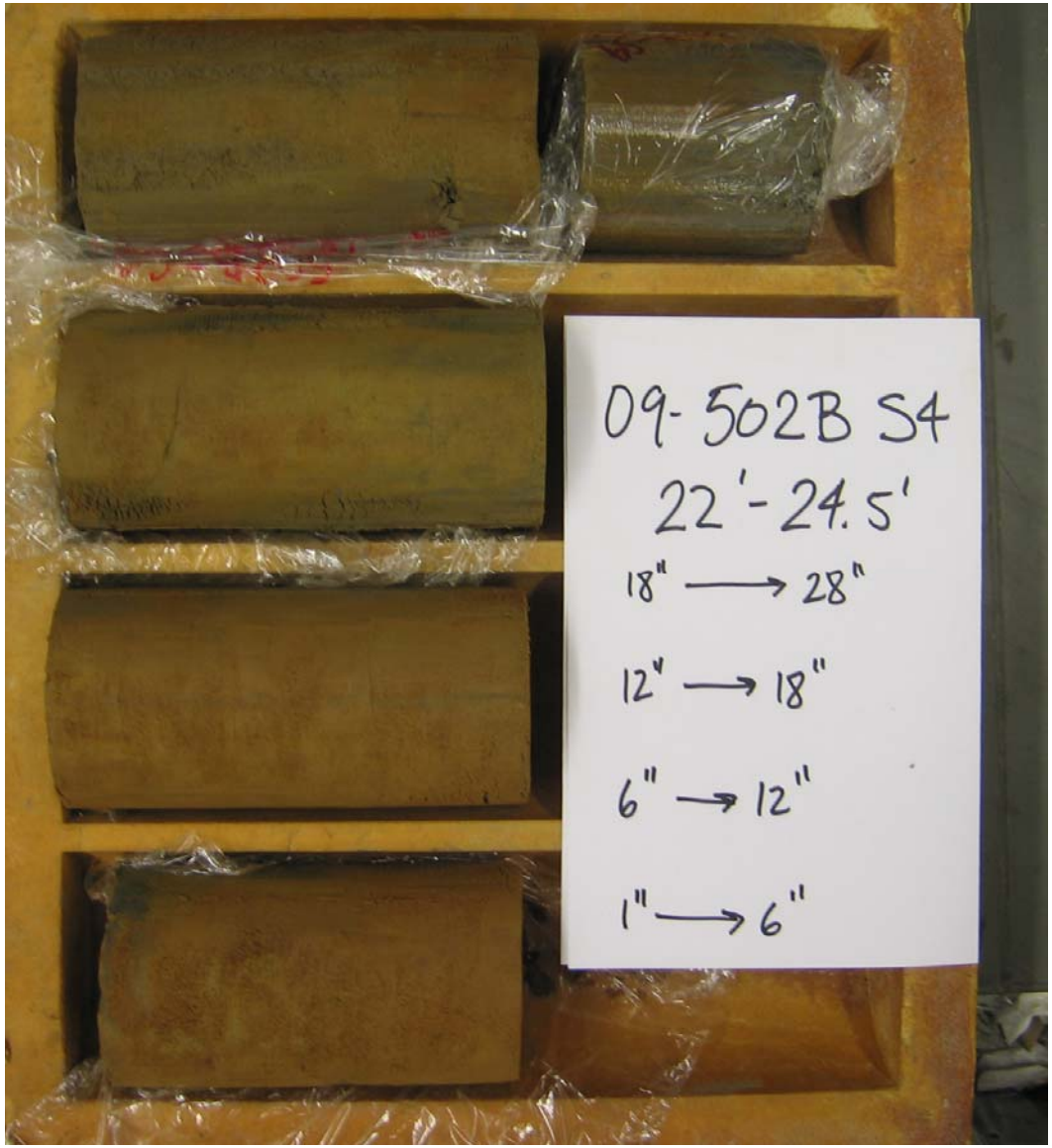


Figure 5.1.26 Photograph of Sample 09-502B S4 22.0 to 24.5 ft.

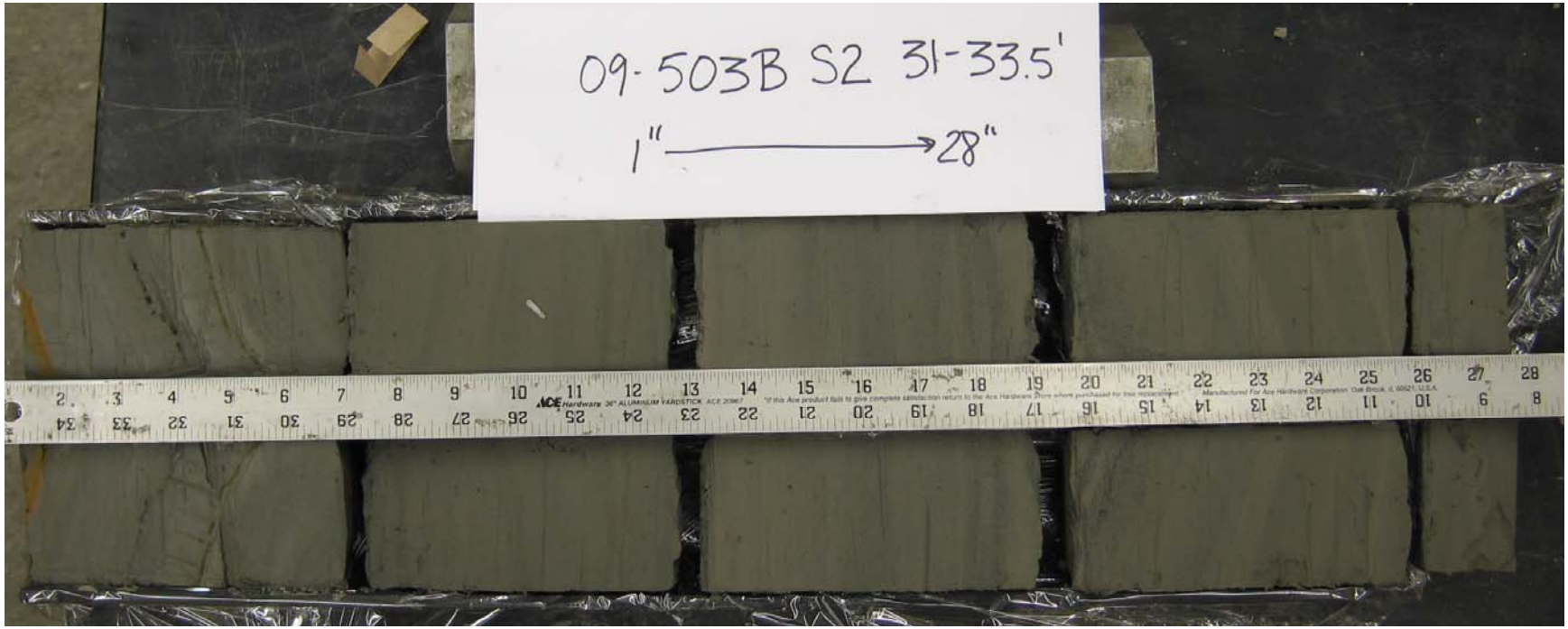


Figure 5.1.27 Photograph of Sample 09-503B S2 31.0 to 33.5 ft.

Figure 5.1.28 Photograph of Sample 09-503B S3 33.5 to 38.0ft.

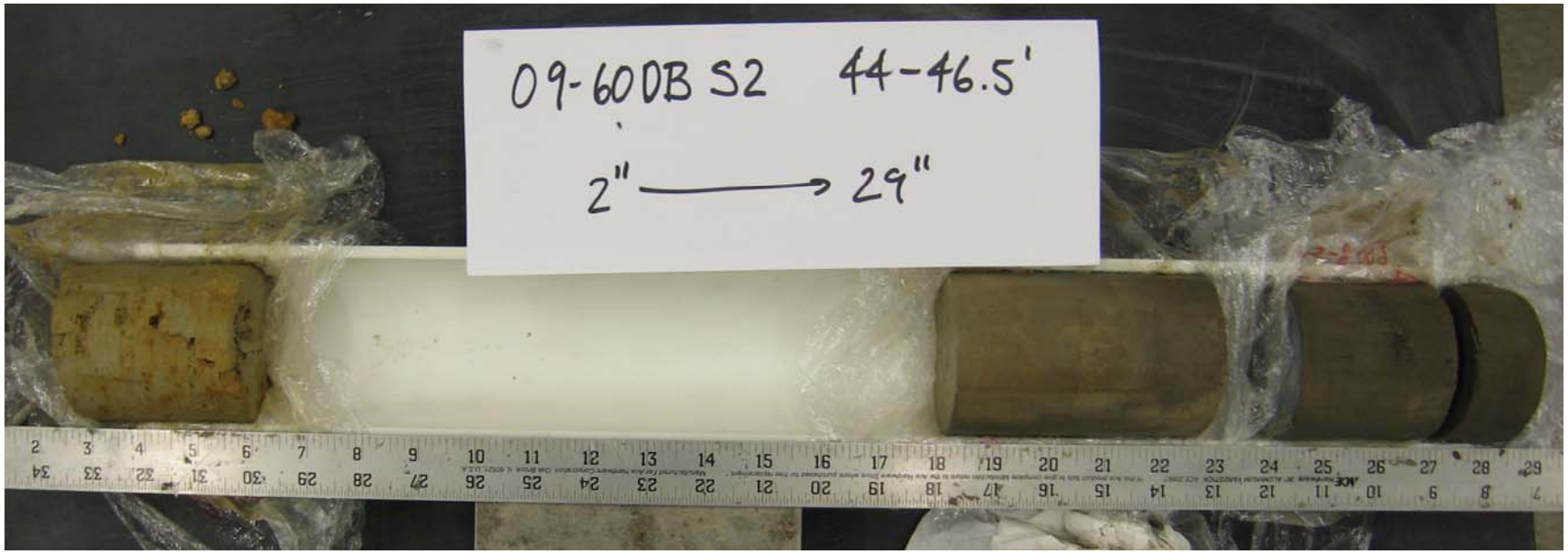


Figure 5.1.29 Photograph of Sample 09-600B S2 44.0 to 46.5 ft.

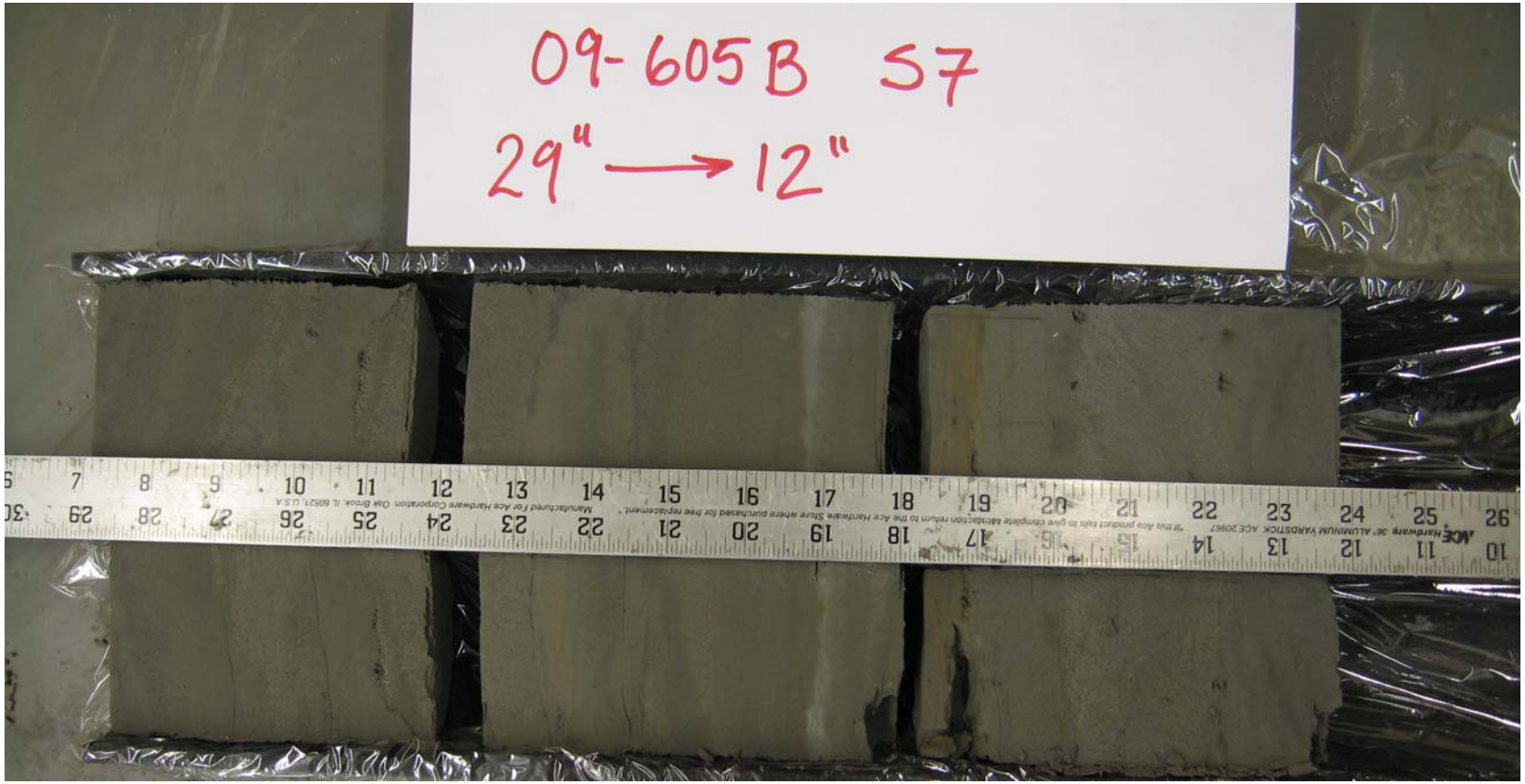


Figure 5.1.30 Photograph 1of Sample 09-605B S7 47.0 to 49.5 ft – 12 to 29 inch section (right to left).

Figure 5.1.31 Photograph 2 of Sample 09-605B S7 47.0 to 49.5 ft – 1 to 12 inch section (left to right).

Figure 5.1.32 Photograph of Sample 09-700B S3 22.5 to 25.0 ft.

Figure 5.1.33 Photograph of Sample 09-800B S4 26.5 to 29.0 ft.

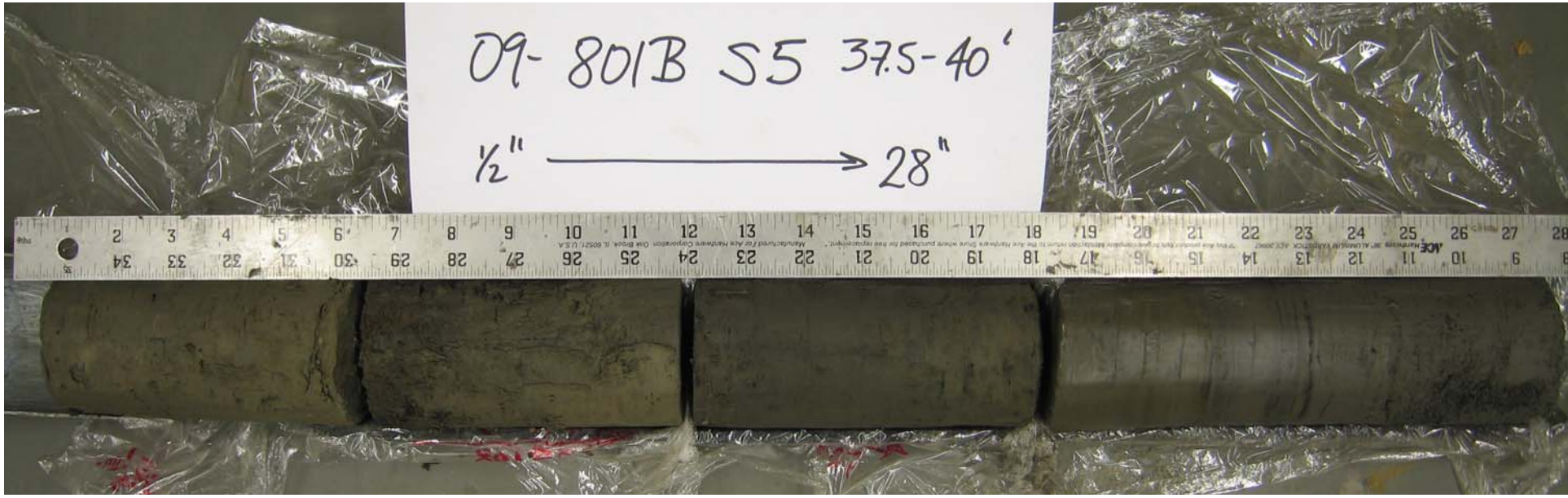


Figure 5.1.34 Photograph of Sample 09-801B S5 37.5 to 40.0 ft.

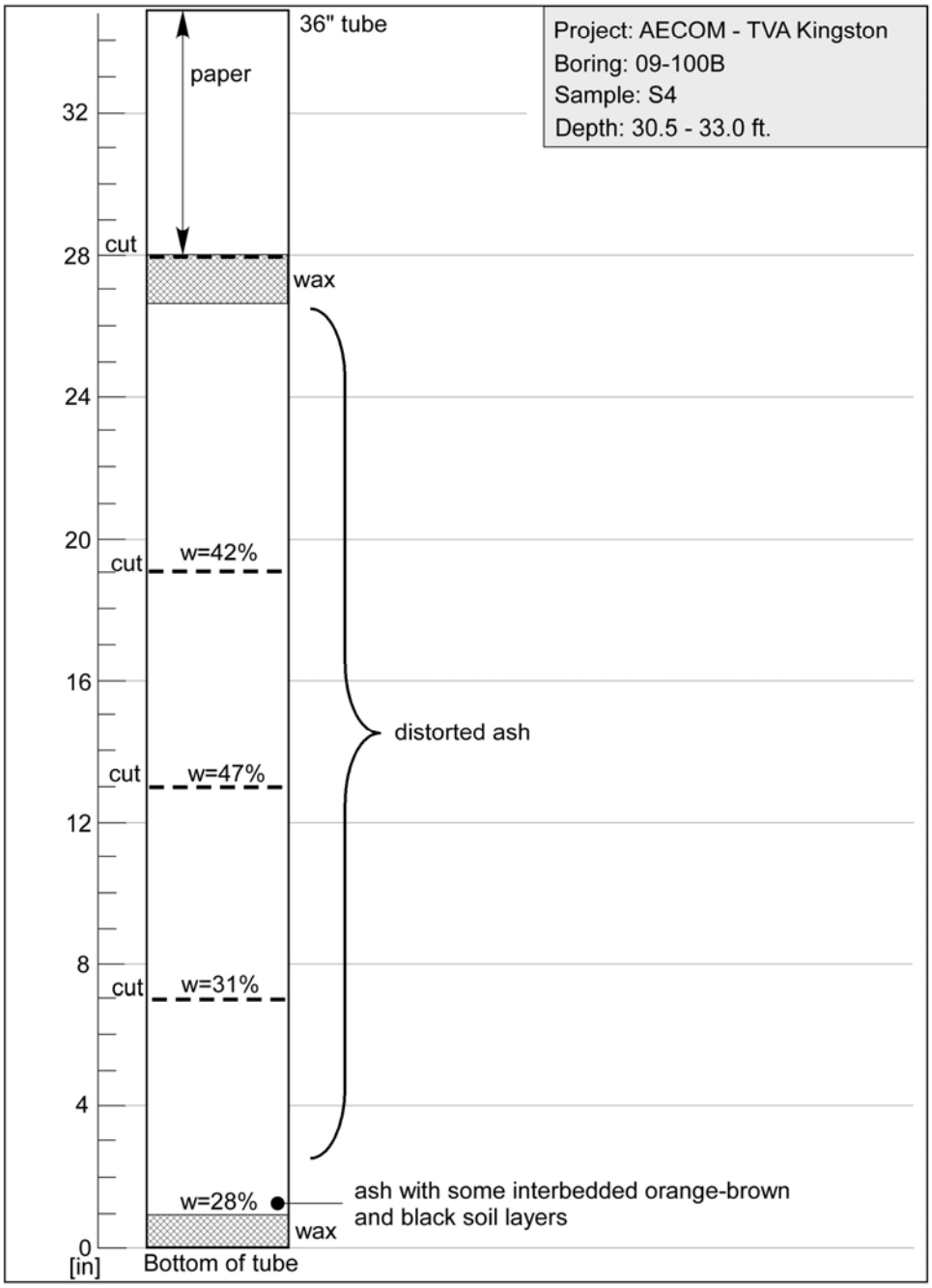


Figure 5.1.35 Sample log and test locations for Sample 09-100B S4 30.5 to 33.0 ft.

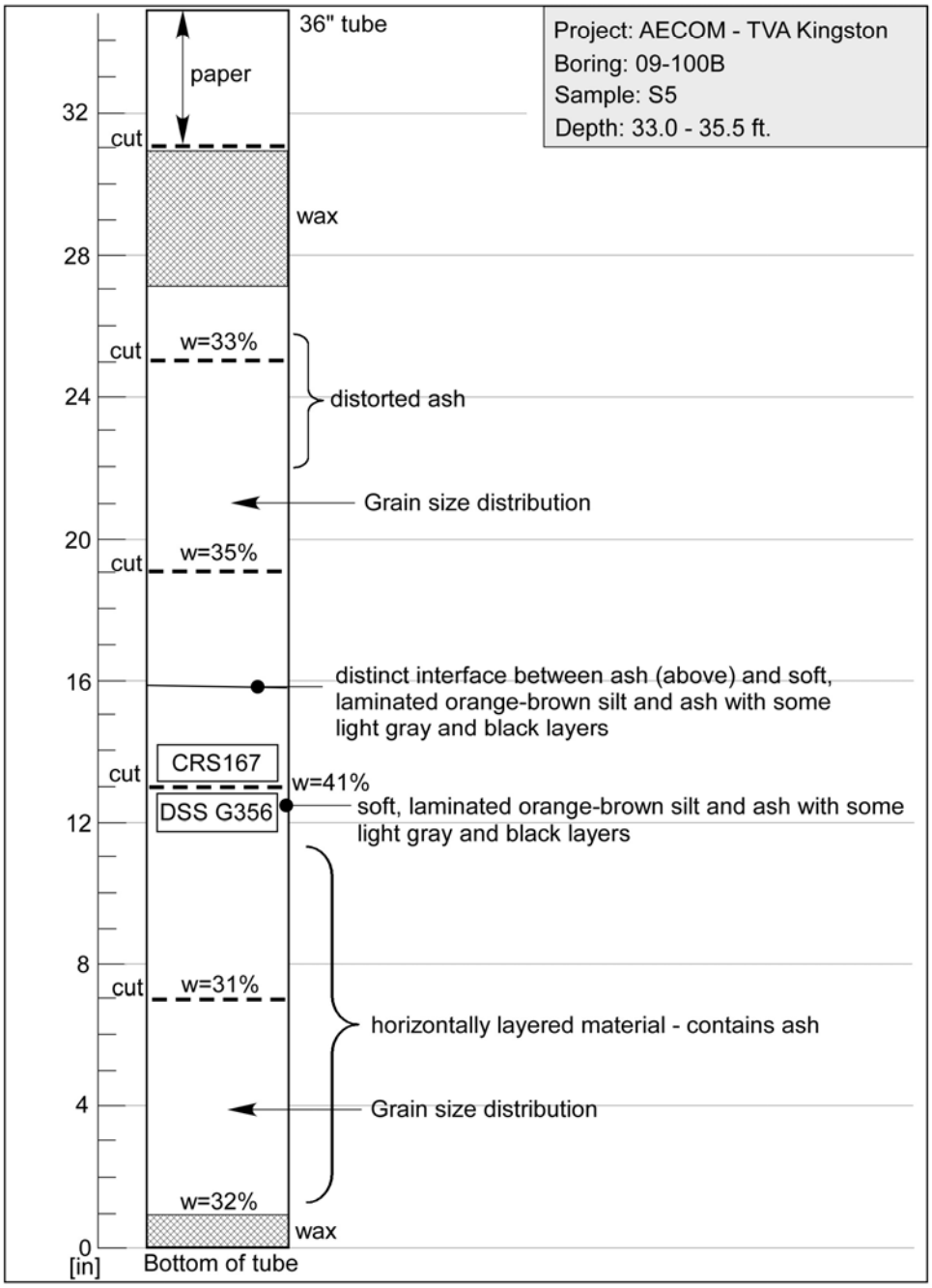


Figure 5.1.36 Sample log and test locations for Sample 09-100B S5 33.0 to 35.5 ft.

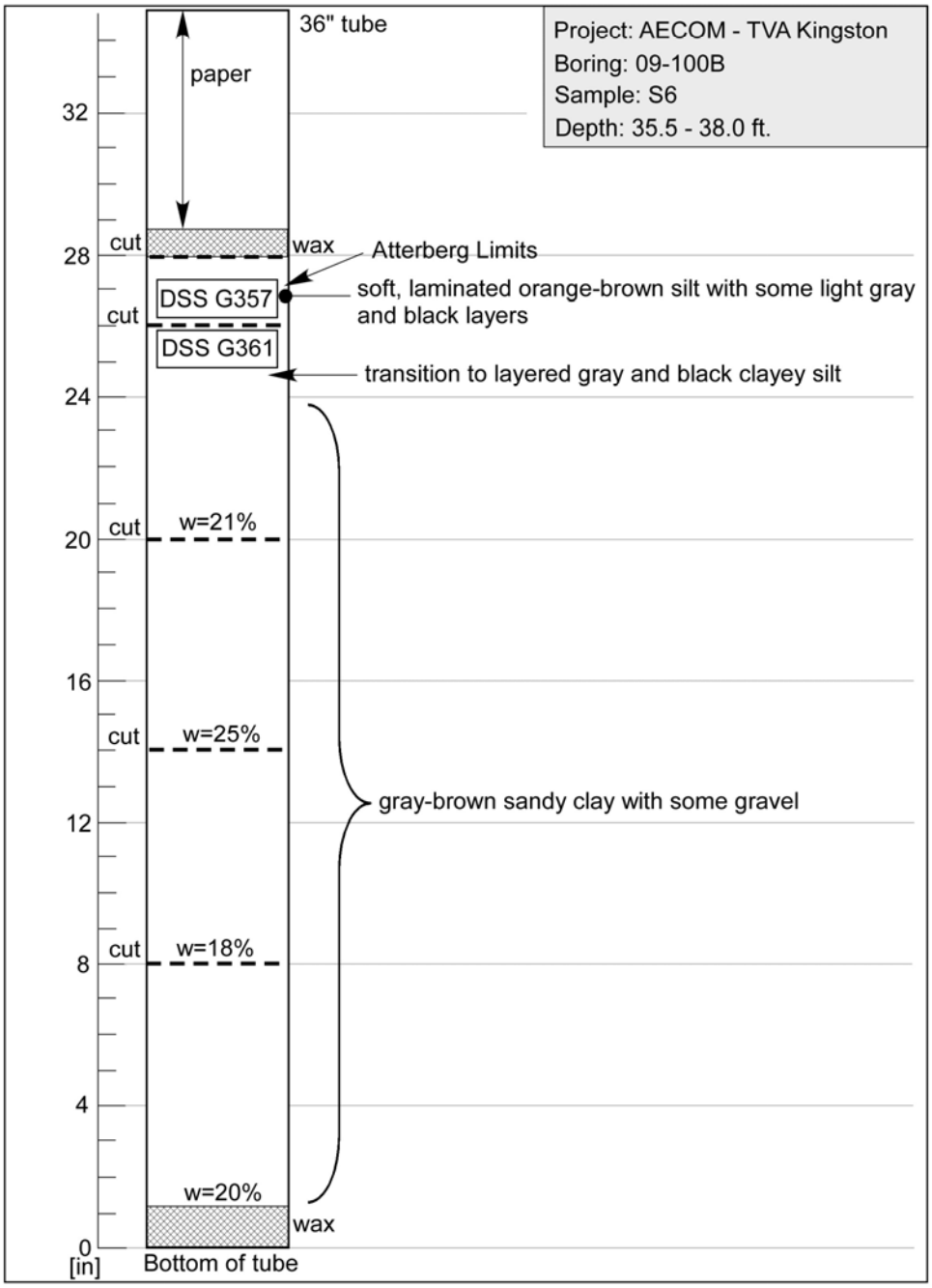


Figure 5.1.37 Sample log and test locations for Sample 09-100B S6 35.5 to 38.0 ft.

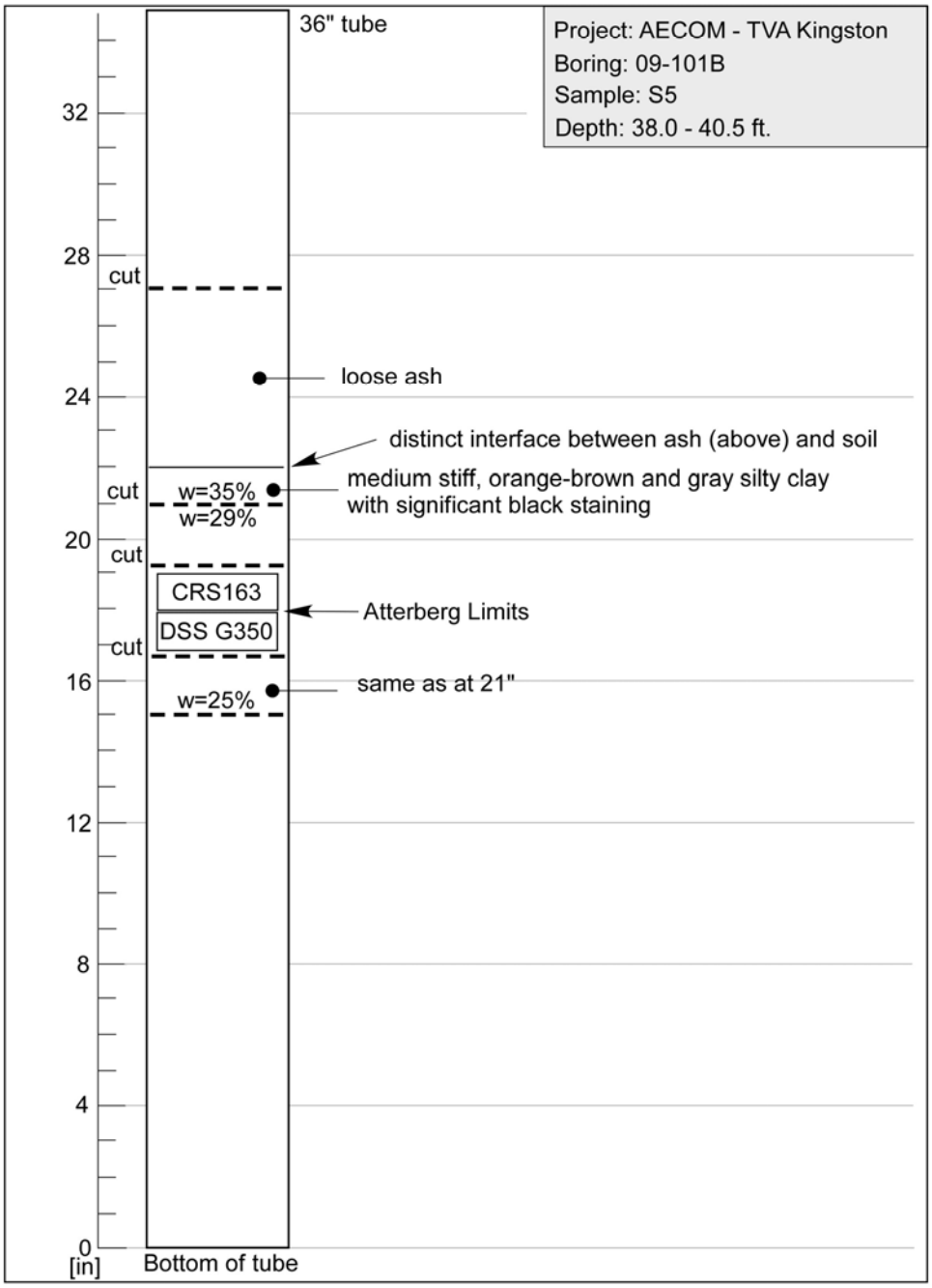


Figure 5.1.38 Sample log and test locations for Sample 09-101B S5 38.0 to 40.5 ft.

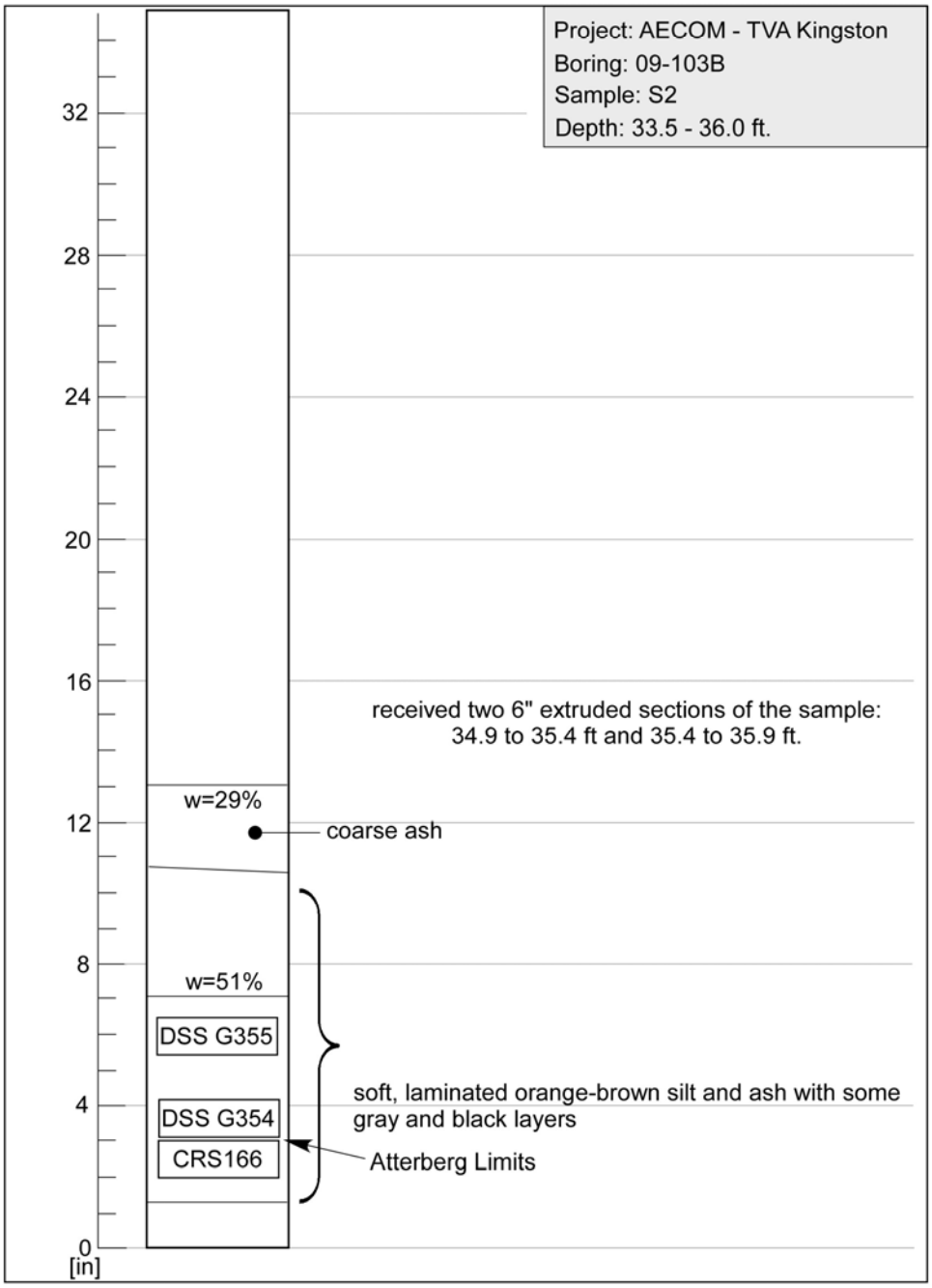


Figure 5.1.39 Sample log and test locations for Sample 09-103B S2 33.5 to 36.0 ft.

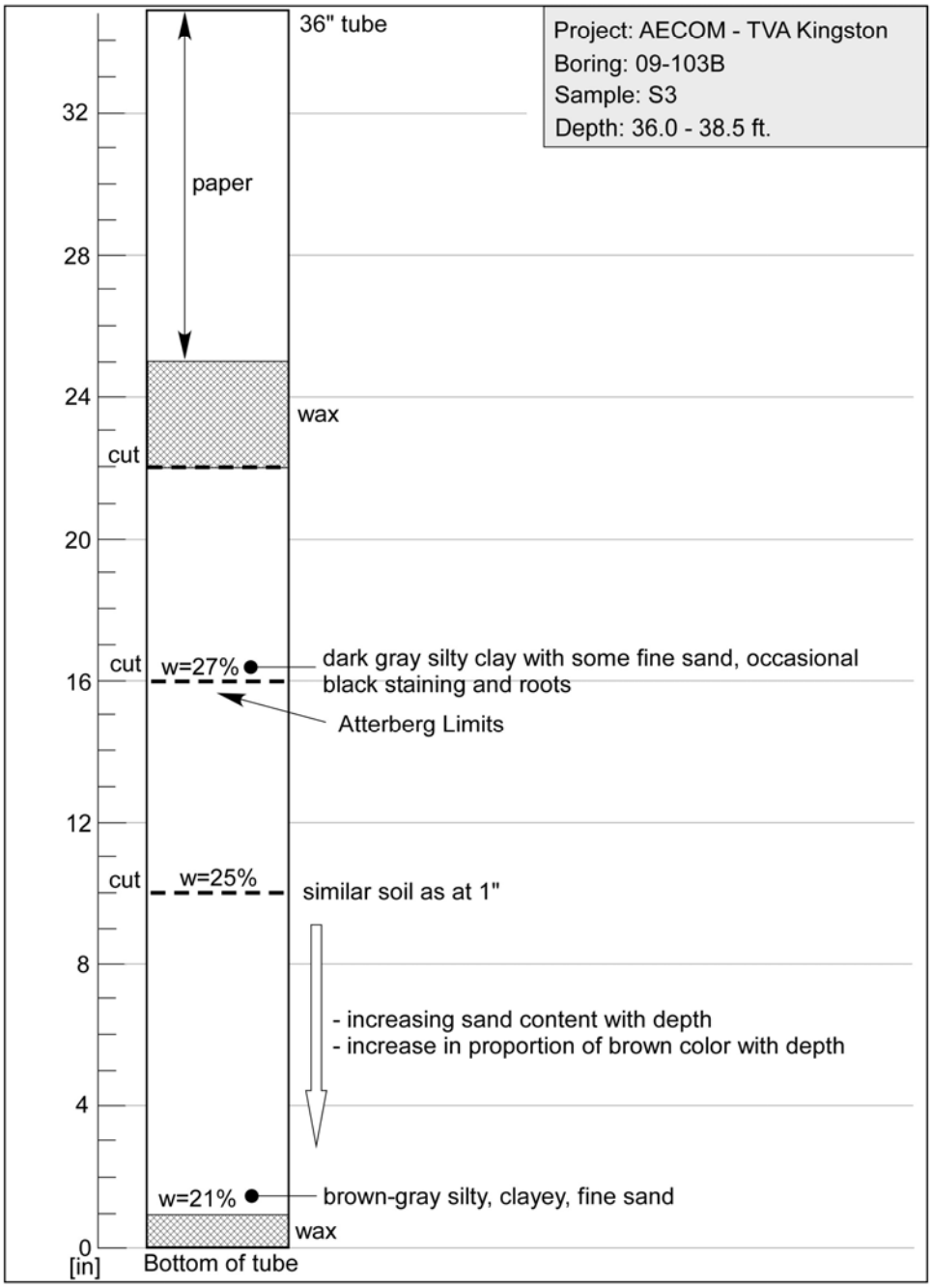


Figure 5.1.40 Sample log and test locations for Sample 09-103B S3 36.0 to 38.5 ft.

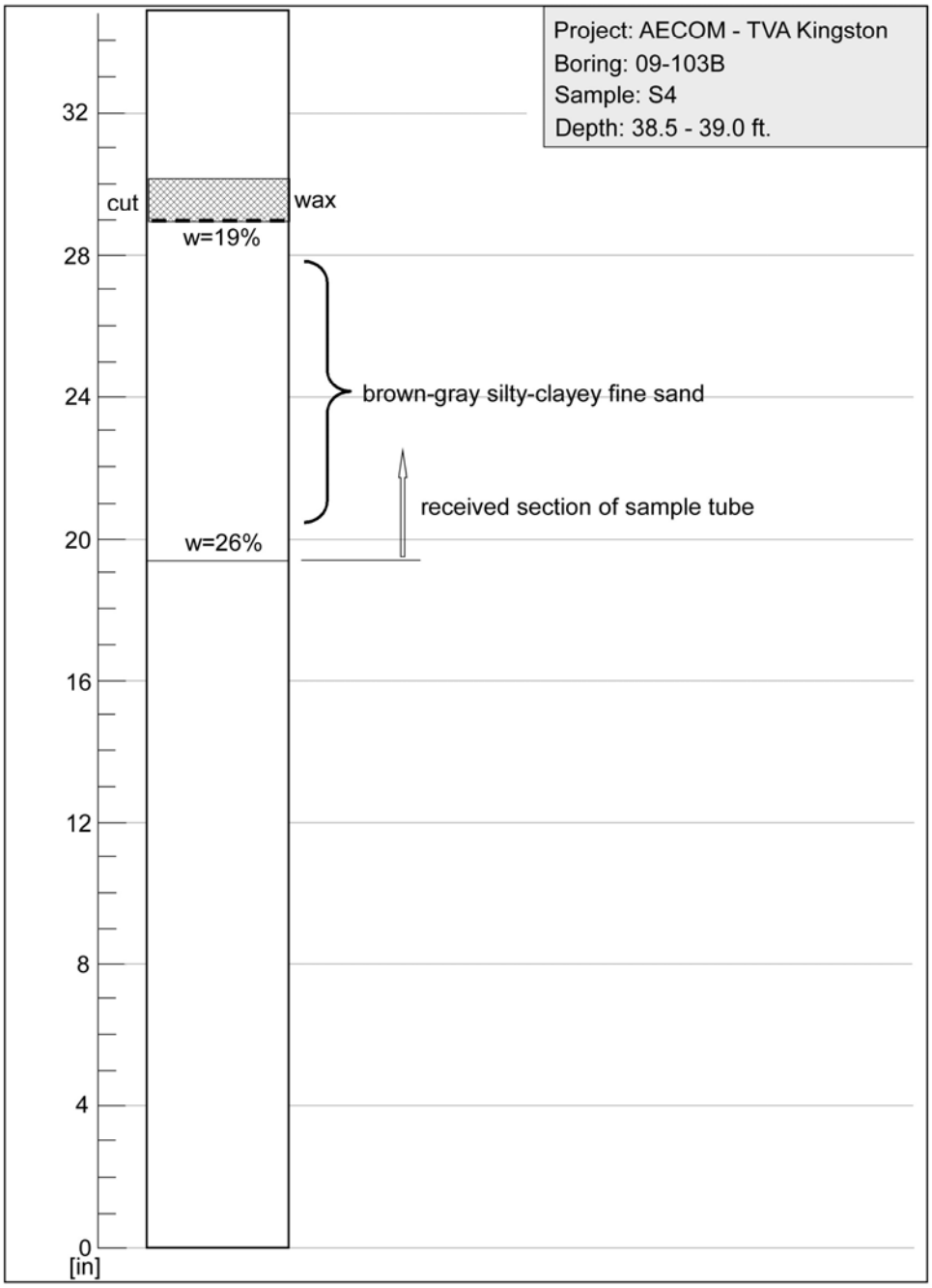


Figure 5.1.41 Sample log and test locations for Sample 09-103B S4 38.5-39.0 ft

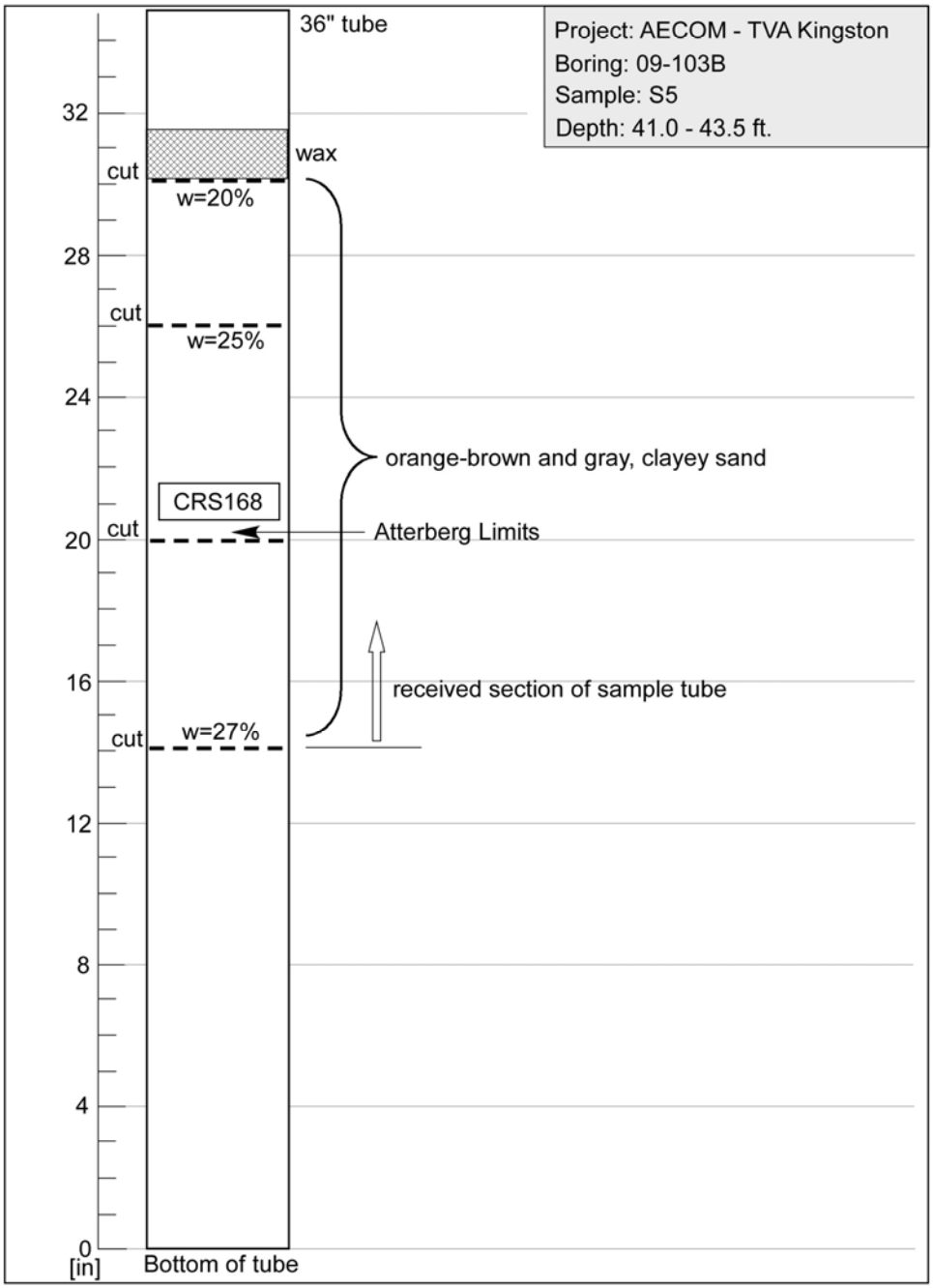


Figure 5.1.42 Sample log and test locations for Sample 09-103B S5 41.0-43.5 ft

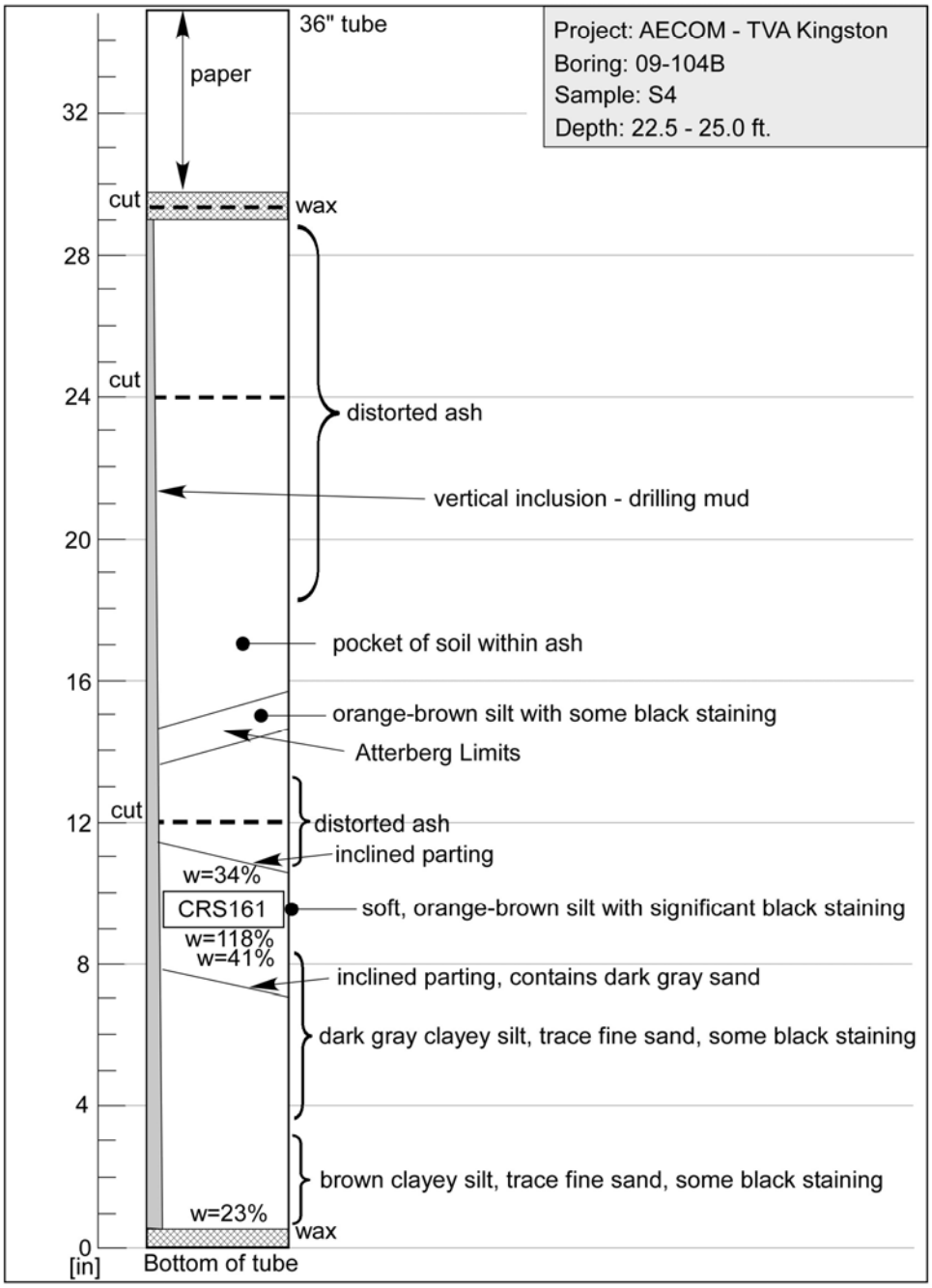


Figure 5.1.43 Sample log and test locations for Sample 09-104B S4 22.5-25.0 ft

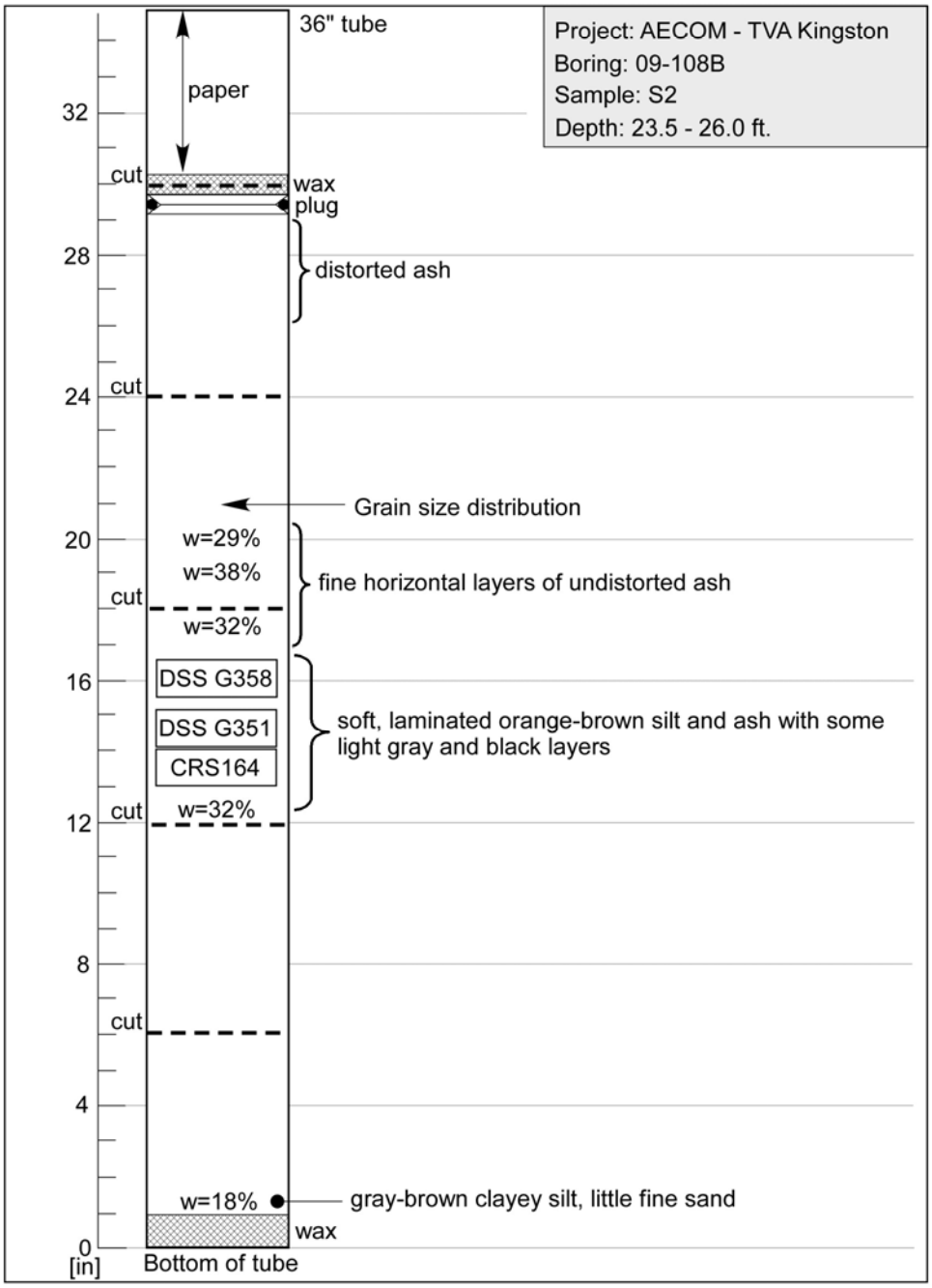


Figure 5.1.44 Sample log and test locations for Sample 09-108B S2 23.5-26.0 ft

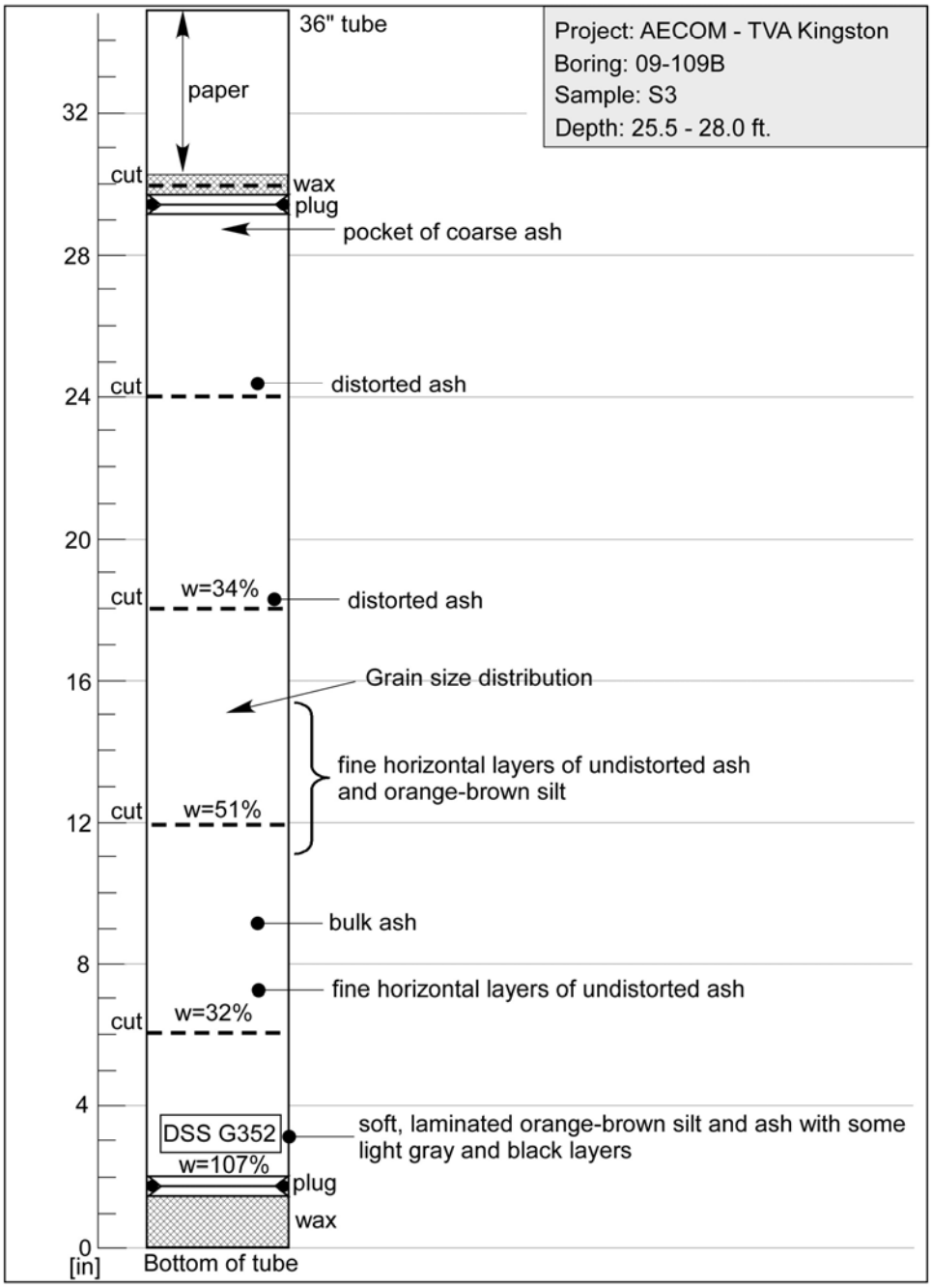


Figure 5.1.45 Sample log and test locations for Sample 09-109B S3 25.5-28.0 ft

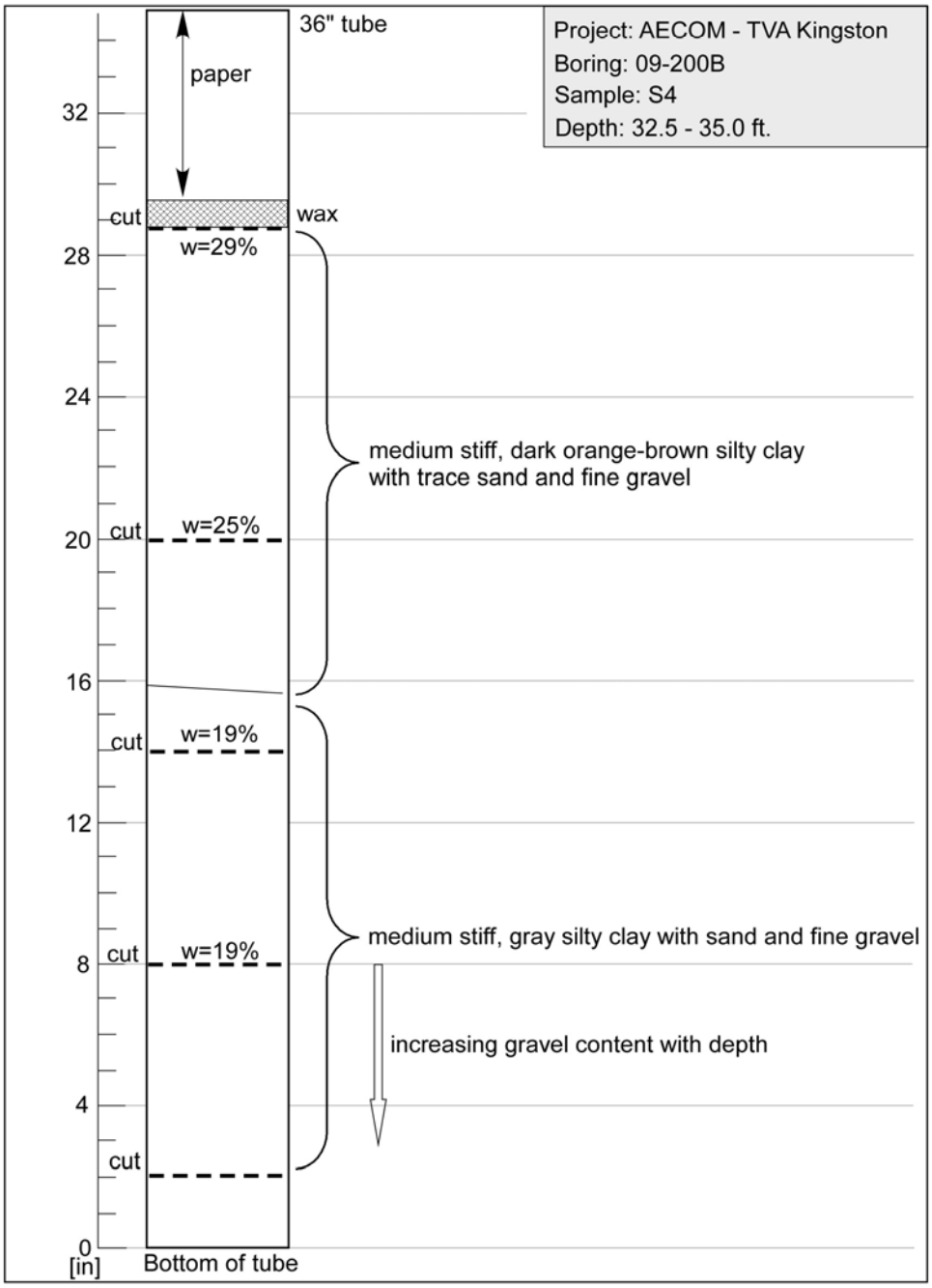


Figure 5.1.46 Sample log and test locations for Sample 09-200B S4 32.5-35.0 ft

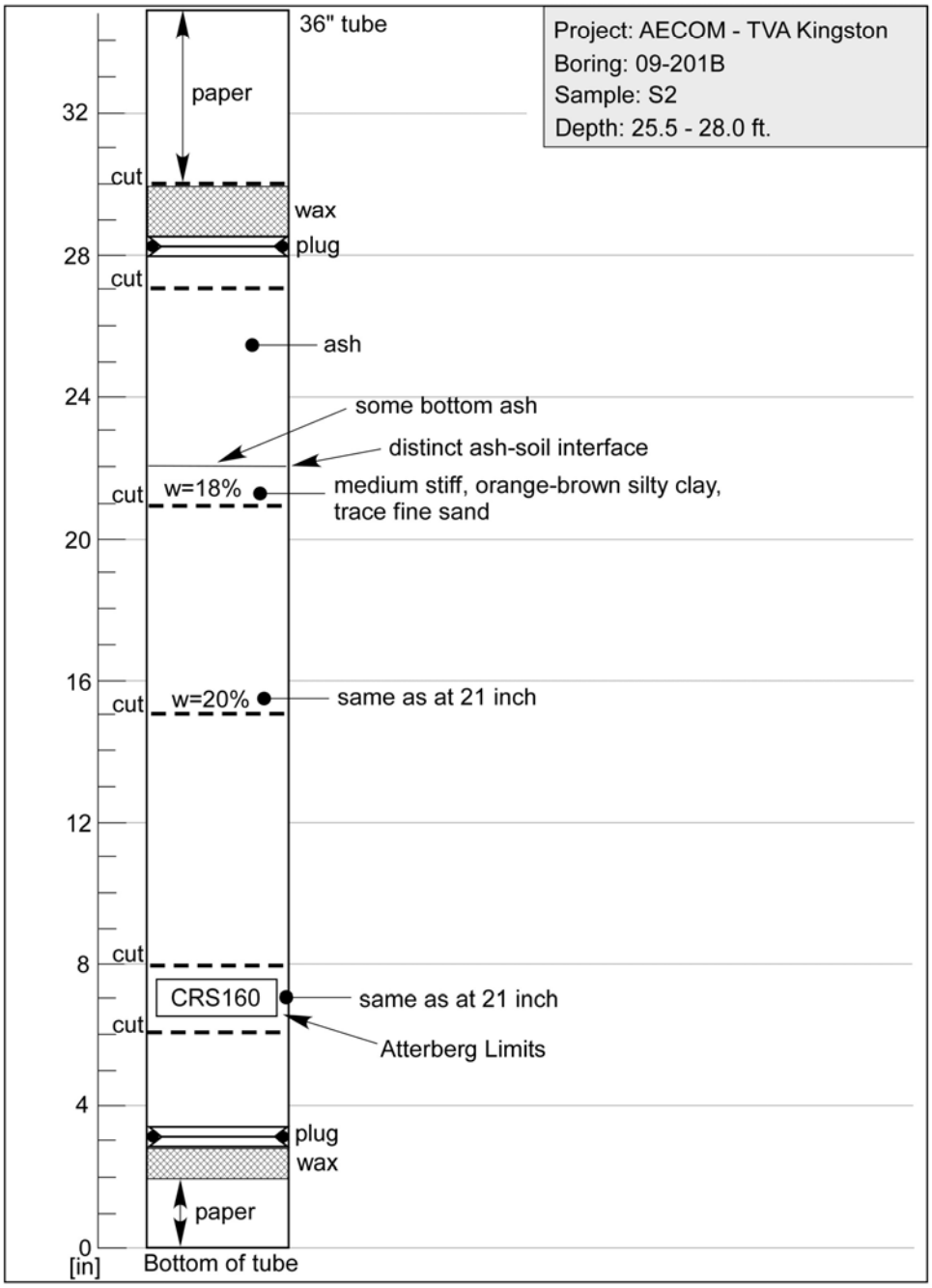


Figure 5.1.47 Sample log and test locations for Sample 09-201B S2 25.5-28.0 ft

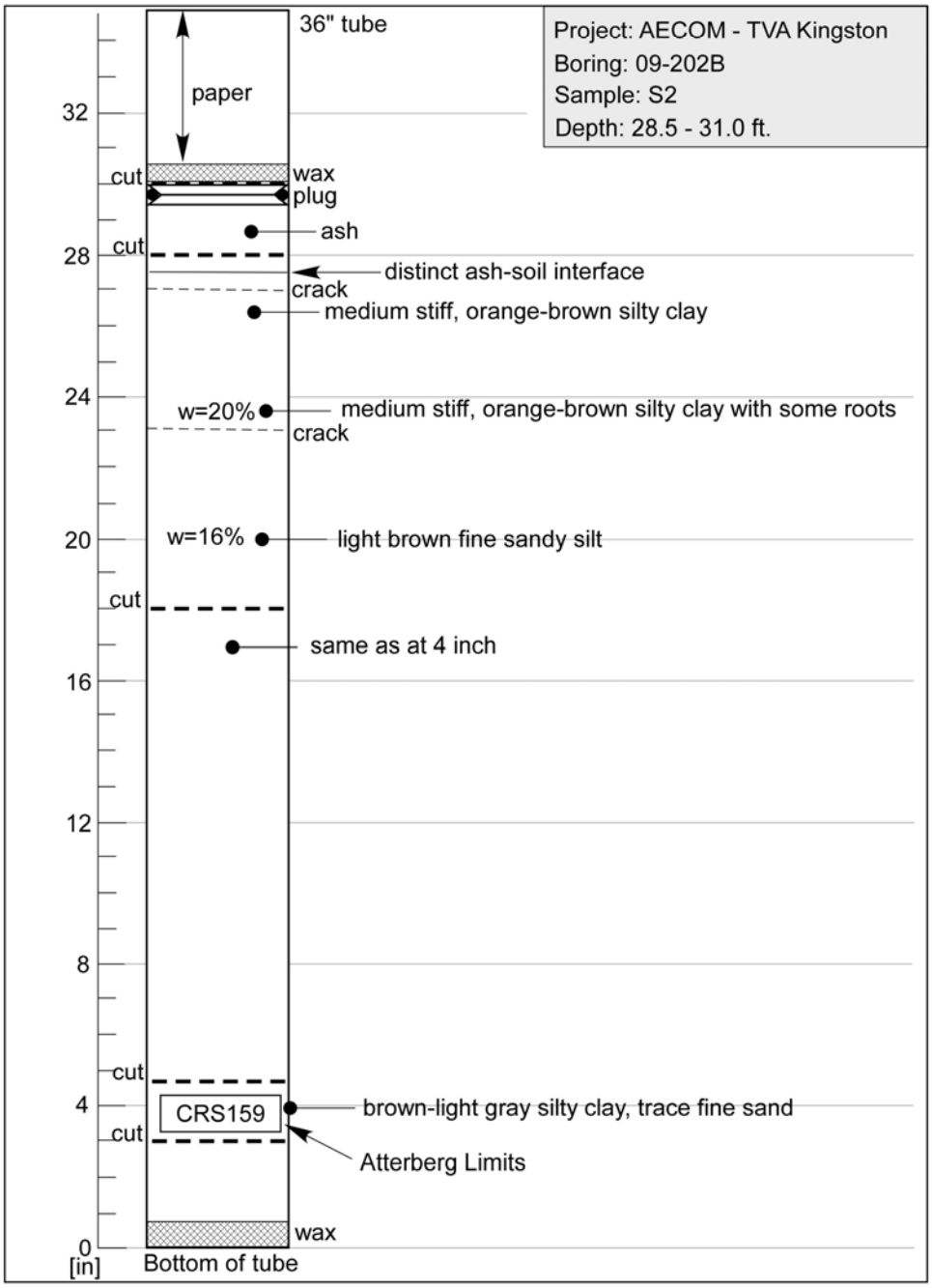


Figure 5.1.48 Sample log and test locations for Sample 09-202B S2 28.5-31.0 ft

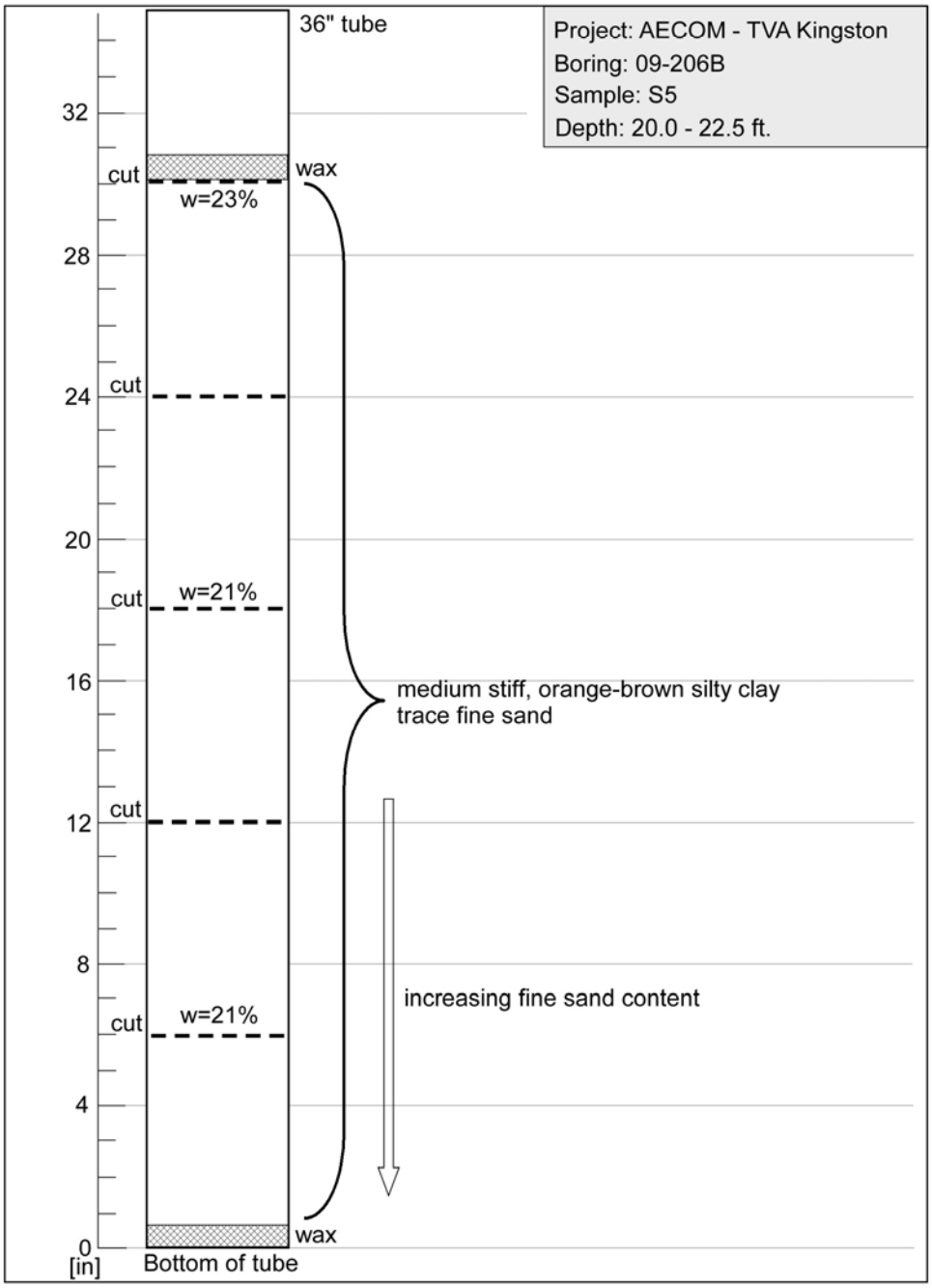


Figure 5.1.49 Sample log and test locations for Sample 09-206B S5 20.0-22.5 ft

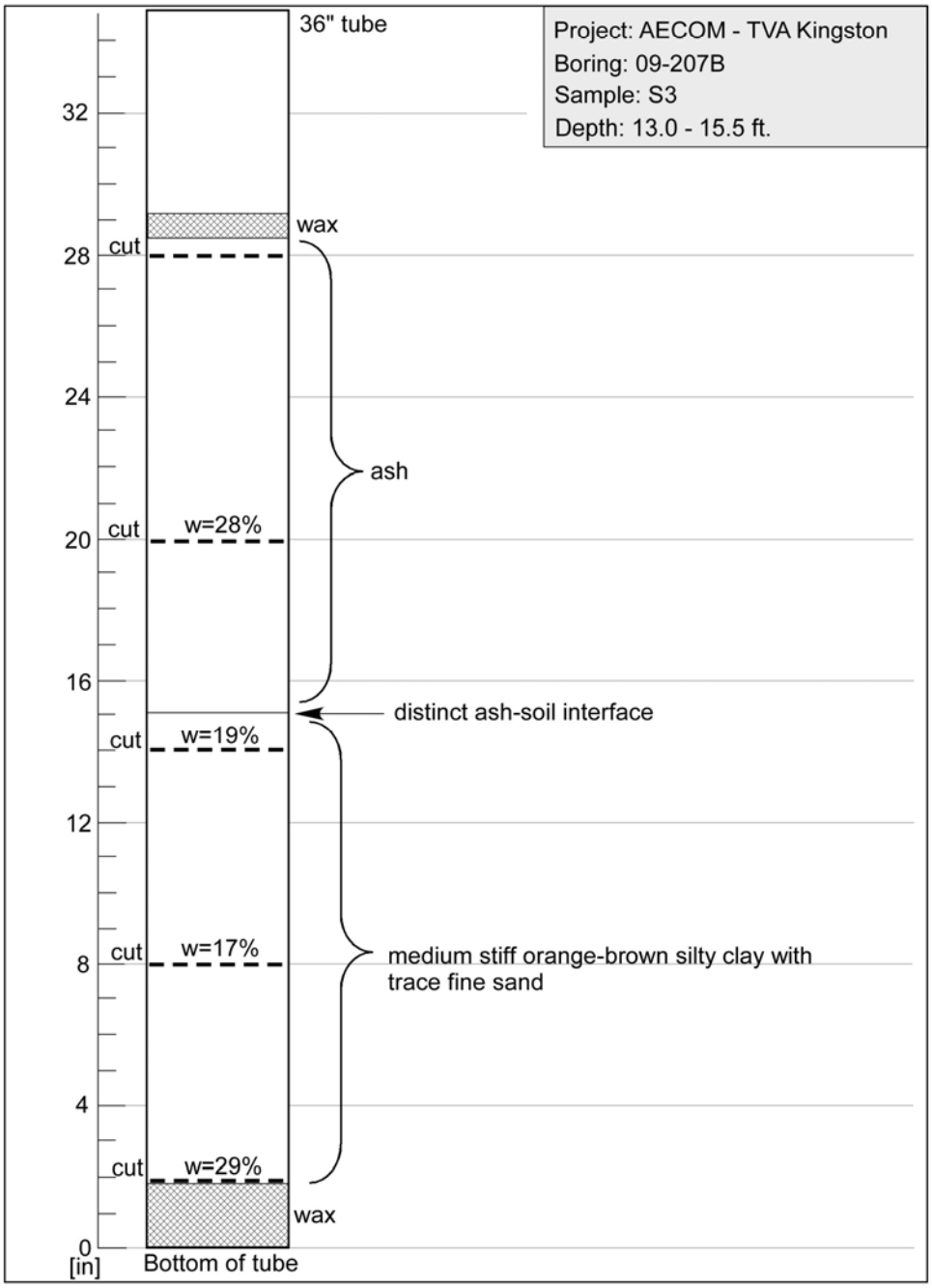


Figure 5.1.50 Sample log and test locations for Sample 09-207B S3 13.0-15.5 ft

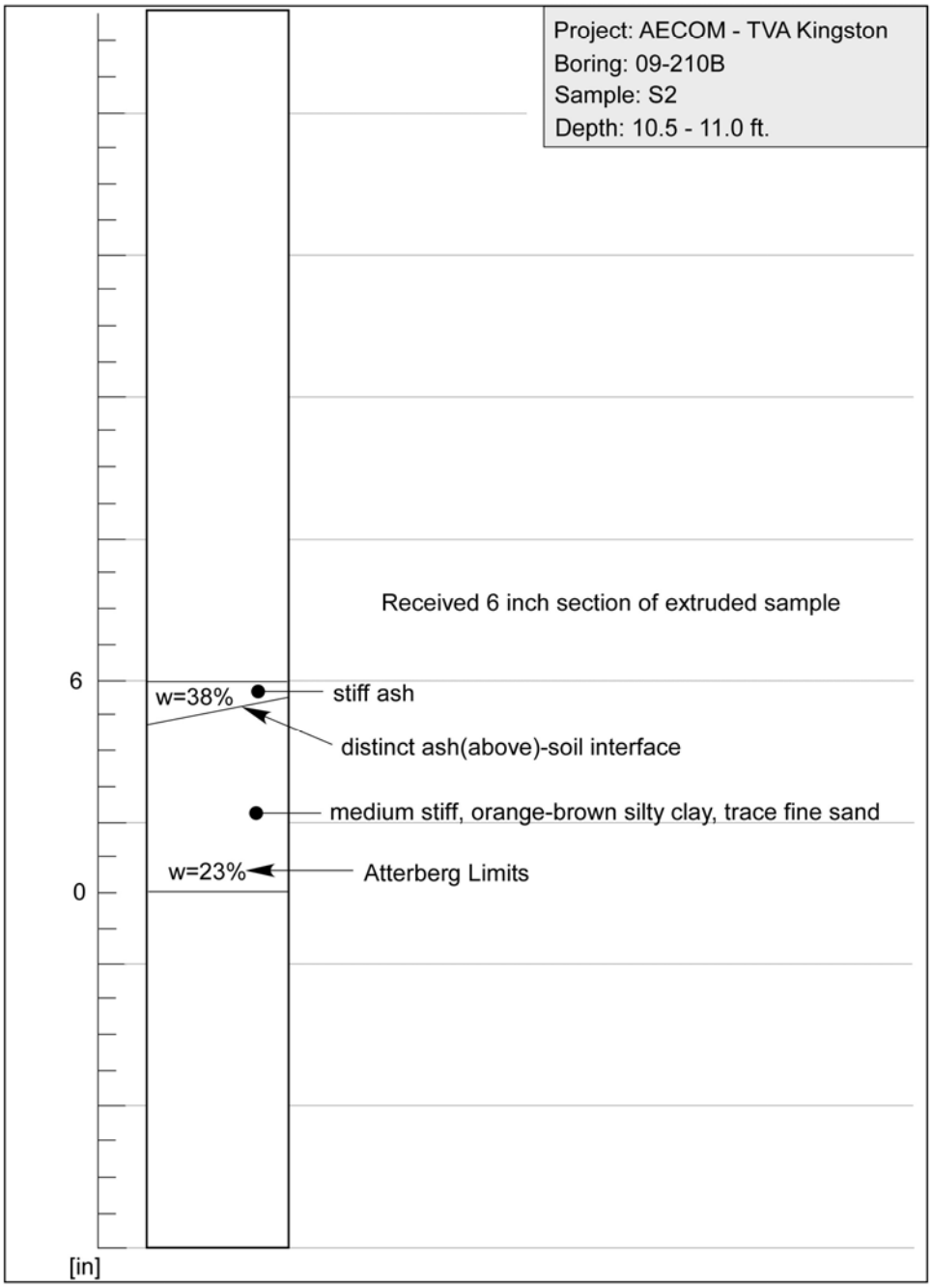


Figure 5.1.51 Sample log and test locations for Sample 09-210B S2 10.5-11.0 ft

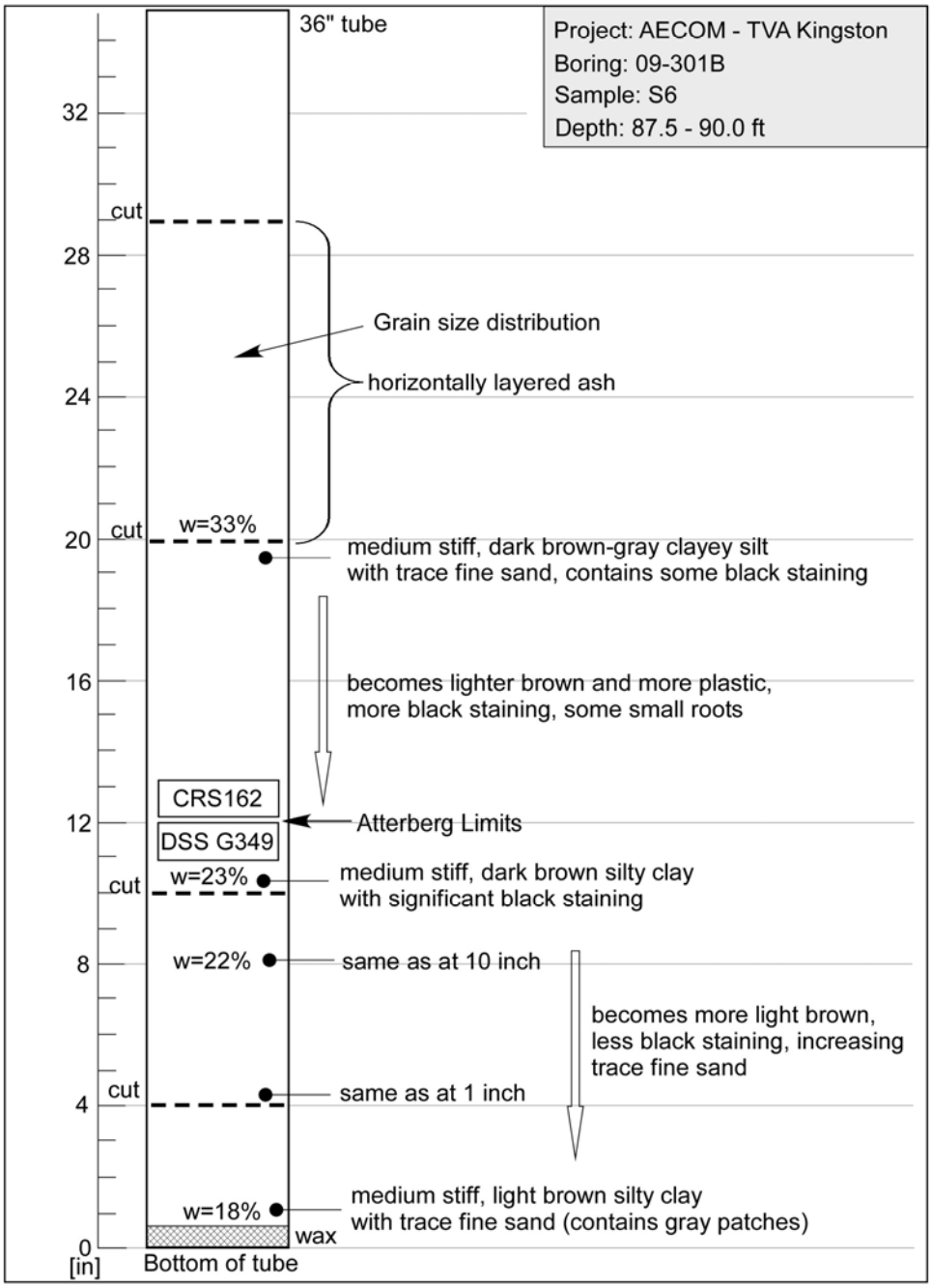


Figure 5.1.52 Sample log and test locations for Sample 09-301B S6 87.5-90.0 ft

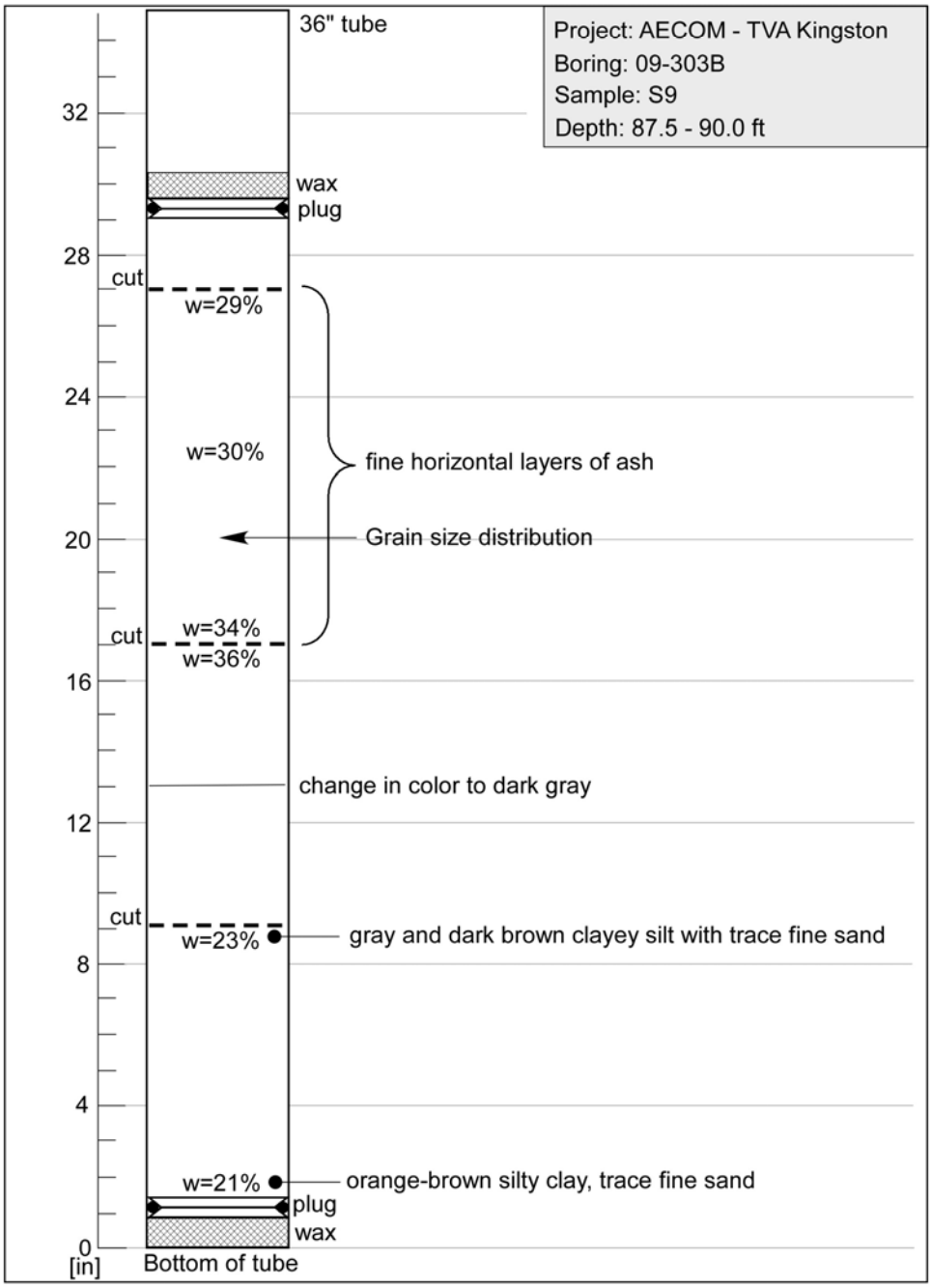


Figure 5.1.53 Sample log and test locations for Sample 09-303B S9 87.5-90.0 ft

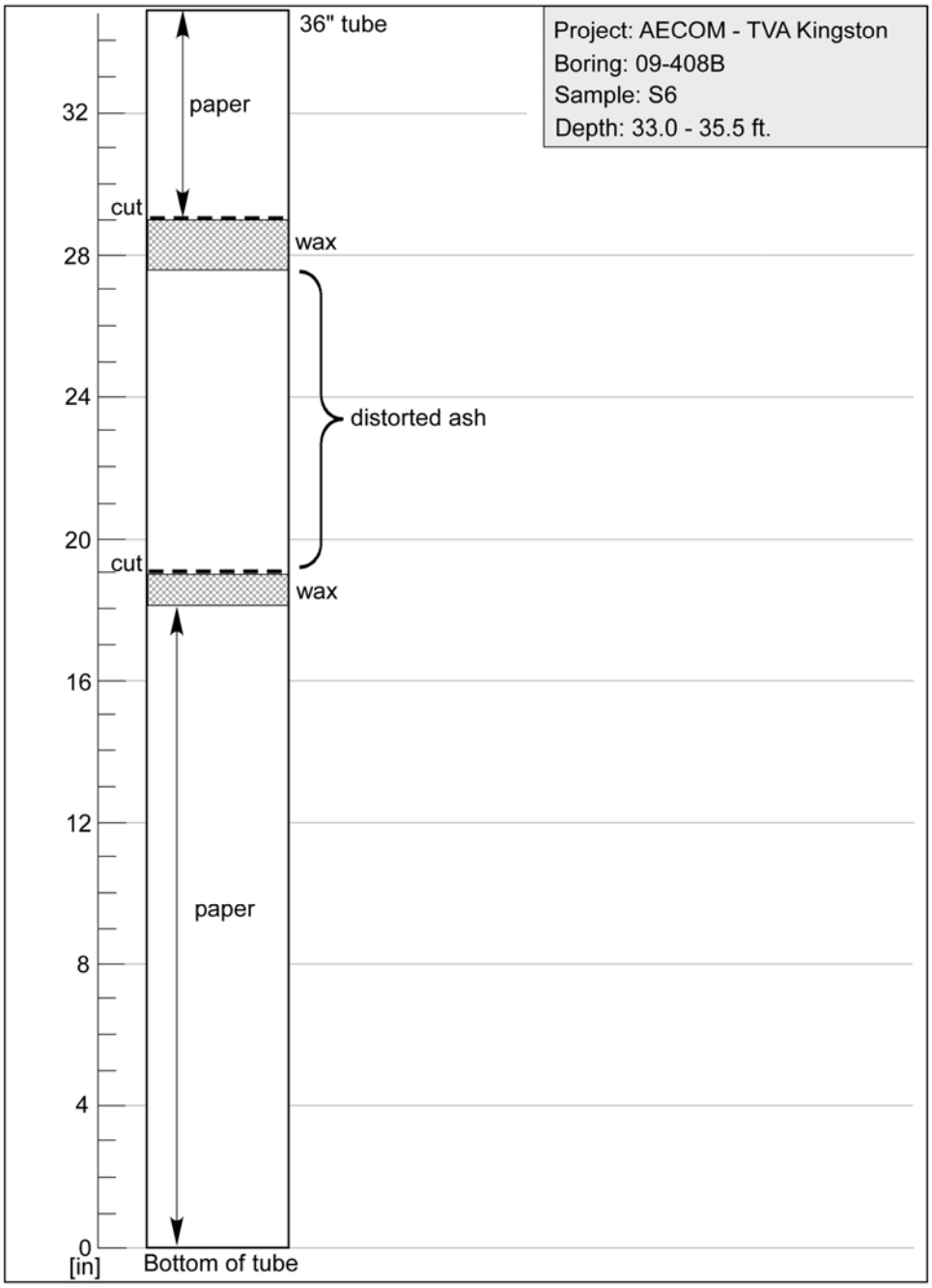


Figure 5.1.54 Sample log and test locations for Sample 09-408B S6 33.0-35.5 ft

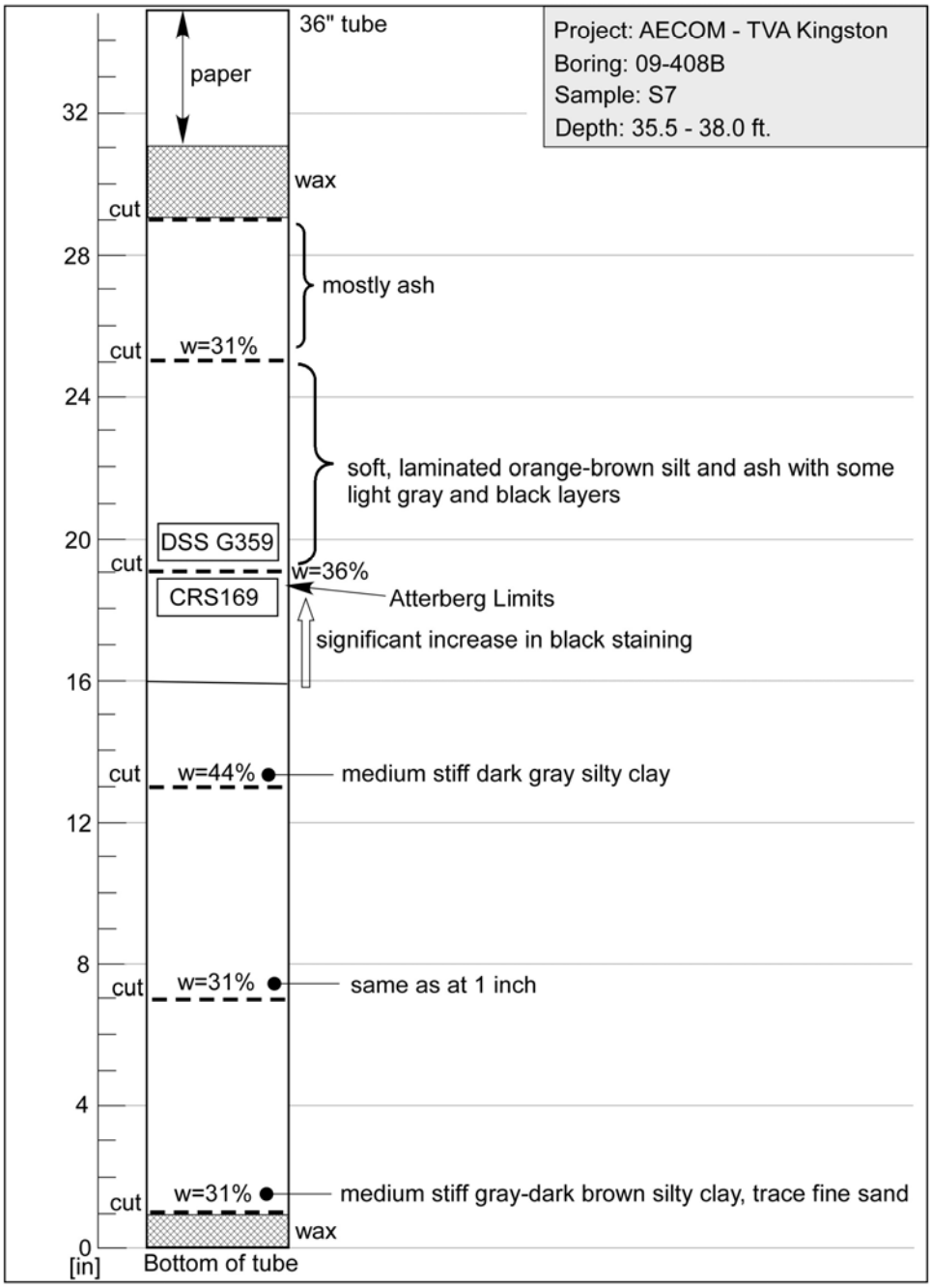


Figure 5.1.55 Sample log and test locations for Sample 09-408B S7 35.5-38.0 ft

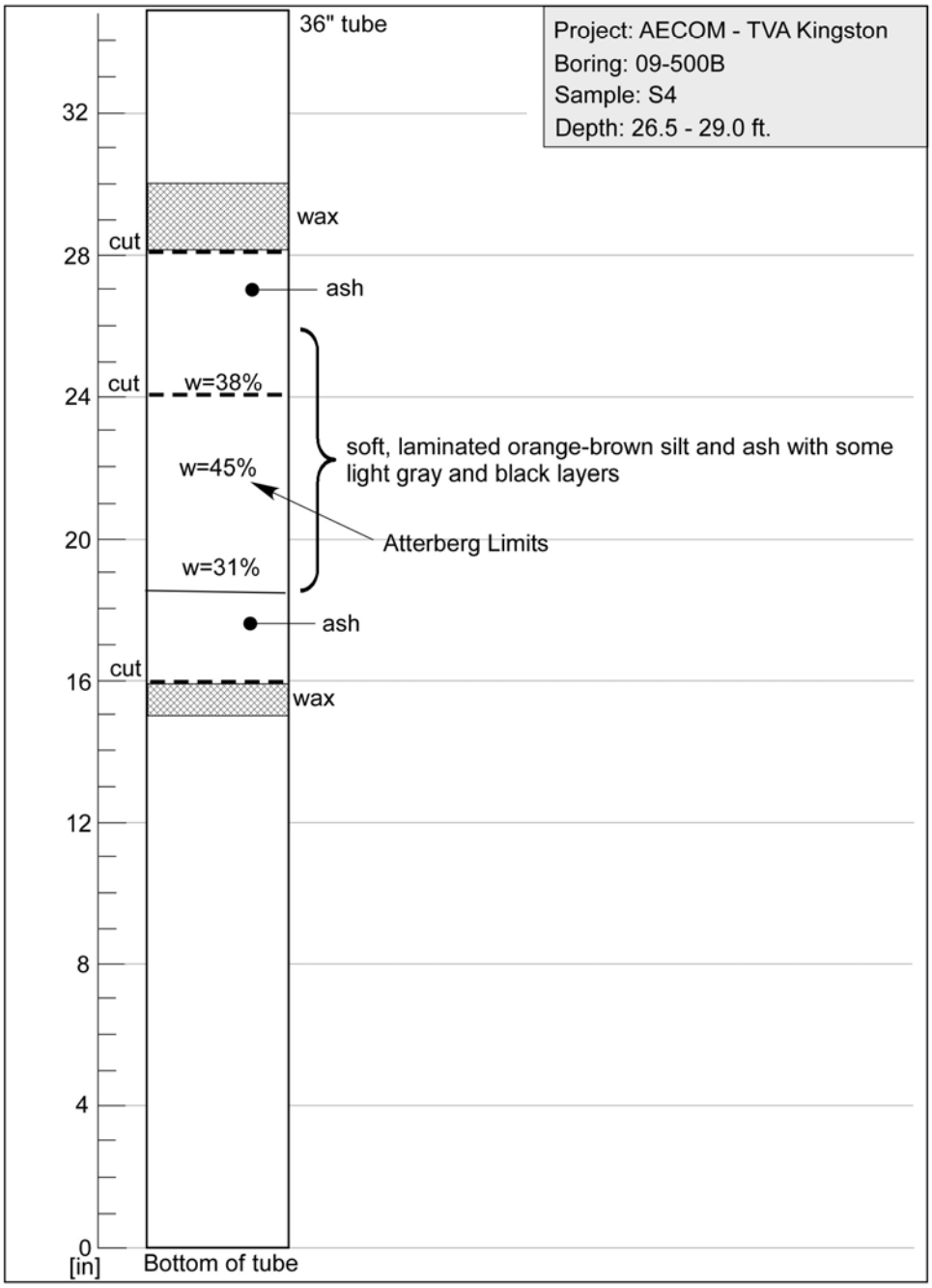


Figure 5.1.56 Sample log and test locations for Sample 09-500B S4 26.5-29.0 ft

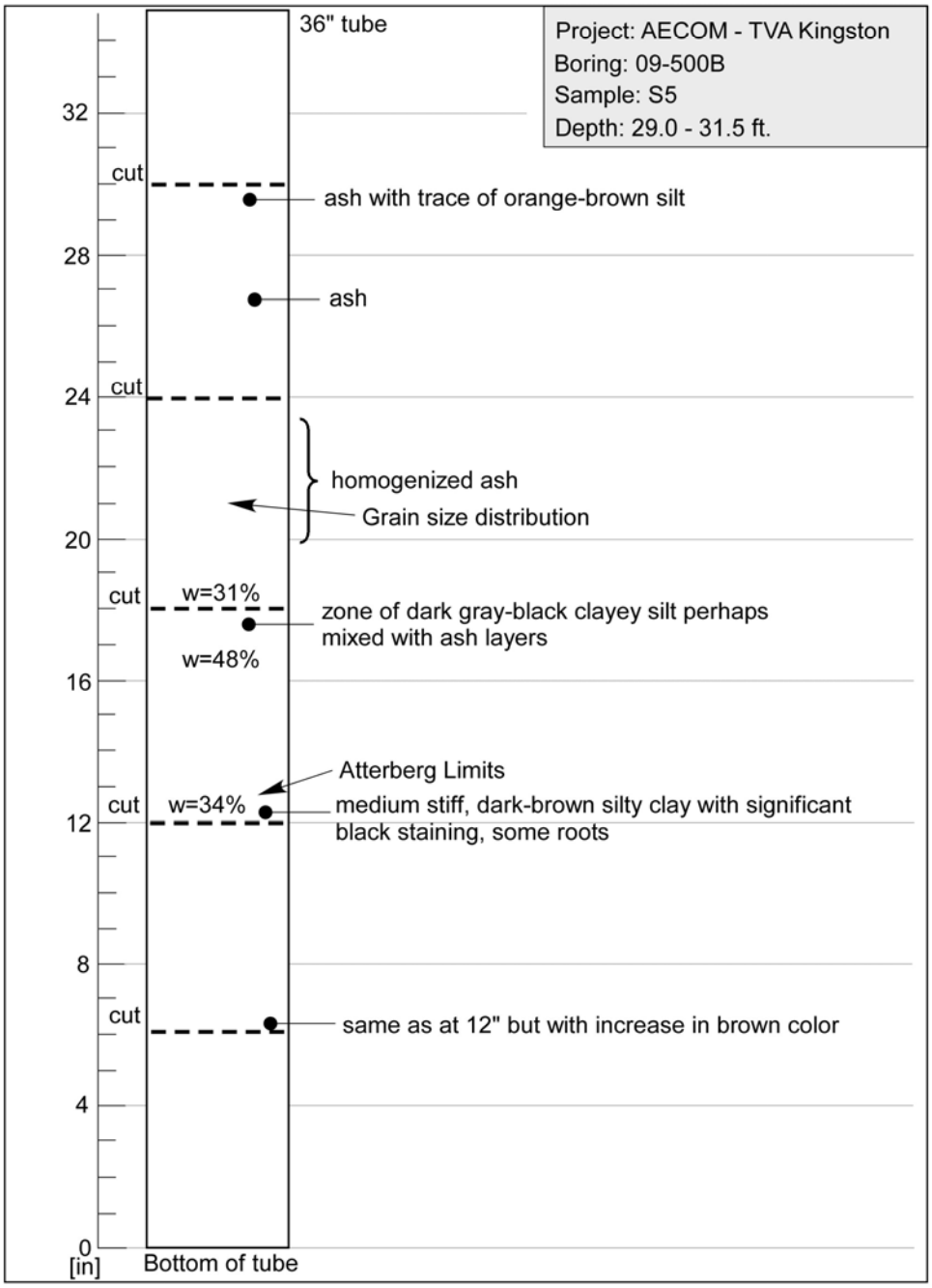


Figure 5.1.57 Sample log and test locations for Sample 09-500B S5 29.0-31.5 ft

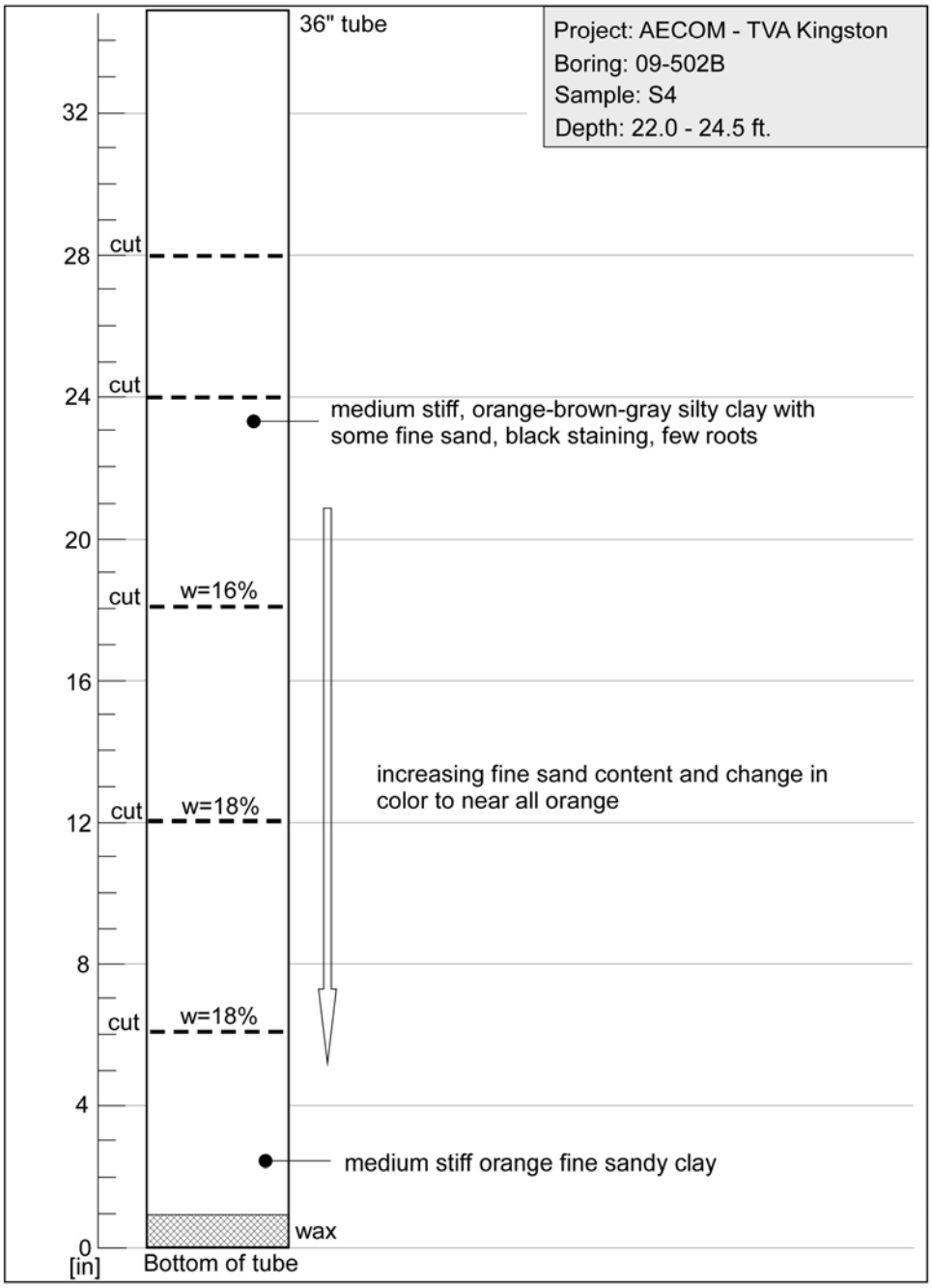


Figure 5.1.58 Sample log and test locations for Sample 09-502B S4 22.0-24.5 ft

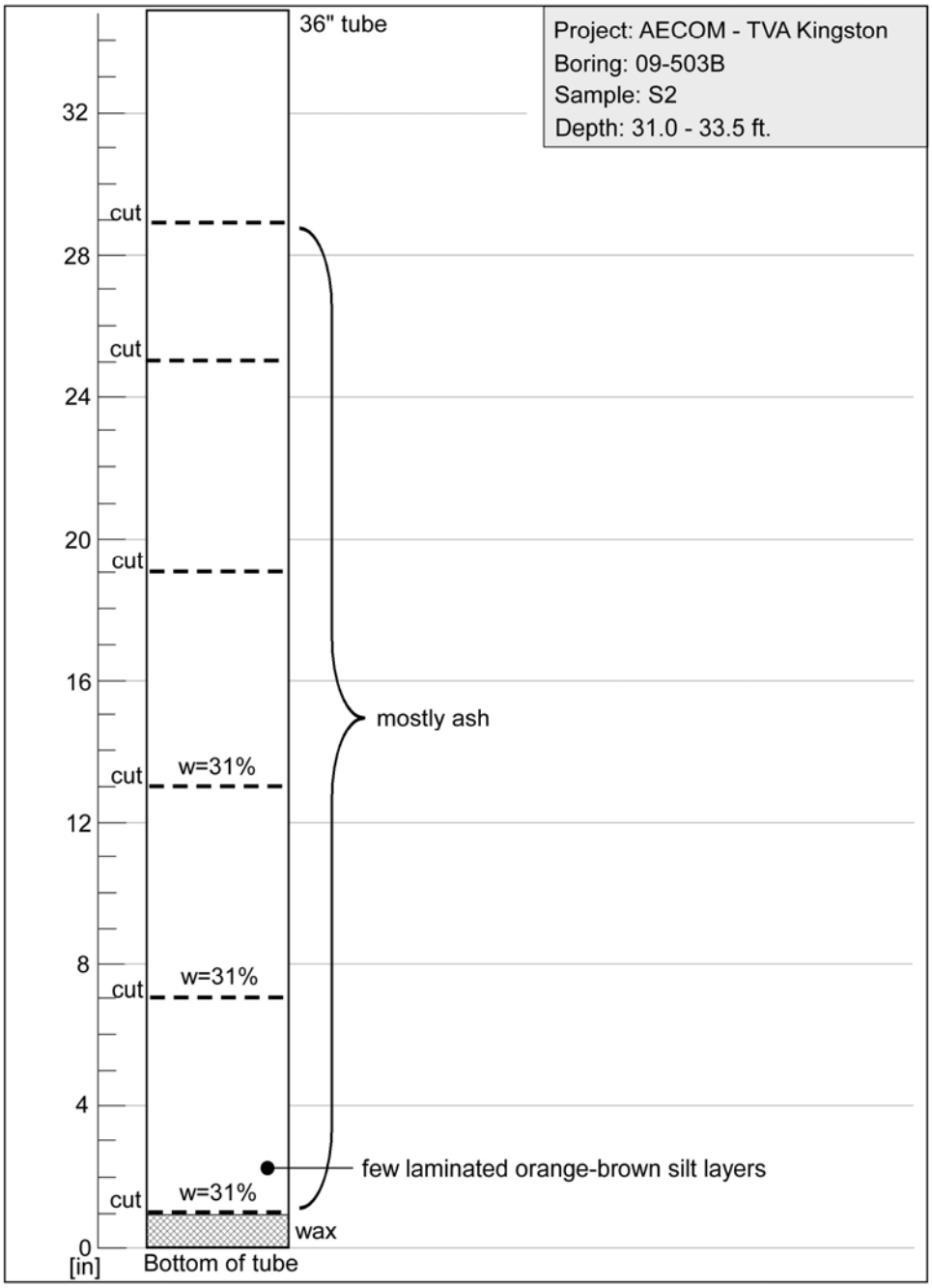


Figure 5.1.59 Sample log and test locations for Sample 09-503B S2 31.0-33.5 ft

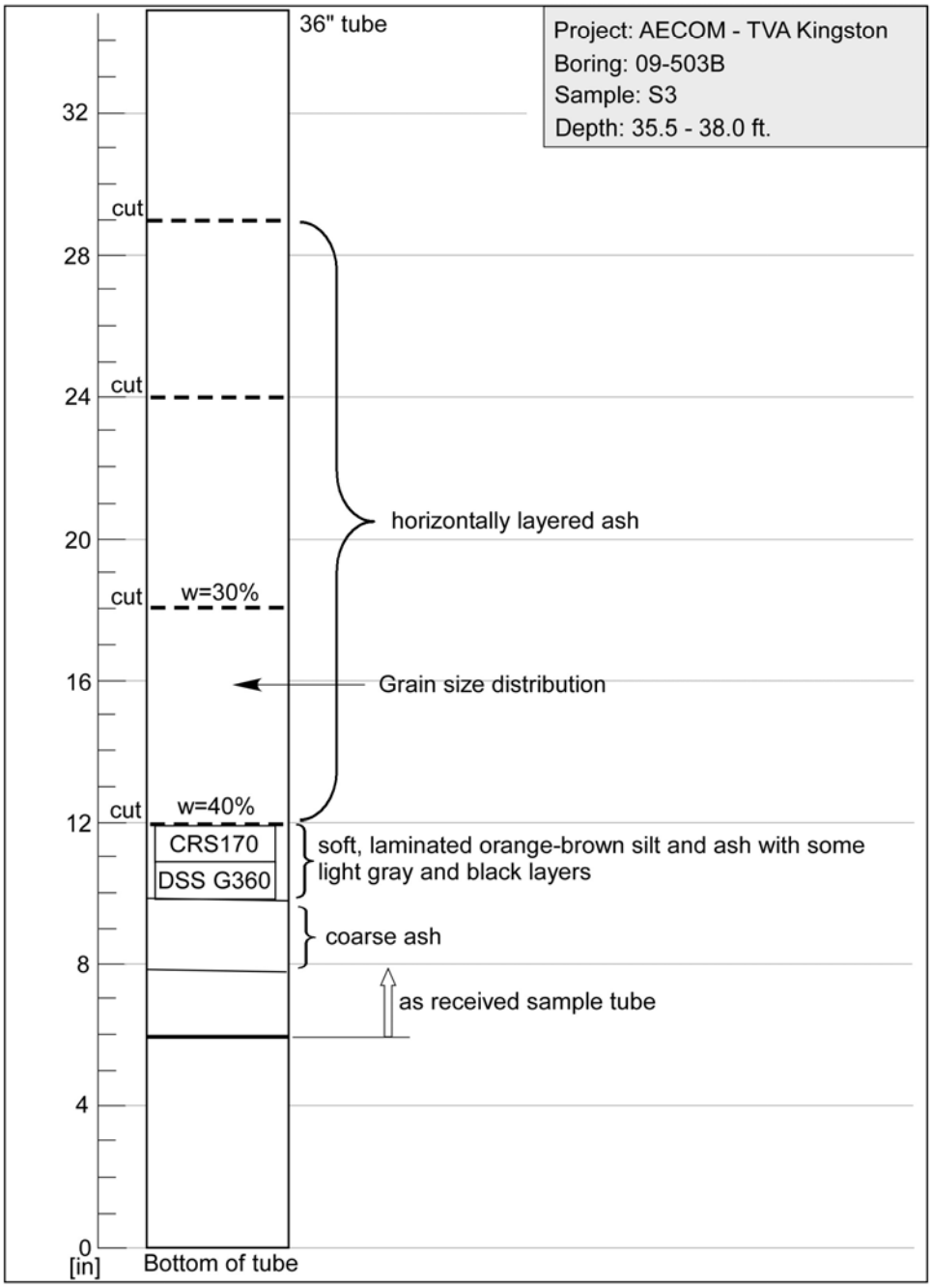


Figure 5.1.60 Sample log and test locations for Sample 09-503B S3 35.5-37.5 ft

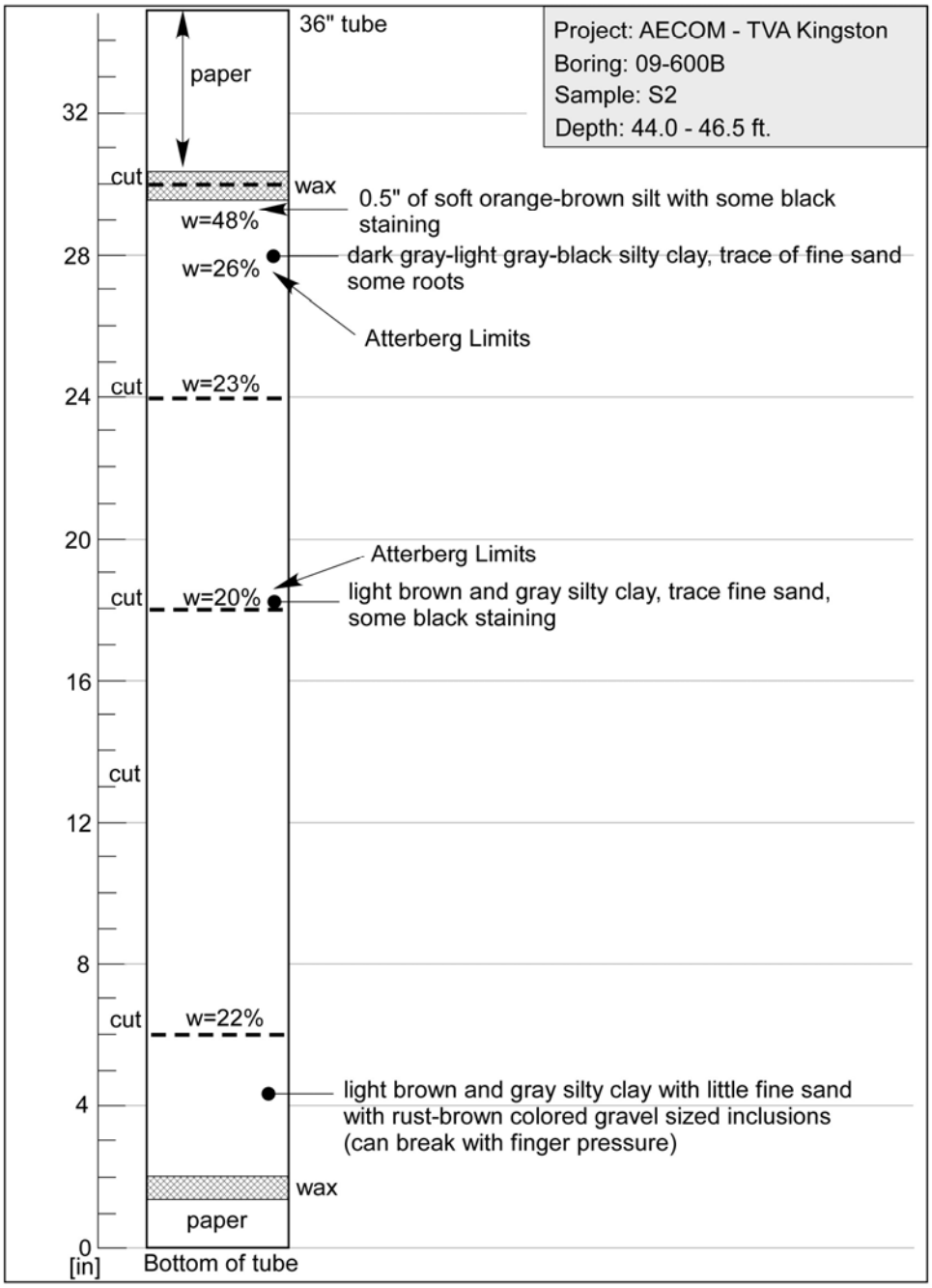


Figure 5.1.61 Sample log and test locations for Sample 09-600B S2 44.0-46.5 ft

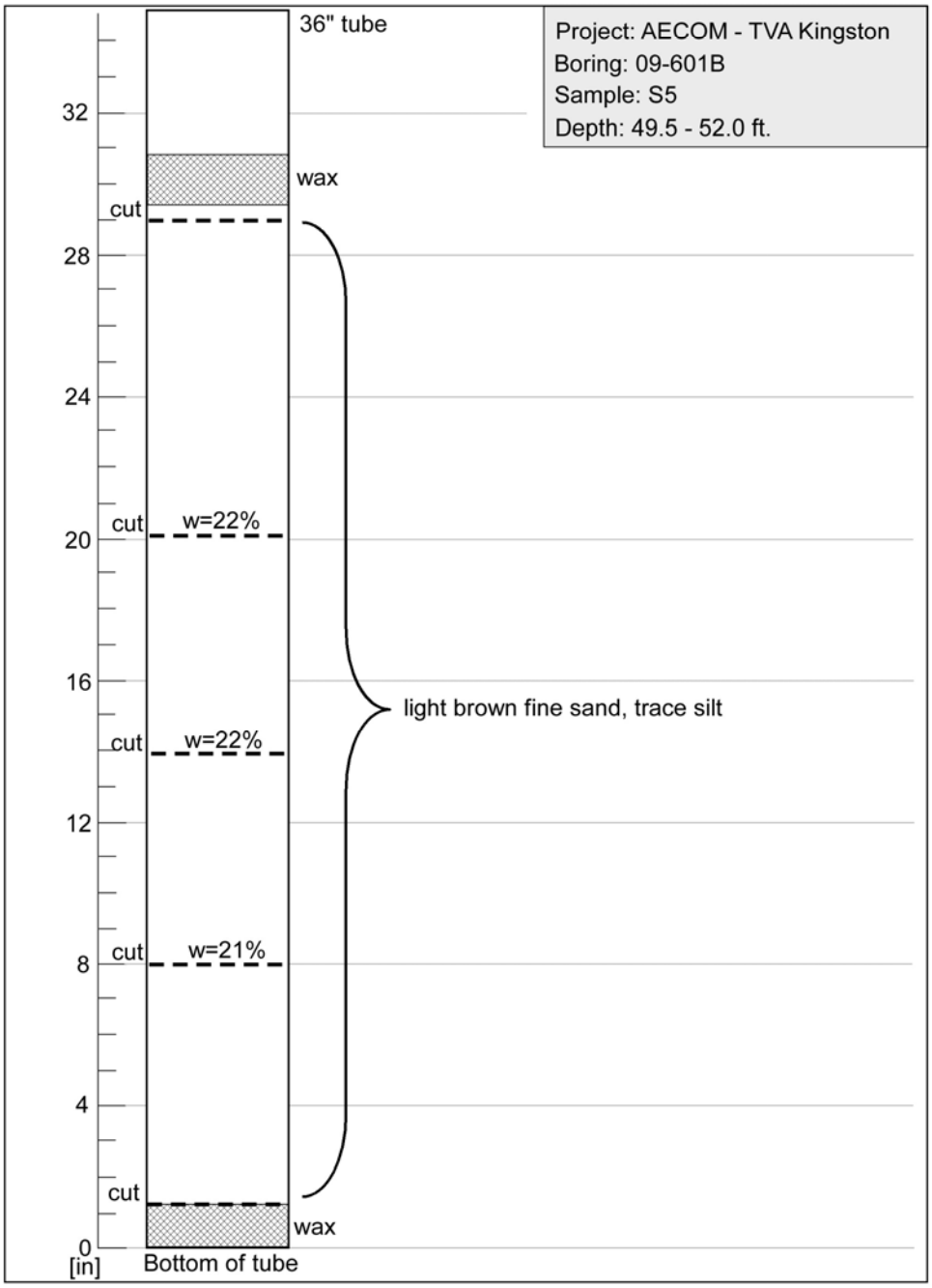


Figure 5.1.62 Sample log and test locations for Sample 09-601B S5 49.5-52.0 ft

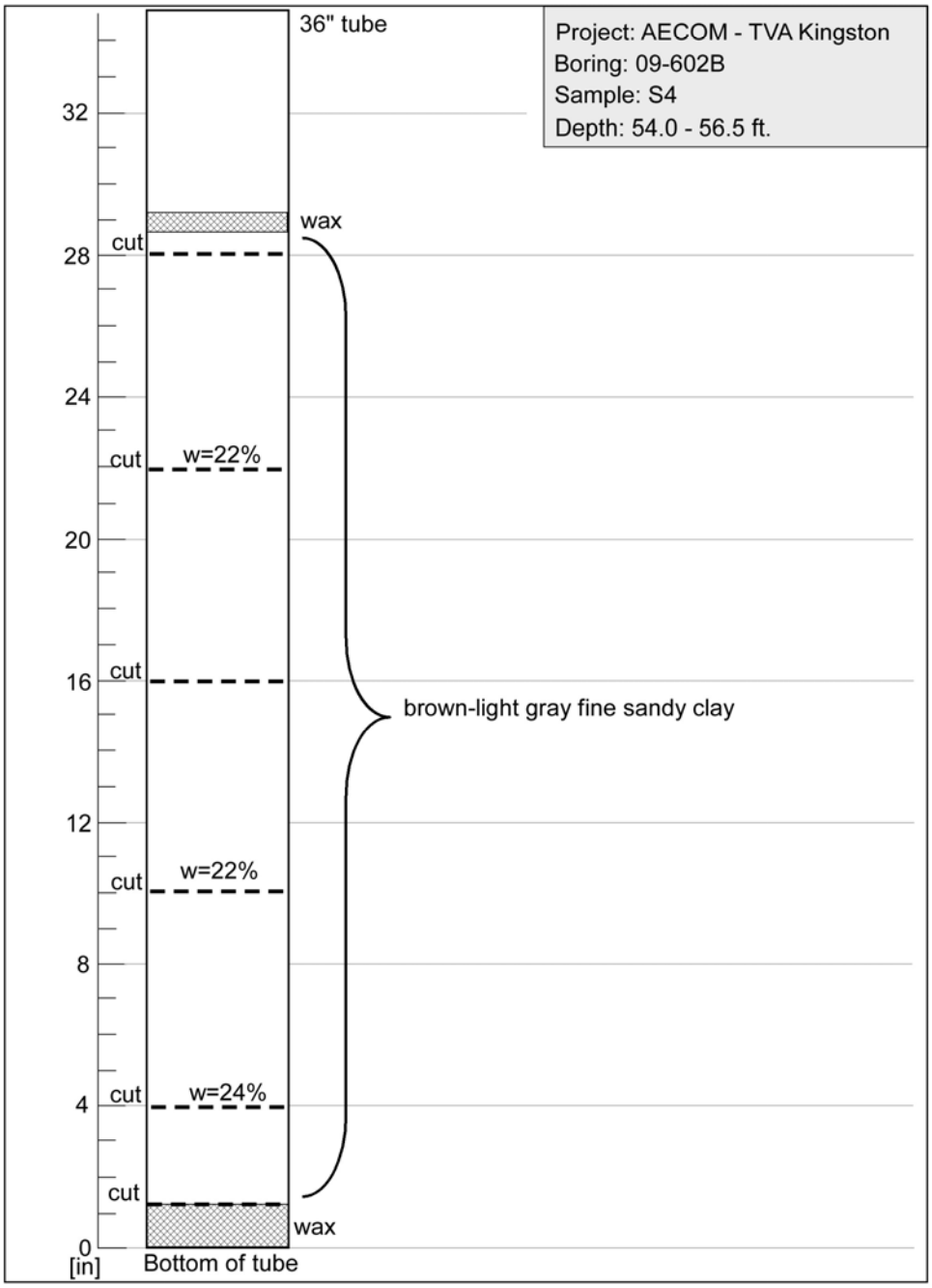


Figure 5.1.63 Sample log and test locations for Sample 09-602B S4 54.0-56.5 ft

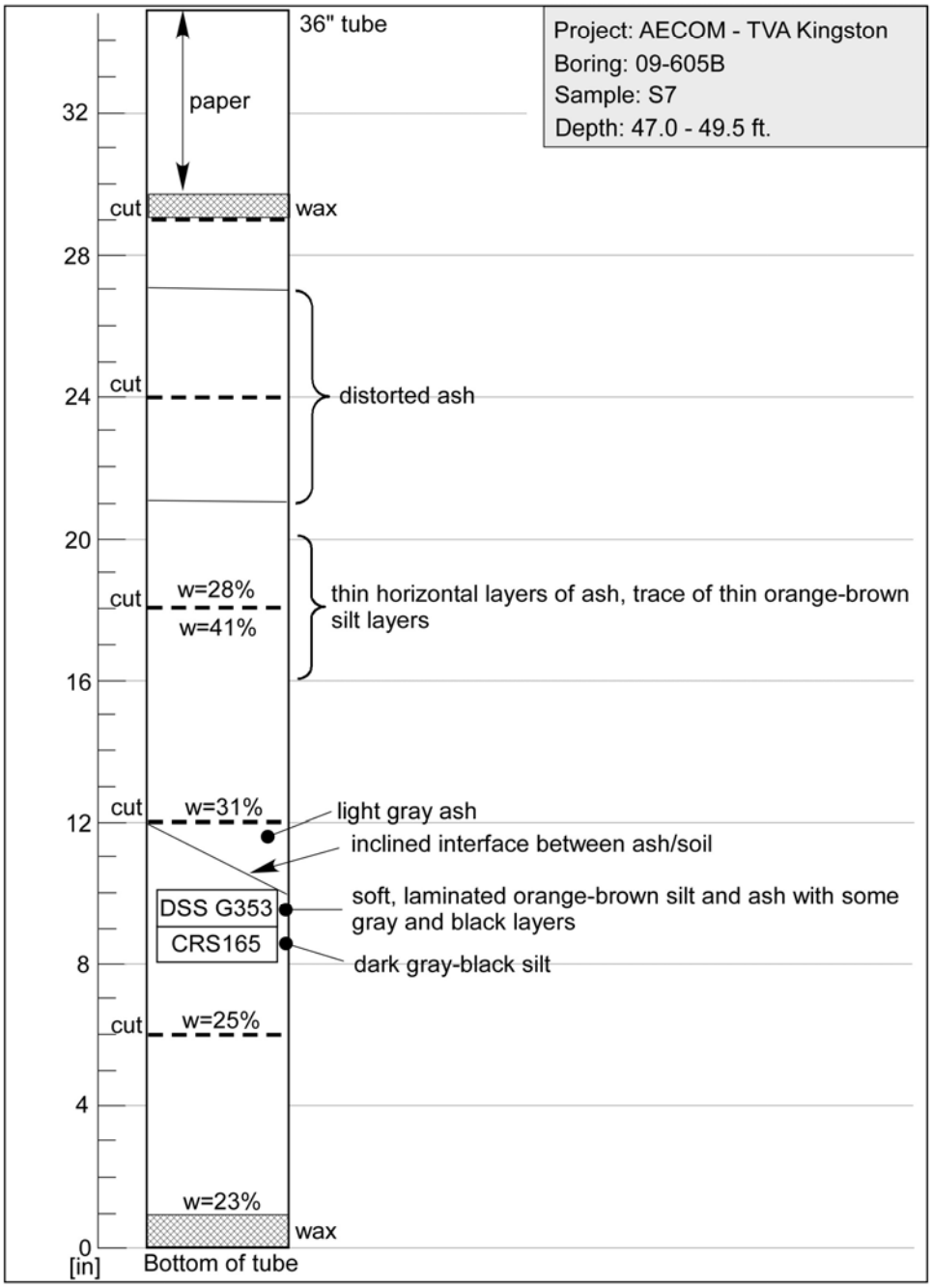


Figure 5.1.64 Sample log and test locations for Sample 09-605B S7 47.0-49.5 ft

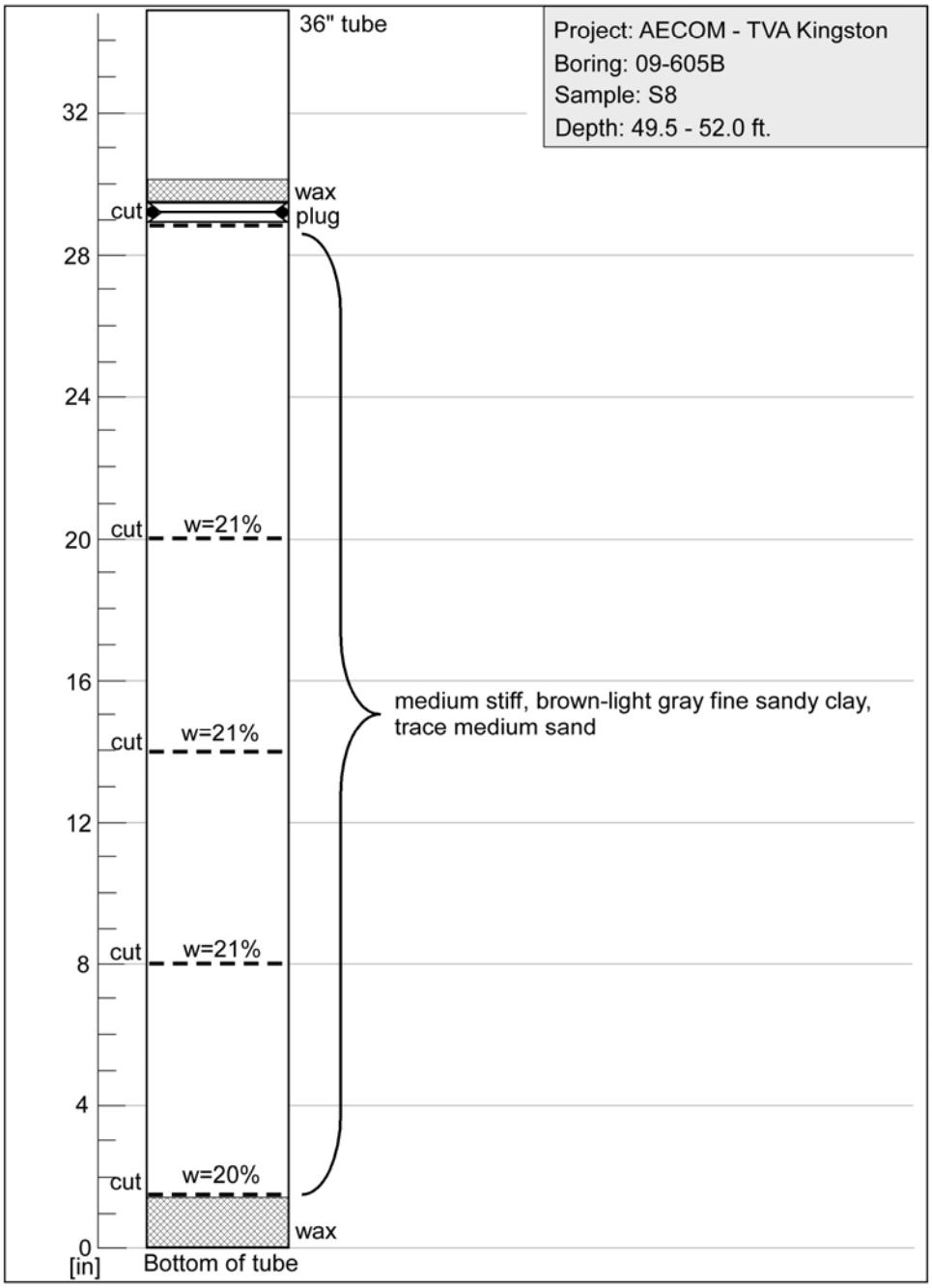


Figure 5.1.65 Sample log and test locations for Sample 09-605B S8 49.5-52.0 ft

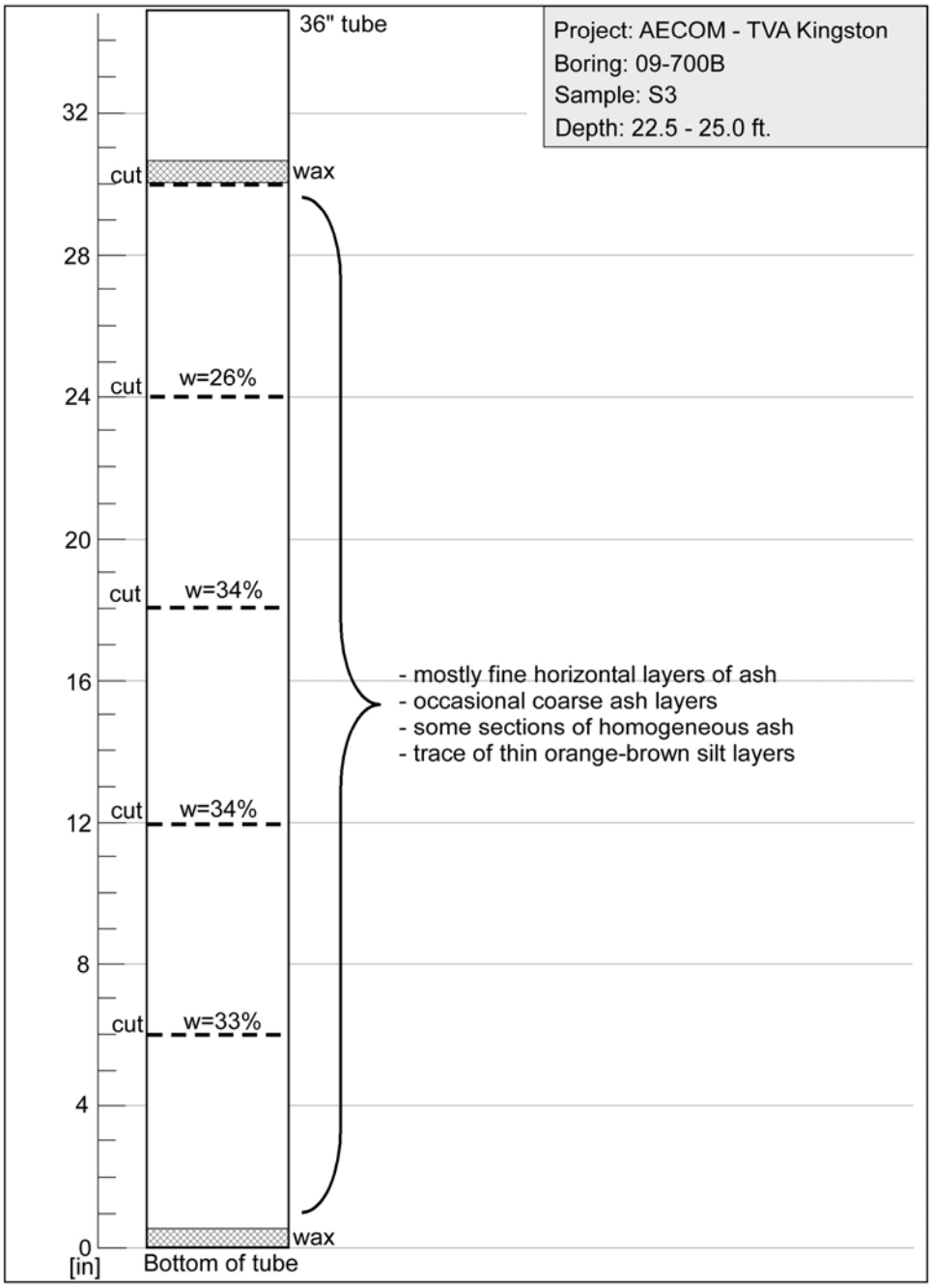


Figure 5.1.66 Sample log and test locations for Sample 09-700B S3 22.5-25.0 ft

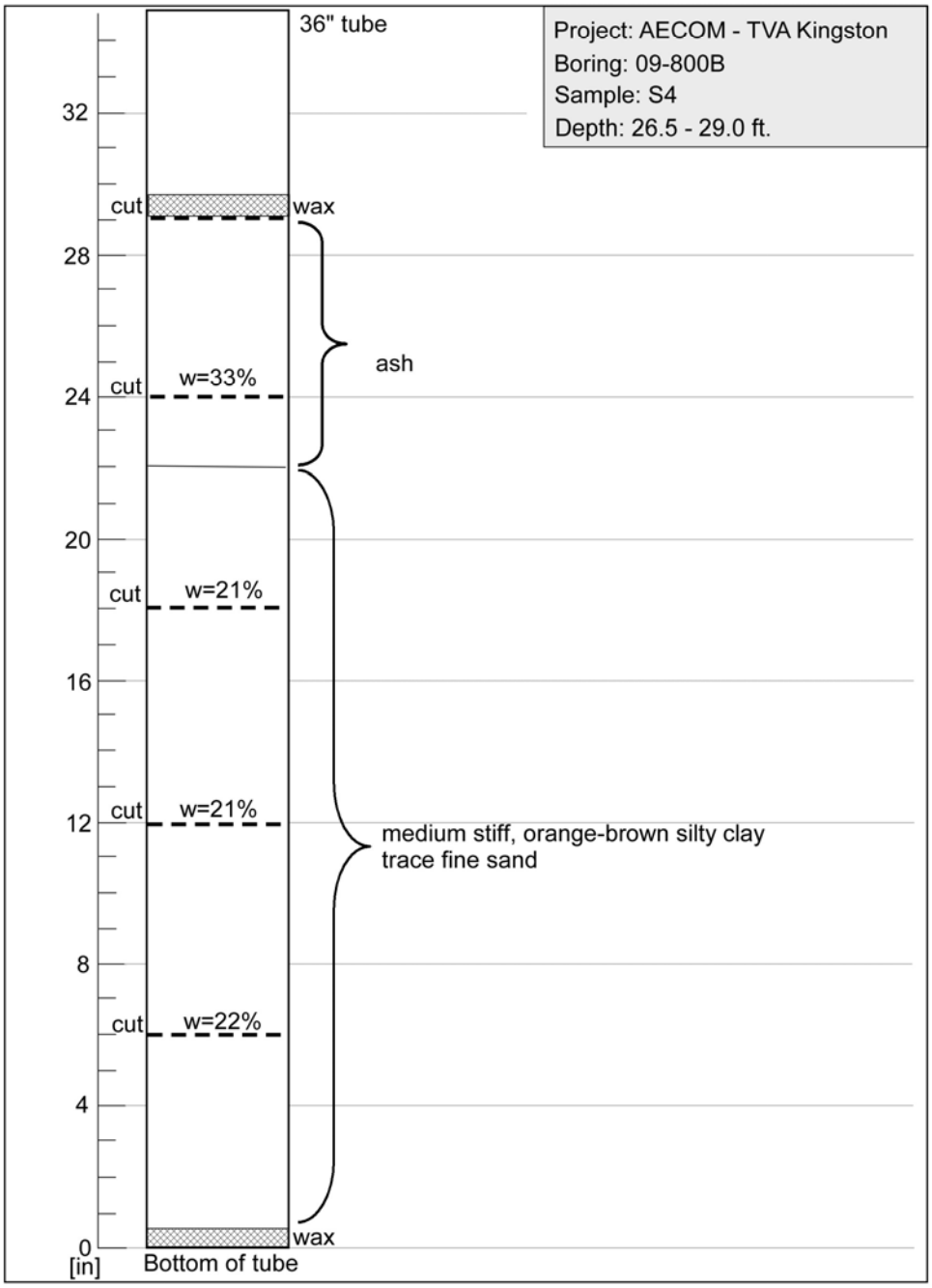


Figure 5.1.67 Sample log and test locations for Sample 09-800B S4 26.5-29.0 ft

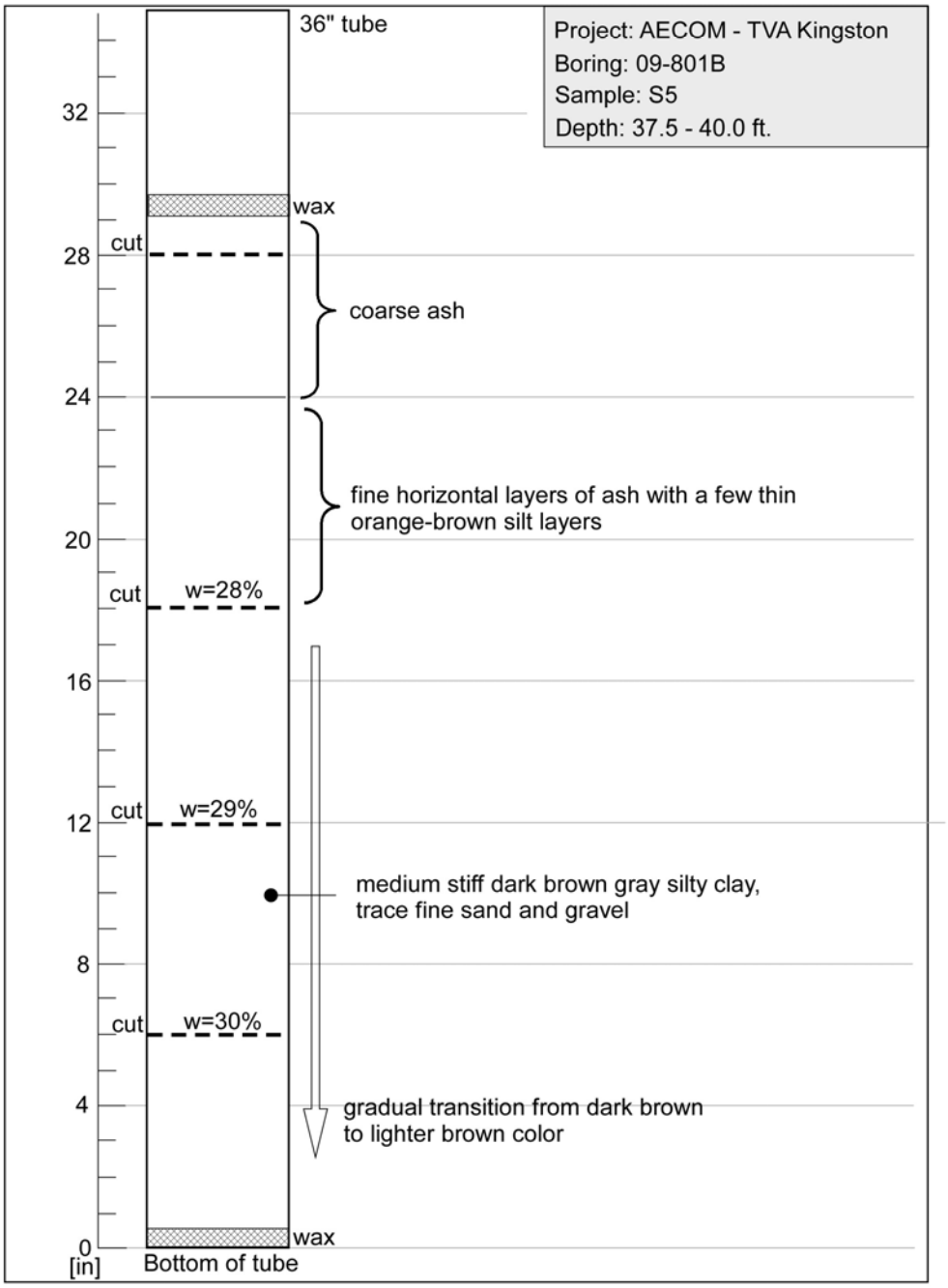


Figure 5.1.68 Sample log and test locations for Sample 09-801B S5 37.5-40.0 ft

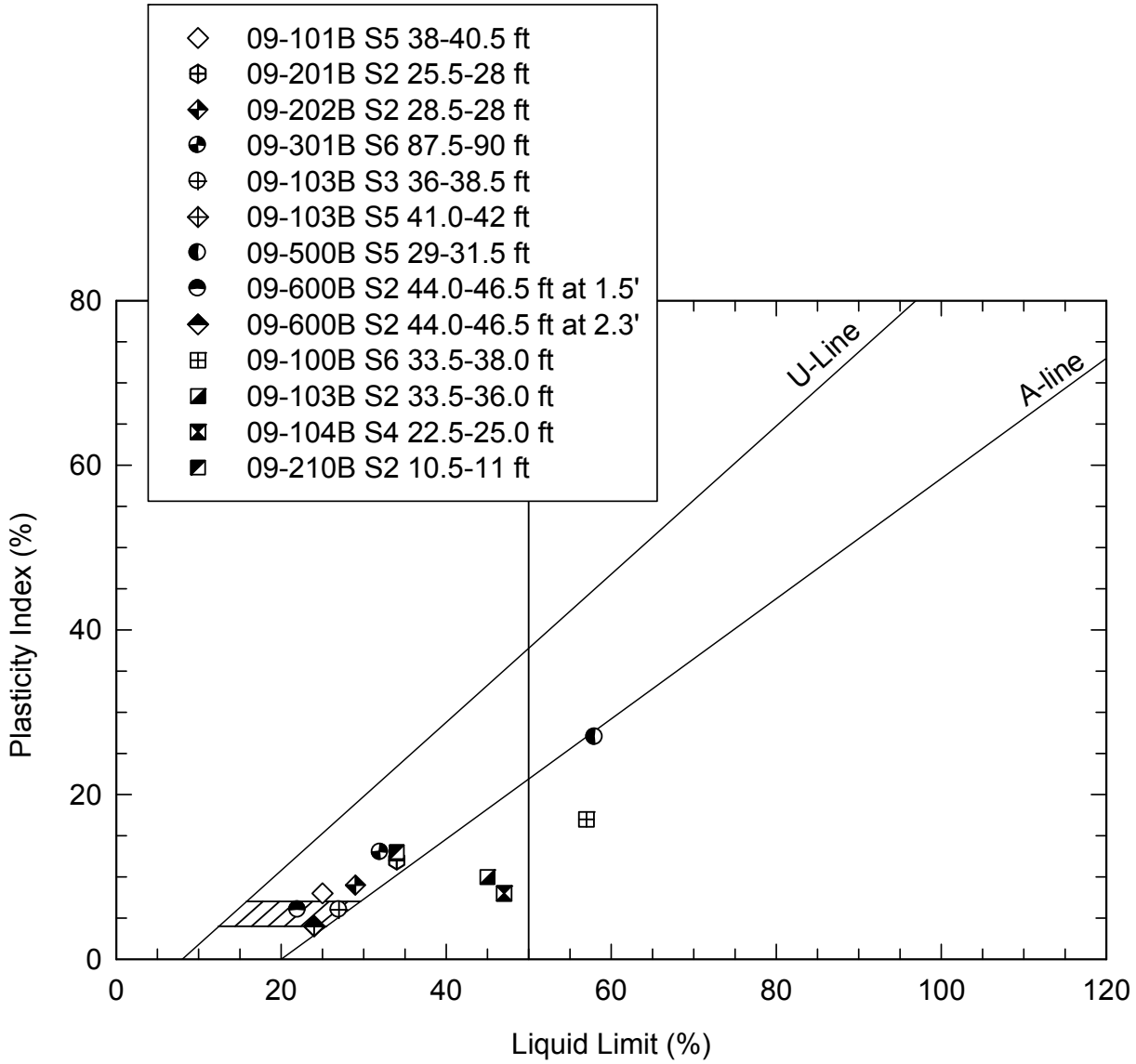


Figure 5.2.1 Plasticity Chart with Atterberg Limits results for selected samples (see Table 3 and sample logs for specific test specimen locations).

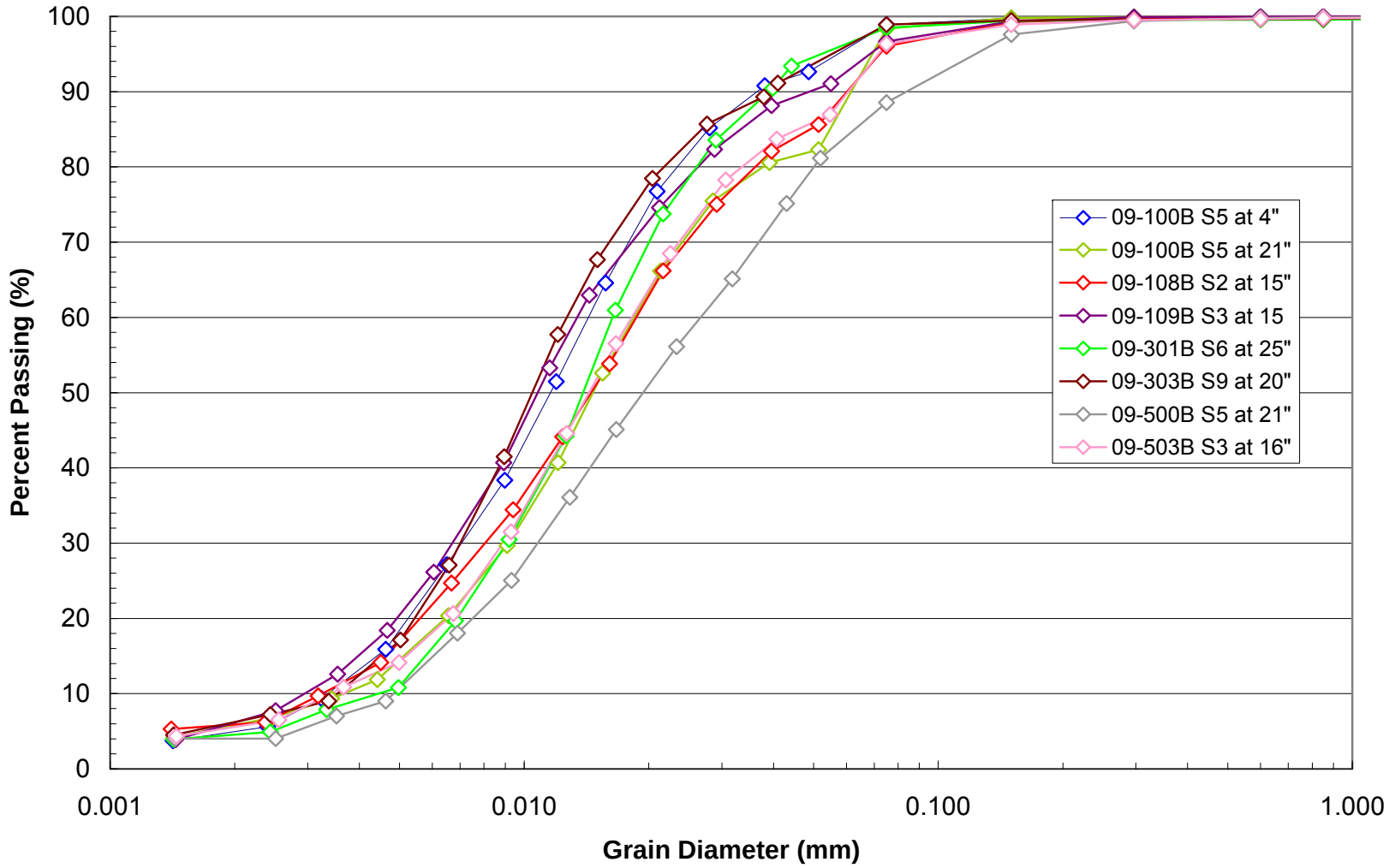


Figure 5.2.2 Percent passing versus grain diameter from grain size distribution test results (see sample logs for specific test specimen location).

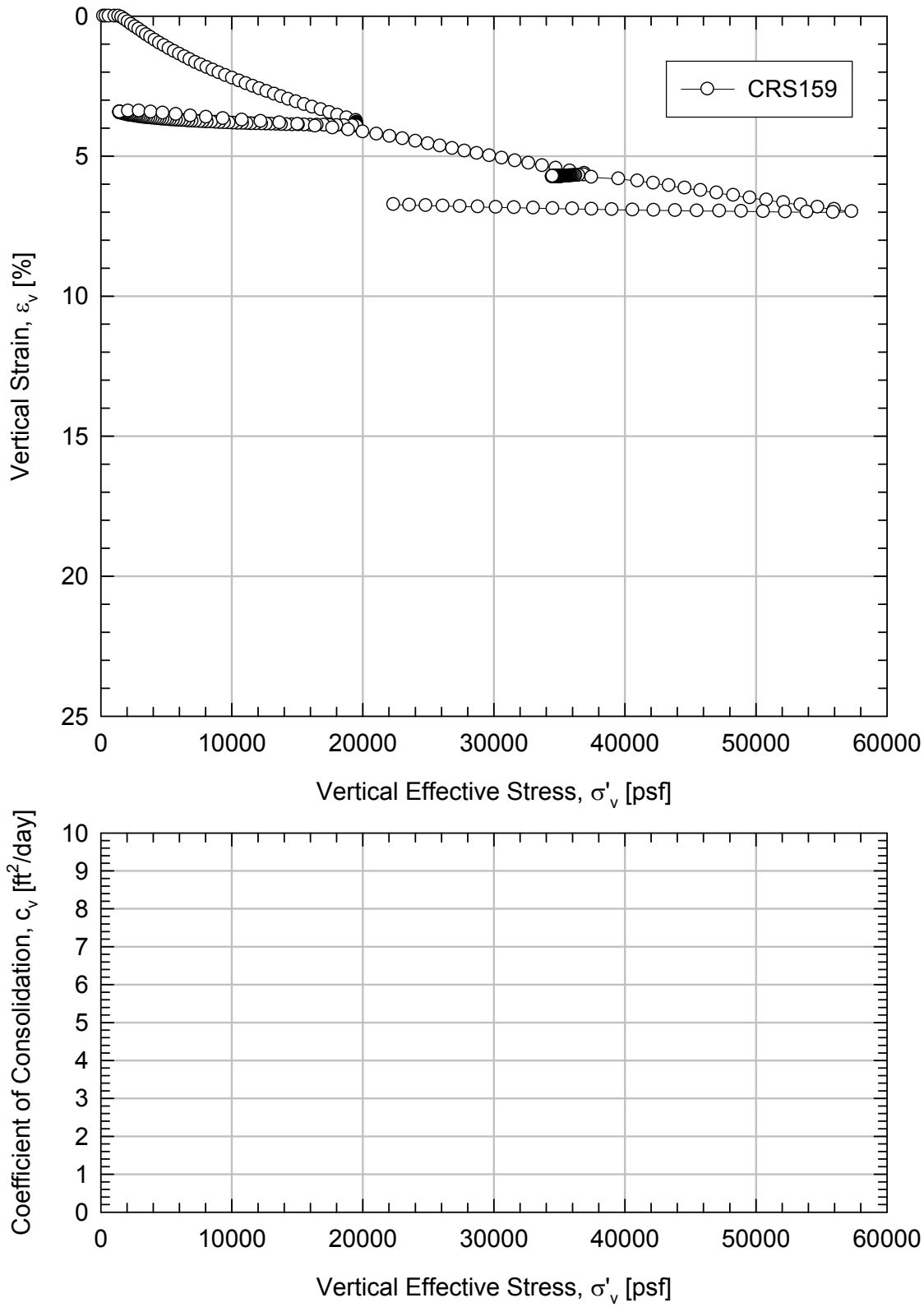


Figure 5.3.1 Vertical strain and coefficient of consolidation versus vertical effective stress for test CRS159 on sample 09-202B S2 28.5-31.0 ft.

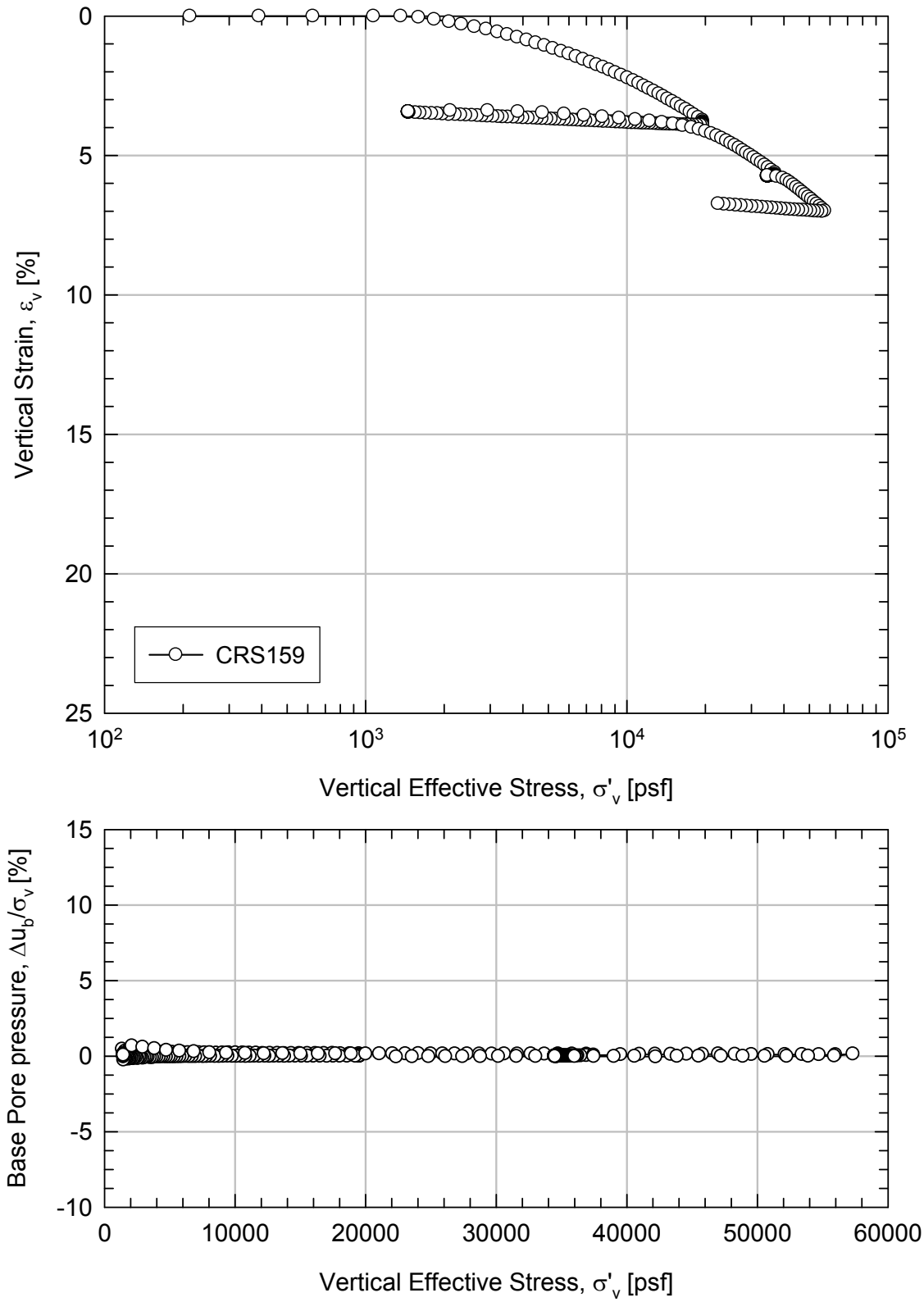


Figure 5.3.2 Vertical strain and normalized base pore pressure versus vertical effective stress for test CRS159 on sample 09-202B S2 28.5-31.0 ft.

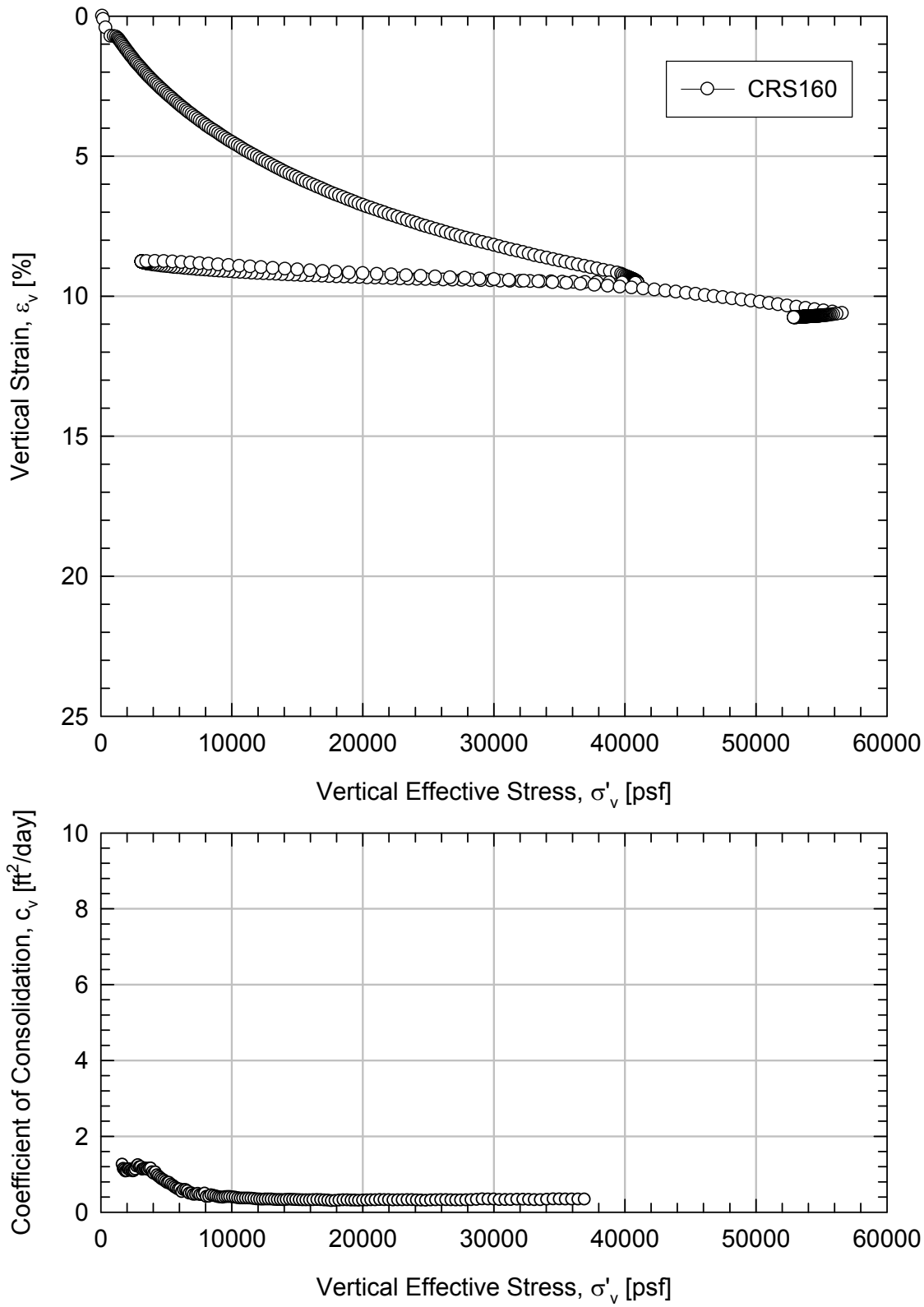


Figure 5.3.3 Vertical strain and coefficient of consolidation versus vertical effective stress for test CRS160 on sample 09-201B S2 25.5-28.0 ft.

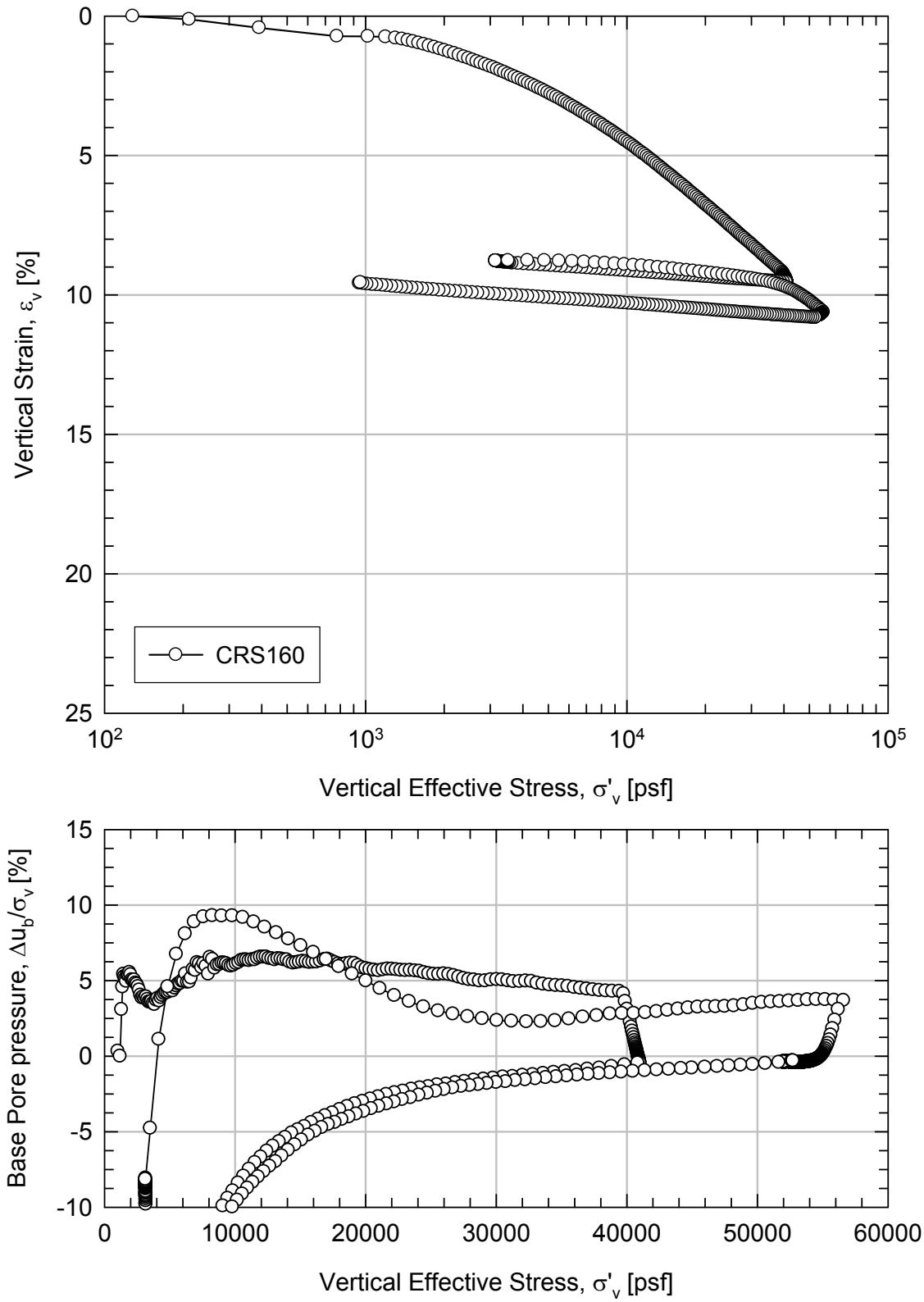


Figure 5.3.4 Vertical strain and normalized base pore pressure versus vertical effective stress for test CRS160 on sample 09-201B S2 25.5-28.0 ft.

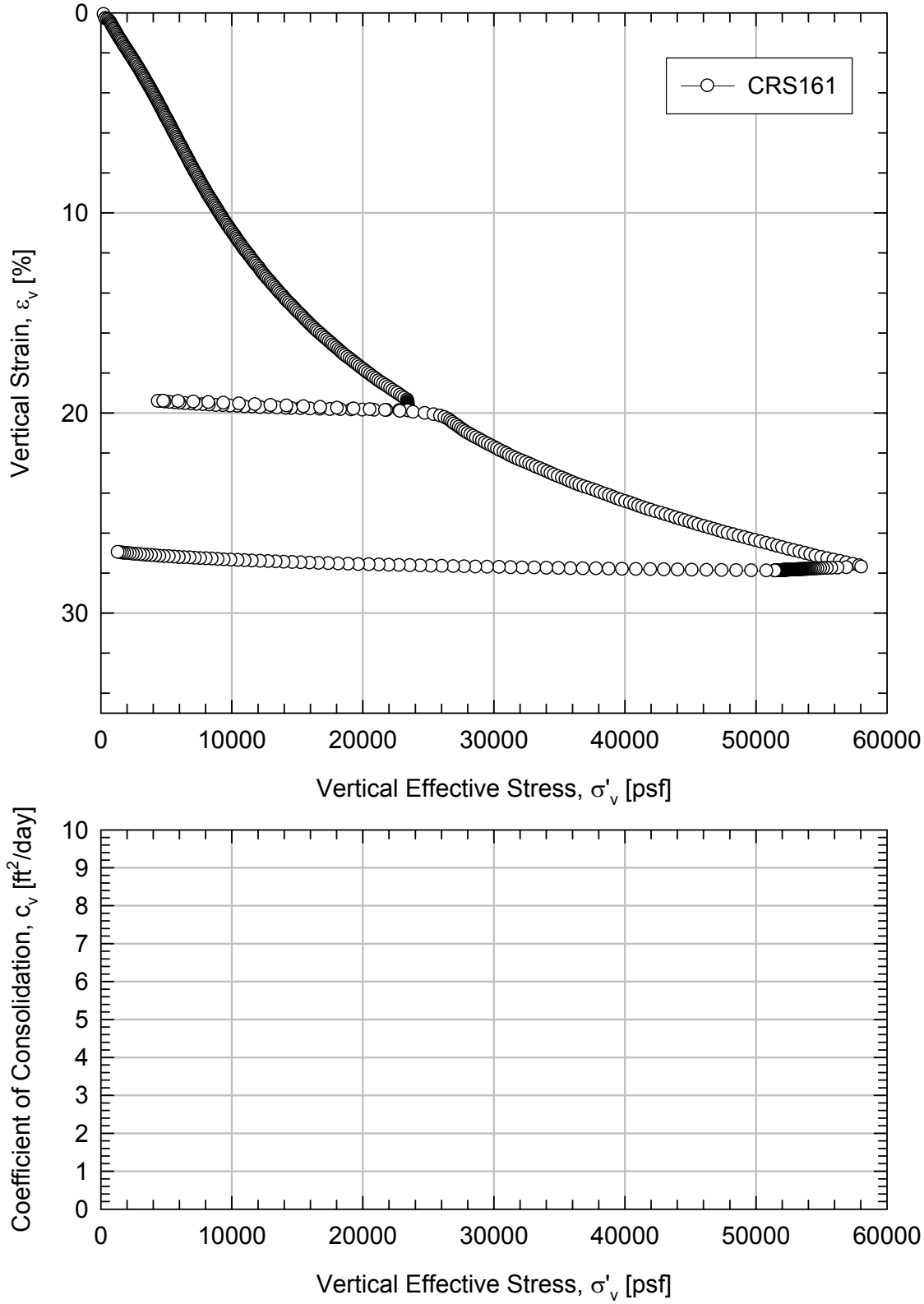


Figure 5.3.5 Vertical strain and coefficient of consolidation versus vertical effective stress for test CRS161 on sample 09-104B S4 22.5-25.0 ft.

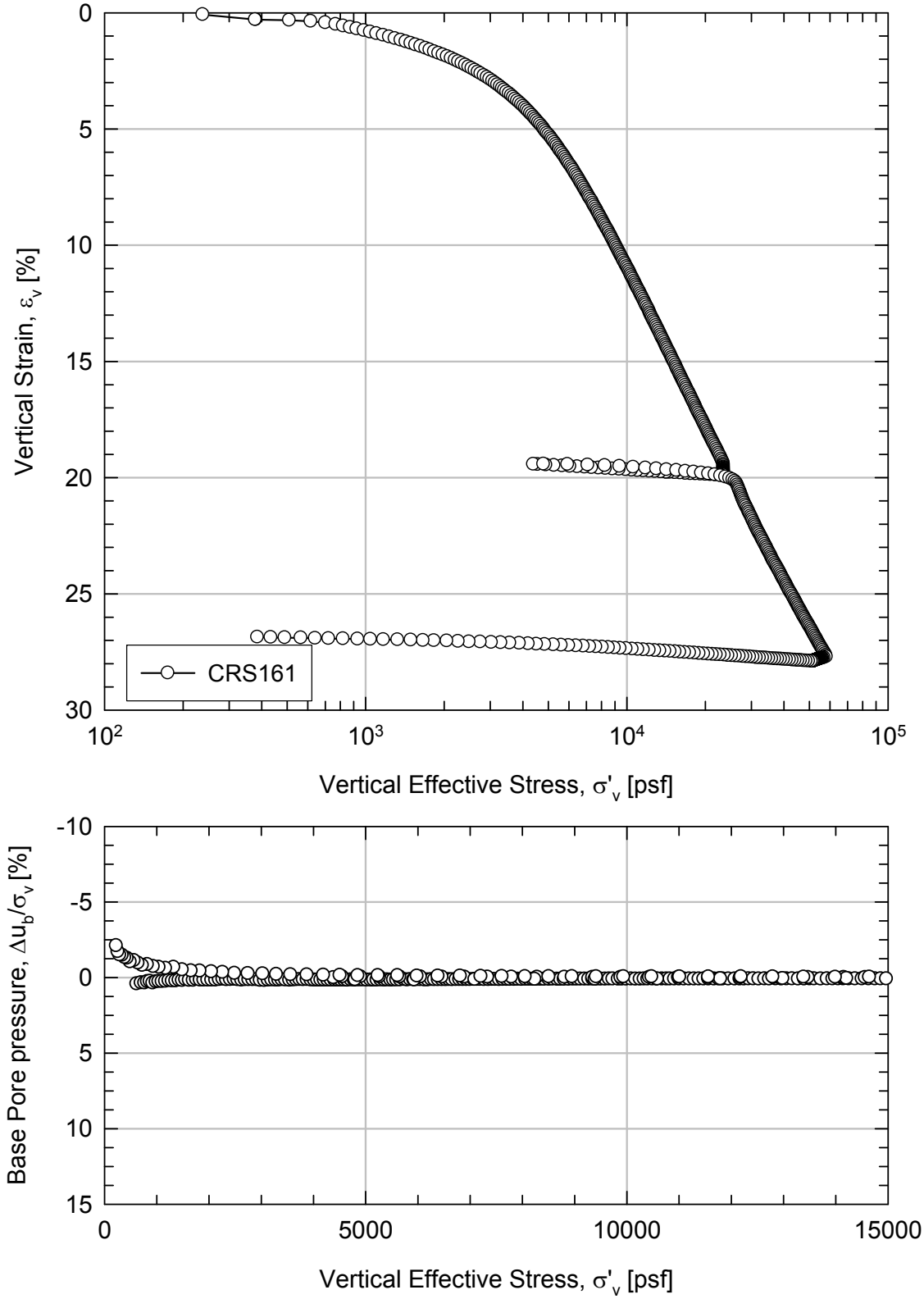


Figure 5.3.6 Vertical strain and normalized base pore pressure versus vertical effective stress for test CRS161 on sample 09-104B S4 22.5-25.0 ft.

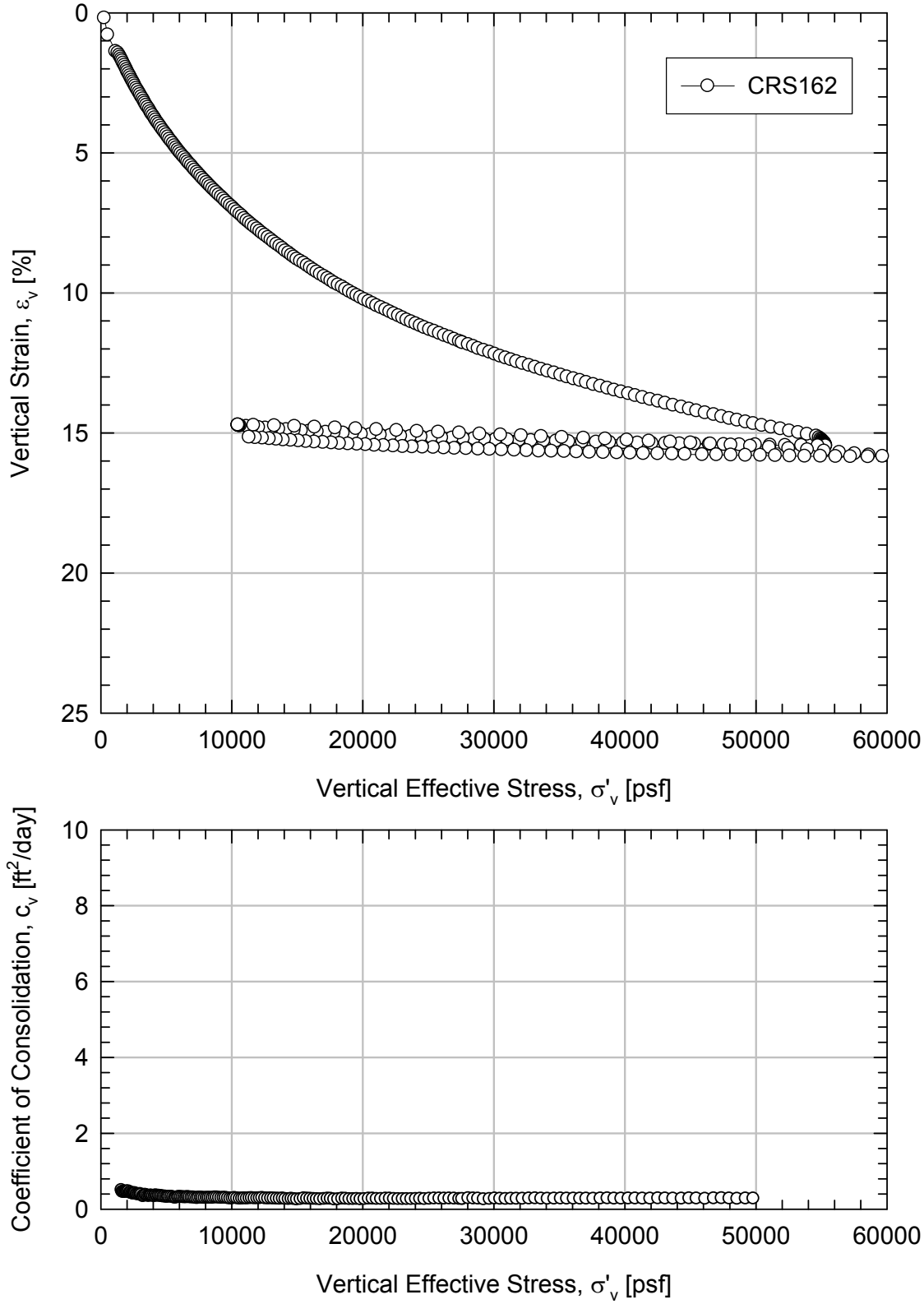


Figure 5.3.7 Vertical strain and coefficient of consolidation versus vertical effective stress for test CRS162 on sample 09-301B S6 87.5-90.0 ft.

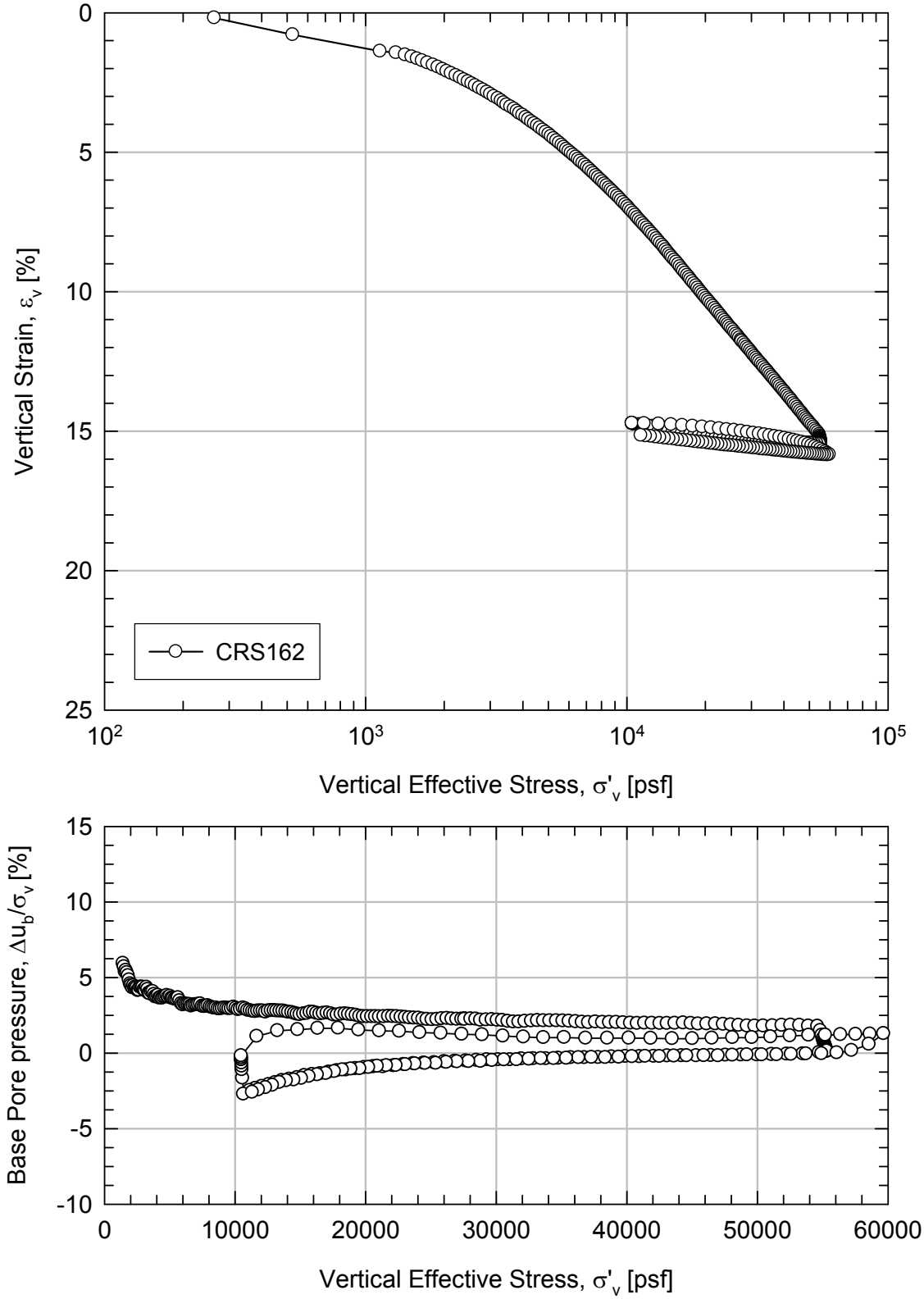


Figure 5.3.8 Vertical strain and normalized base pore pressure versus vertical effective stress for test CRS162 on sample 09-301B S6 87.5-90.0 ft.

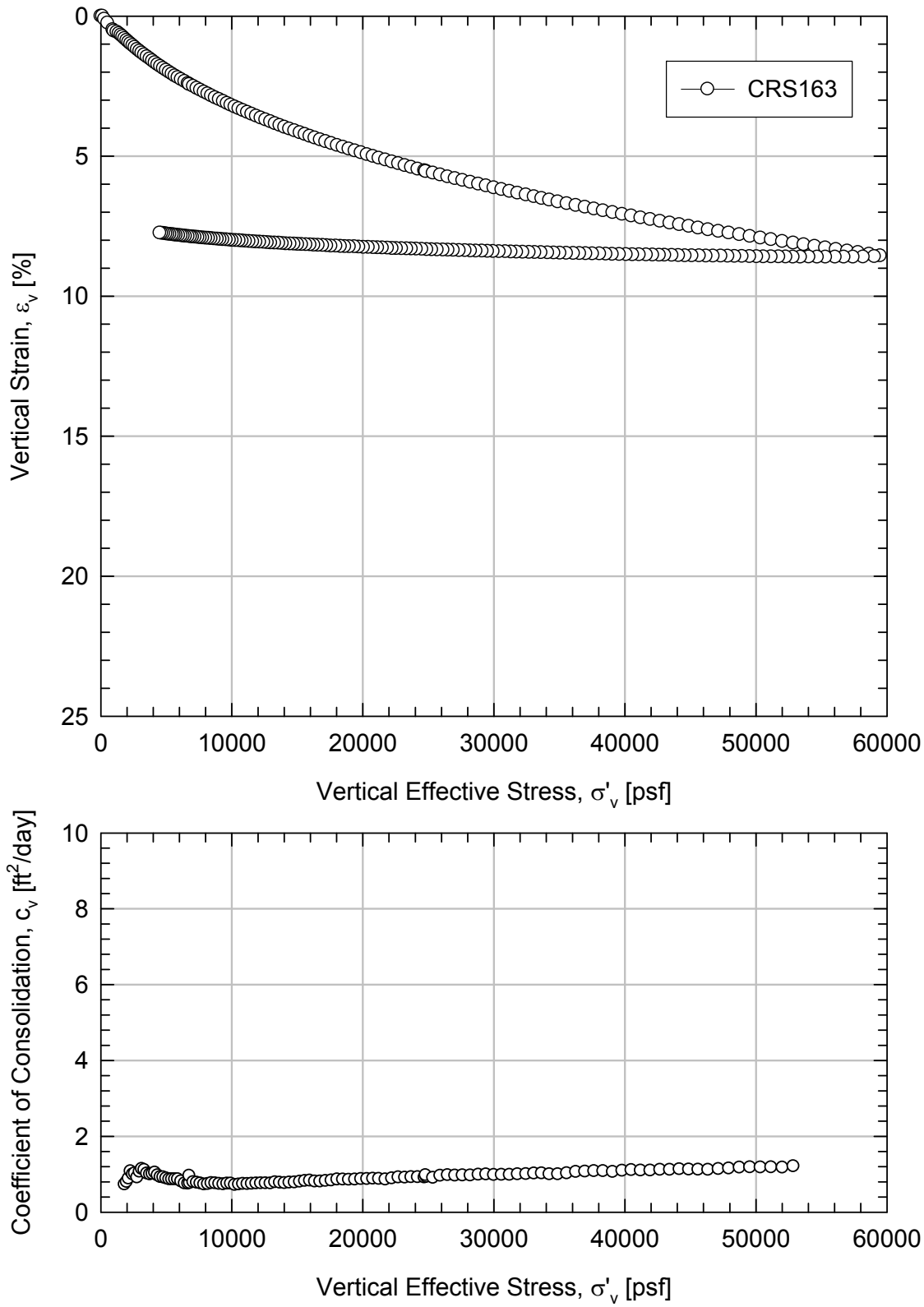


Figure 5.3.9 Vertical strain and coefficient of consolidation versus vertical effective stress for test CRS163 on sample 09-101B S5 38.0-40.5 ft.

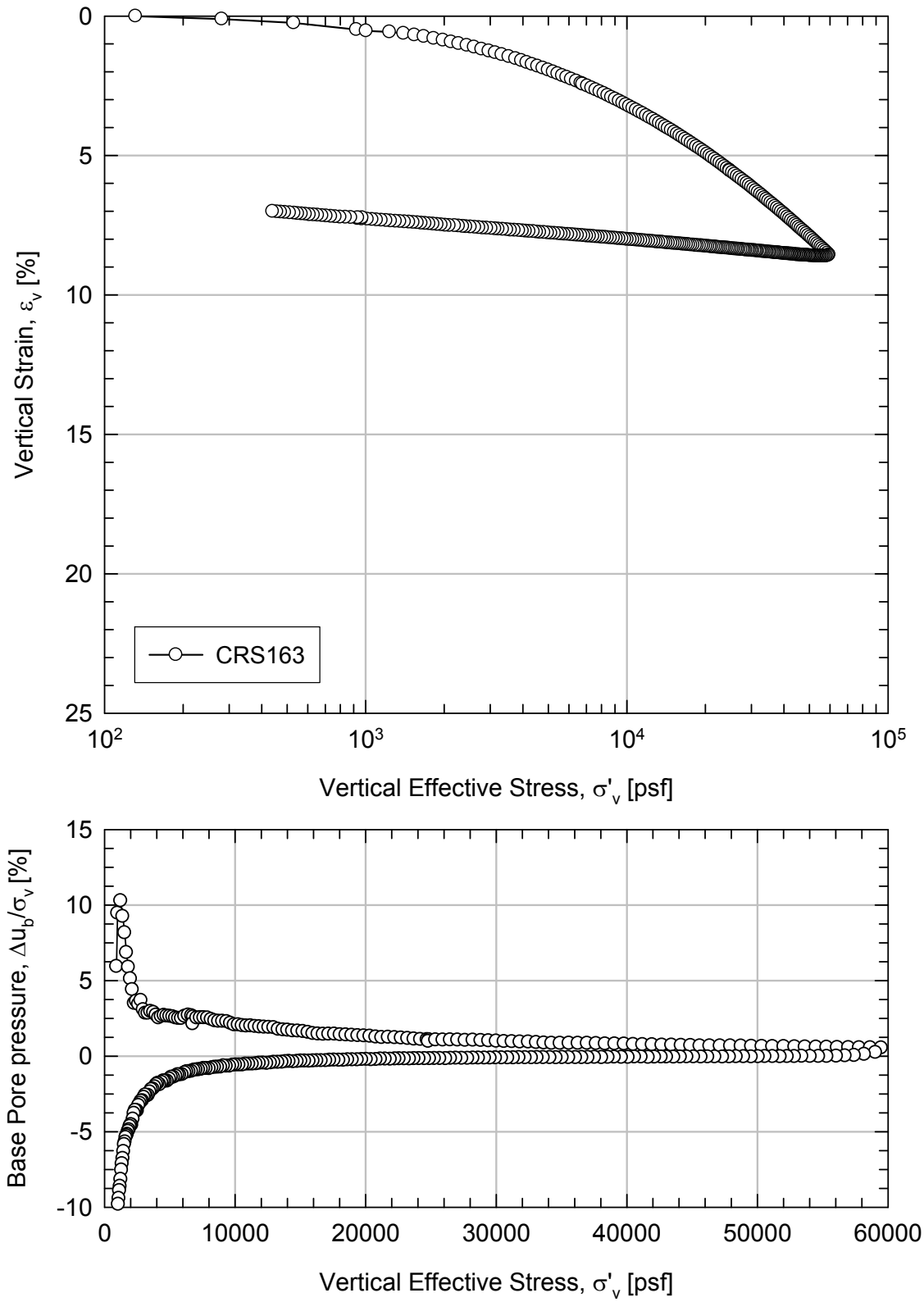


Figure 5.3.10 Vertical strain and normalized base pore pressure versus vertical effective stress for test CRS163 on sample 09-101B S5 38.0-40.5 ft.

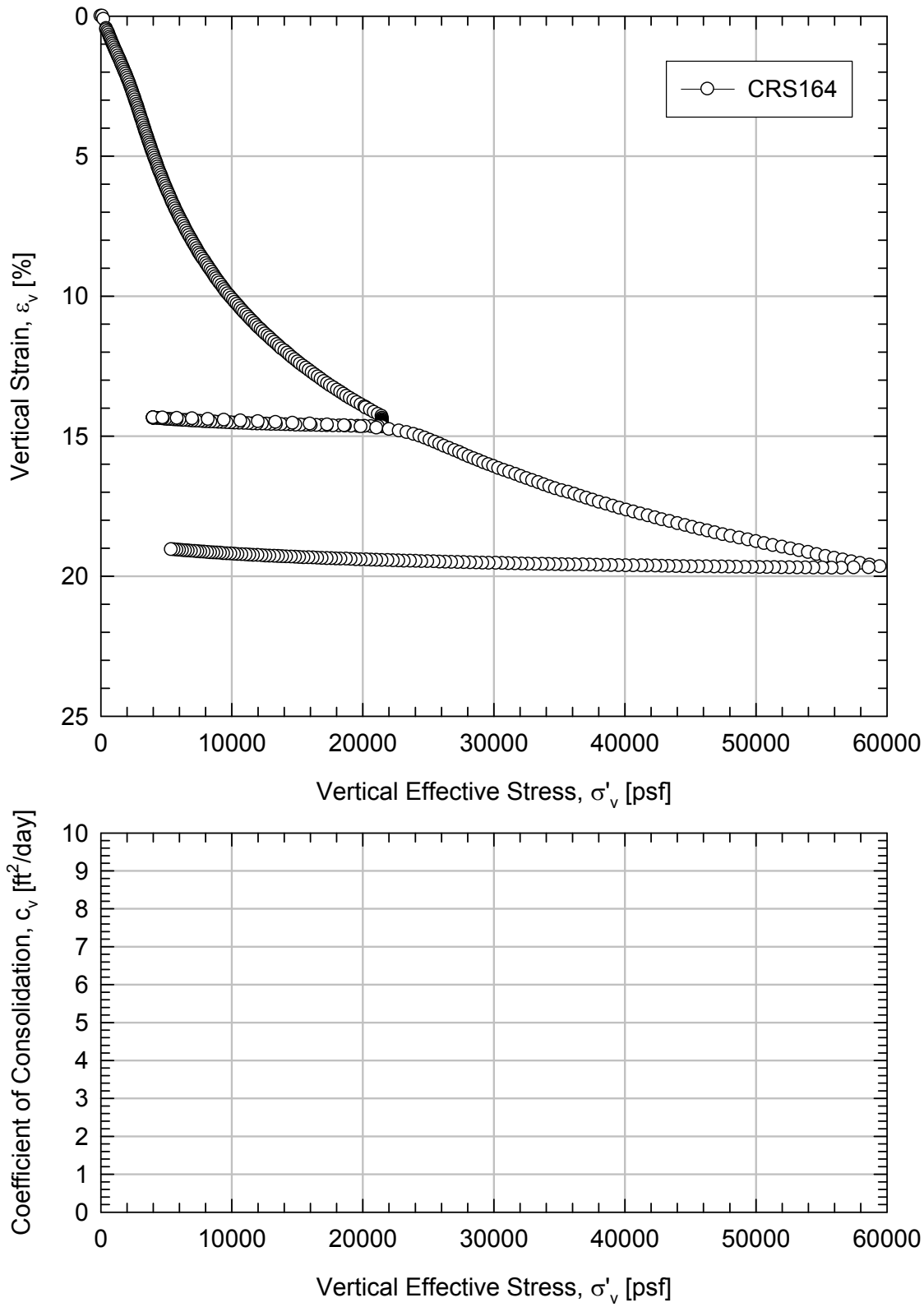


Figure 5.3.11 Vertical strain and coefficient of consolidation versus vertical effective stress for test CRS164 on sample 09-108B S2 23.5-26.0 ft.

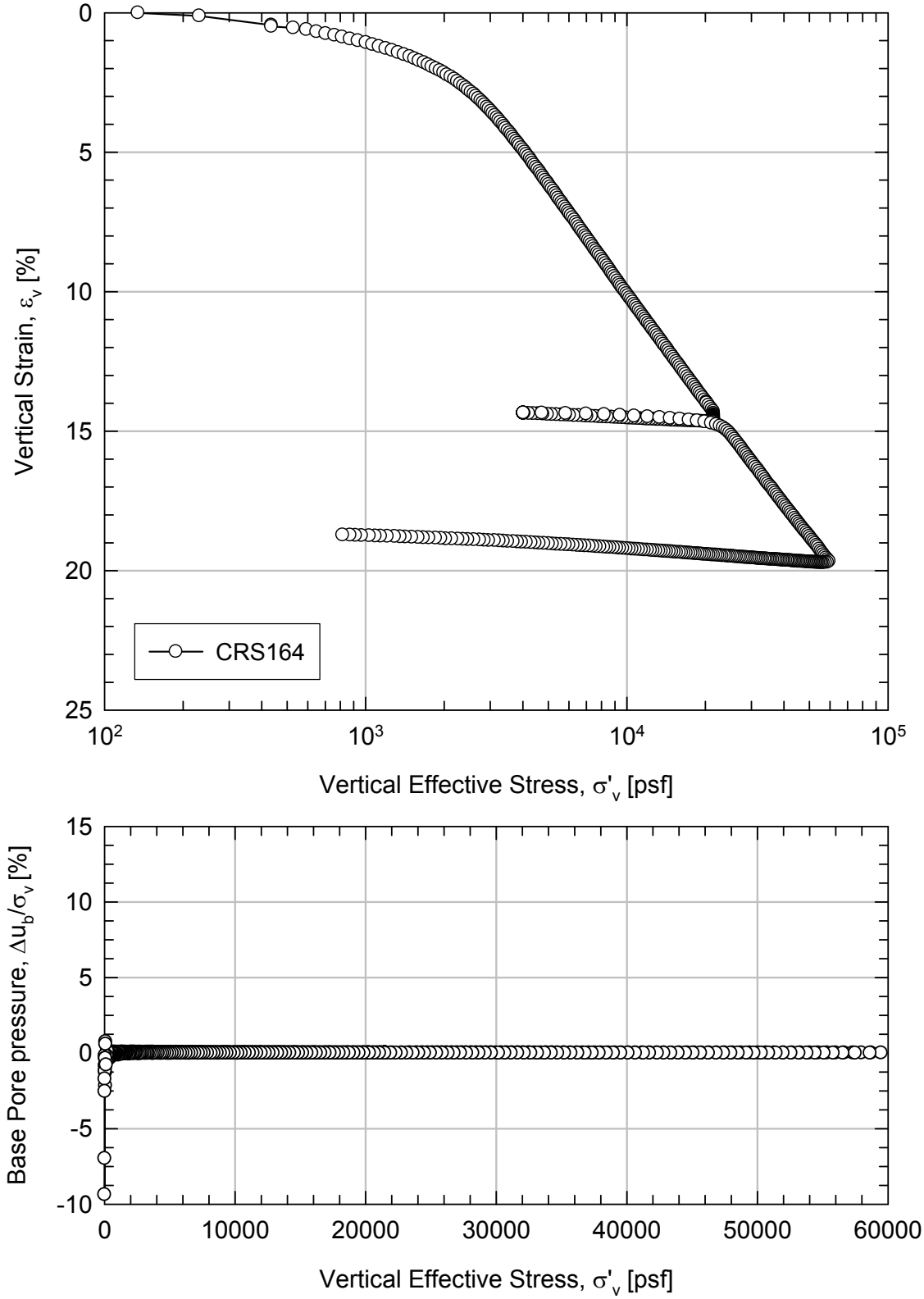


Figure 5.3.12 Vertical strain and normalized base pore pressure versus vertical effective stress for test CRS164 on sample 09-108B S2 23.5-26.0 ft.

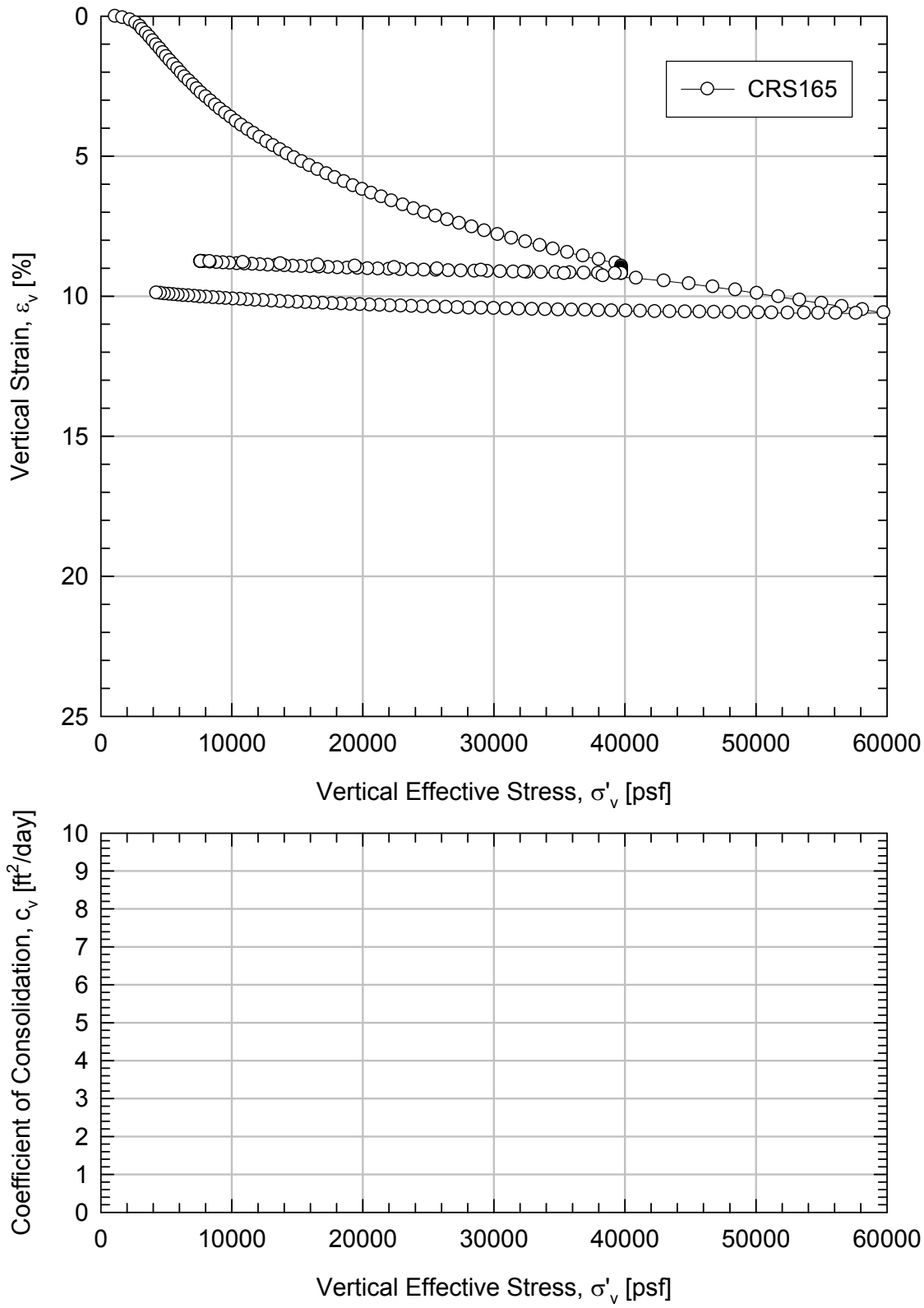


Figure 5.3.13 Vertical strain and coefficient of consolidation versus vertical effective stress for test CRS165 on sample 09-605B S7 47.0-49.5 ft.

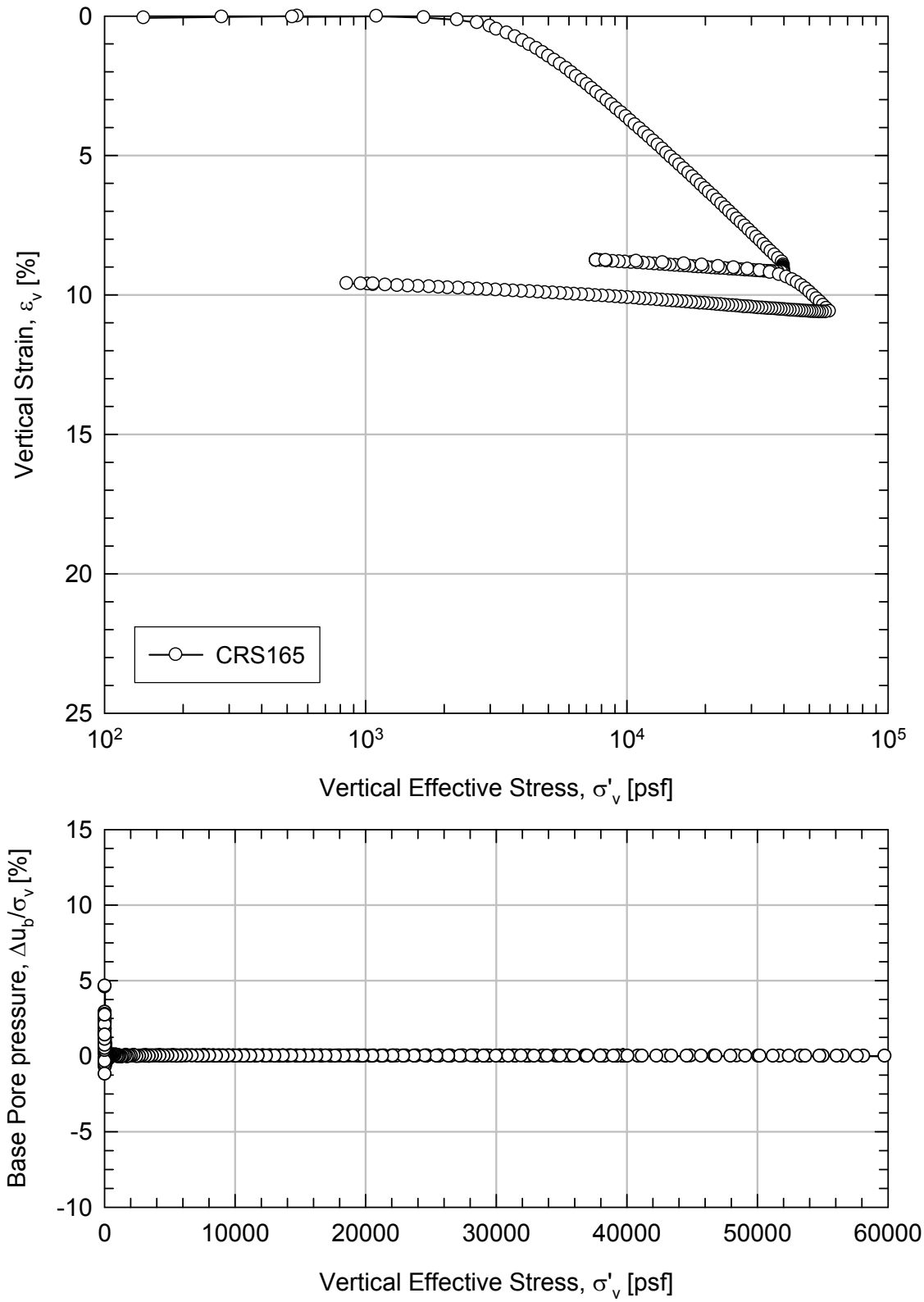


Figure 5.3.14 Vertical strain and normalized base pore pressure versus vertical effective stress for test CRS165 on sample 09-605B S7 47.0-49.5 ft.

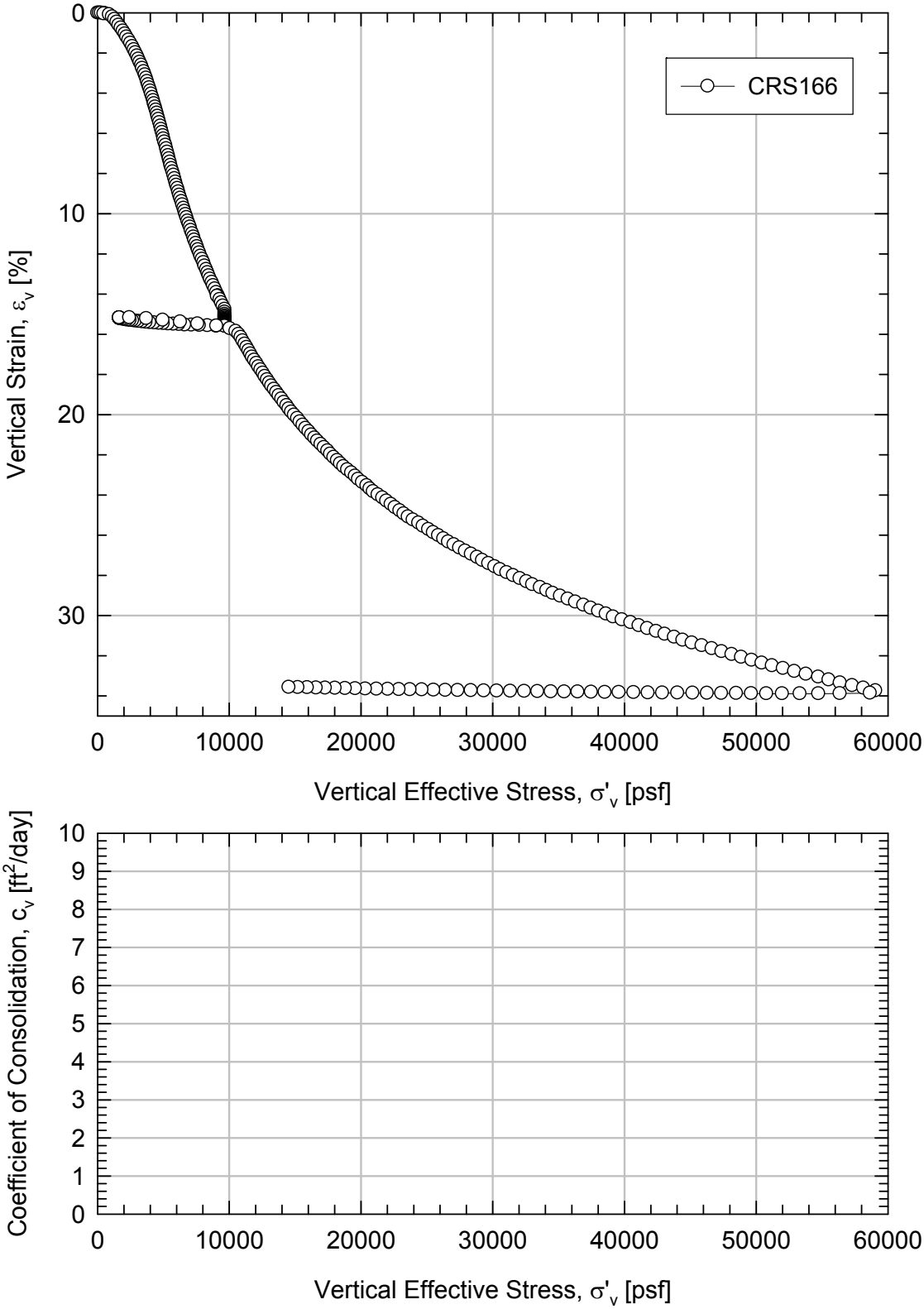


Figure 5.3.15 Vertical strain and coefficient of consolidation versus vertical effective stress for test CRS166 on sample 09-103B S2 33.5-36.0 ft.

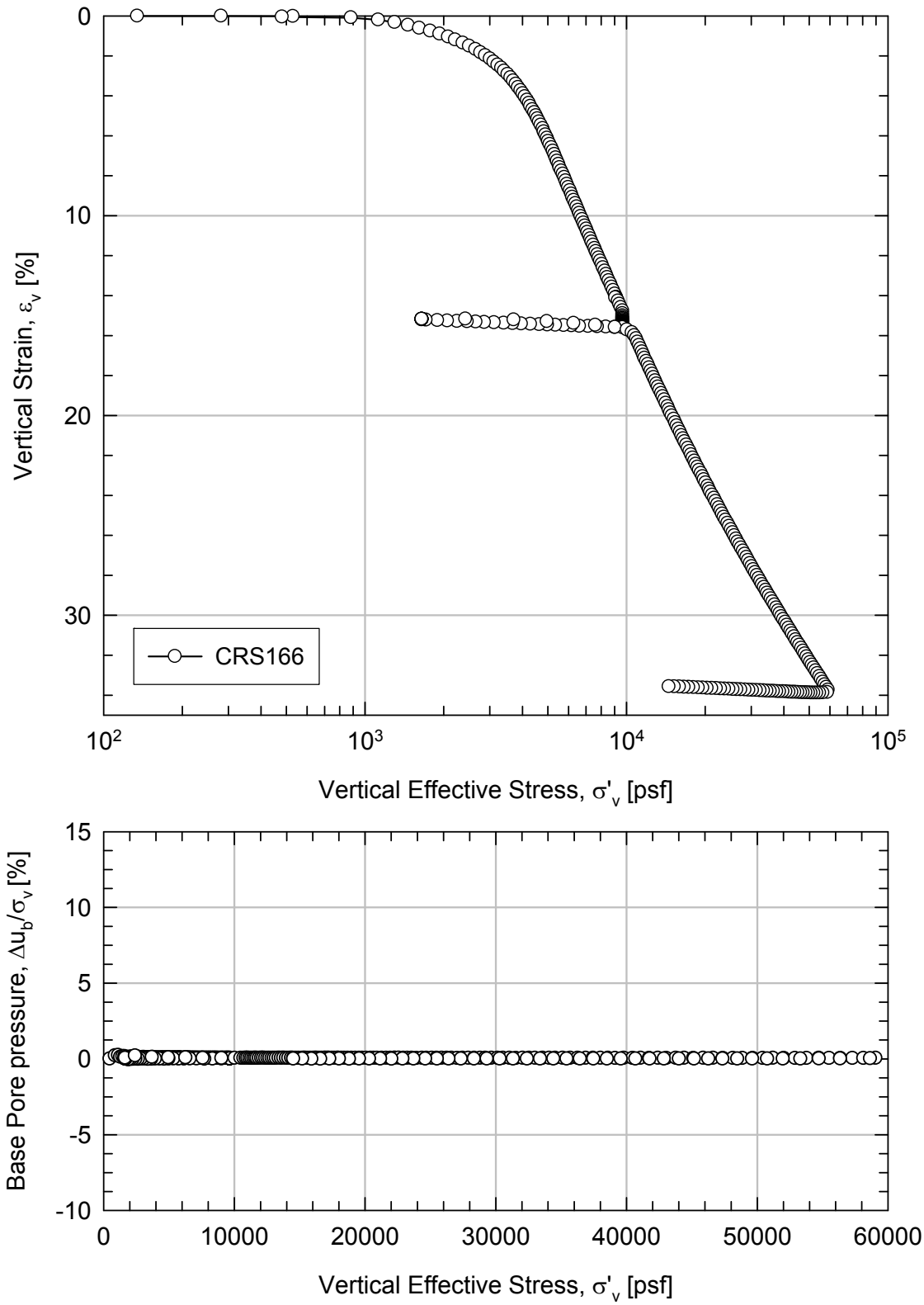


Figure 5.3.16 Vertical strain and normalized base pore pressure versus vertical effective stress for test CRS166 on sample 09-103B S2 33.5-36.0 ft.

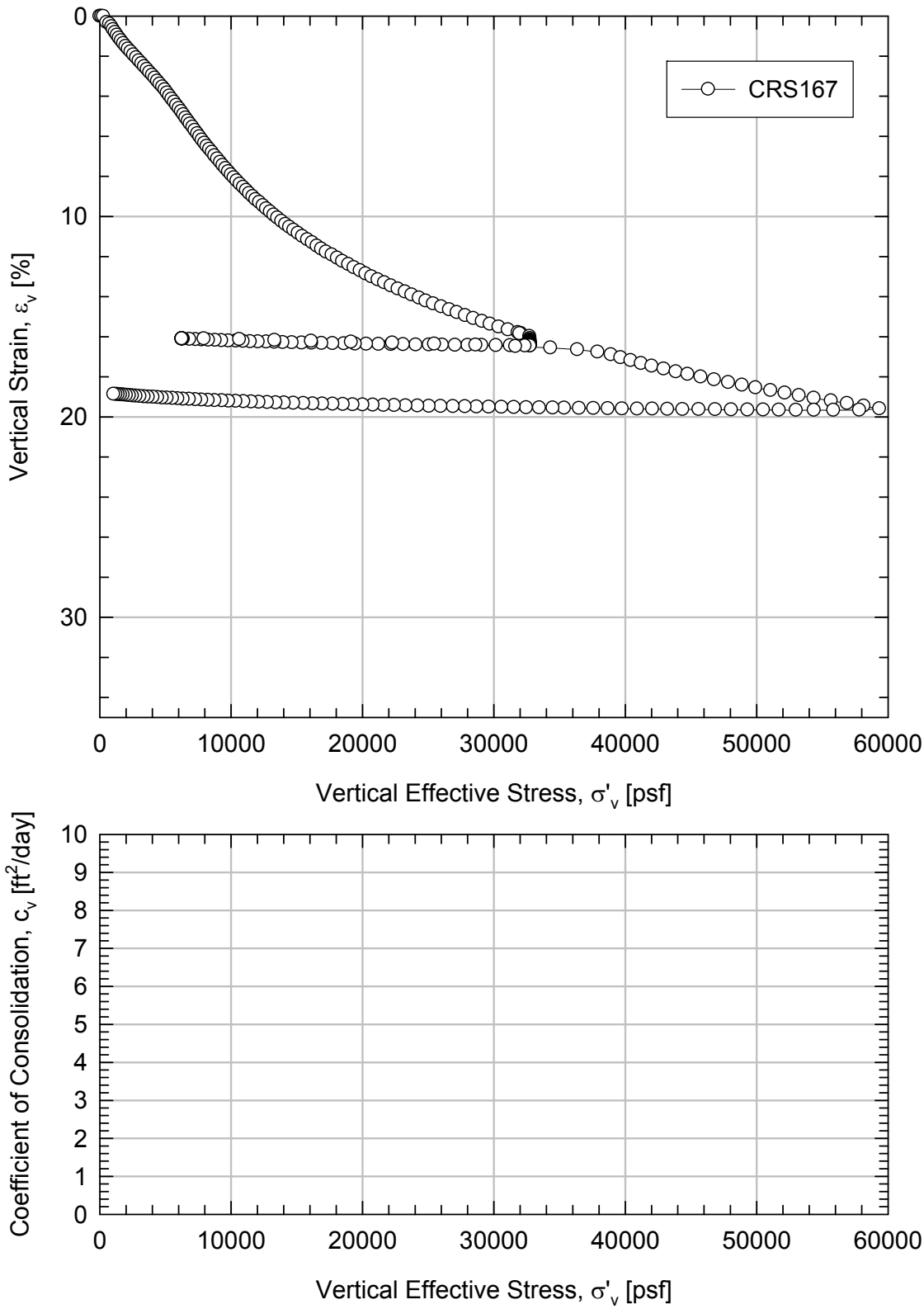


Figure 5.3.17 Vertical strain and coefficient of consolidation versus vertical effective stress for test CRS167 on sample 09-100B S5 33.0-35.5 ft.

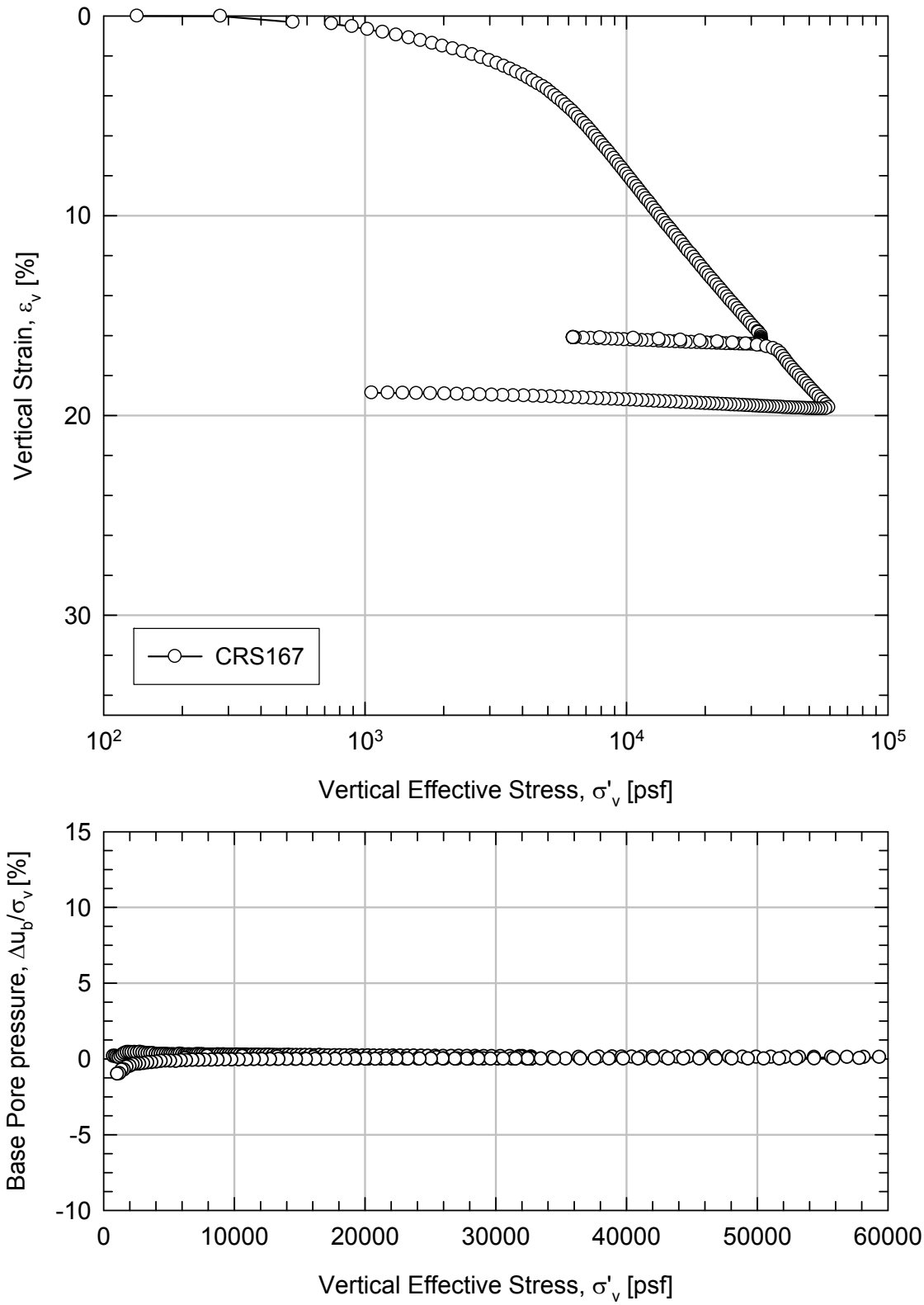


Figure 5.3.18 Vertical strain and normalized base pore pressure versus vertical effective stress for test CRS167 on sample 09-100B S5 33.0-35.5 ft.

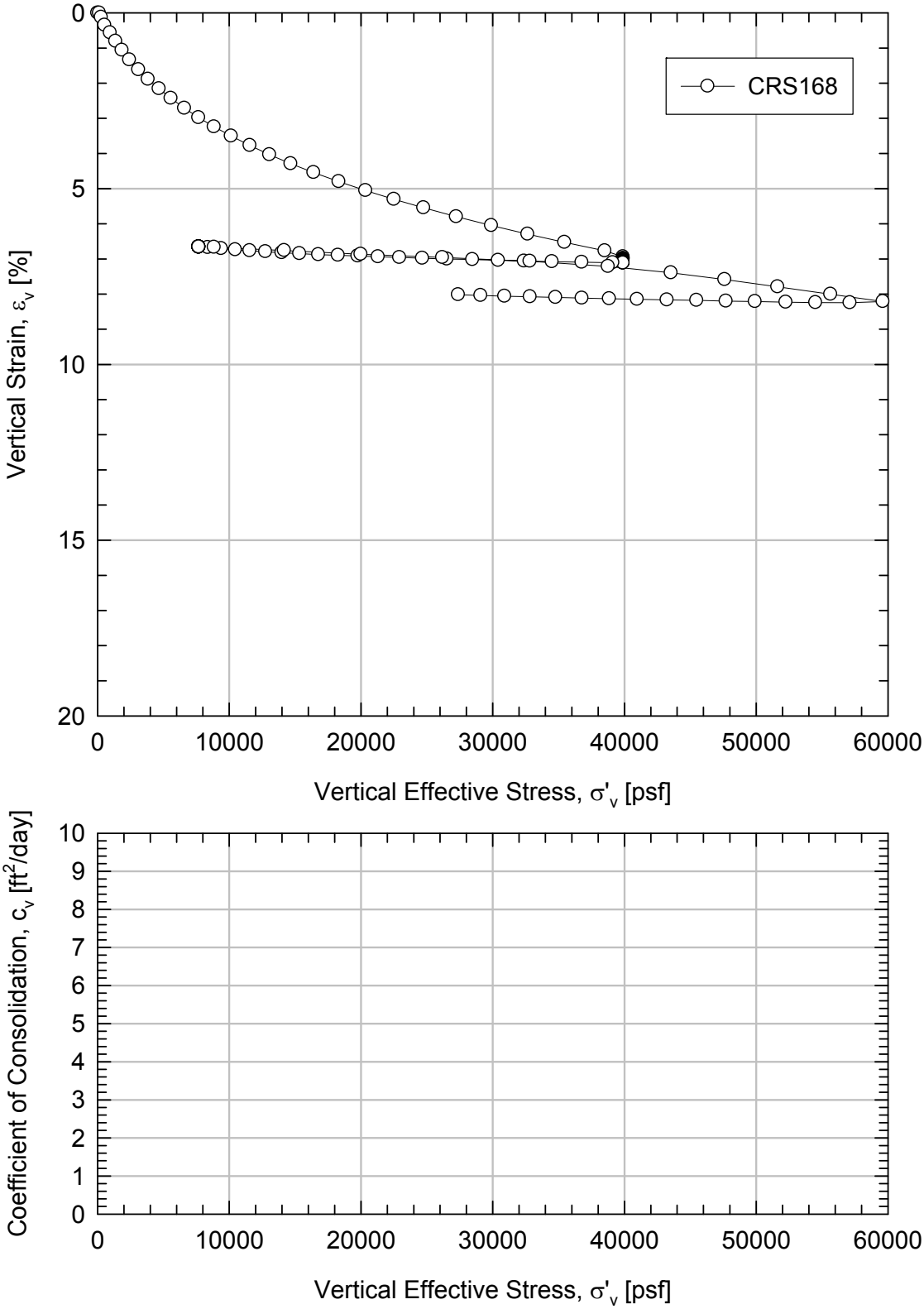


Figure 5.3.19 Vertical strain and coefficient of consolidation versus vertical effective stress for test CRS168 on sample 09-103B S5 41.0-43.5 ft.

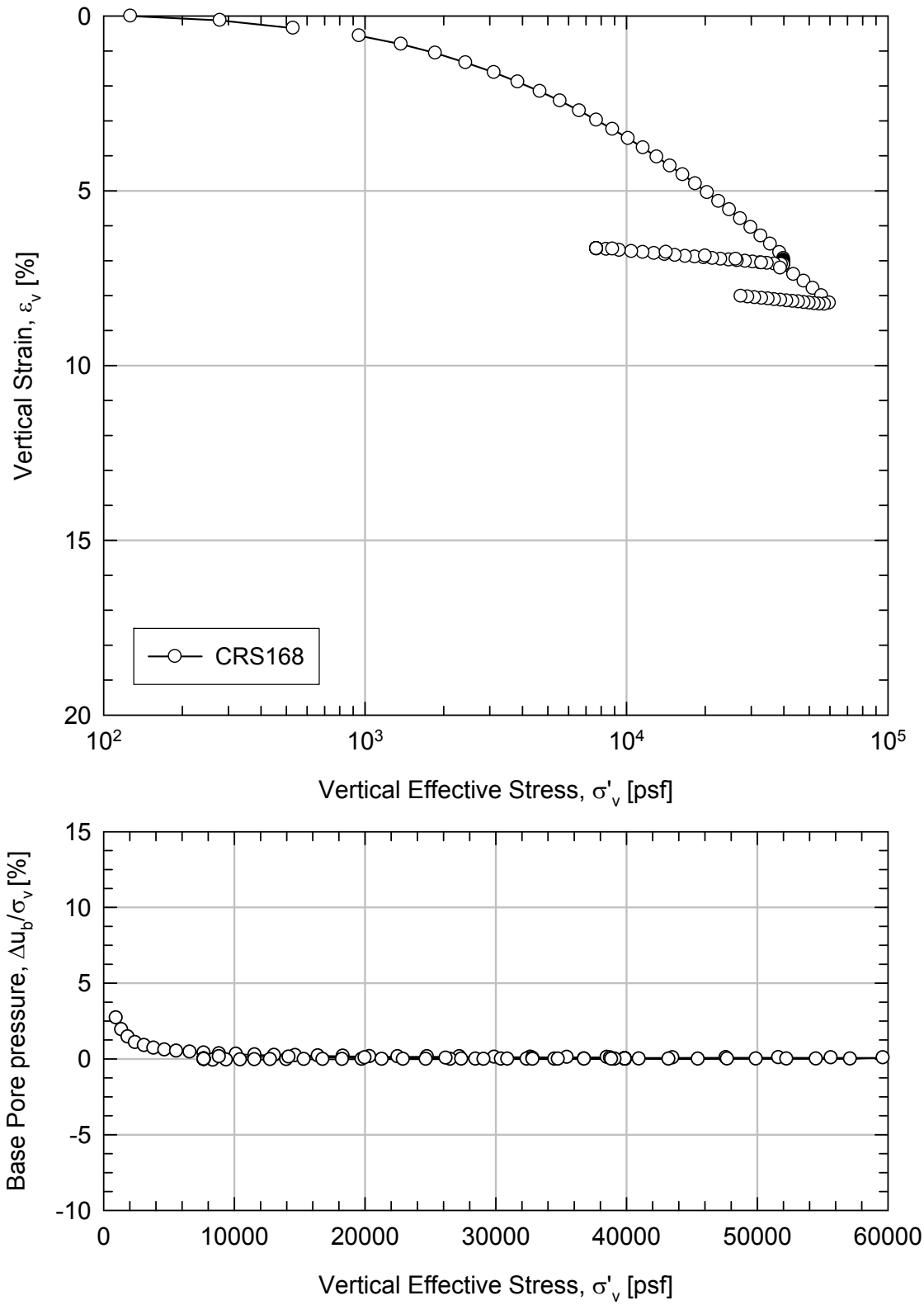


Figure 5.3.20 Vertical strain and normalized base pore pressure versus vertical effective stress for test CRS168 on sample 09-103B S5 41.0-43.5 ft.

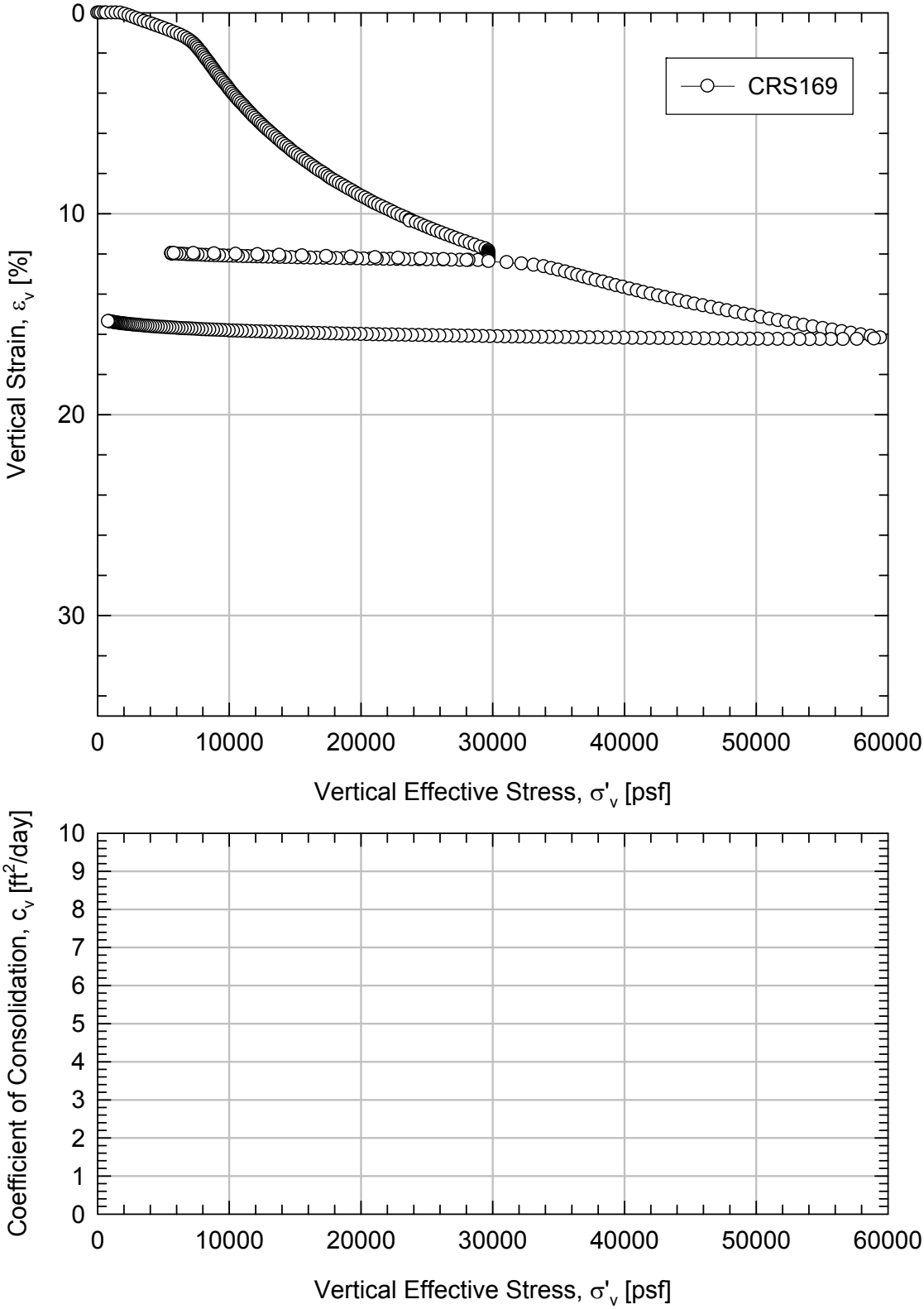


Figure 5.3.21 Vertical strain and coefficient of consolidation versus vertical effective stress for test CRS169 on sample 09-408B S7 35.5-38.0 ft.

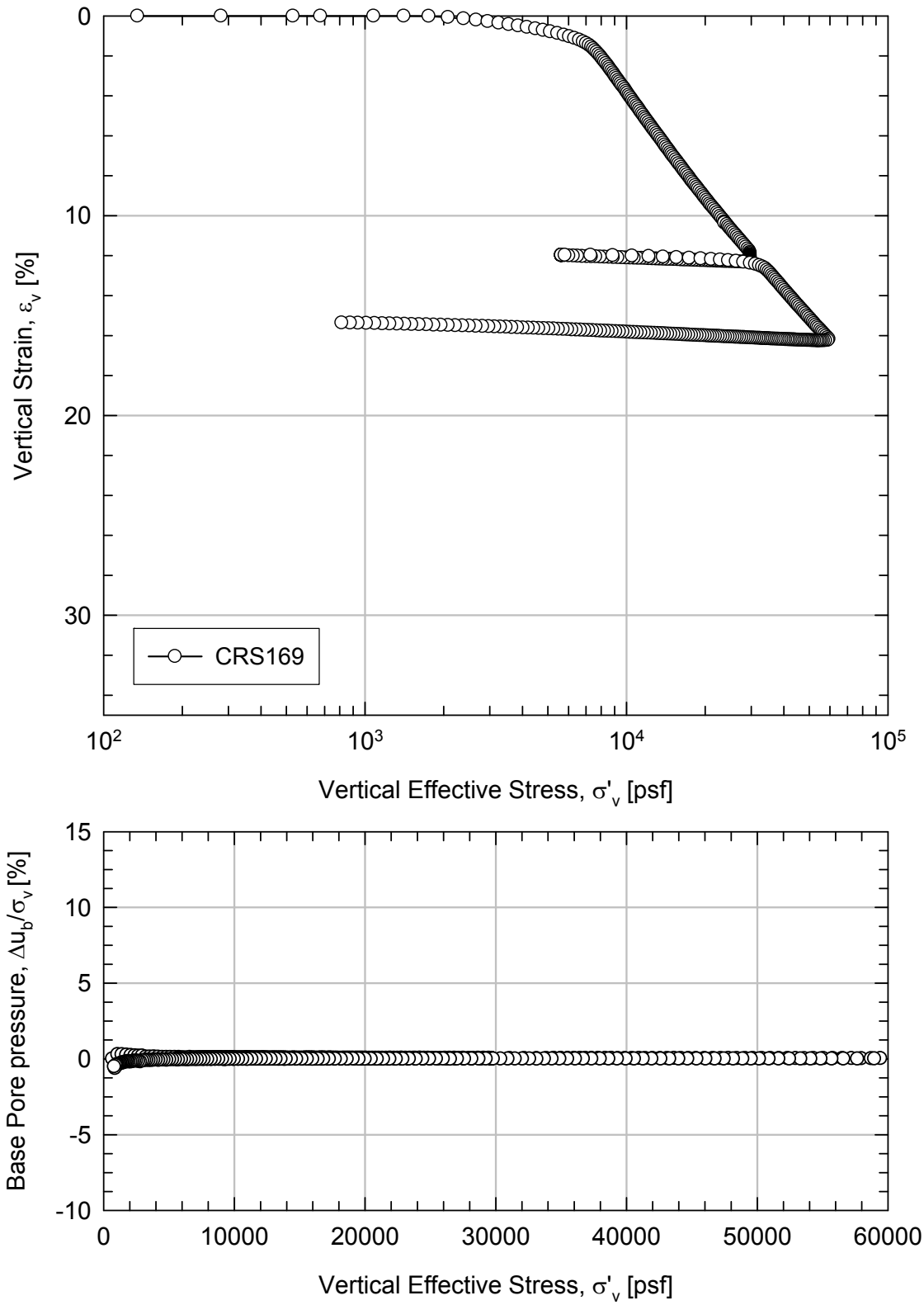


Figure 5.3.22 Vertical strain and normalized base pore pressure versus vertical effective stress for test CRS169 on sample 09-408B S7 35.5-38.0 ft.

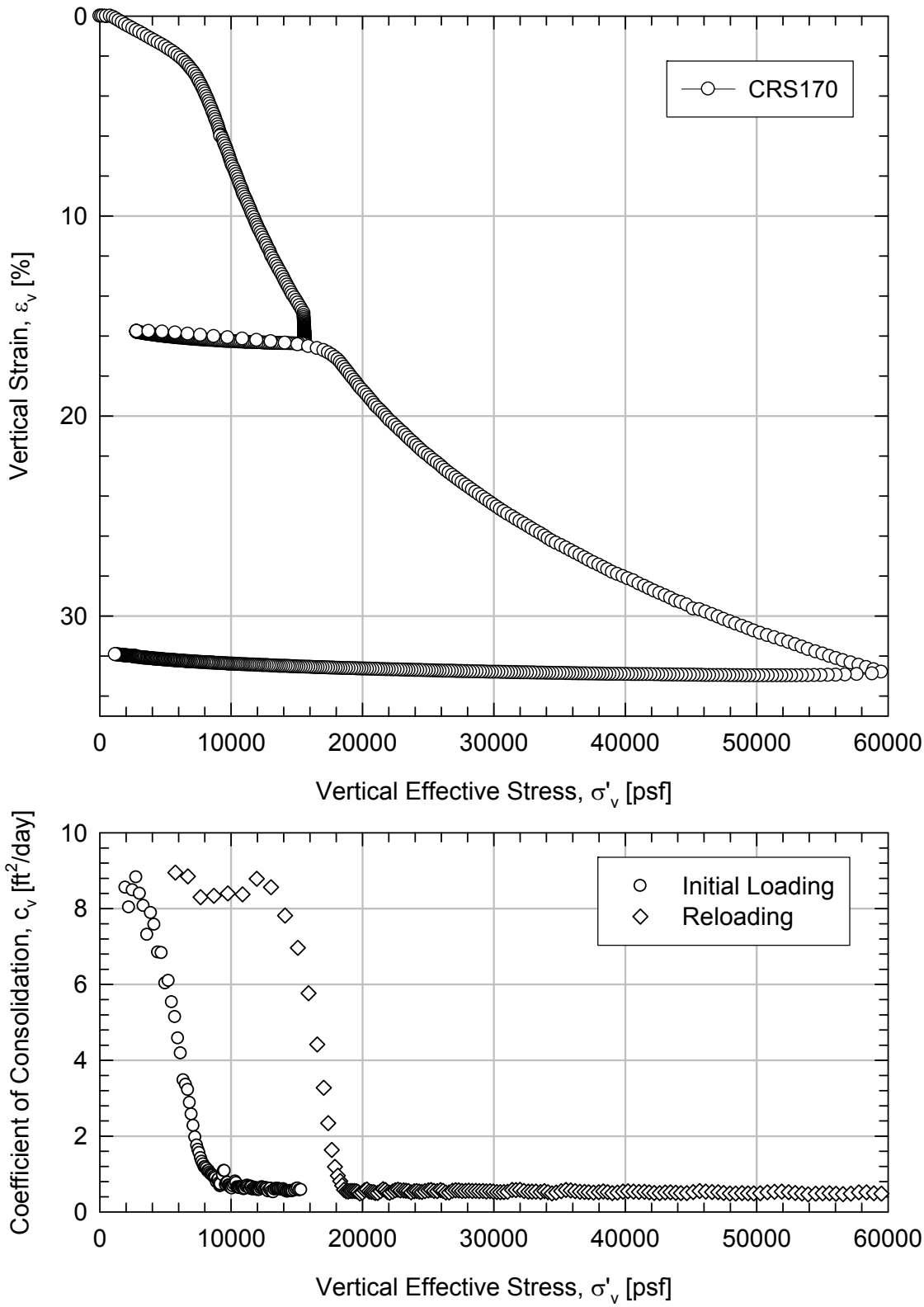


Figure 5.3.23 Vertical strain and coefficient of consolidation versus vertical effective stress for test CRS170 on sample 09-503B S3 35.5-38.0 ft.

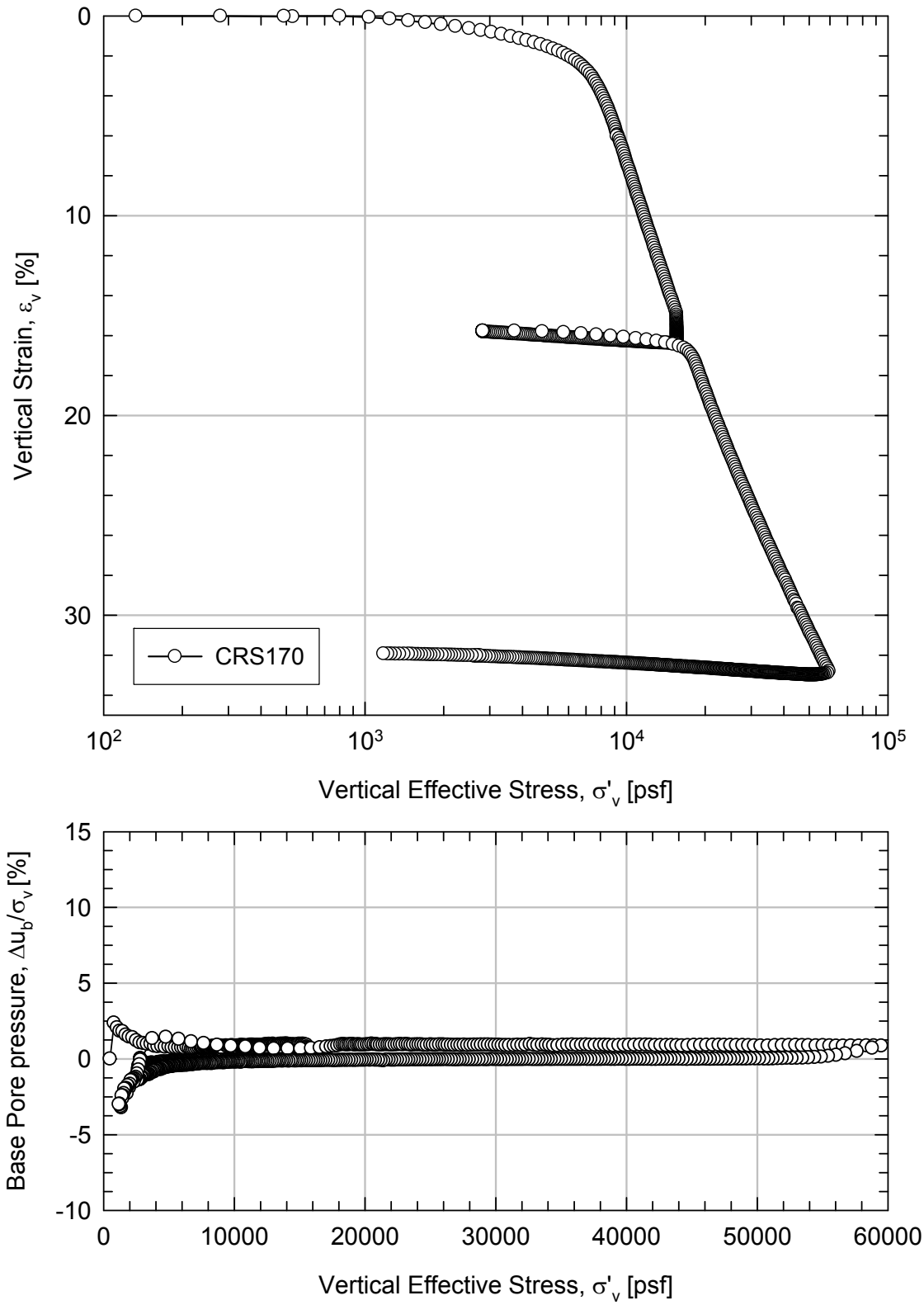


Figure 5.3.24 Vertical strain and normalized base pore pressure versus vertical effective stress for test CRS170 on sample 09-503B S3 35.5-38.0 ft.

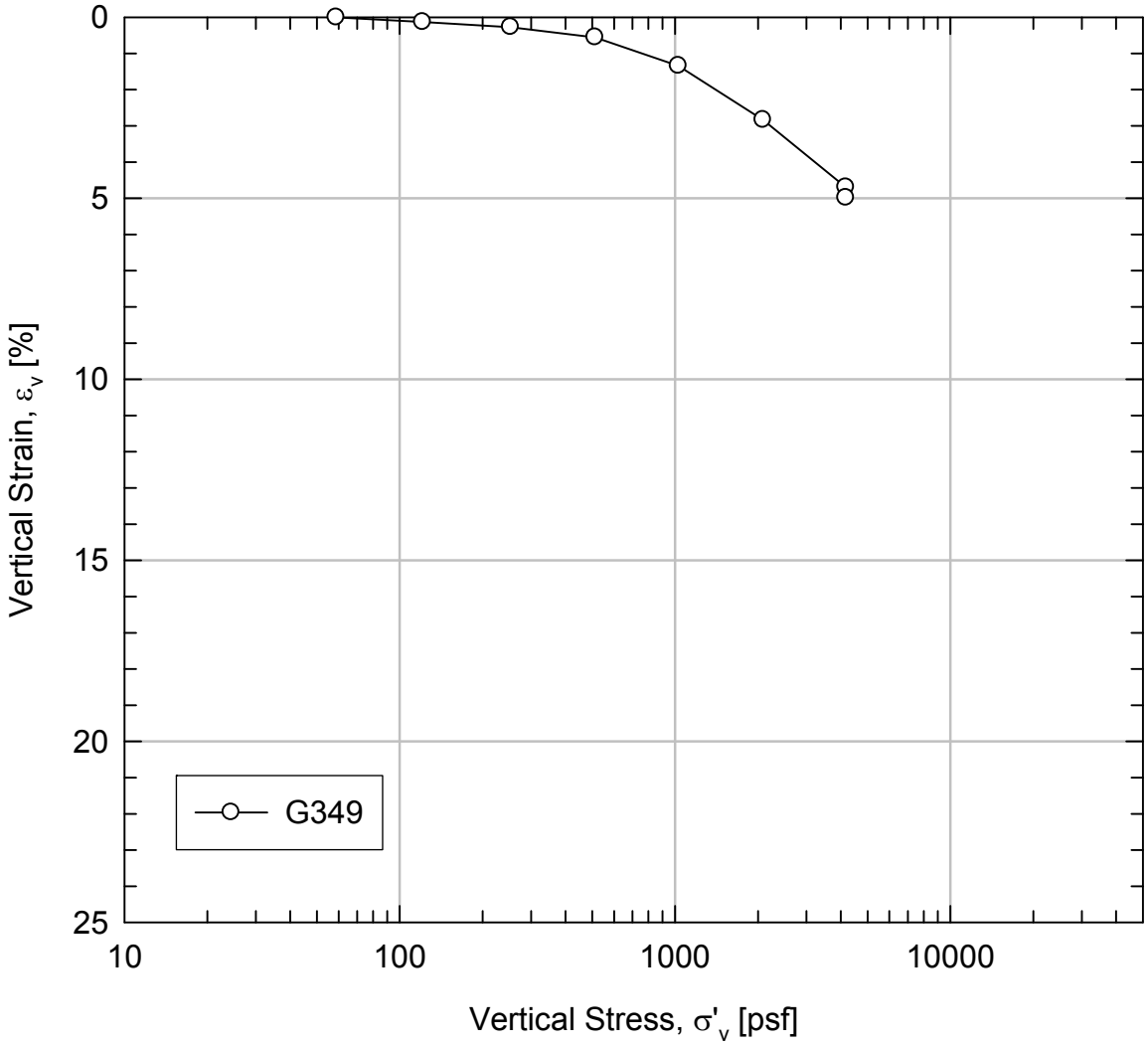


Figure 5.4.1 Vertical strain versus vertical effective stress for test DSS G349 on sample 09-301B S6 87.5-90.0 ft

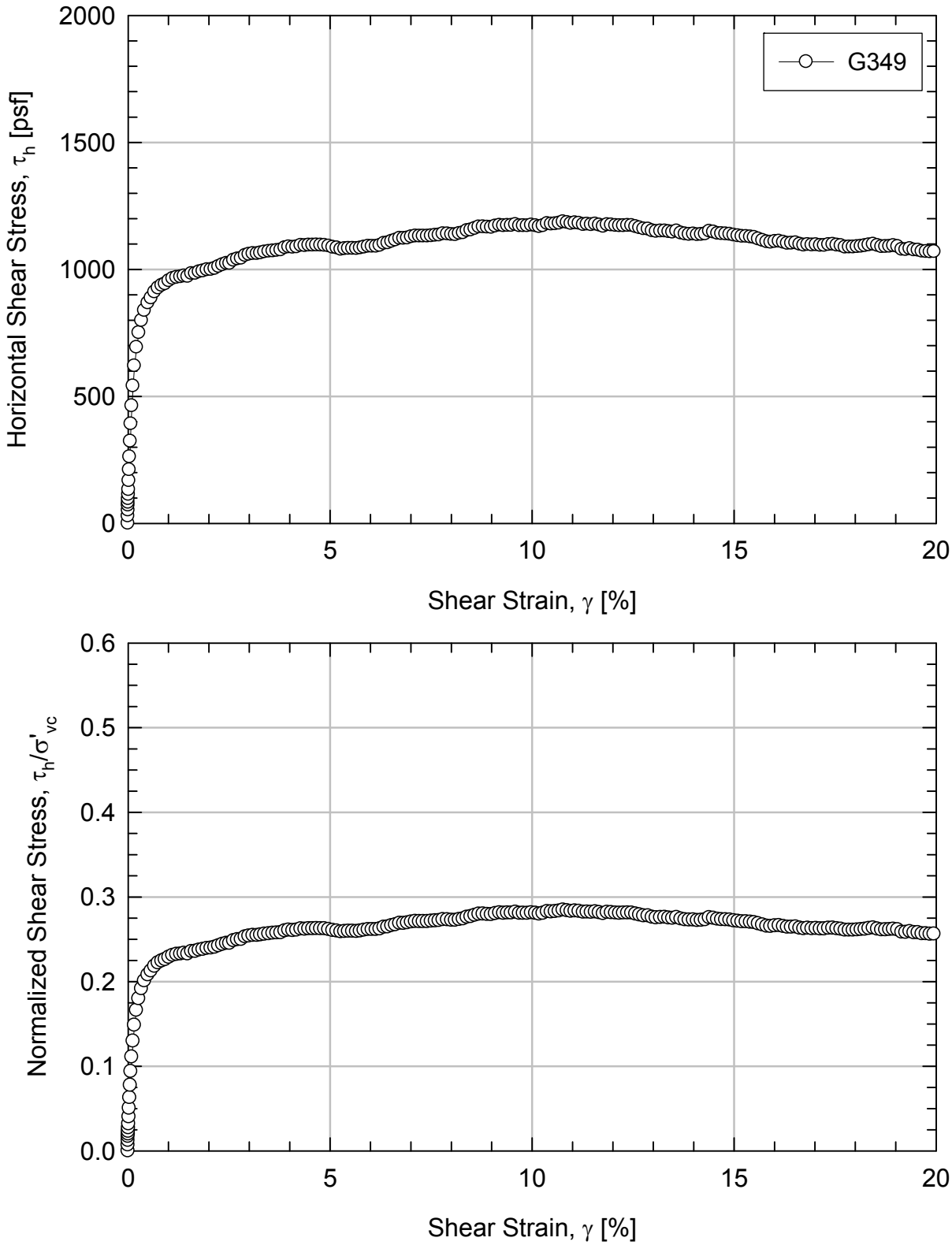


Figure 5.4.2 Horizontal shear stress and normalized shear stress versus shear strain for test DSS G349 on sample 09-301B S6 87.5-90.0 ft

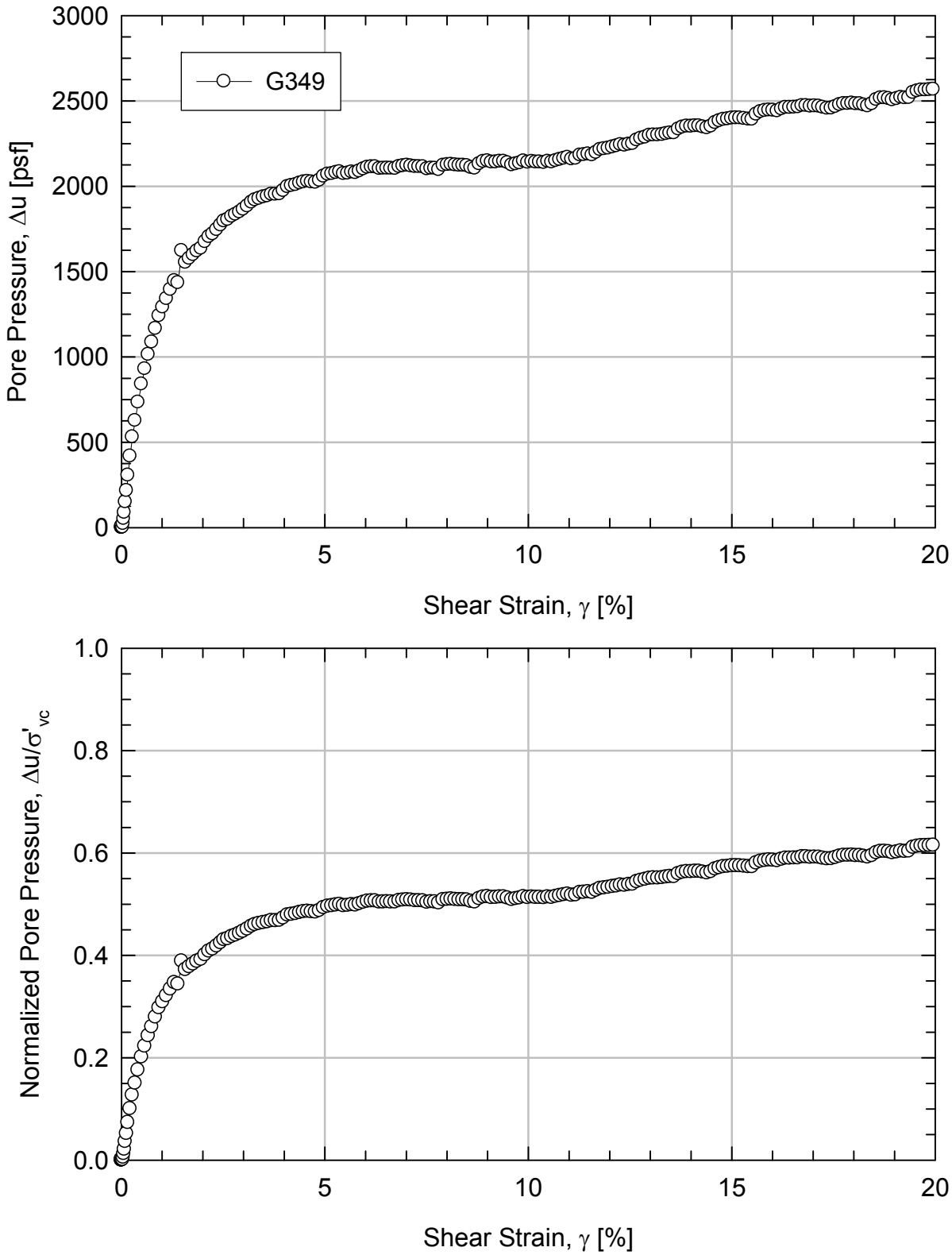


Figure 5.4.3 Pore pressure and normalized pore pressure versus shear strain for test DSS G349 on sample 09-301B S6 87.5-90.0 ft

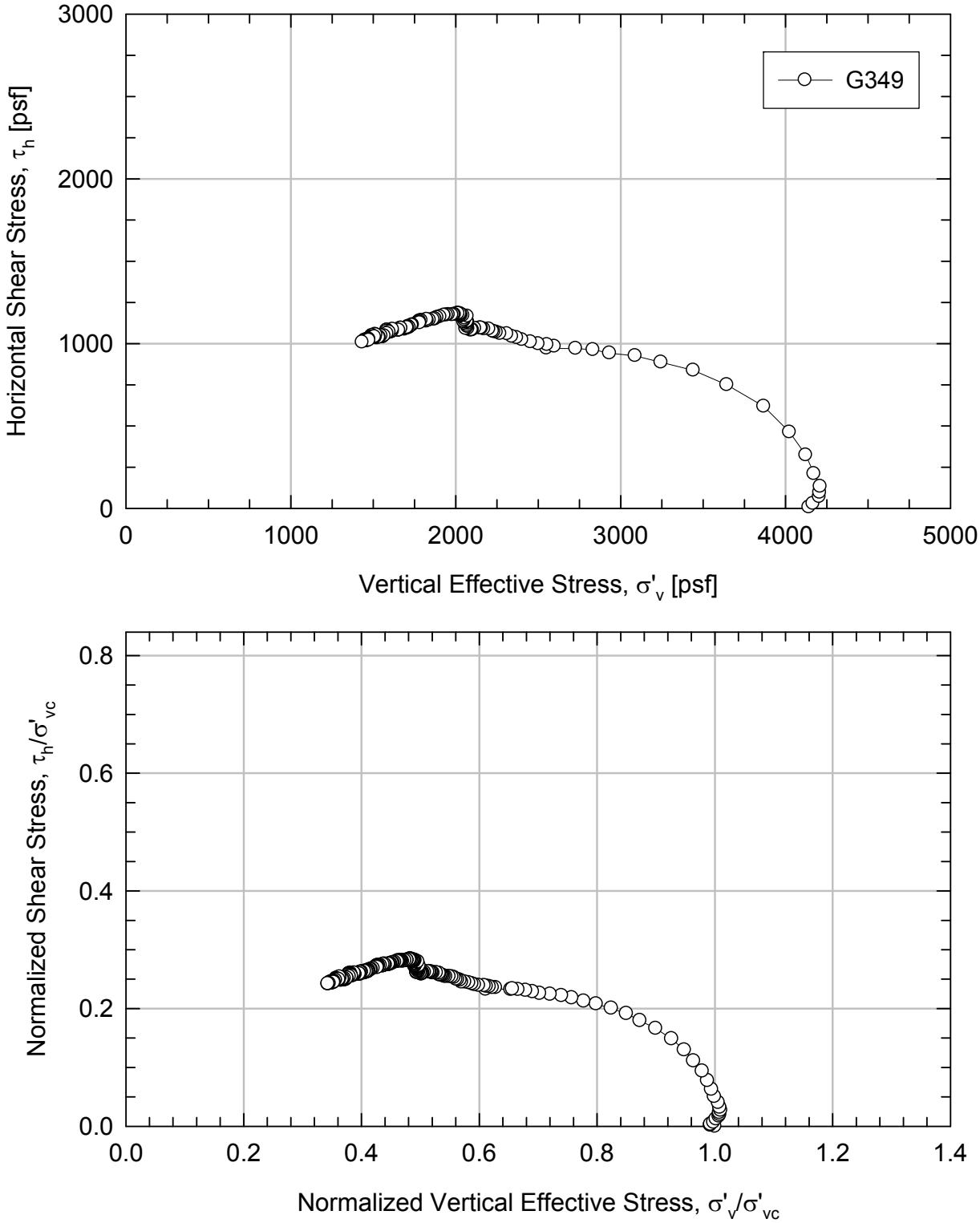


Figure 5.4.4 Horizontal shear stress versus vertical effective stress and normalized shear stress versus normalized vertical effective stress for test DSS G349 on sample 09-301B S6 87.5-90.0 ft.

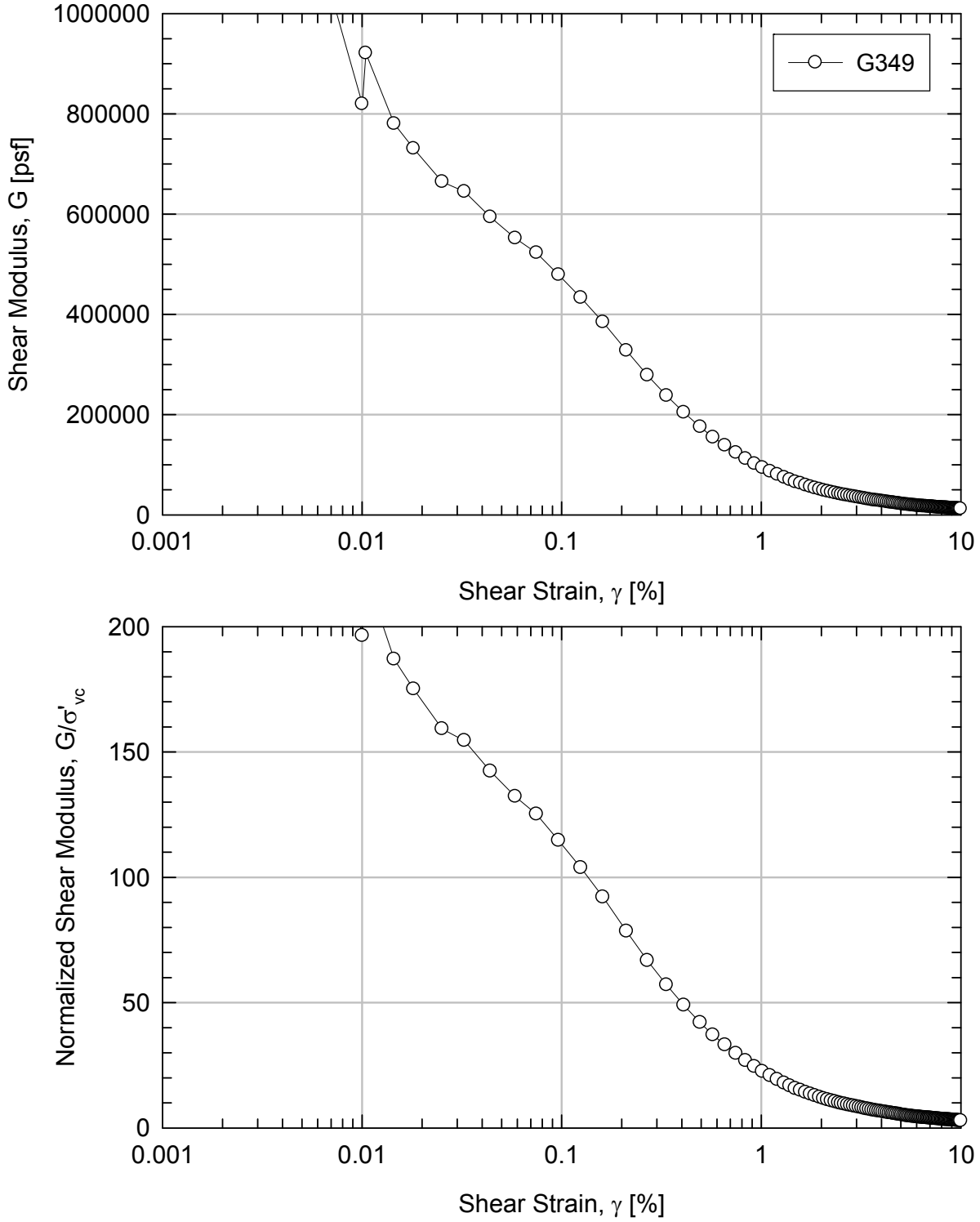


Figure 5.4.5 Shear modulus and normalized shear modulus versus shear strain for test DSS G349 on sample 09-301B S6 87.5-90.0 ft

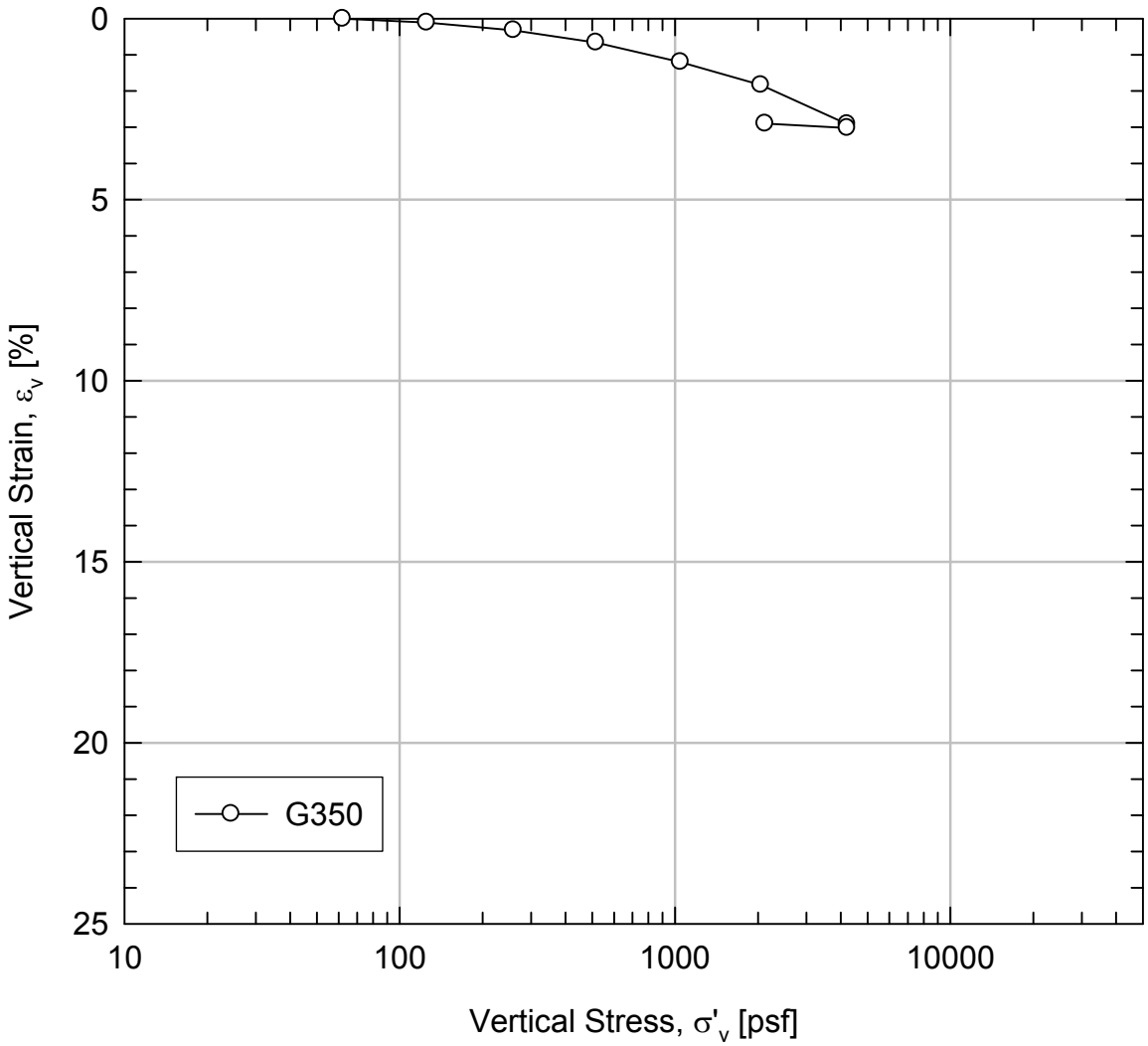


Figure 5.4.6 Vertical strain versus vertical effective stress for test DSS G350 on sample 09-101B S5 38.0-40.5 ft.

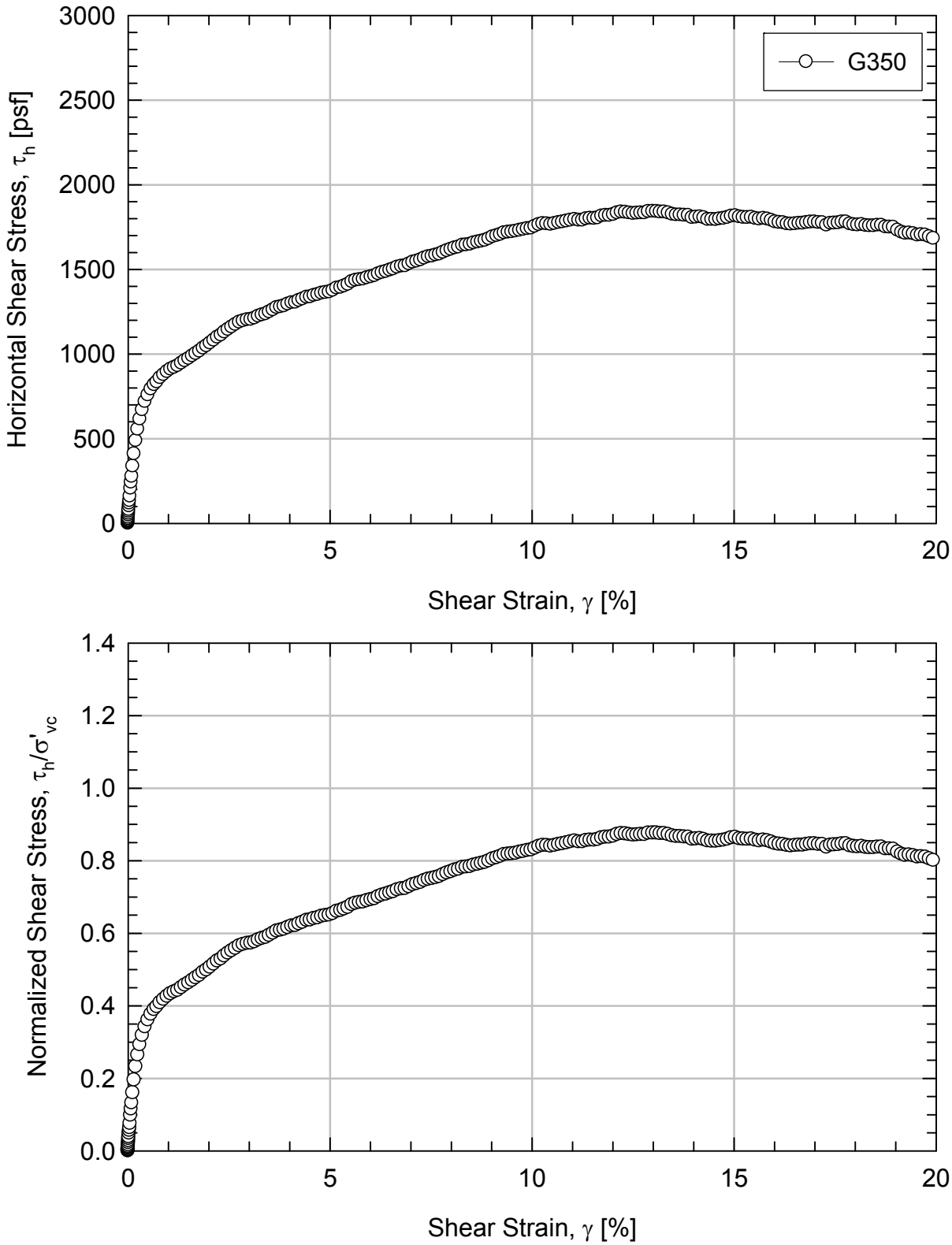


Figure 5.4.7 Horizontal shear stress and normalized shear stress versus shear strain for test DSS G350 on sample 09-101B S5 38.0-40.5 ft.

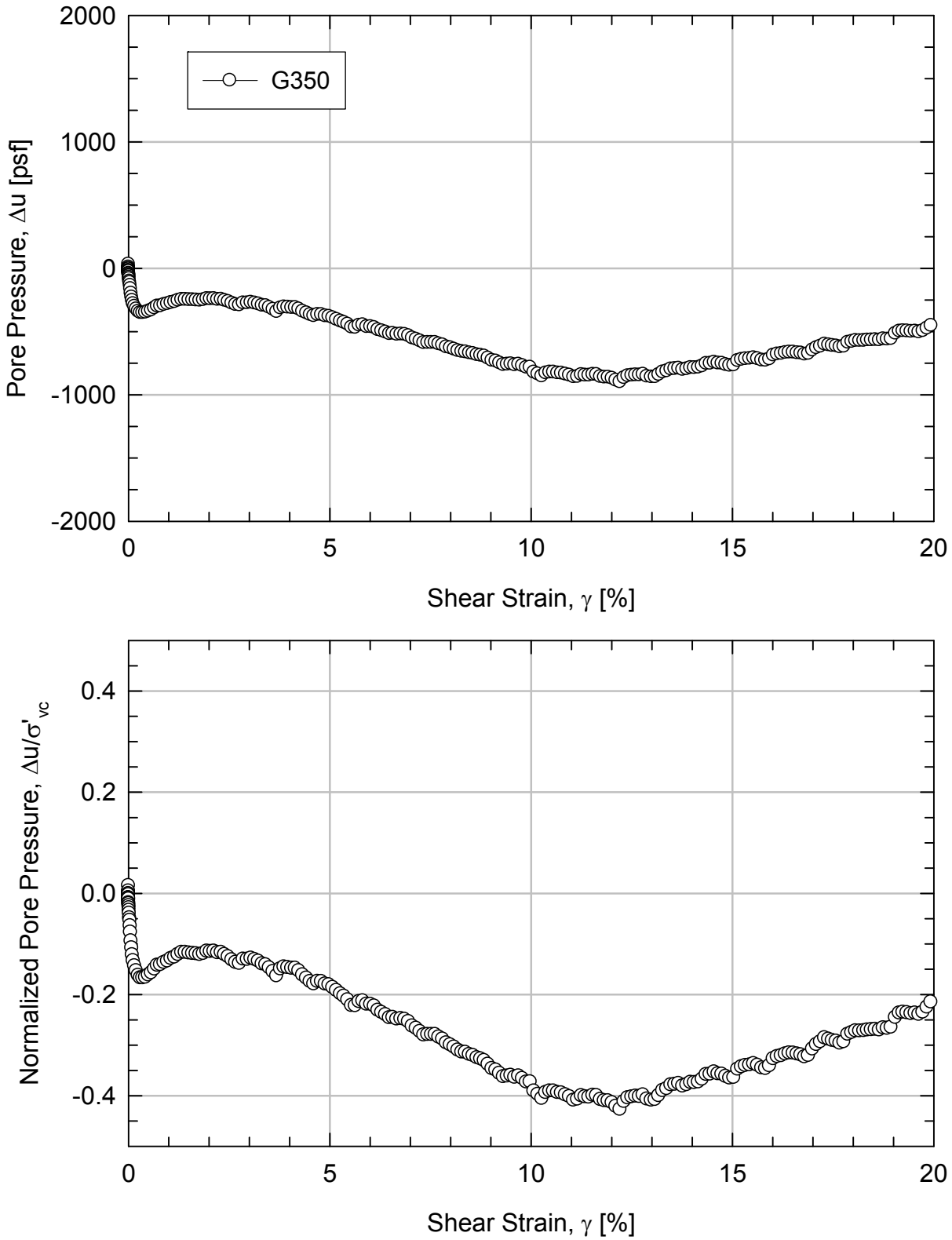


Figure 5.4.8 Pore pressure and normalized pore pressure versus shear strain for test DSS G350 on sample 09-101B S5 38.0-40.5 ft.

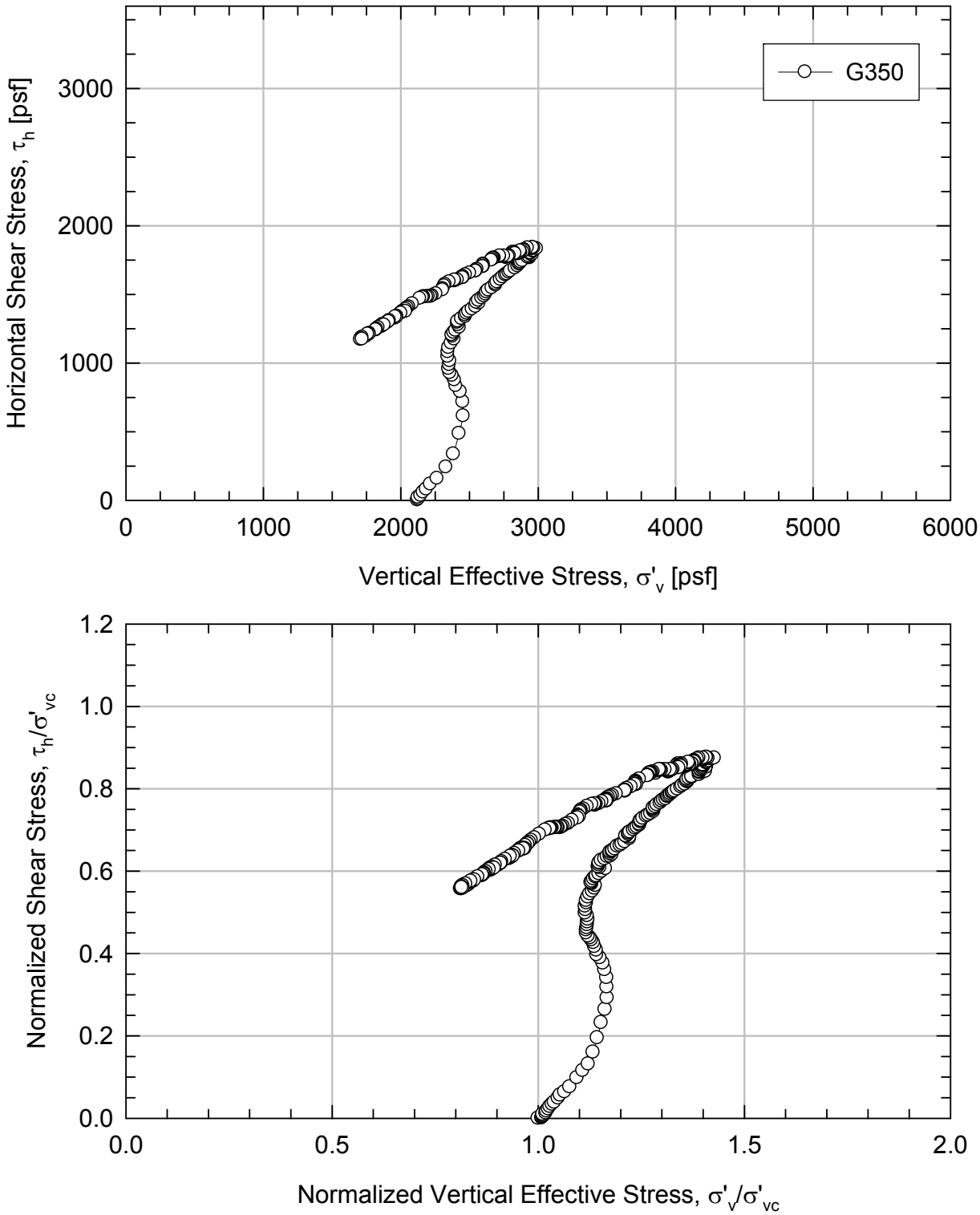


Figure 5.4.9 Horizontal shear stress versus vertical effective stress and normalized shear stress versus normalized vertical effective stress for test DSS G350 on sample 09-101B S5 38.0-40.5 ft.

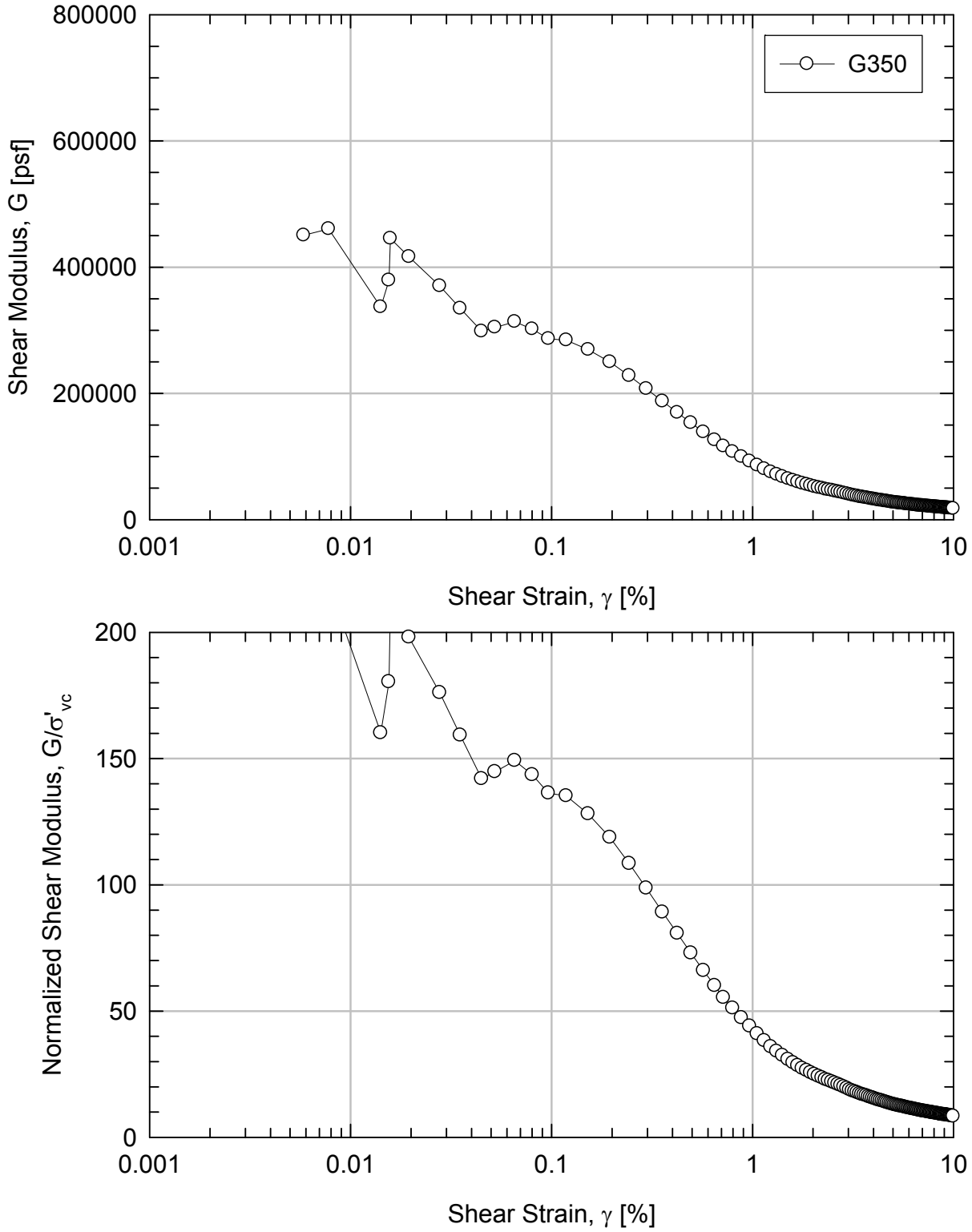


Figure 5.4.10 Shear modulus and normalized shear modulus versus shear strain for test DSS G350 on sample 09-101B S5 38.0-40.5 ft.

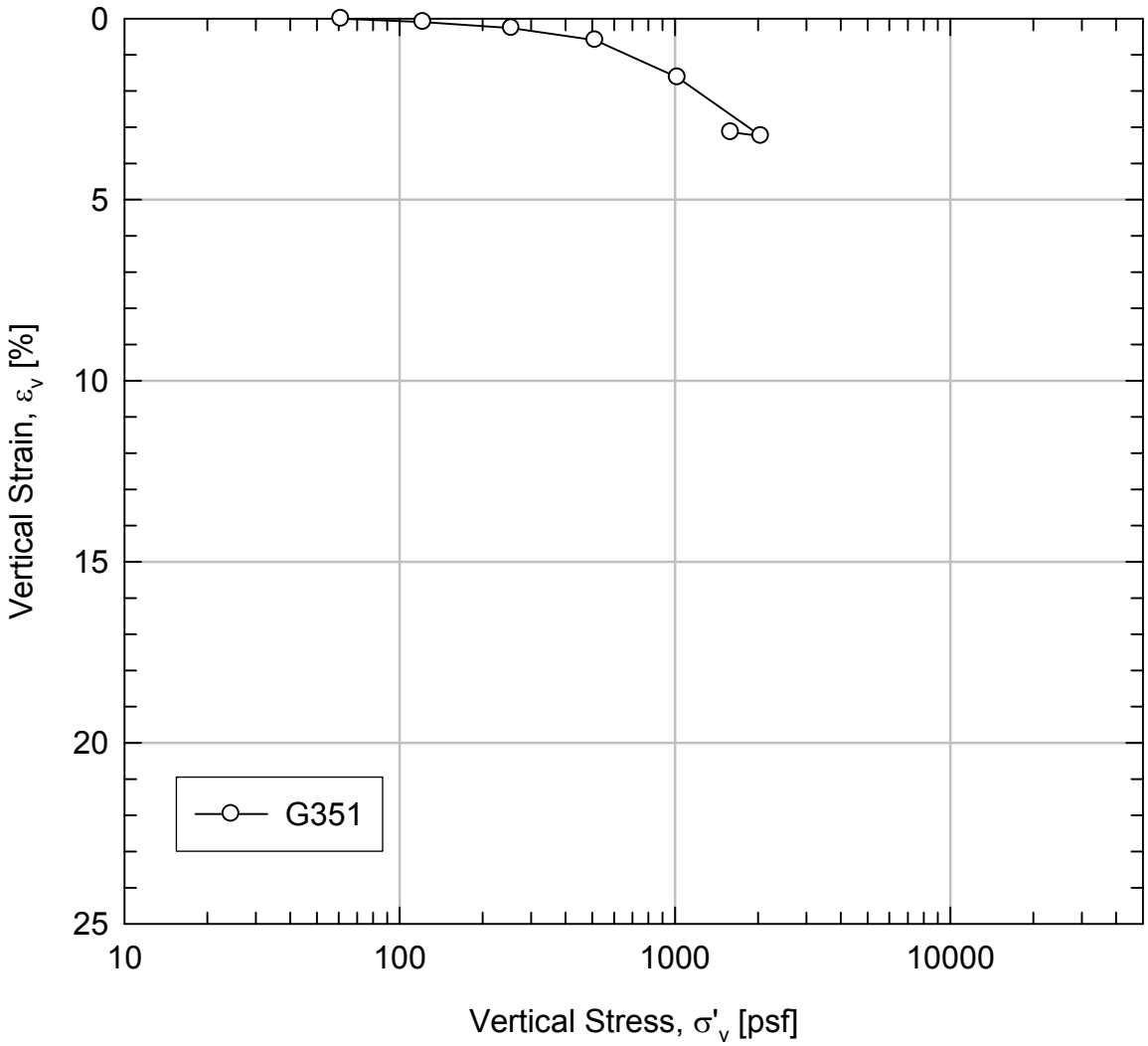


Figure 5.4.11 Vertical strain versus vertical effective stress for test DSS G351 on sample 09-108B S2 23.5-26.0 ft.

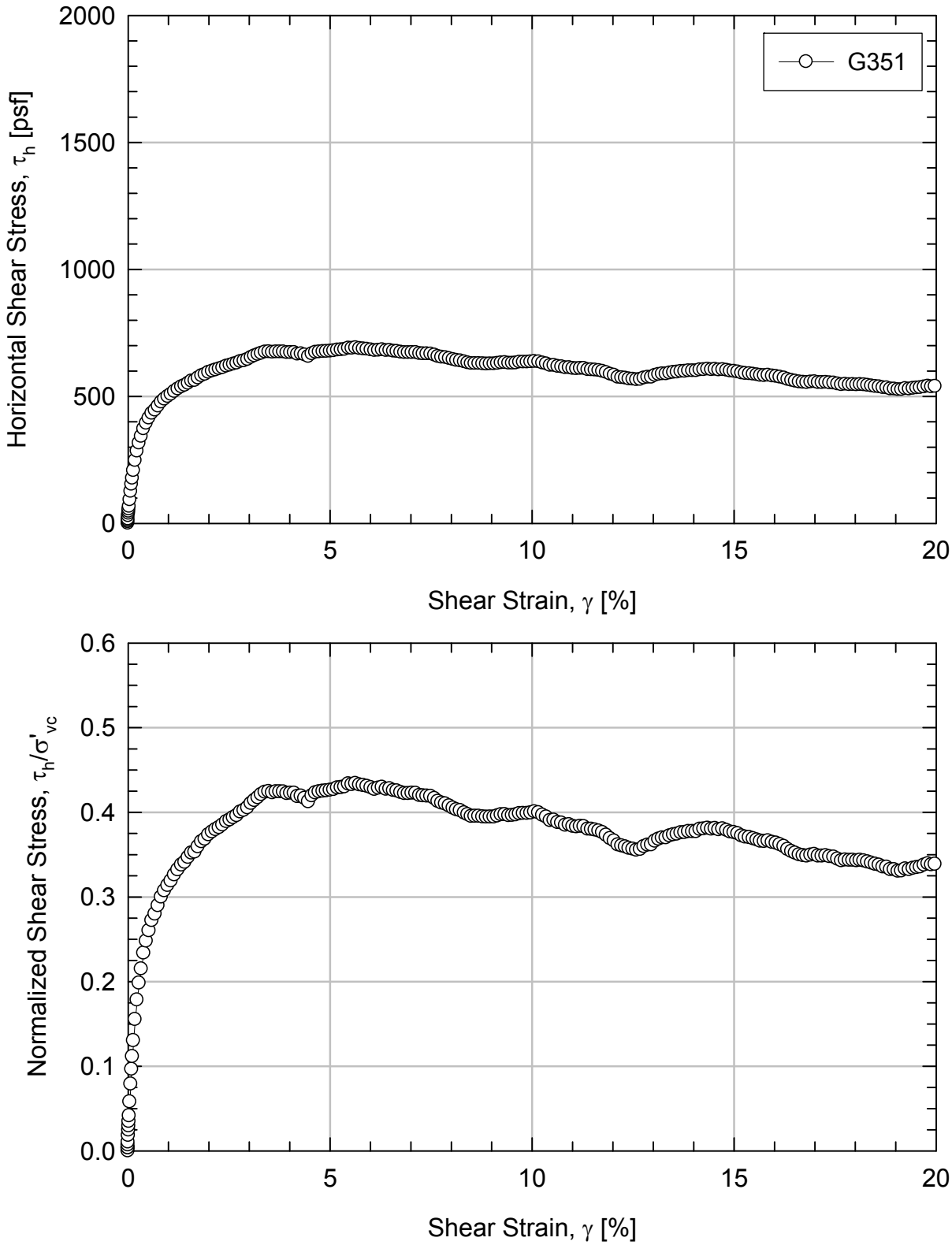


Figure 5.4.12 Horizontal shear stress and normalized shear stress versus shear strain for test DSS G351 on sample 09-108B S2 23.5-26.0 ft.

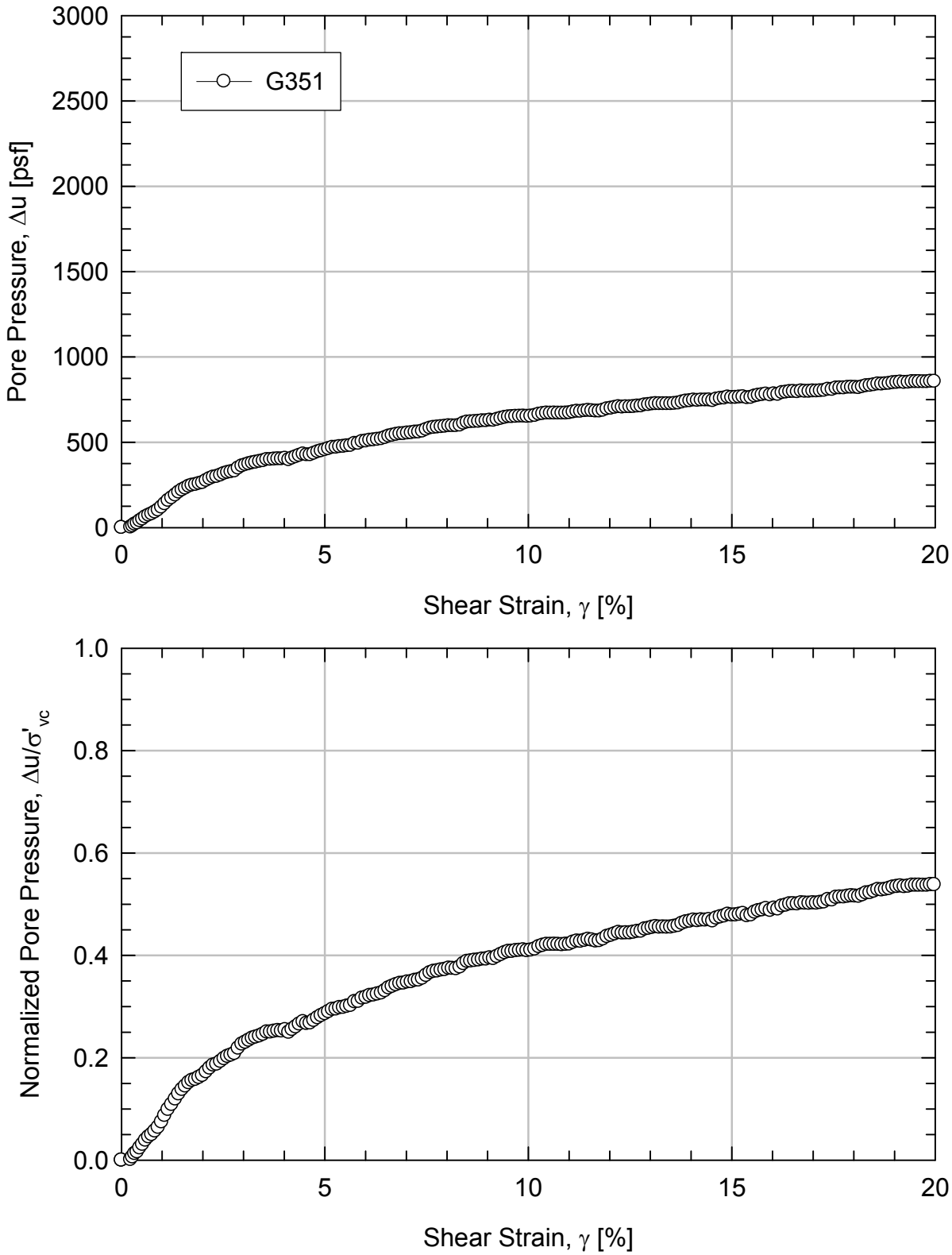


Figure 5.4.13 Pore pressure and normalized pore pressure versus shear strain for test DSS G351 on sample 09-108B S2 23.5-26.0 ft.

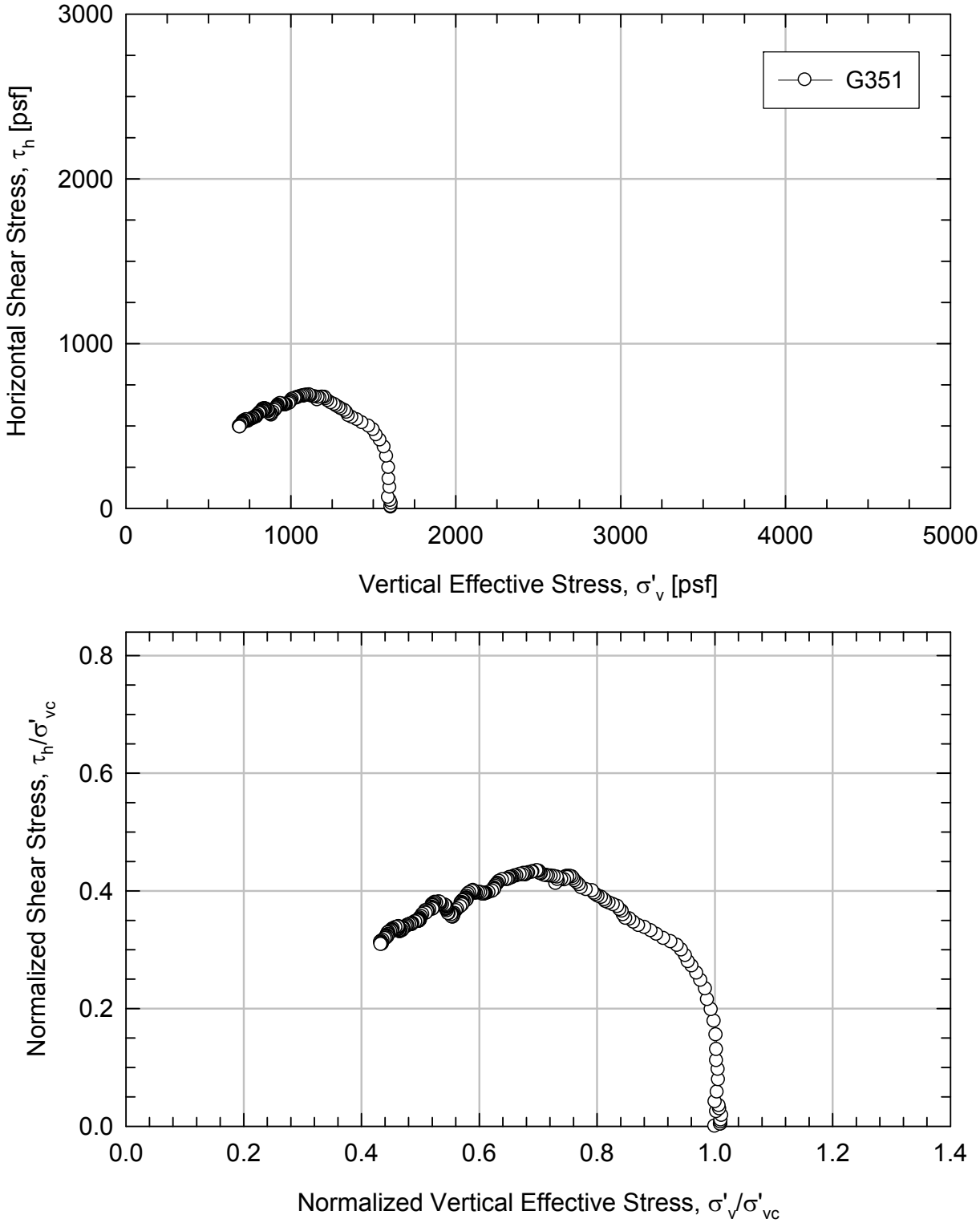


Figure 5.4.14 Horizontal shear stress versus vertical effective stress and normalized shear stress versus normalized vertical effective stress for test DSS G351 on sample 09-108B S2 23.5-26.0 ft.

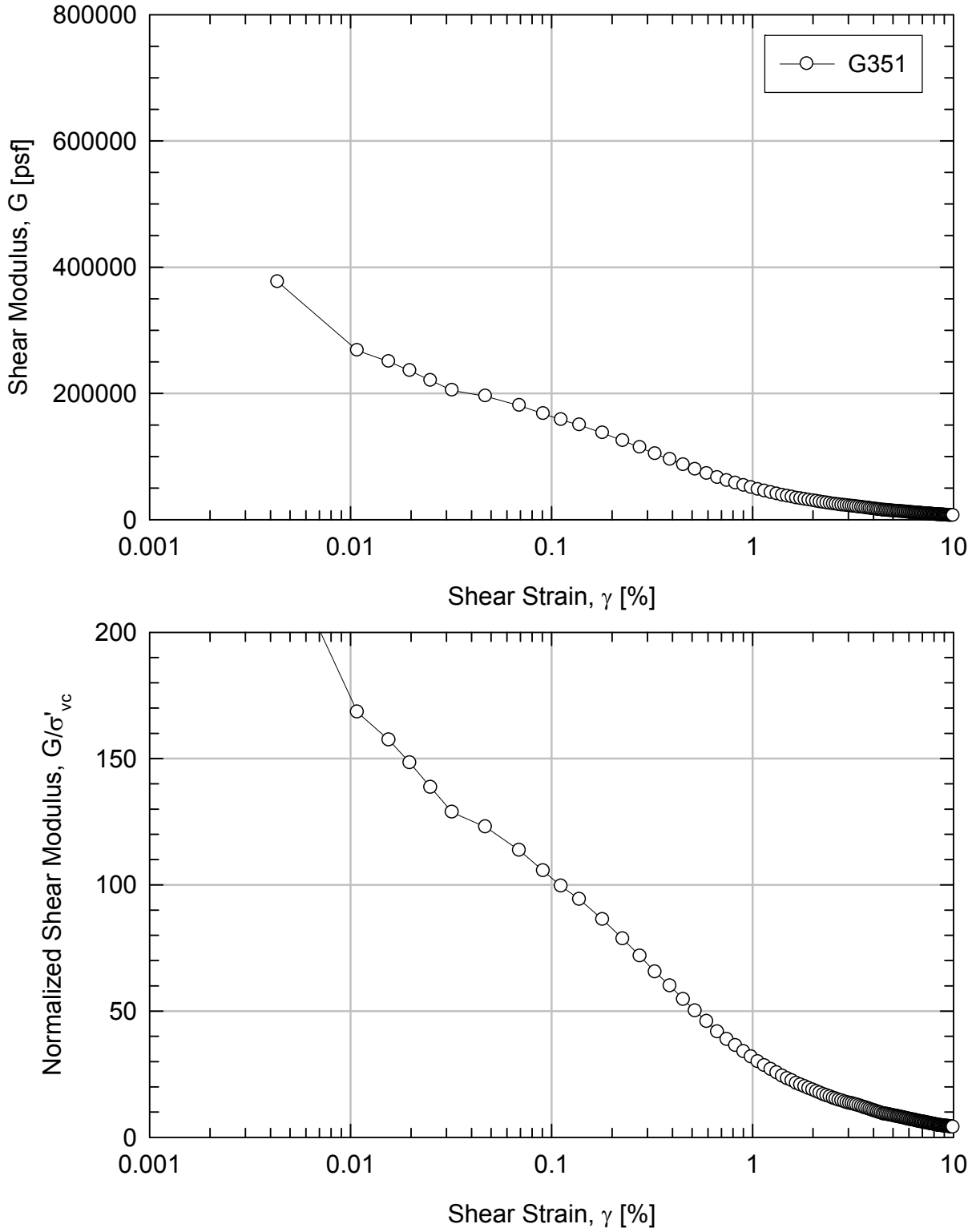


Figure 5.4.15 Shear modulus and normalized shear modulus versus shear strain for test DSS G351 on sample 09-108B S2 23.5-26.0 ft.

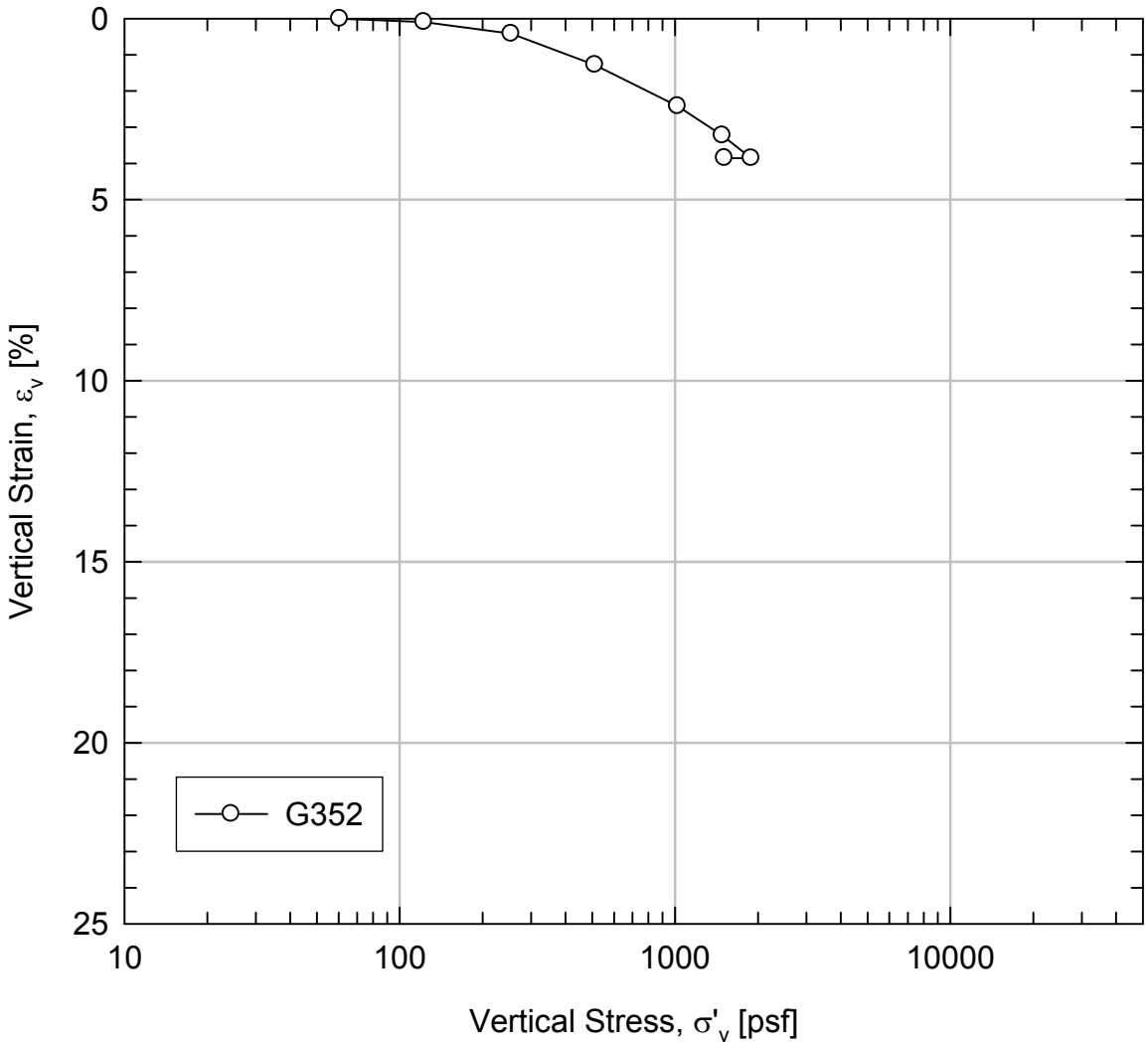


Figure 5.4.16 Vertical strain versus vertical effective stress for test DSS G352 on sample 09-109B S3 25.5-28.0 ft.

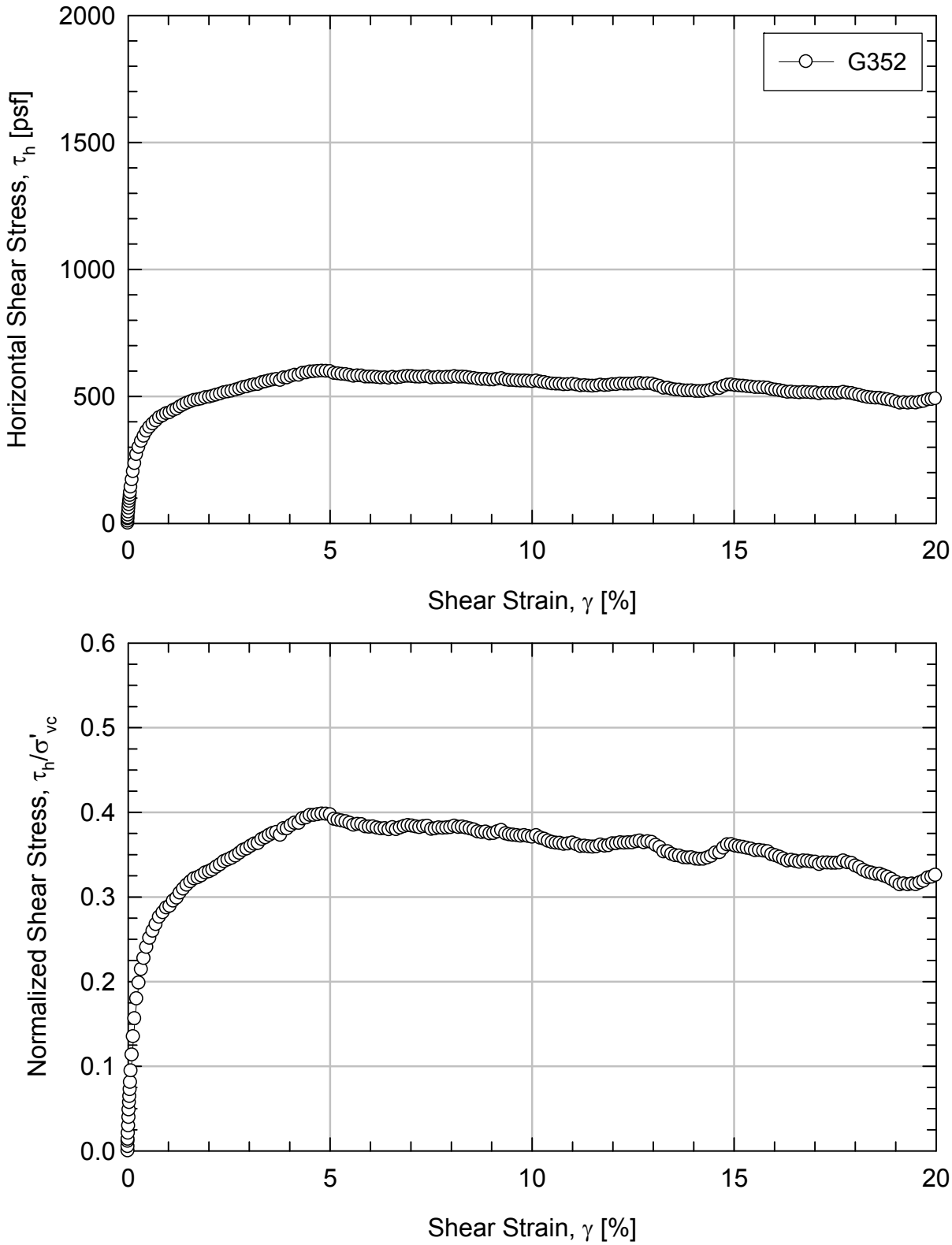


Figure 5.4.17 Horizontal shear stress and normalized shear stress versus shear strain for test DSS G352 on sample 09-109B S3 25.5-28.0 ft.

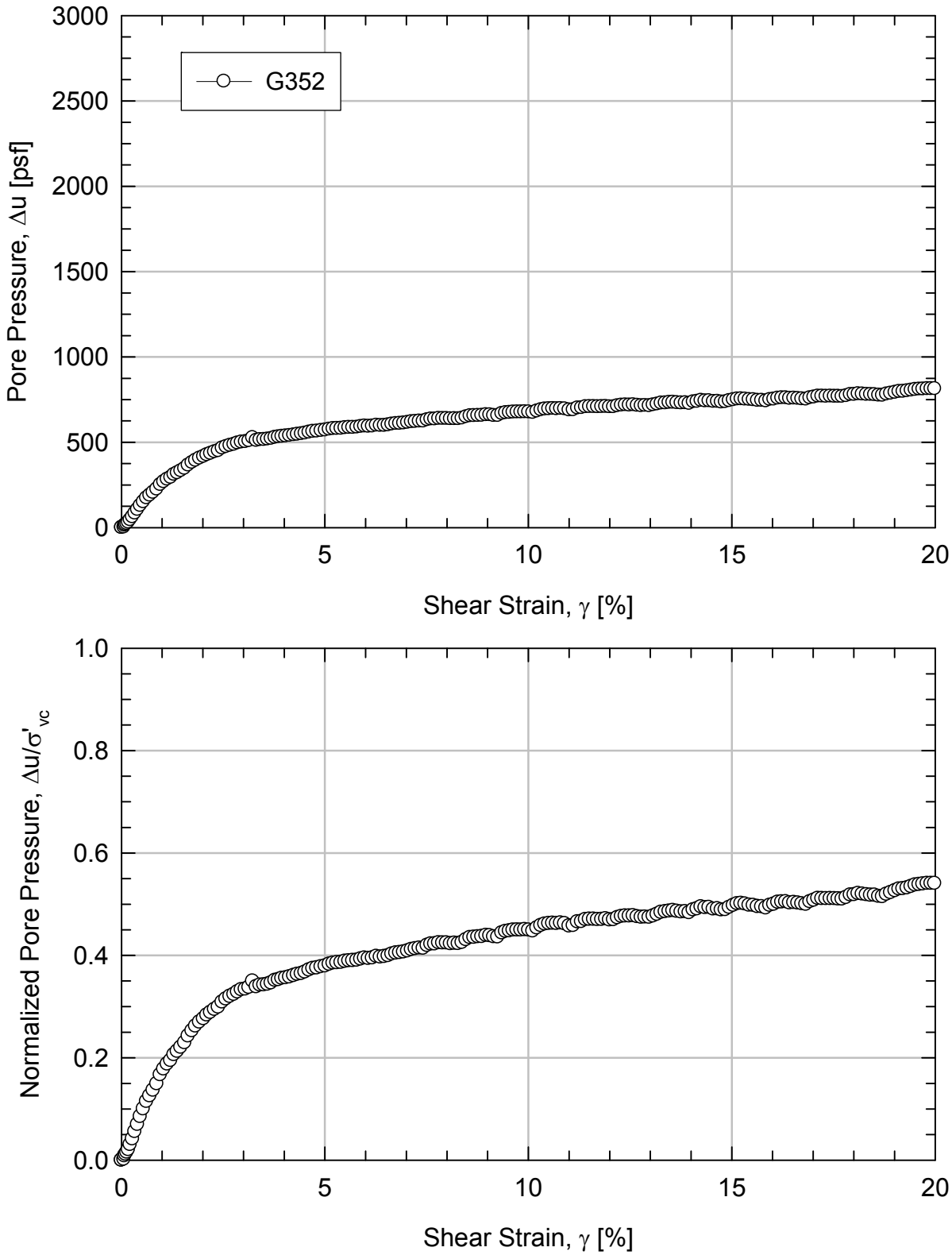


Figure 5.4.18 Pore pressure and normalized pore pressure versus shear strain for test DSS G352 on sample 09-109B S3 25.5-28.0 ft.

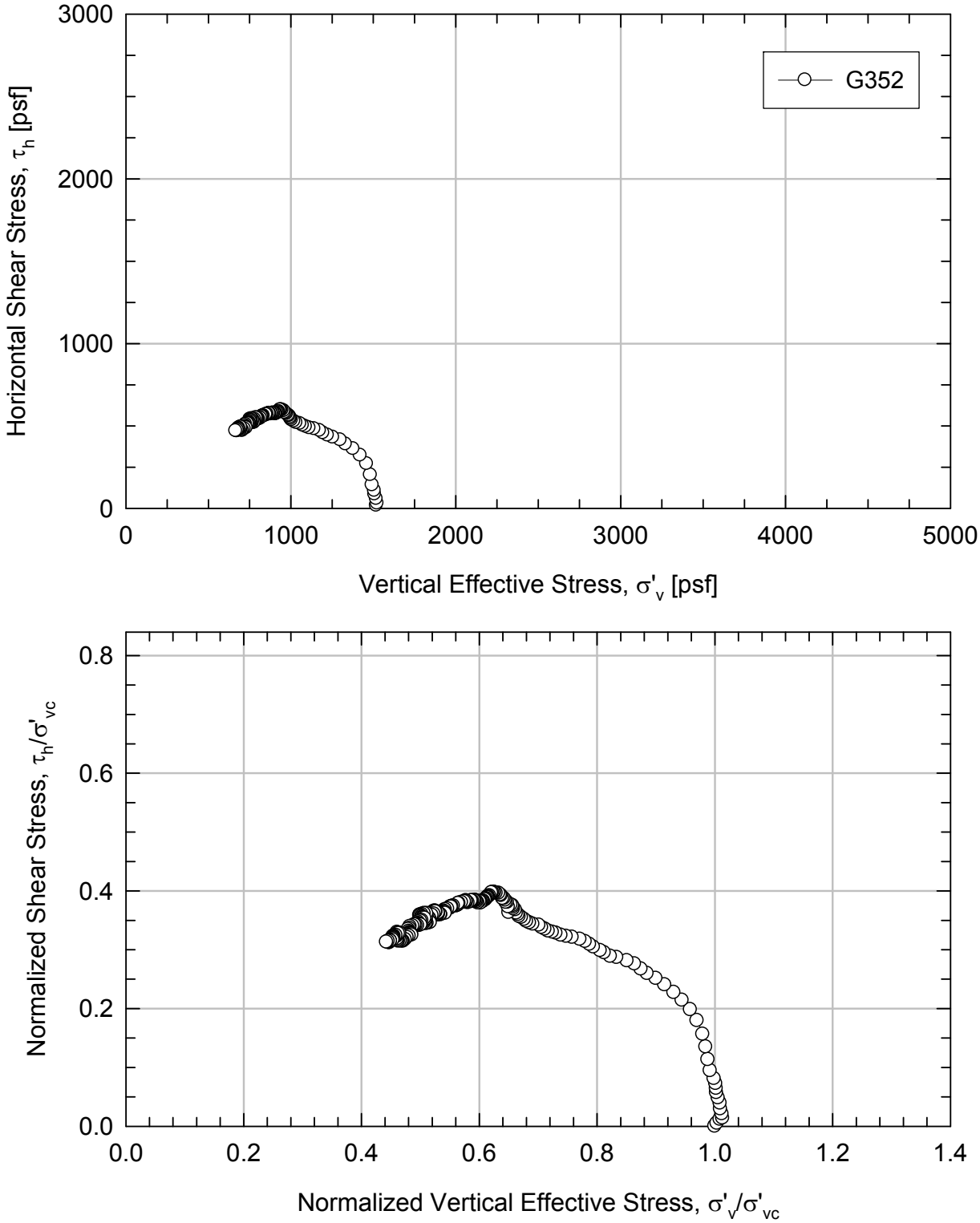


Figure 5.4.19 Horizontal shear stress versus vertical effective stress and normalized shear stress versus normalized vertical effective stress for test DSS G352 on sample 09-109B S3 25.5-28.0 ft.

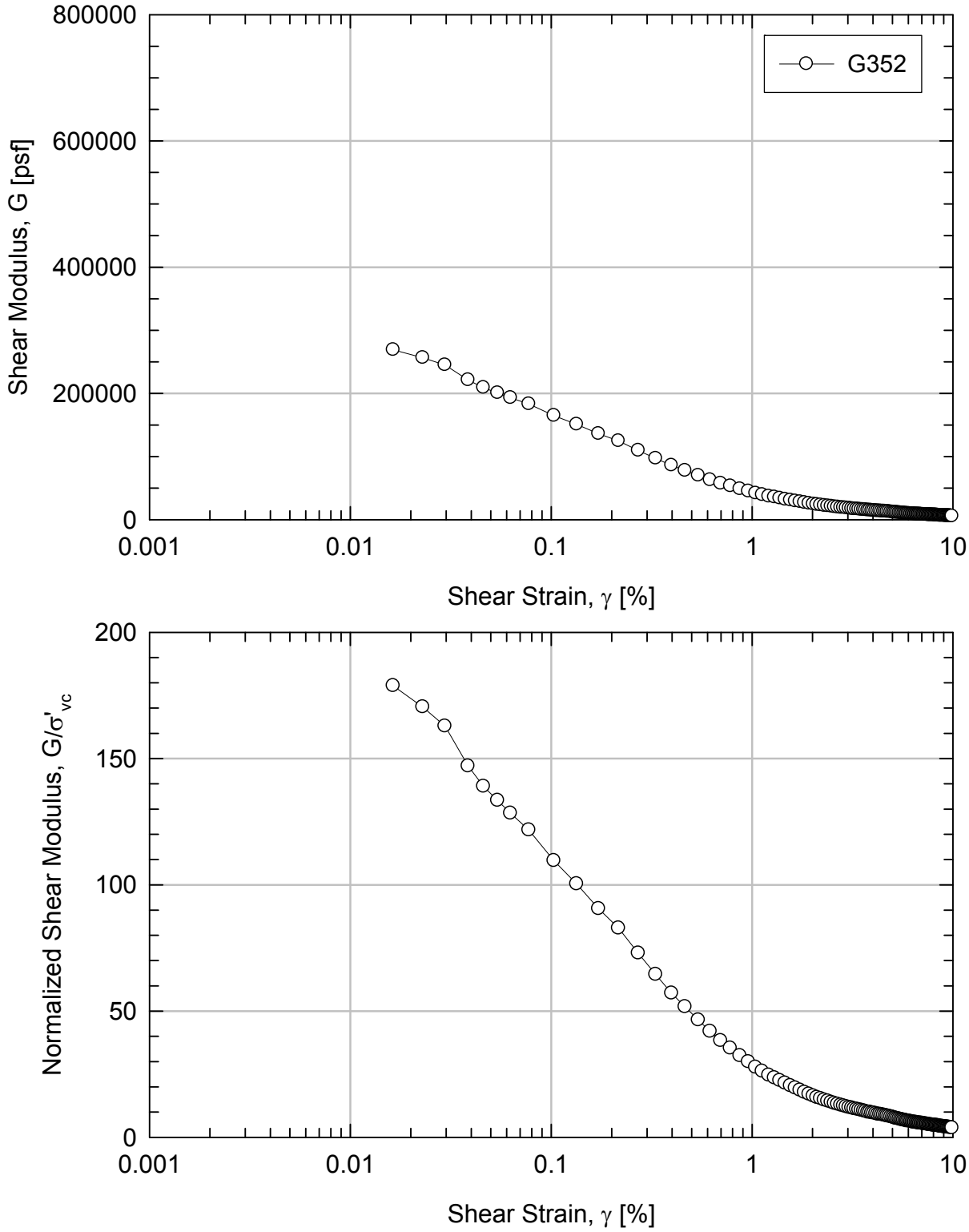


Figure 5.4.20 Shear modulus and normalized shear modulus versus shear strain for test DSS G352 on sample 09-109B S3 25.5-28.0 ft.

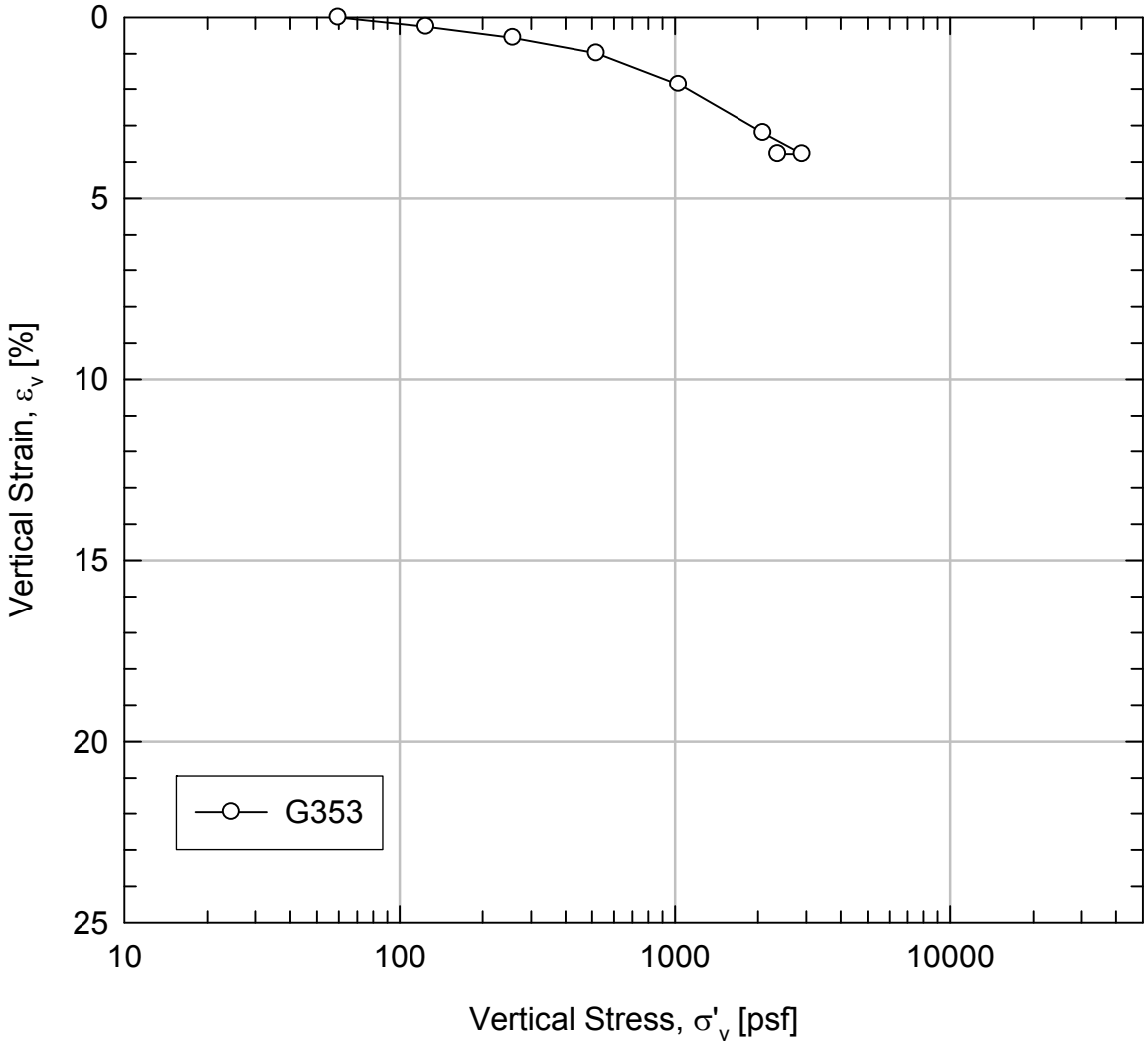


Figure 5.4.21 Vertical strain versus vertical effective stress for test DSS G353 on sample 09-605B S7 47.5-49.5 ft.

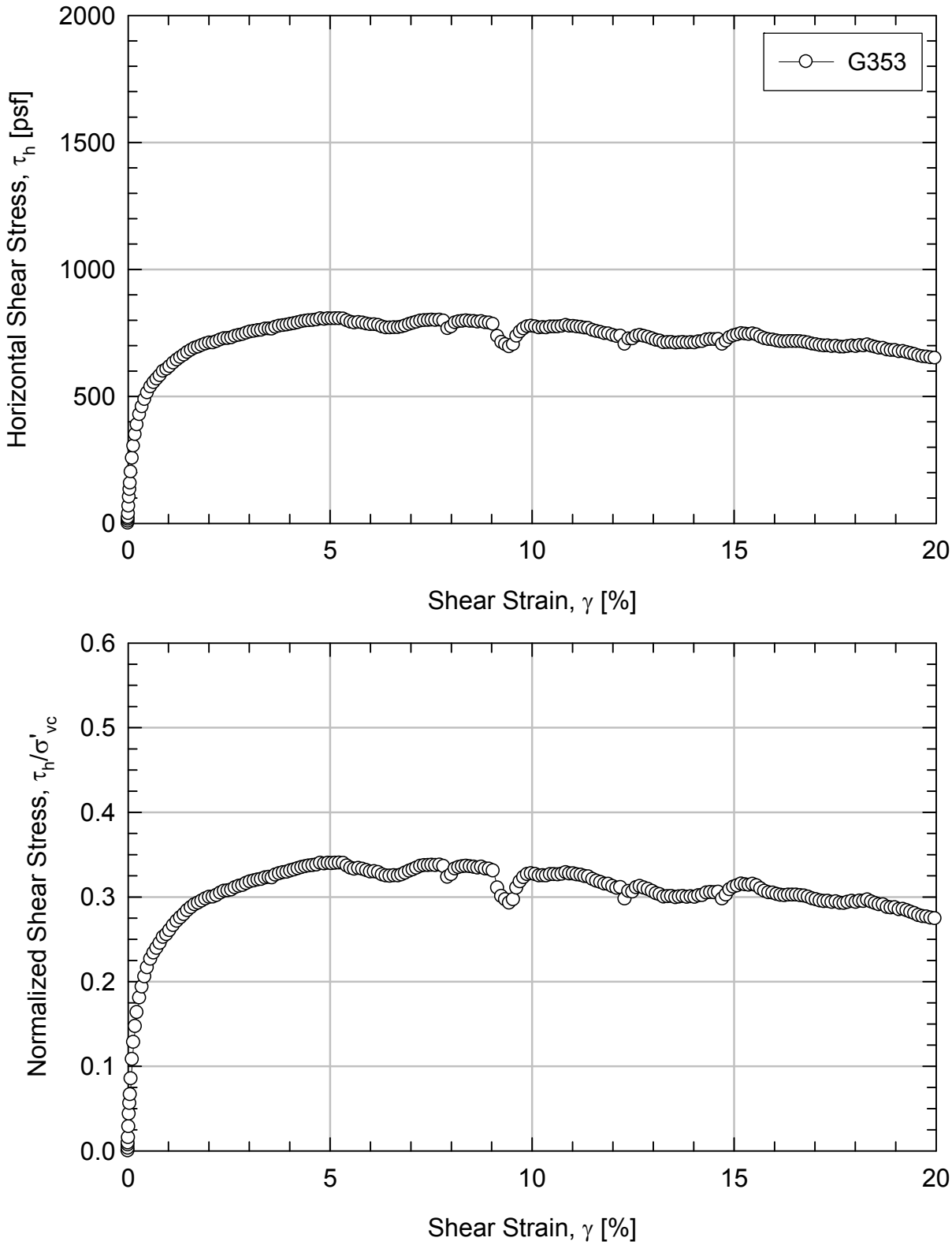


Figure 5.4.22 Horizontal shear stress and normalized shear stress versus shear strain for test DSS G353 on sample 09-605B S7 47.5-49.5 ft.

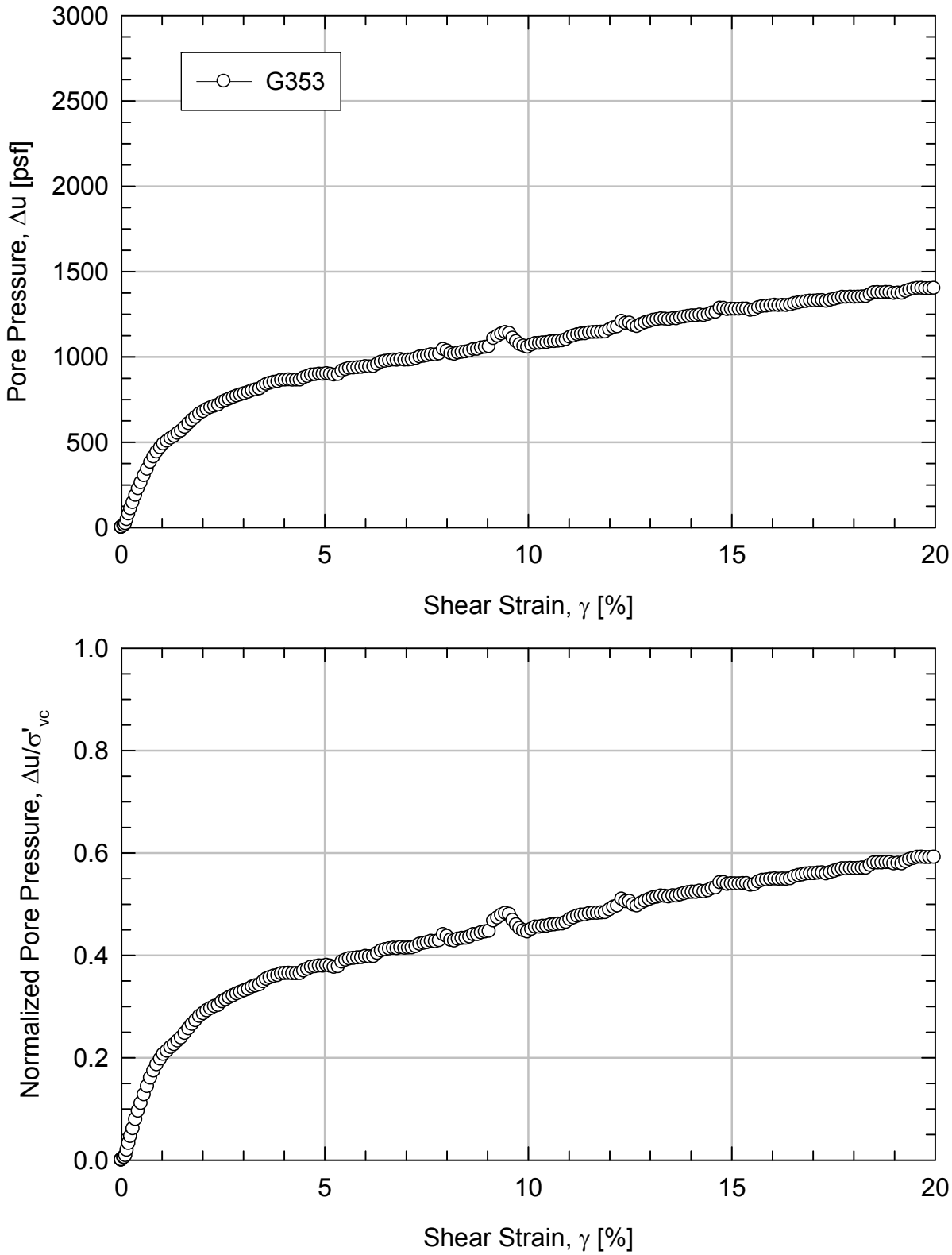


Figure 5.4.23 Pore pressure and normalized pore pressure versus shear strain for test DSS G353 on sample 09-605B S7 47.5-49.5 ft.

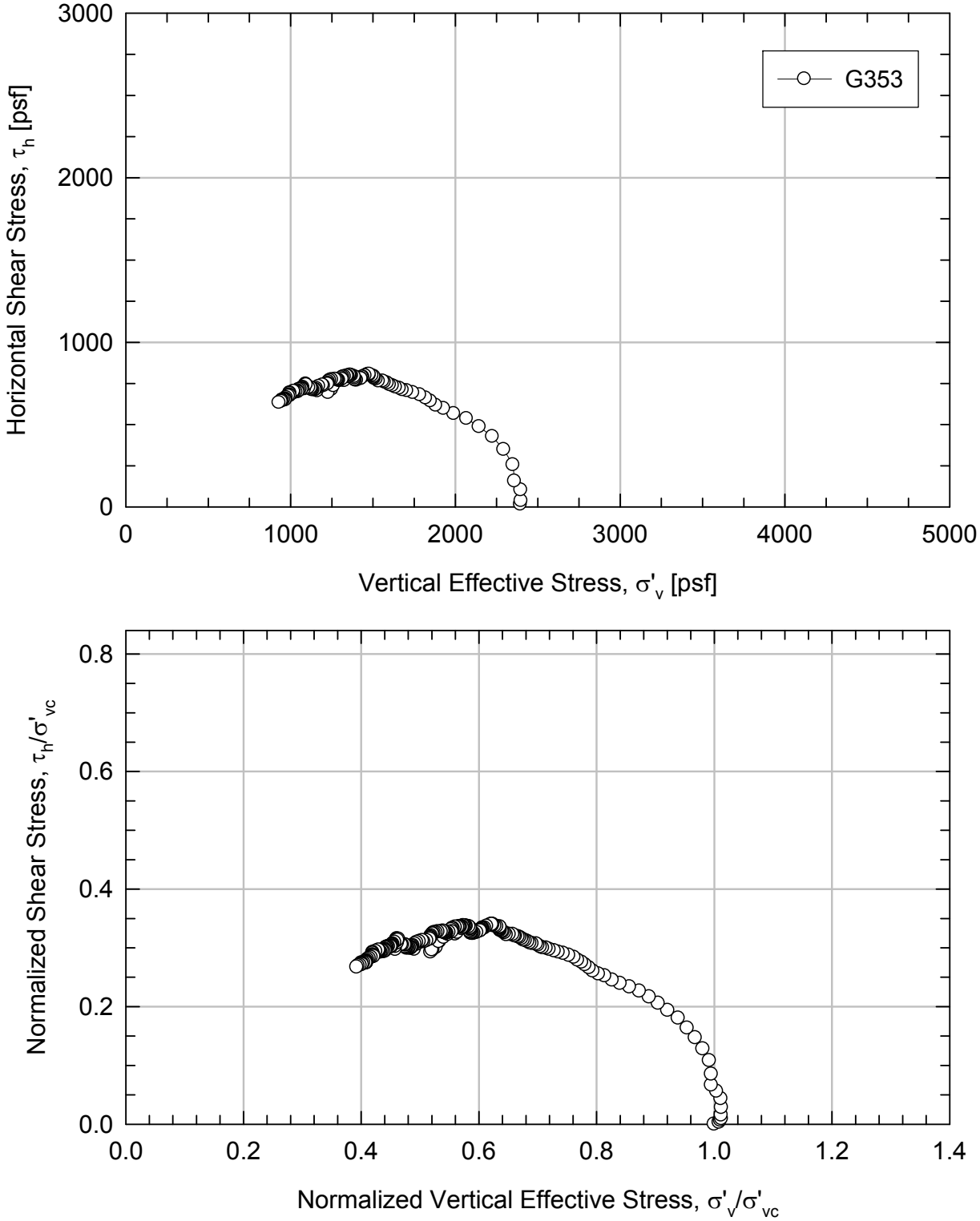


Figure 5.4.24 Horizontal shear stress versus vertical effective stress and normalized shear stress versus normalized vertical effective stress for test DSS G353 on sample 09-605B S7 47.5-49.5 ft.

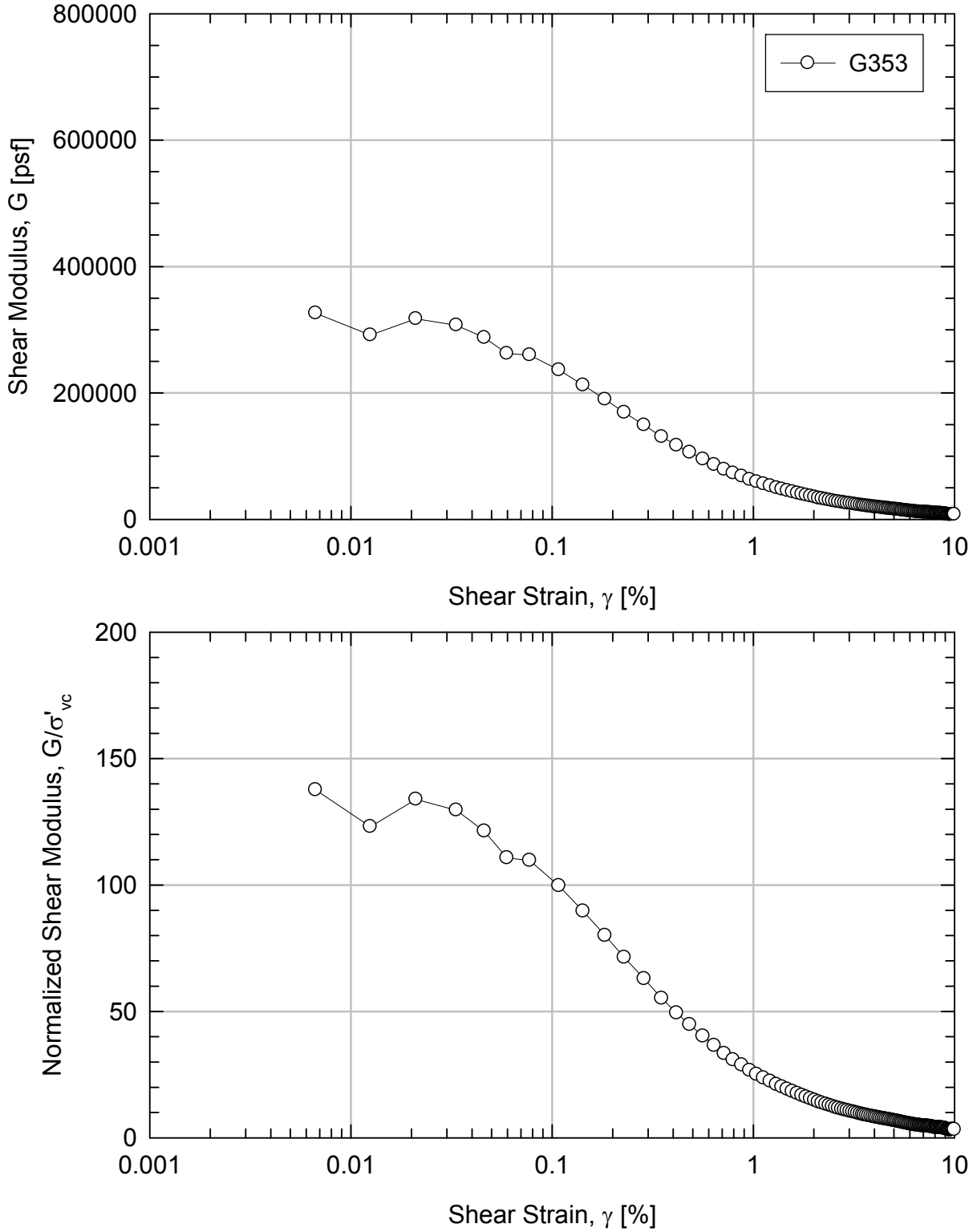


Figure 5.4.25 Shear modulus and normalized shear modulus versus shear strain for test DSS G353 on sample 09-605B S7 47.5-49.5 ft.

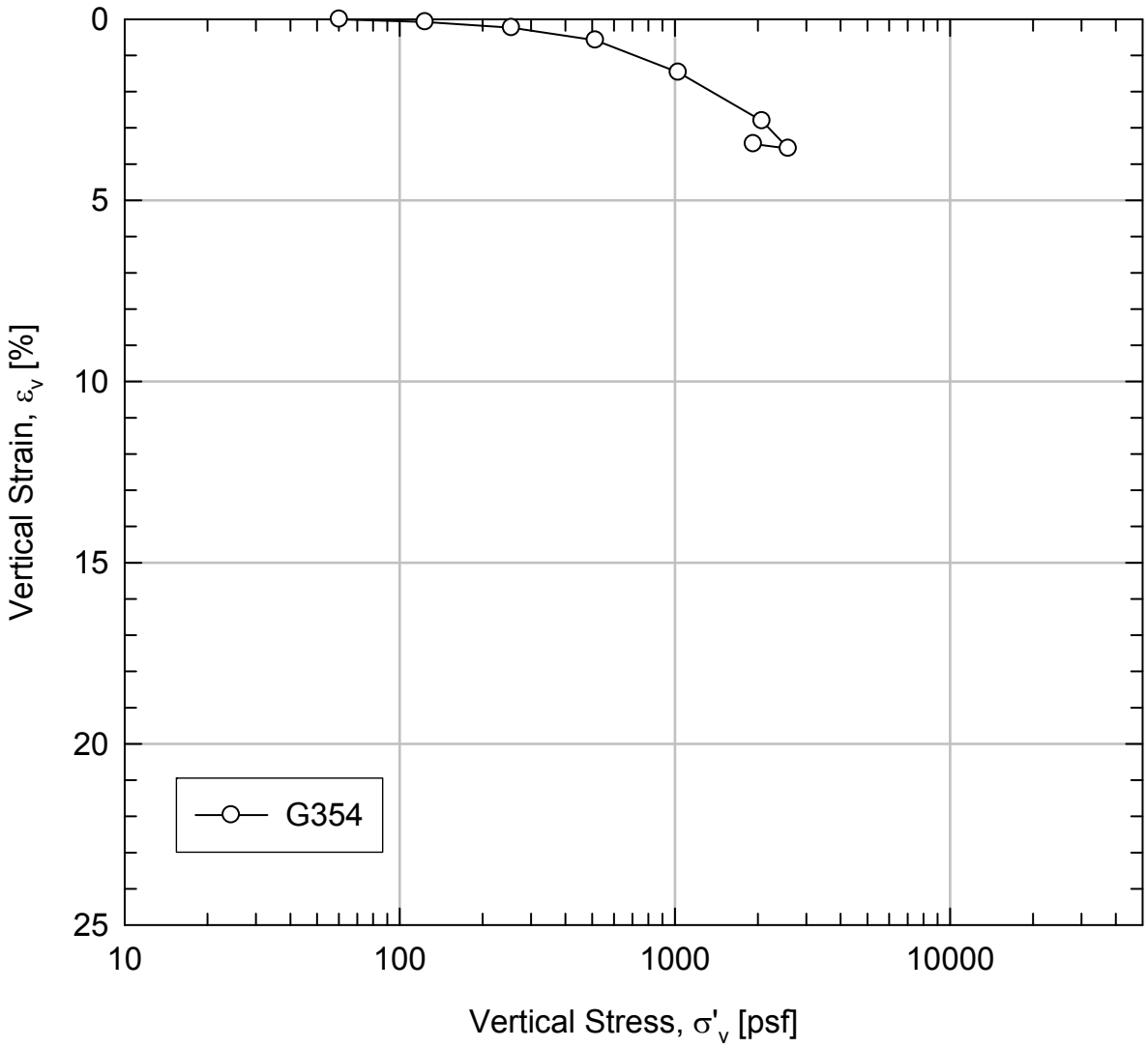


Figure 5.4.26 Vertical strain versus vertical effective stress for test DSS G354 on sample 09-103B S2 33.5-36.0 ft.

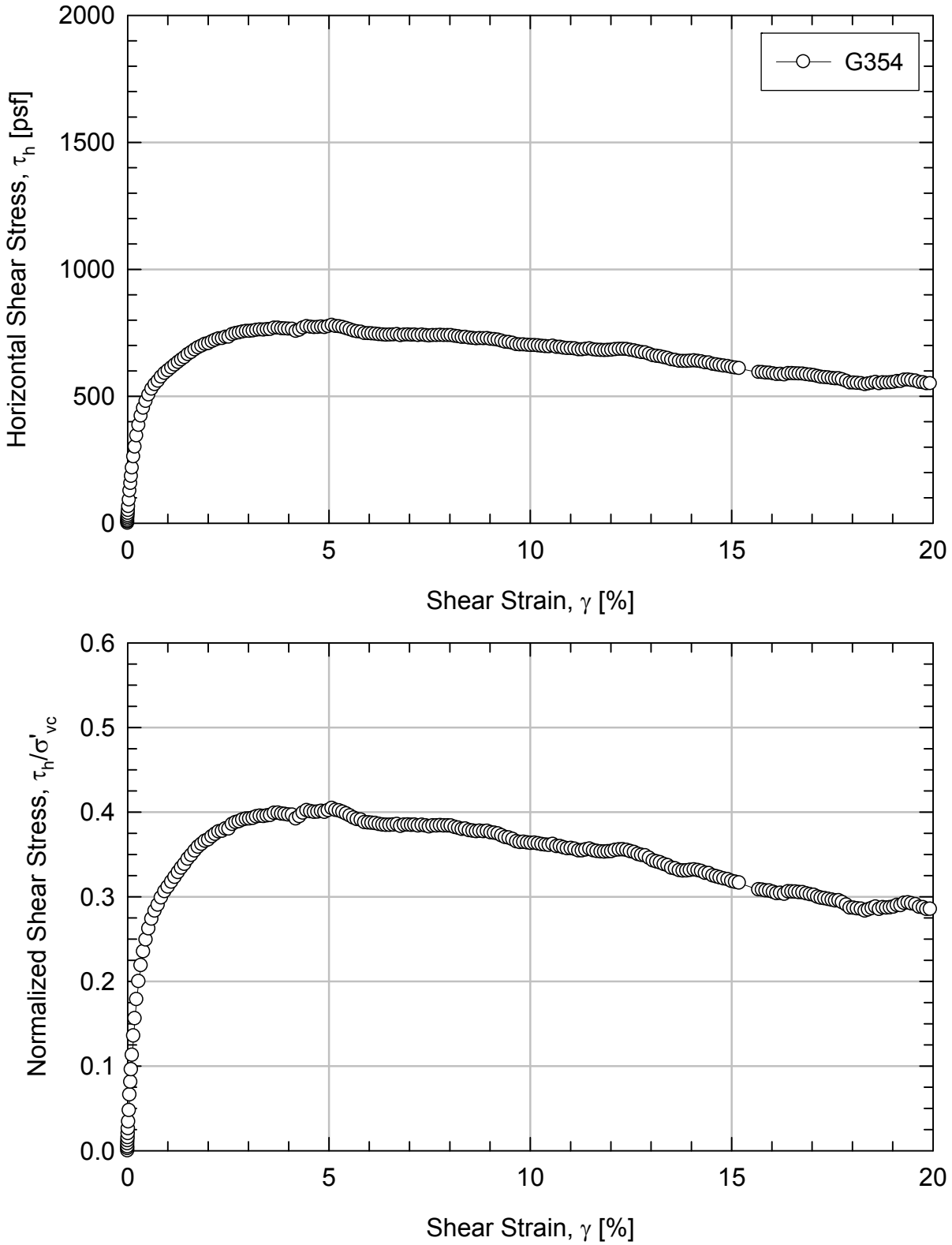


Figure 5.4.27 Horizontal shear stress and normalized shear stress versus shear strain for test DSS G354 on sample 09-103B S2 33.5-36.0 ft.

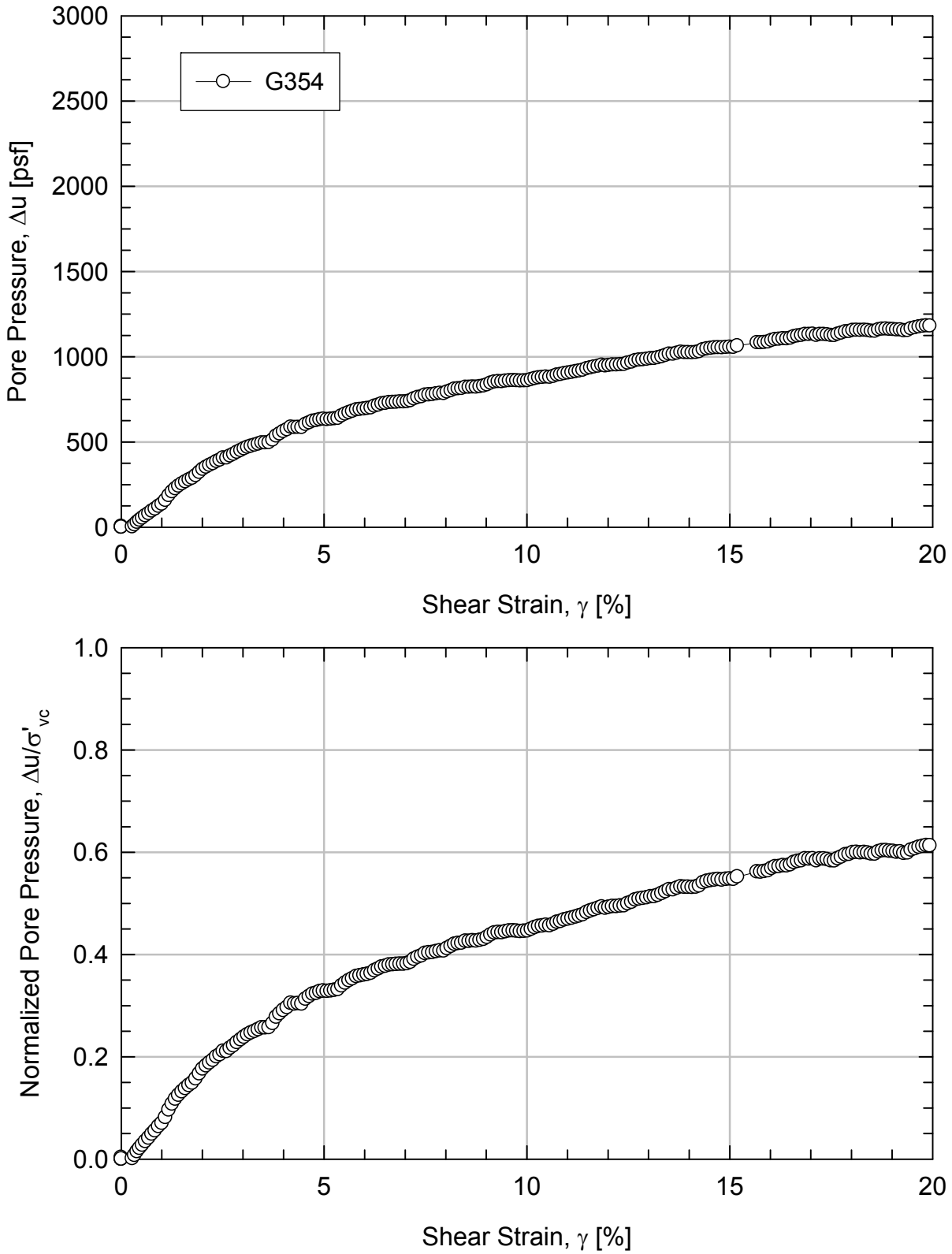


Figure 5.4.28 Pore pressure and normalized pore pressure versus shear strain for test DSS G354 on sample 09-103B S2 33.5-36.0 ft.

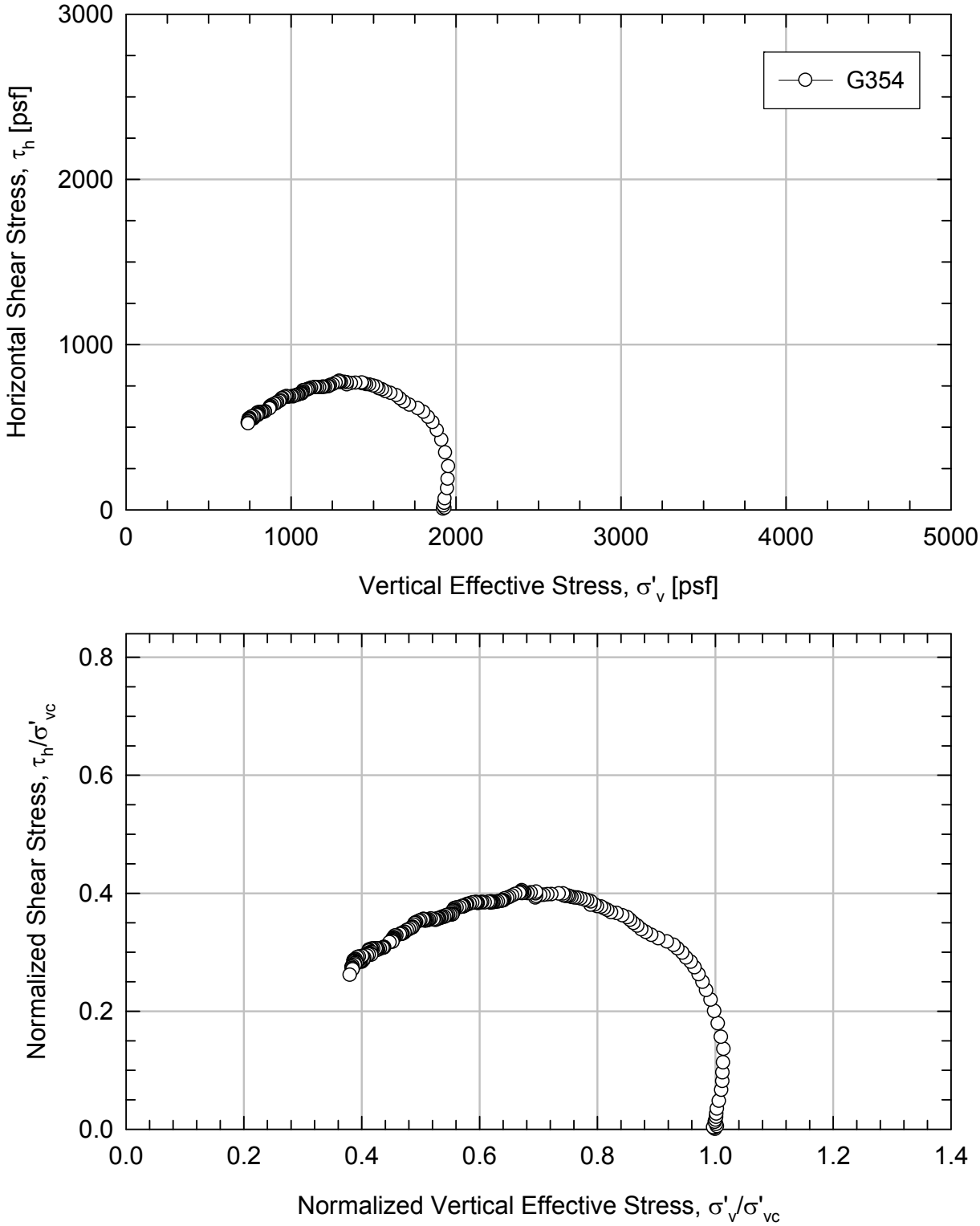


Figure 5.4.29 Horizontal shear stress versus vertical effective stress and normalized shear stress versus normalized vertical effective stress for test DSS G354 on sample 09-103B S2 33.5-36.0 ft.

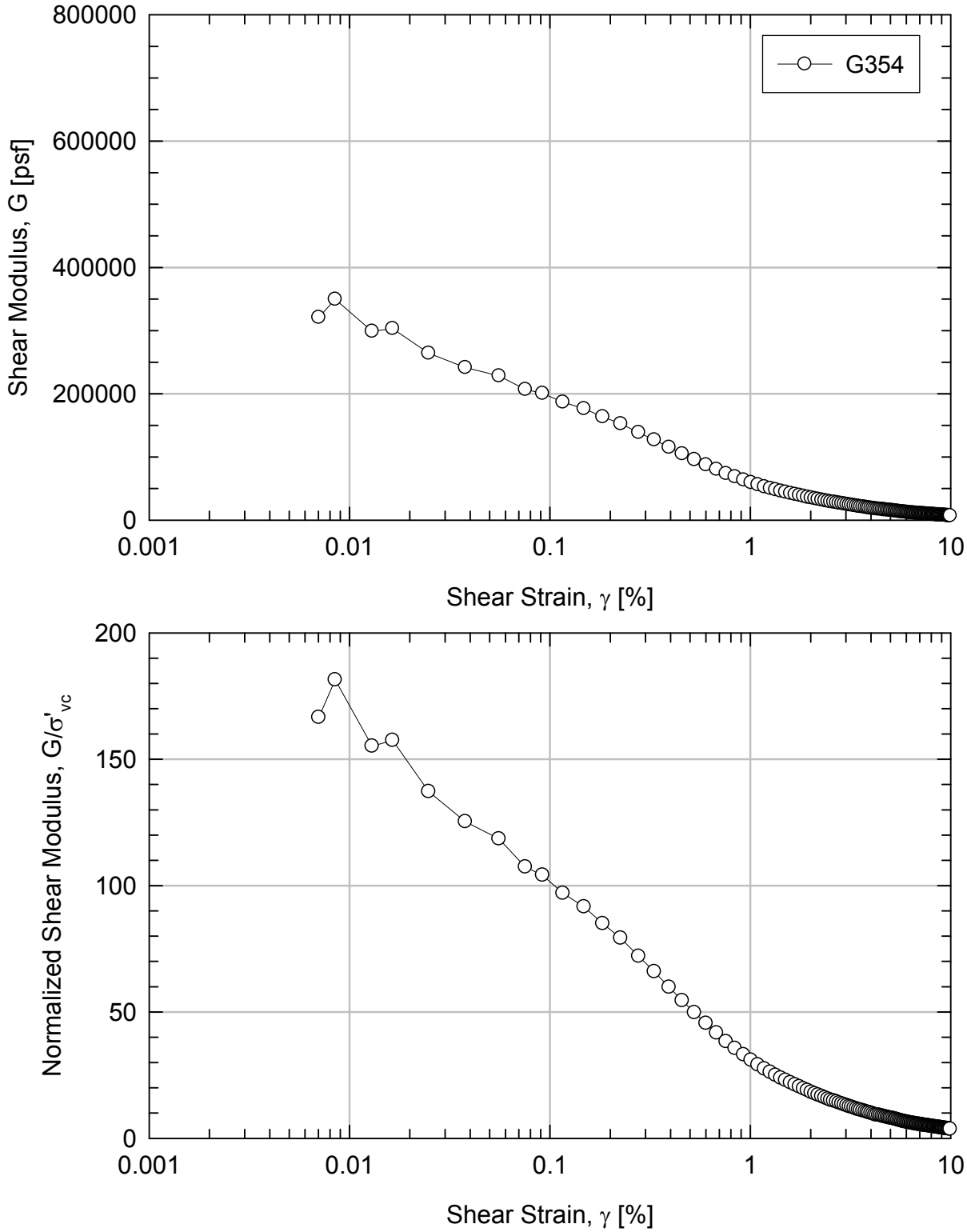


Figure 5.4.30 Shear modulus and normalized shear modulus versus shear strain for test DSS G354 on sample 09-103B S2 33.5-36.0 ft.

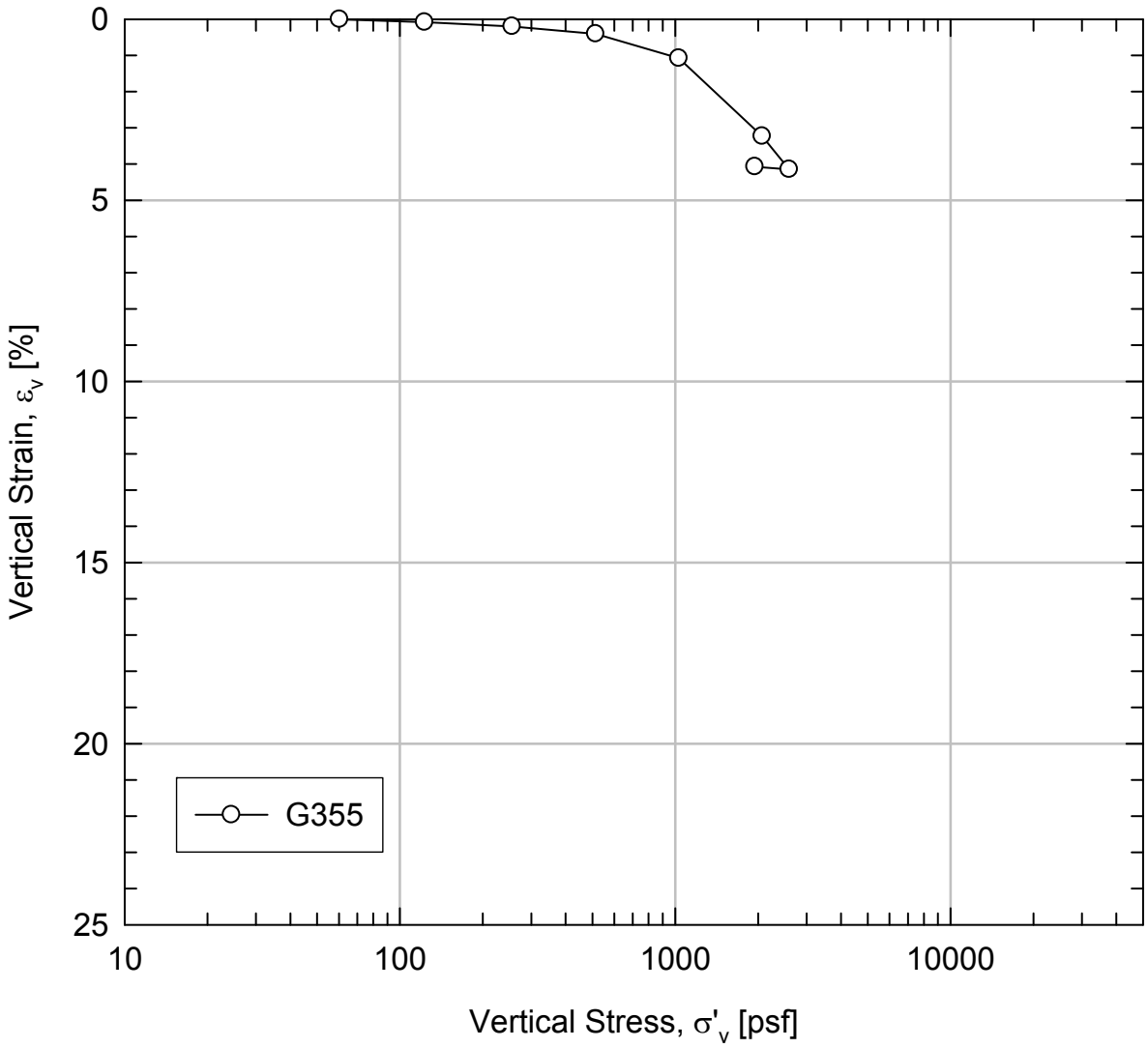


Figure 5.4.31 Vertical strain versus vertical effective stress for test DSS G355 on sample 09-103B S2 33.5-36.0 ft

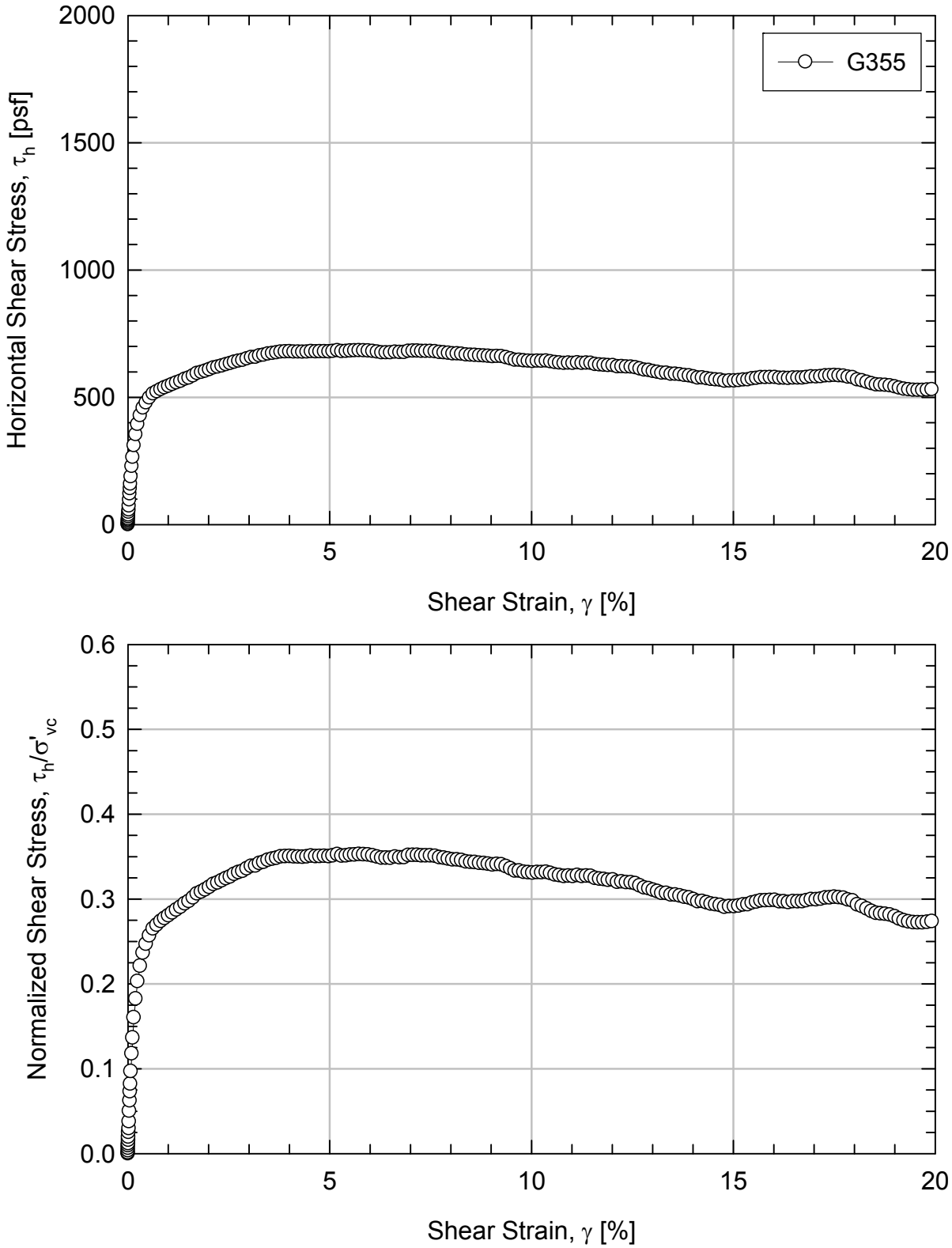


Figure 5.4.32 Horizontal shear stress and normalized shear stress versus shear strain for test DSS G355 on sample 09-103B S2 33.5-36.0 ft

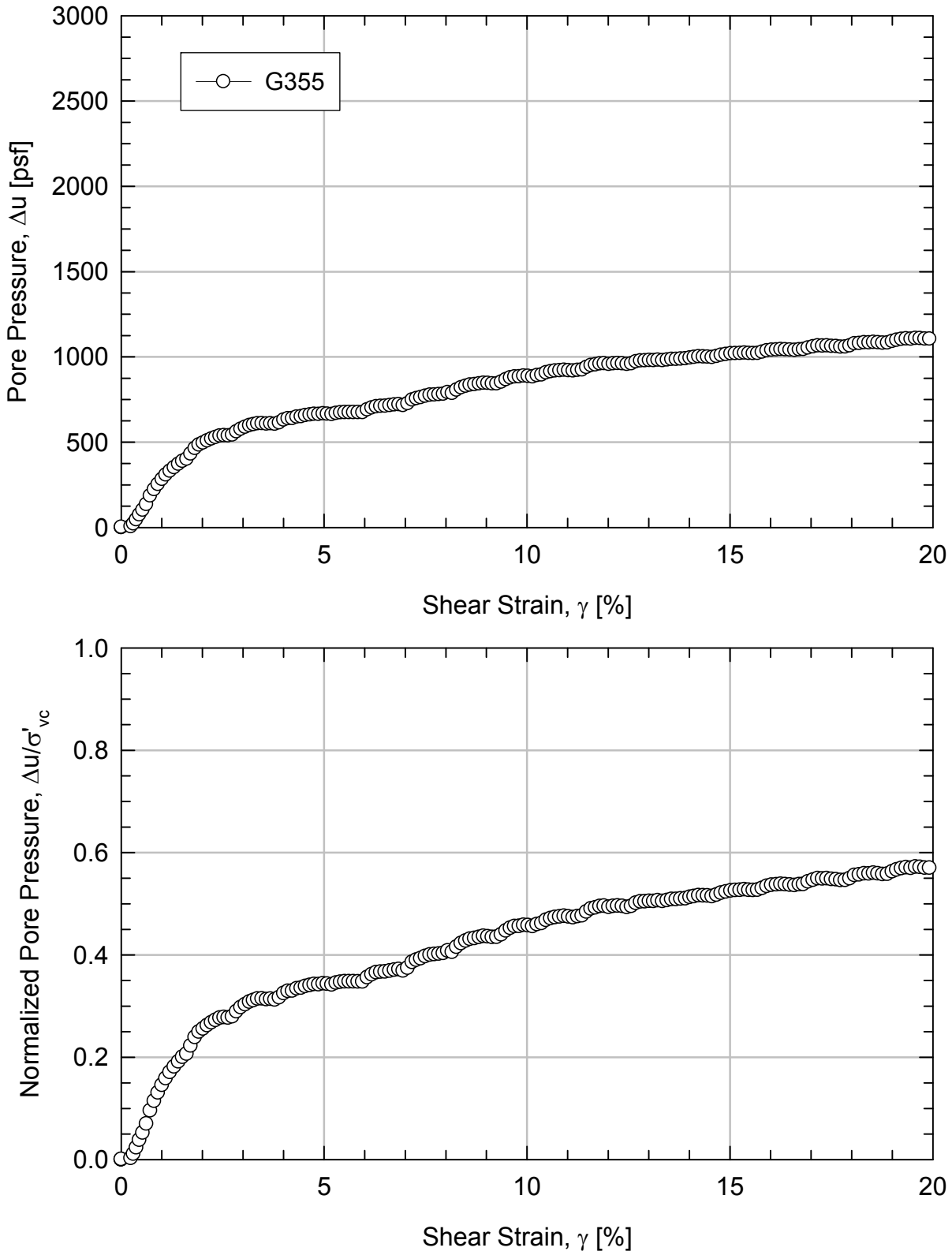


Figure 5.4.33 Pore pressure and normalized pore pressure versus shear strain for test DSS G355 on sample 09-103B S2 33.5-36.0 ft

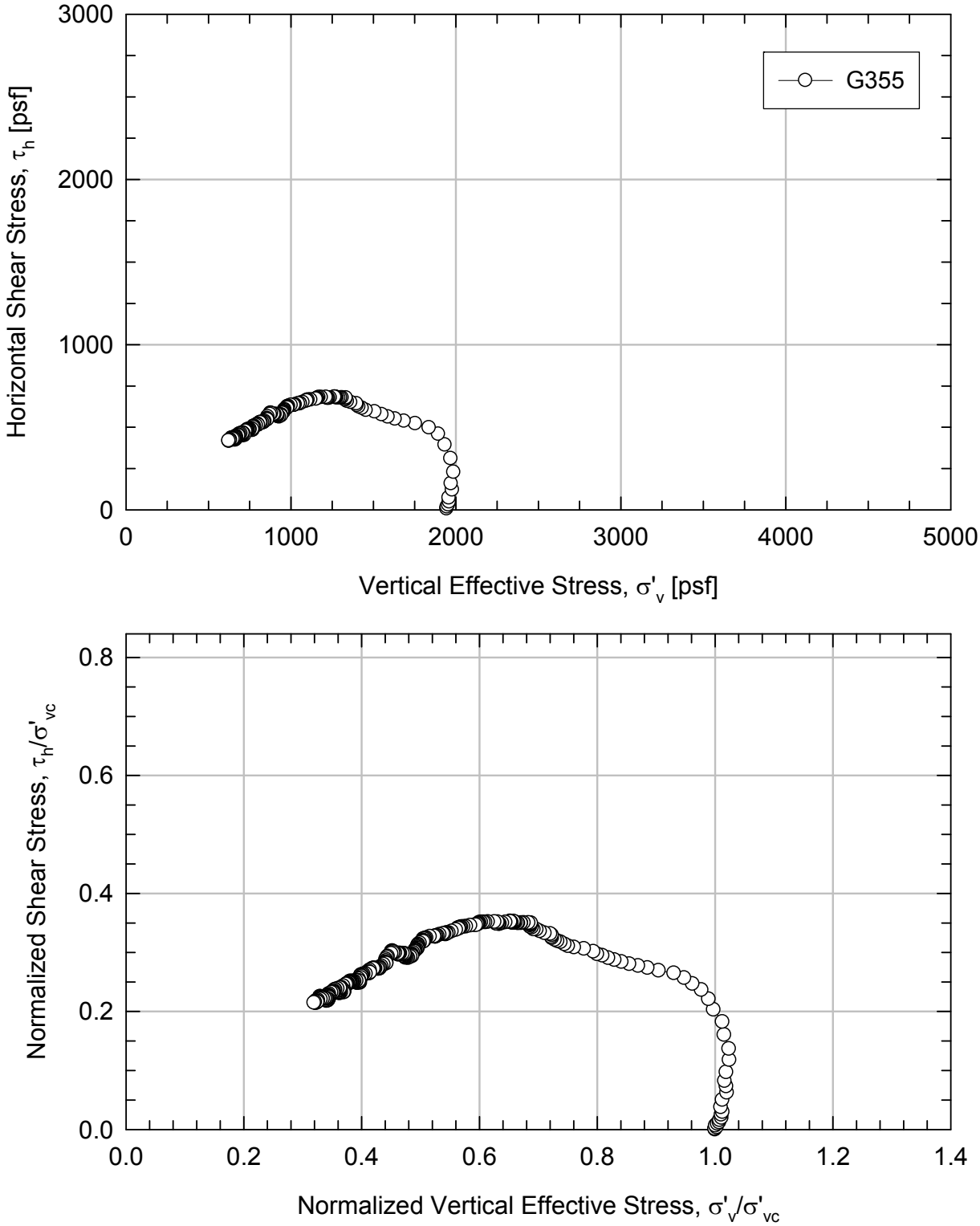


Figure 5.4.34 Horizontal shear stress versus vertical effective stress and normalized shear stress versus normalized vertical effective stress for test DSS G355 on sample 09-103B S2 33.5-36.0 ft

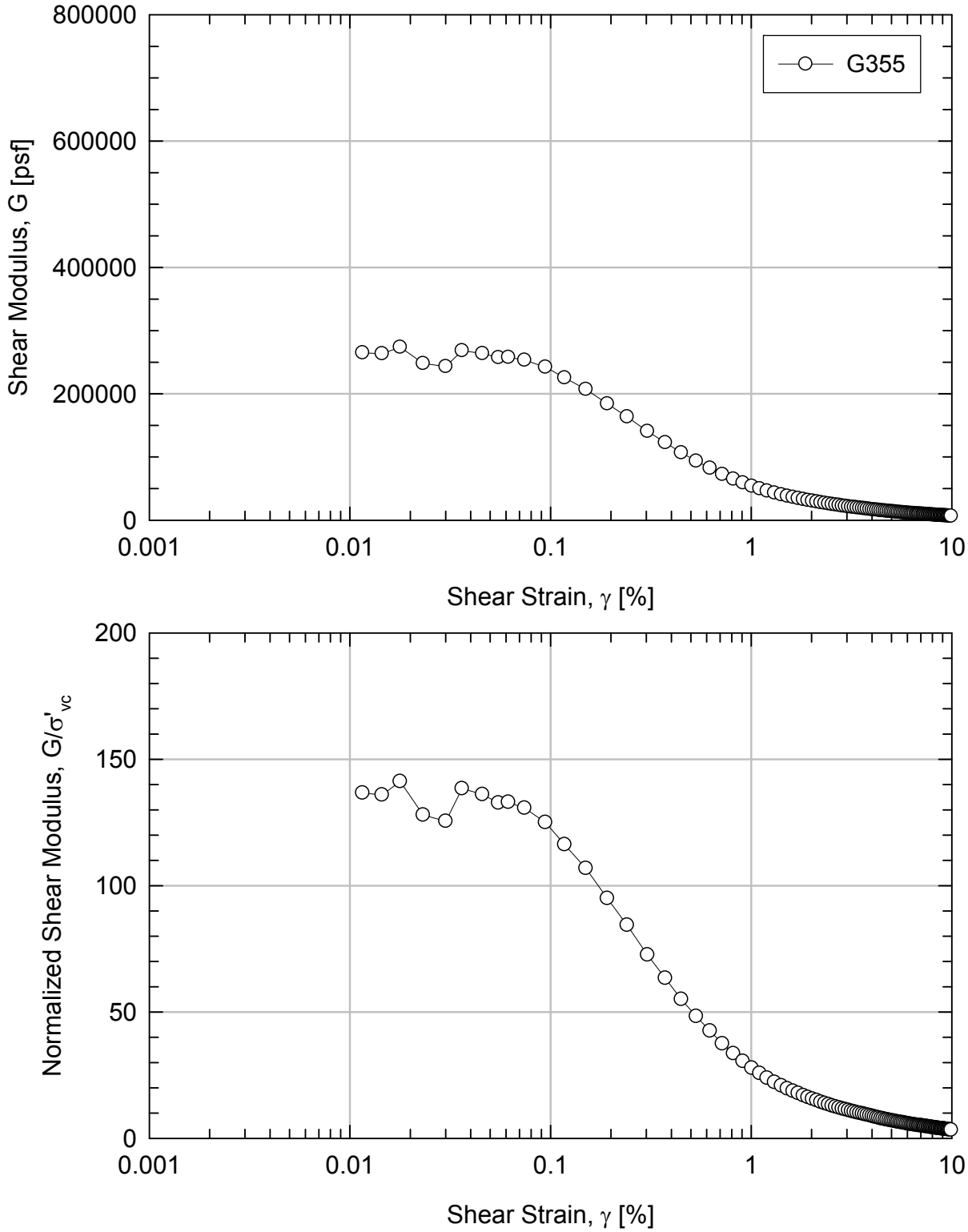


Figure 5.4.35 Shear modulus and normalized shear modulus versus shear strain for test DSS G355 on sample 09-103B S2 33.5-36.0 ft

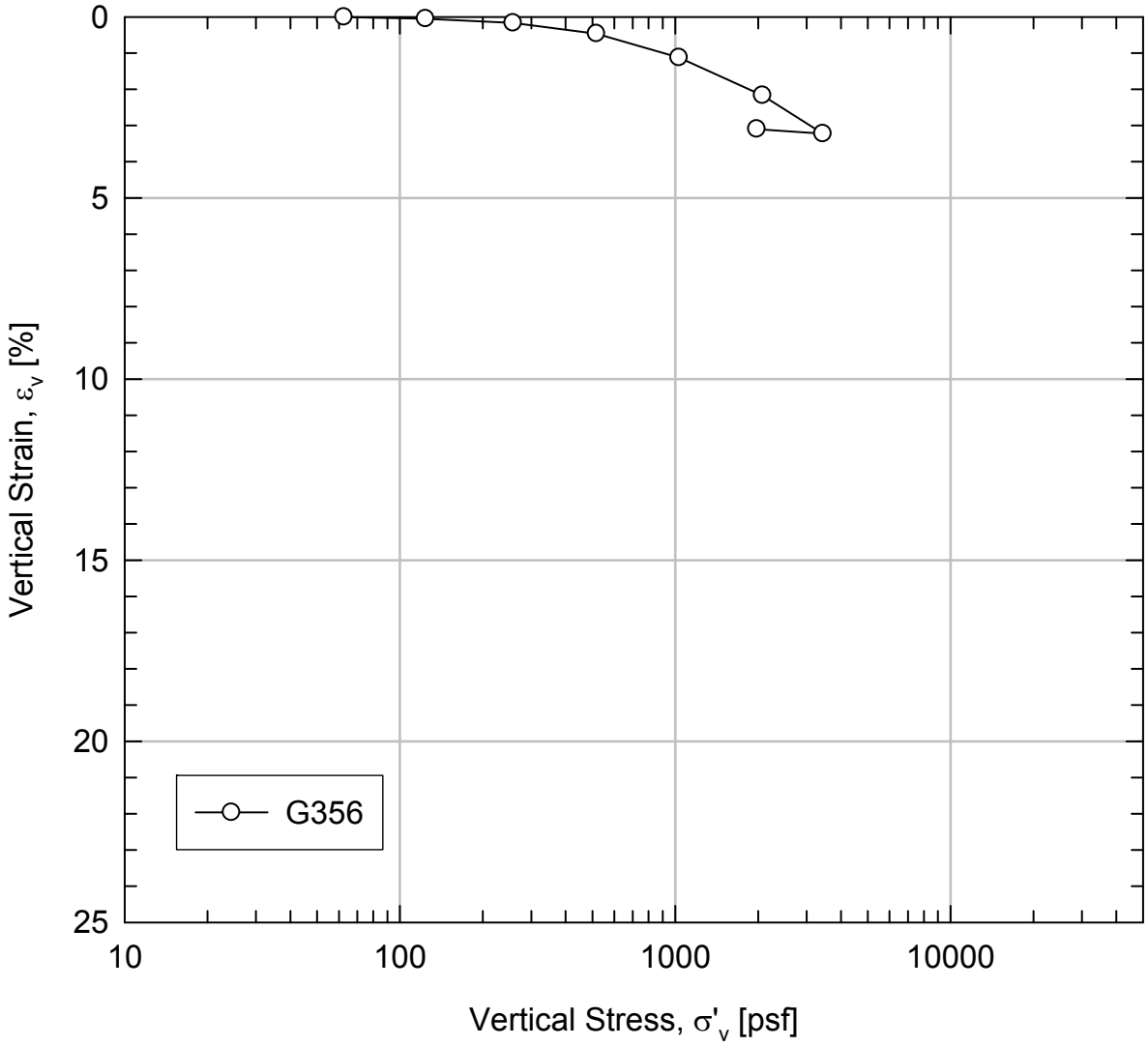


Figure 5.4.36 Vertical strain versus vertical effective stress for test DSS G356 on sample 09-100B S5 33.0-35.5 ft

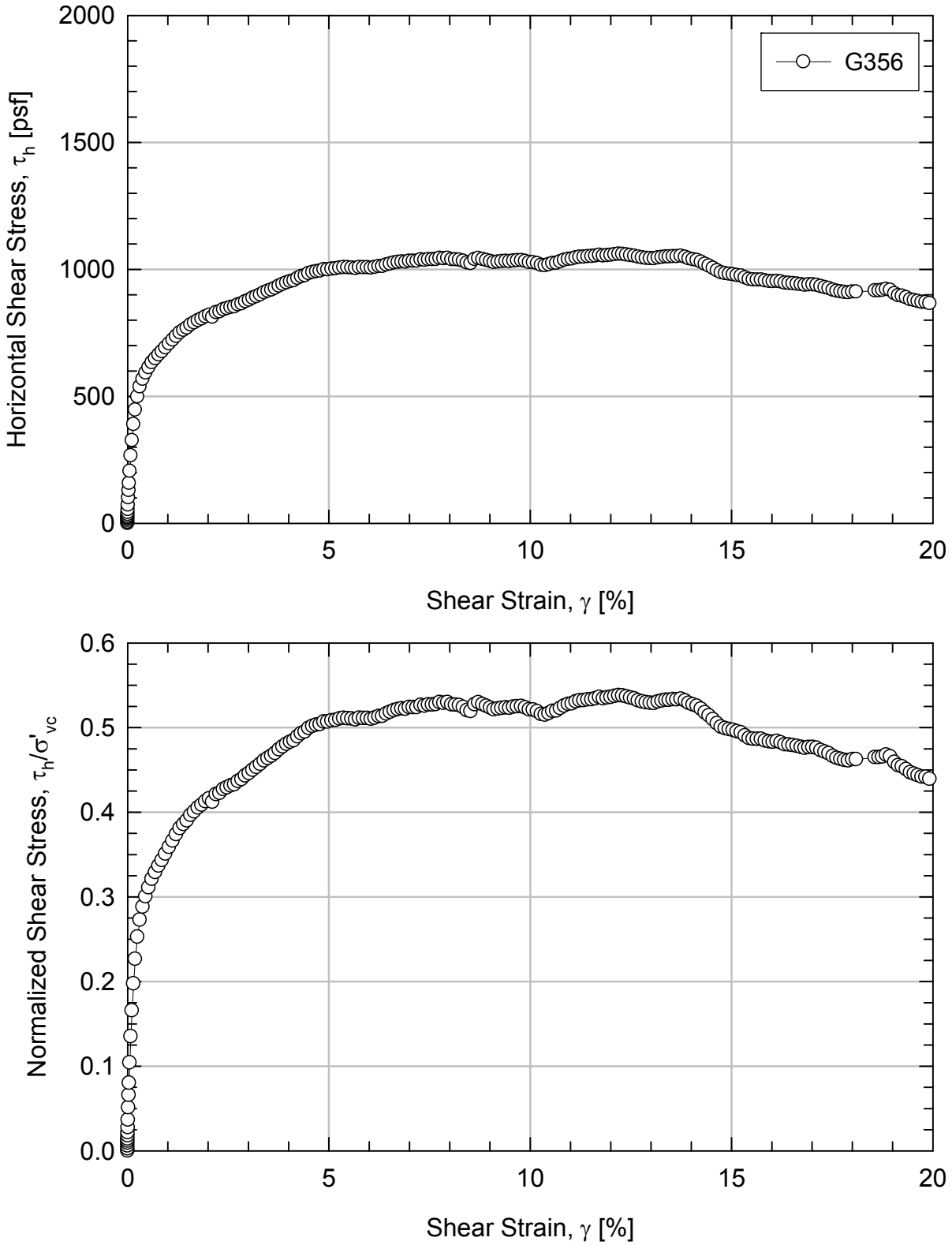


Figure 5.4.37 Horizontal shear stress and normalized shear stress versus shear strain for test DSS G356 on sample 09-100B S5 33.0-35.5 ft

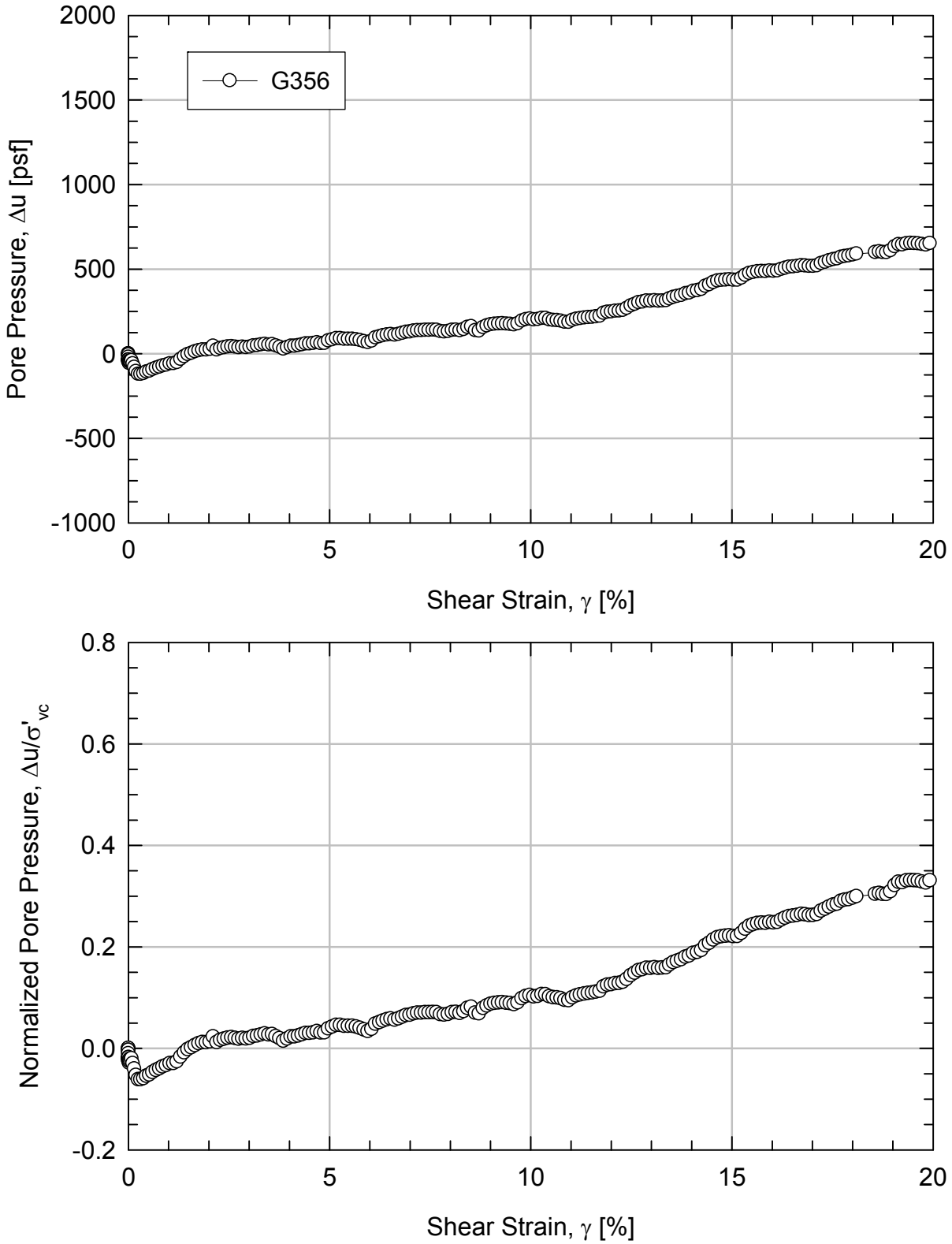


Figure 5.4.38 Pore pressure and normalized pore pressure versus shear strain for test DSS G356 on sample 09-100B S5 33.0-35.5 ft

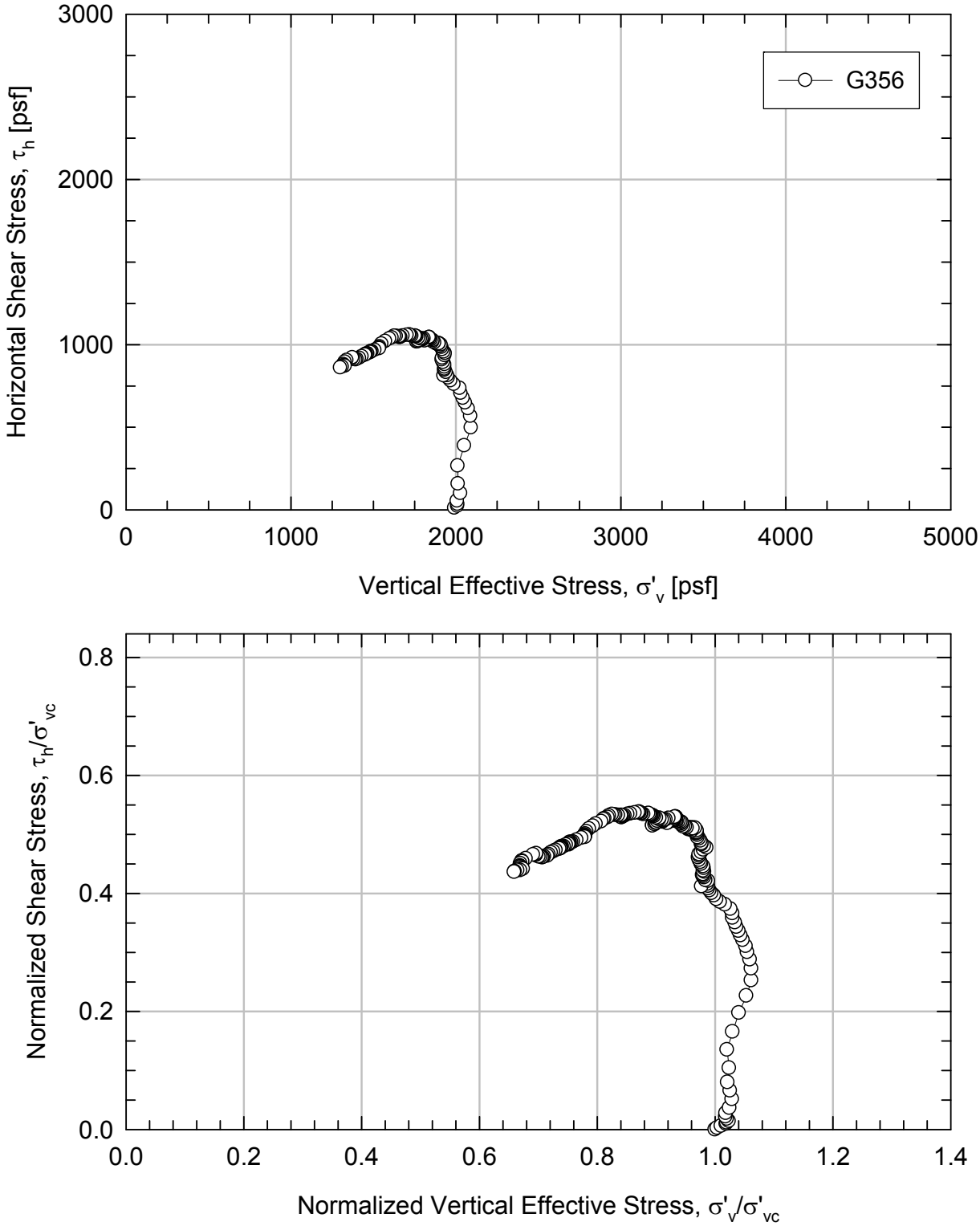


Figure 5.4.39 Horizontal shear stress versus vertical effective stress and normalized shear stress versus normalized vertical effective stress for test DSS G356 on sample 09-100B S5 33.0-35.5 ft

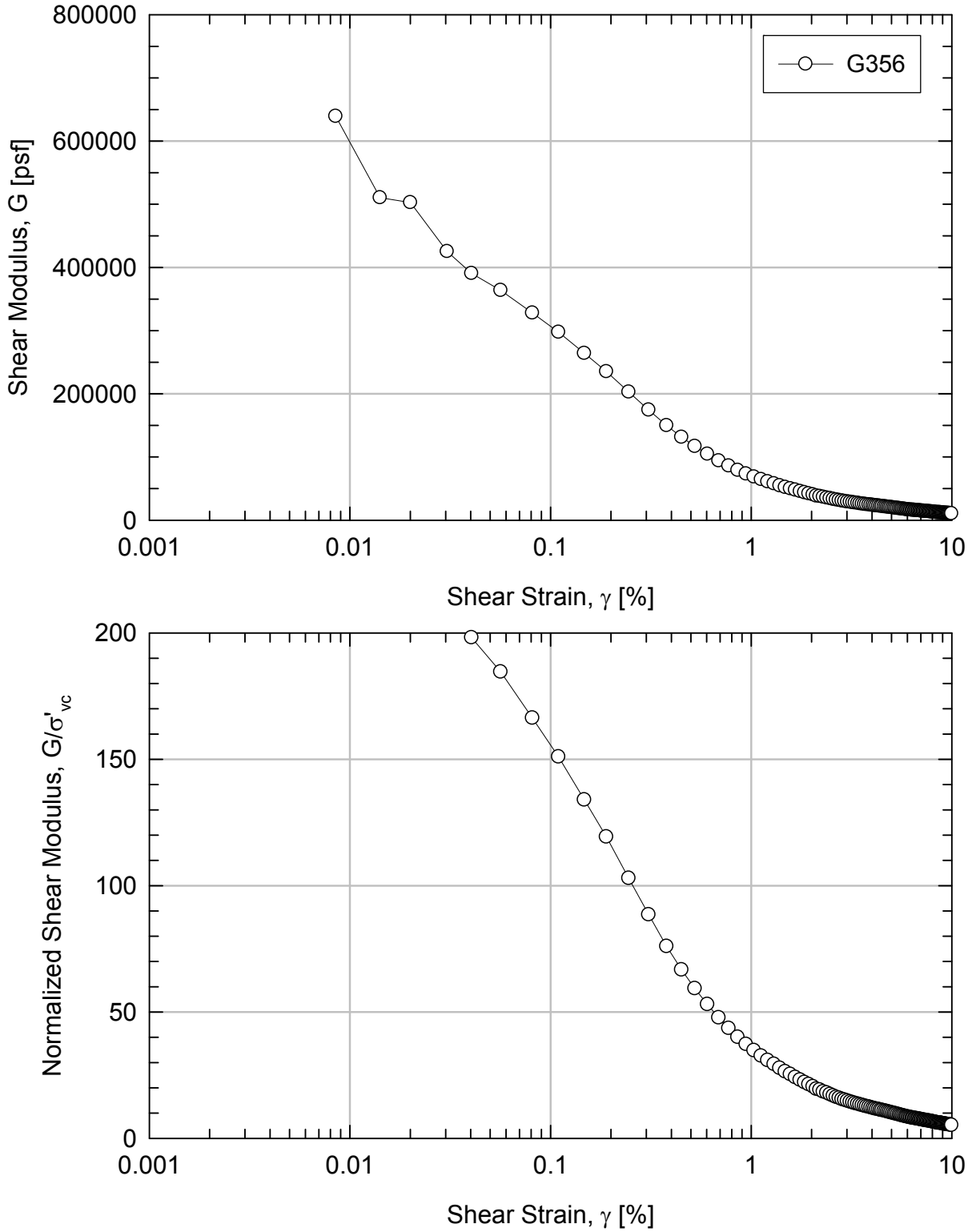


Figure 5.4.40 Shear modulus and normalized shear modulus versus shear strain for test DSS G356 on sample 09-100B S5 33.0-35.5 ft

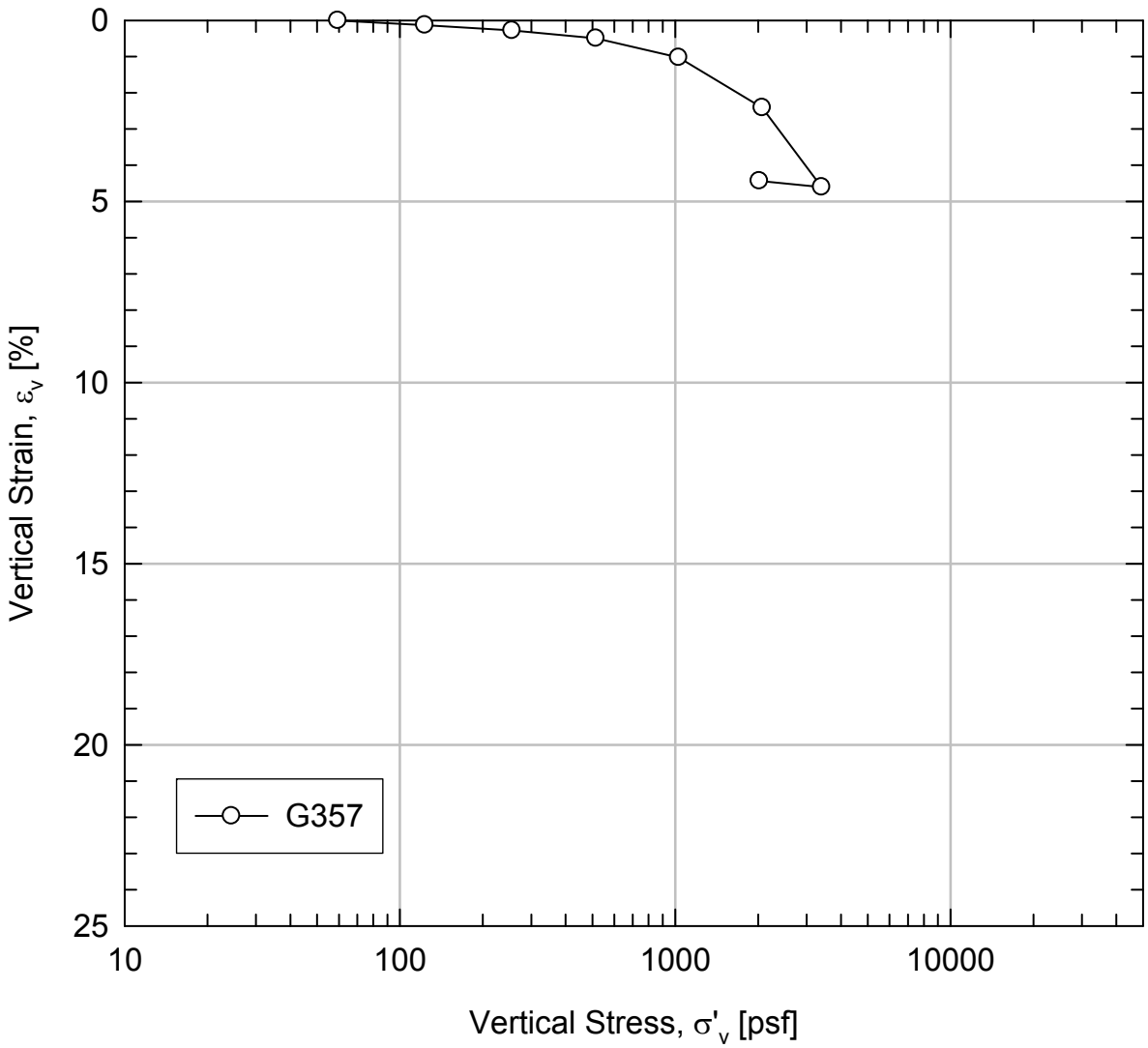


Figure 5.4.41 Vertical strain versus vertical effective stress for test DSS G357 on sample 09-100B S6 35.5-38.0 ft

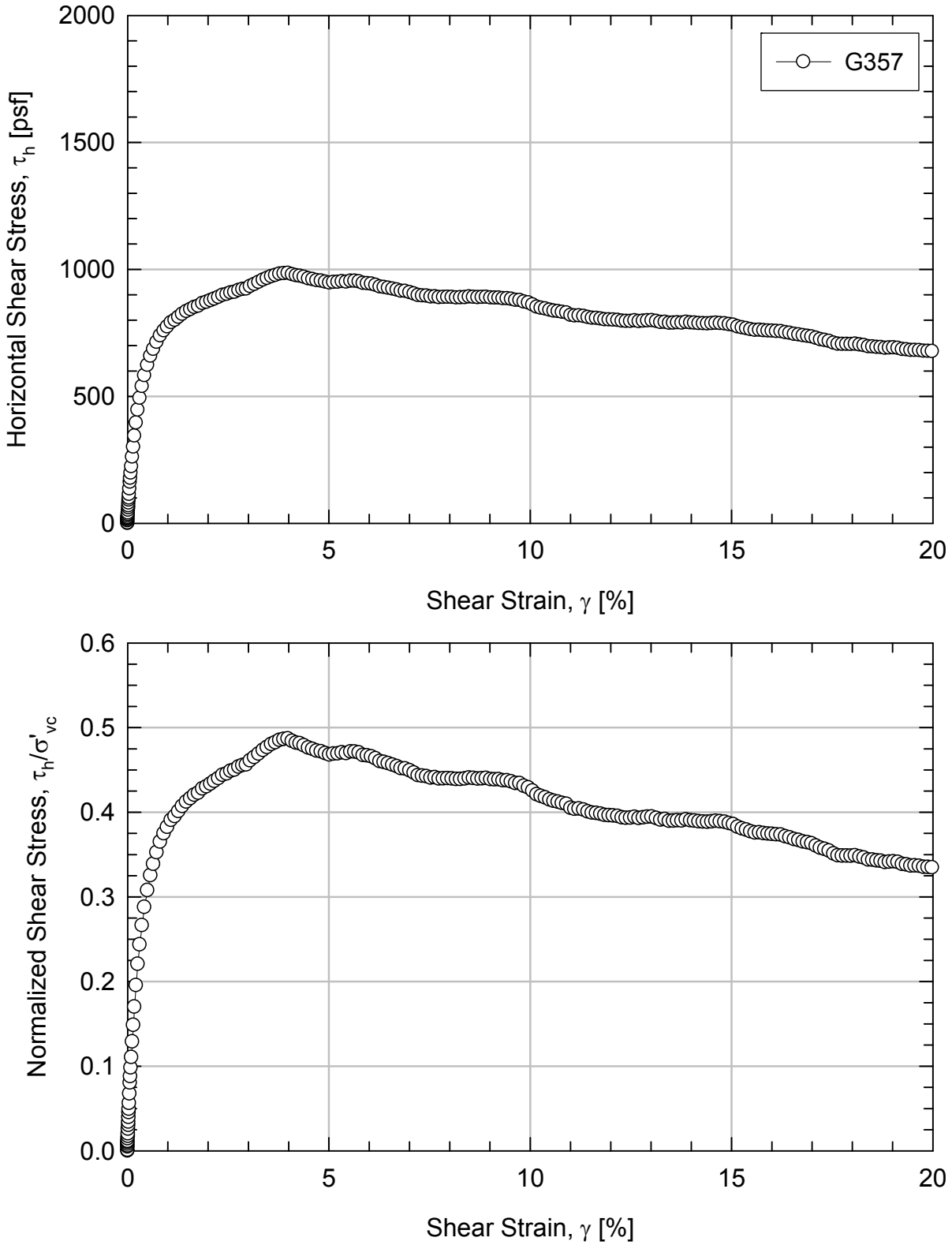


Figure 5.4.42 Horizontal shear stress and normalized shear stress versus shear strain for test DSS G357 on sample 09-100B S6 35.5-38.0 ft

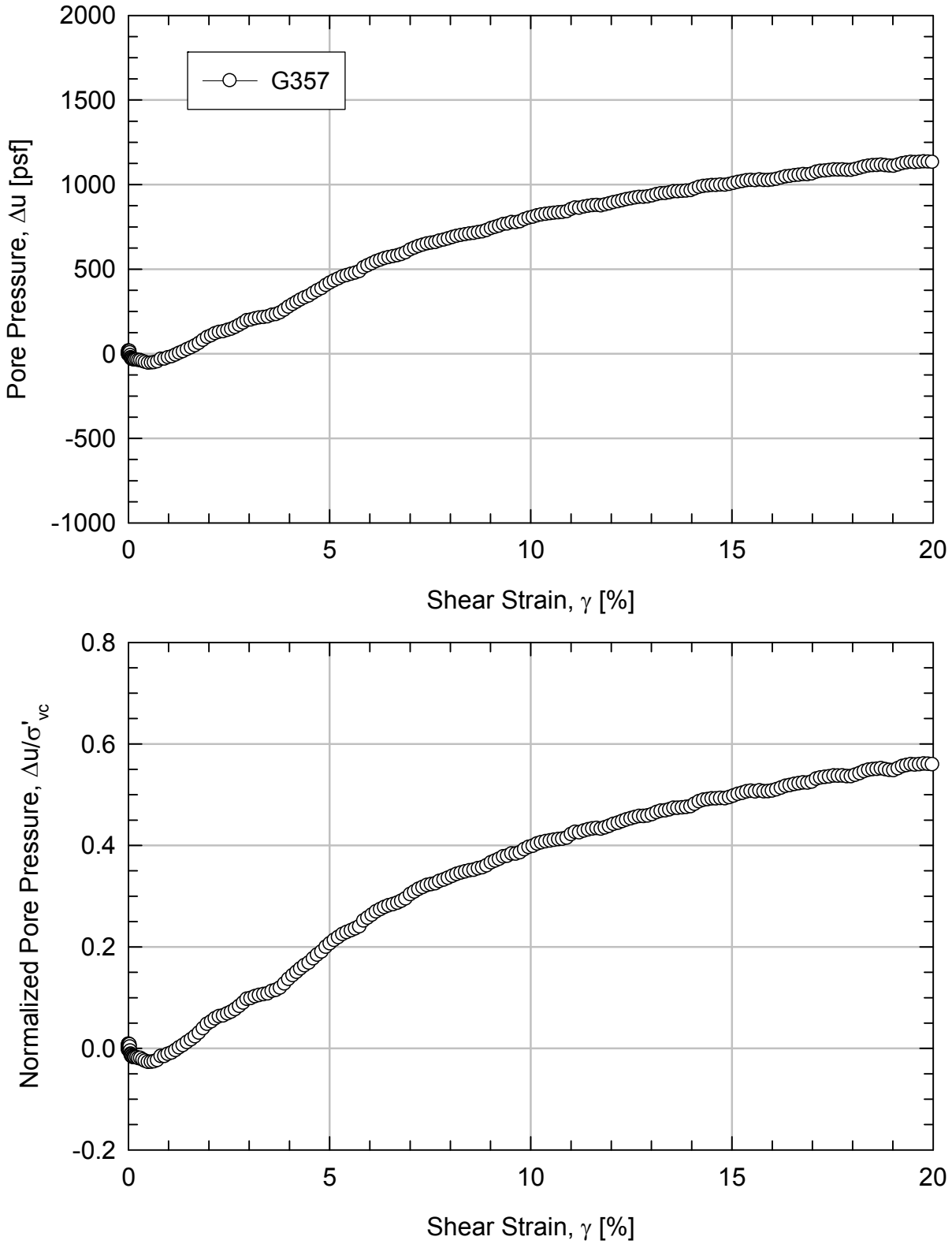


Figure 5.4.43 Pore pressure and normalized pore pressure versus shear strain for test DSS G357 on sample 09-100B S6 35.5-38.0 ft

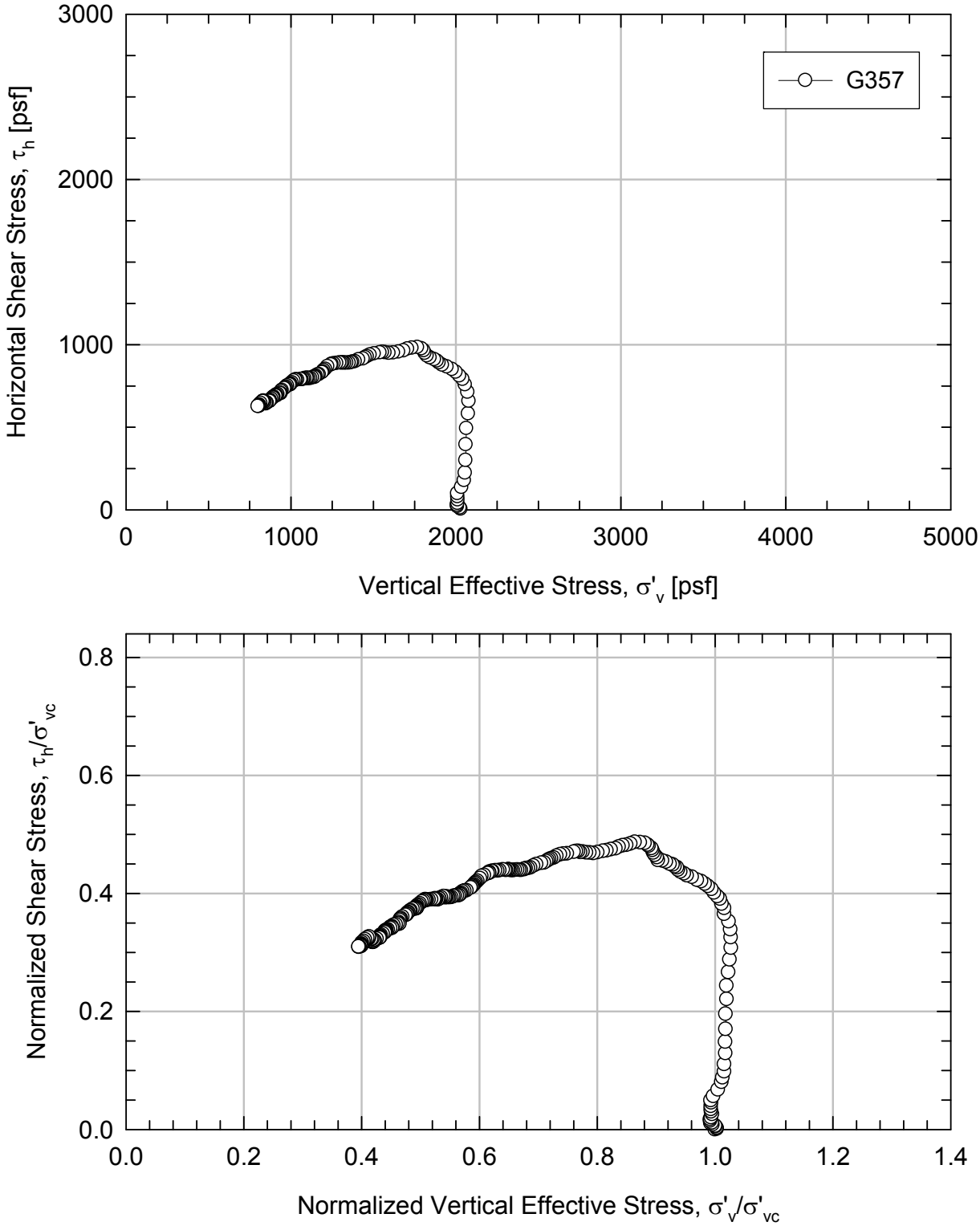


Figure 5.4.44 Horizontal shear stress versus vertical effective stress and normalized shear stress versus normalized vertical effective stress for test DSS G357 on sample 09-100B S6 35.5-38.0 ft

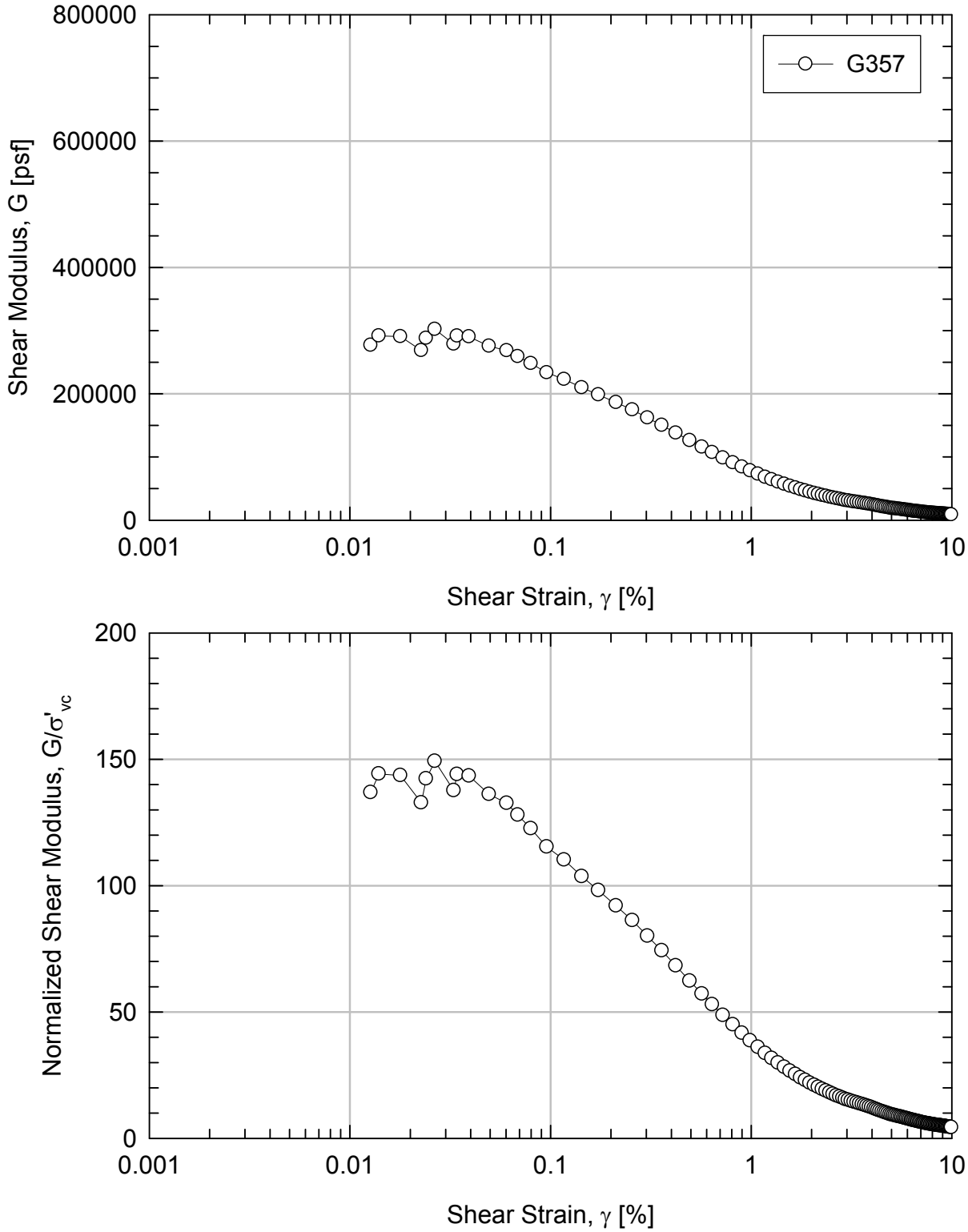


Figure 5.4.45 Shear modulus and normalized shear modulus versus shear strain for test DSS G357 on sample 09-100B S6 35.5-38.0 ft

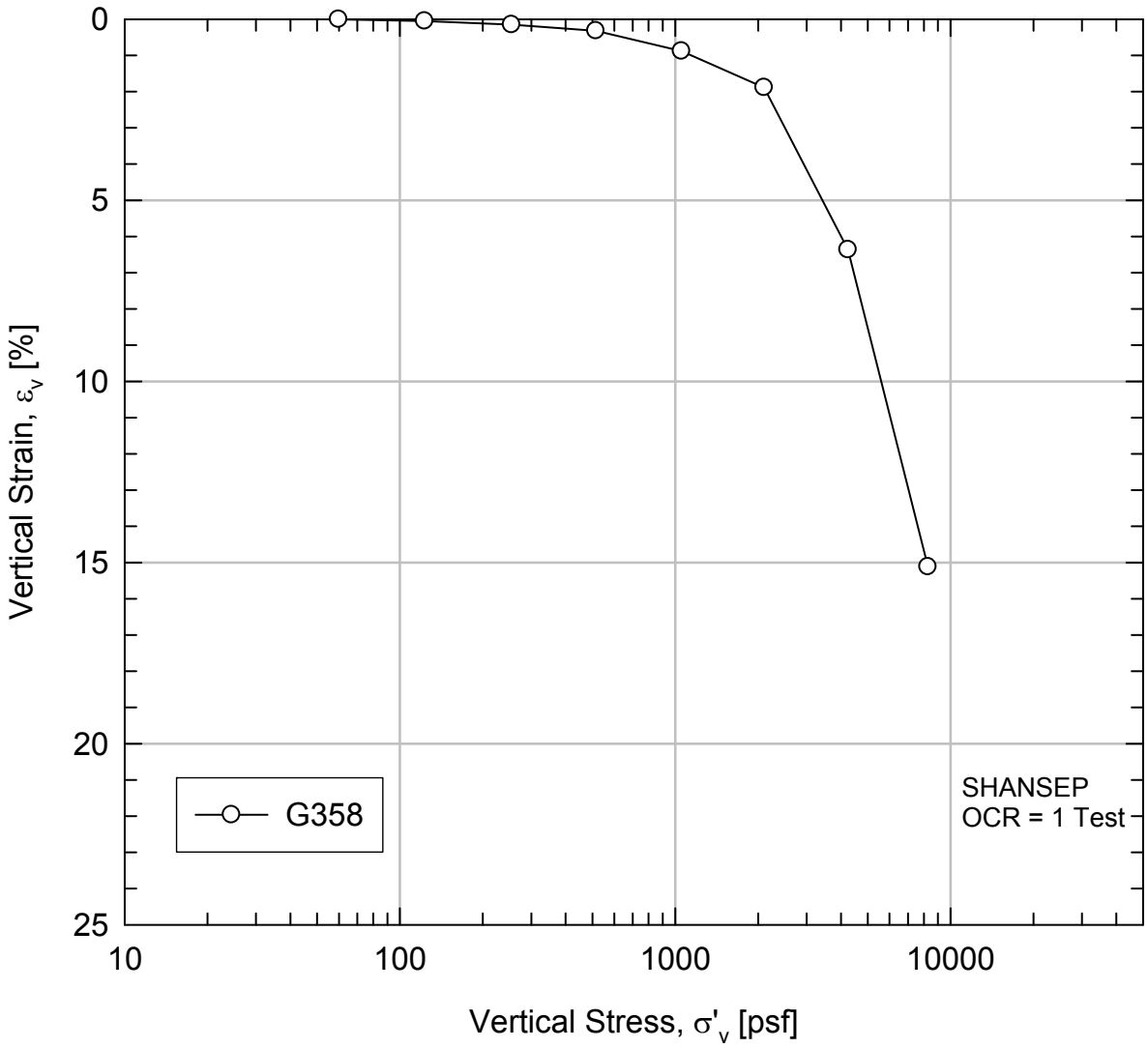


Figure 5.4.46 Vertical strain versus vertical effective stress for test DSS G358 on sample 09-108B S2 23.5-26.0 ft

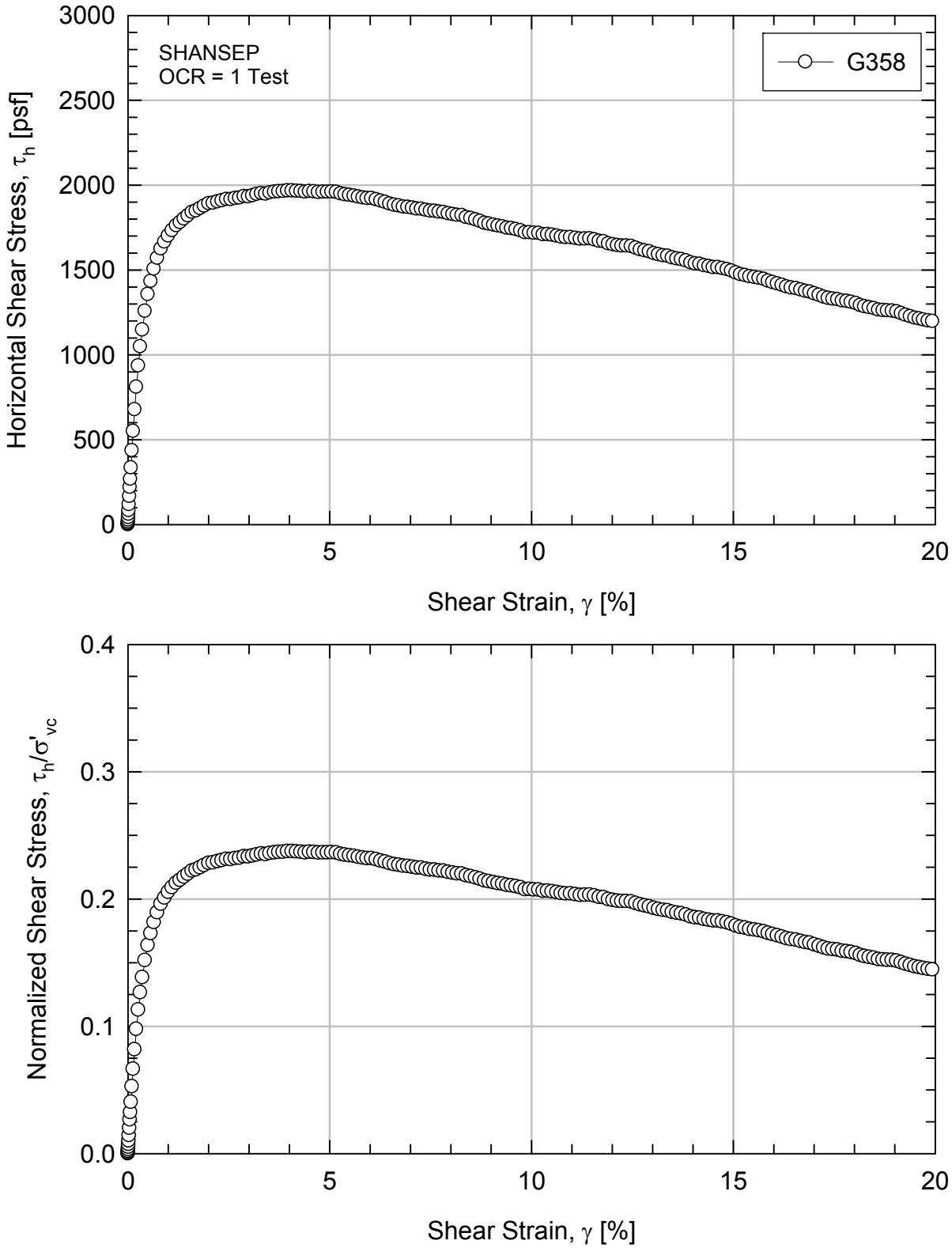


Figure 5.4.47 Horizontal shear stress and normalized shear stress versus shear strain for test DSS G358 on sample 09-108B S2 23.5-26.0 ft

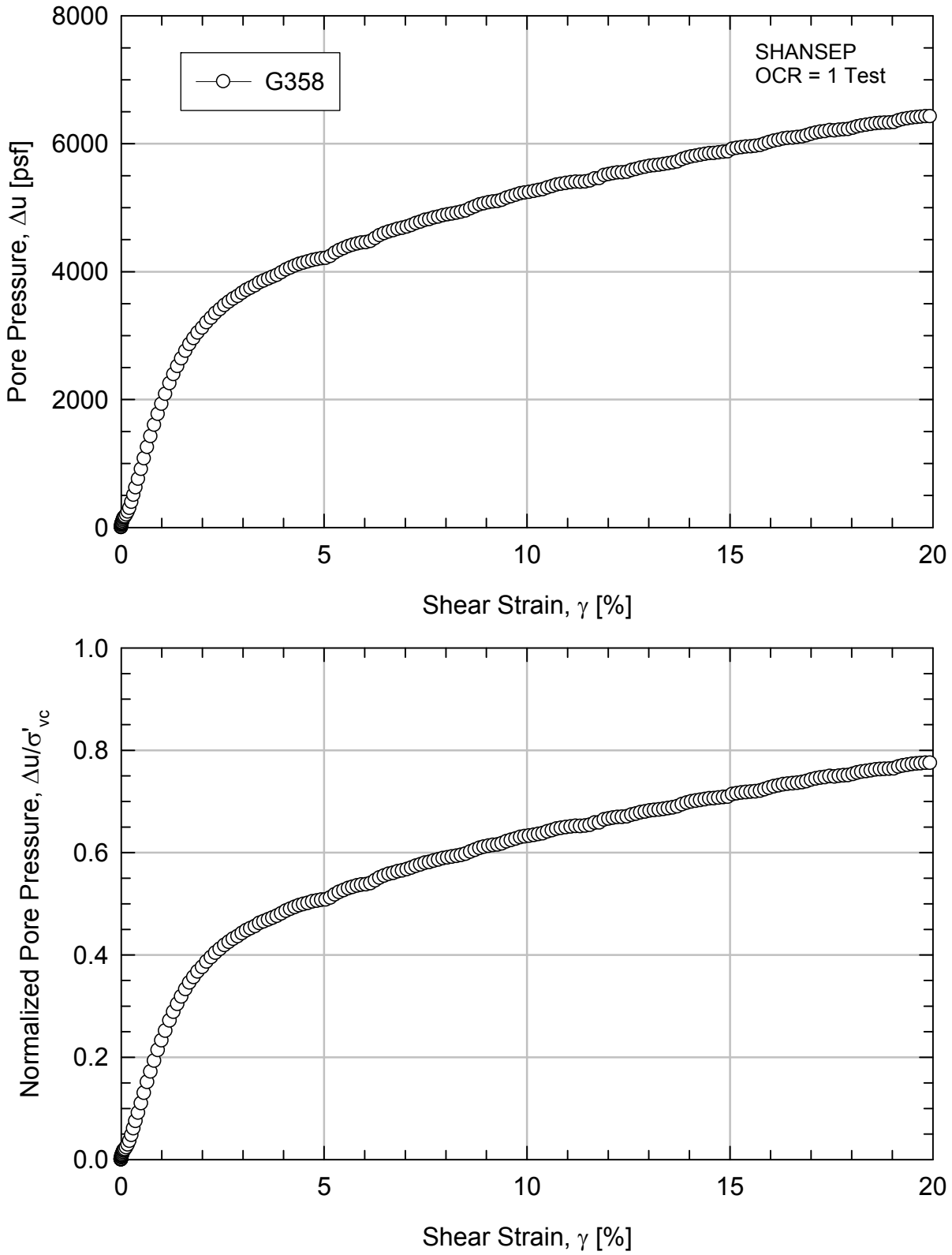


Figure 5.4.48 Pore pressure and normalized pore pressure versus shear strain for test DSS G358 on sample 09-108B S2 23.5-26.0 ft

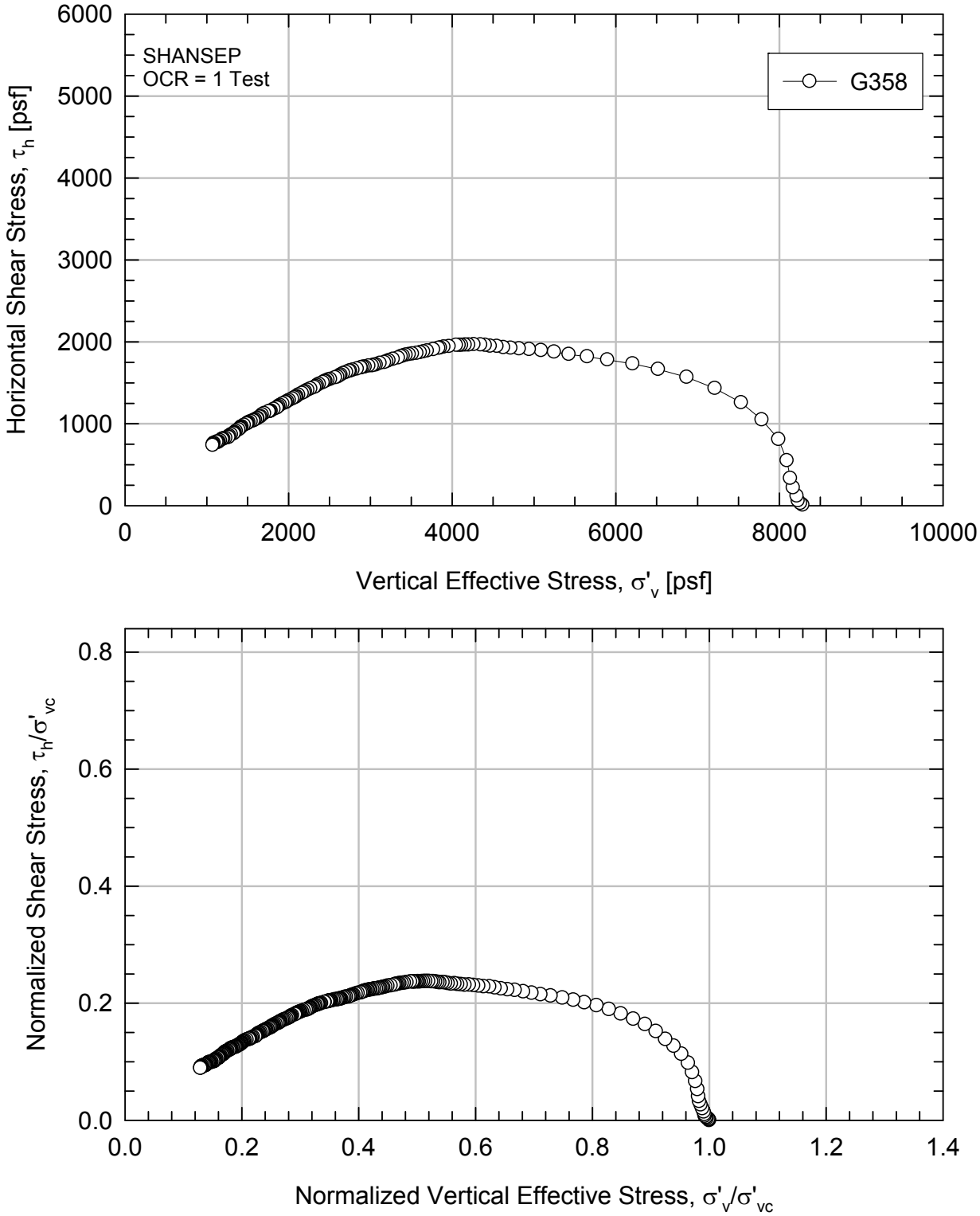


Figure 5.4.49 Horizontal shear stress versus vertical effective stress and normalized shear stress versus normalized vertical effective stress for test DSS G358 on sample 09-108B S2 23.5-26.0 ft

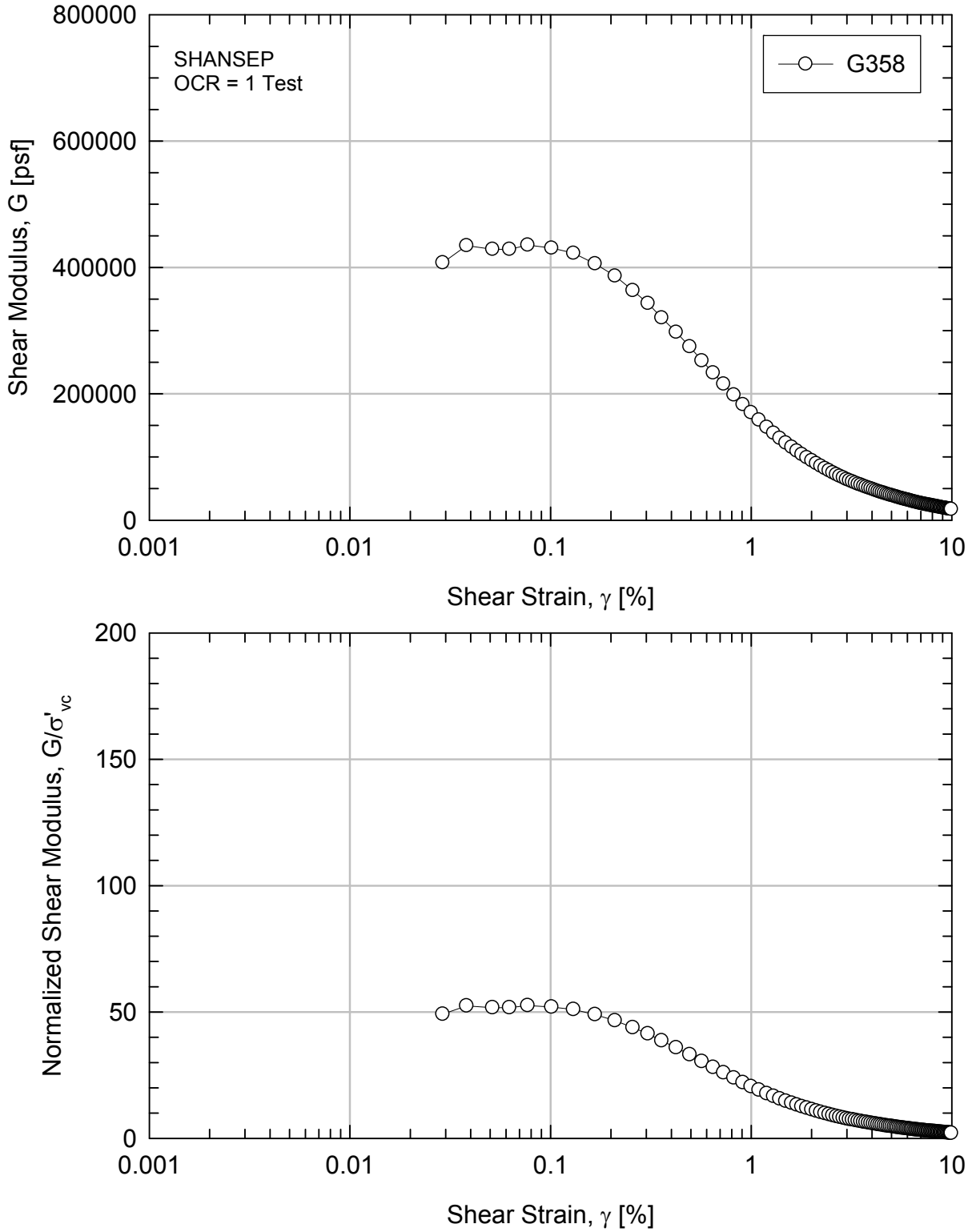


Figure 5.4.50 Shear modulus and normalized shear modulus versus shear strain for test DSS G358 on sample 09-108B S2 23.5-26.0 ft

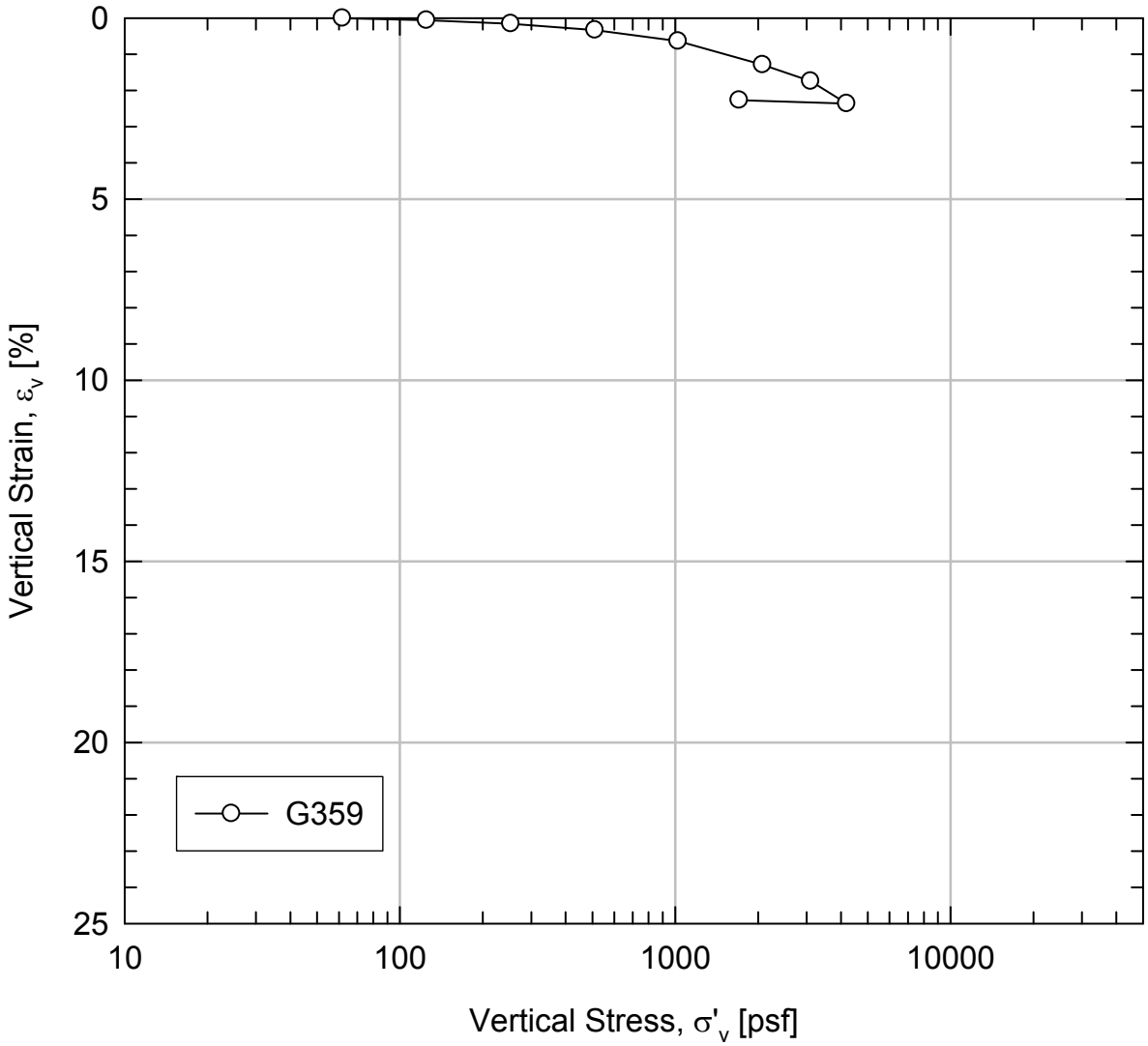


Figure 5.4.51 Vertical strain versus vertical effective stress for test DSS G359 on sample 09-408B S7 35.5-38.0 ft

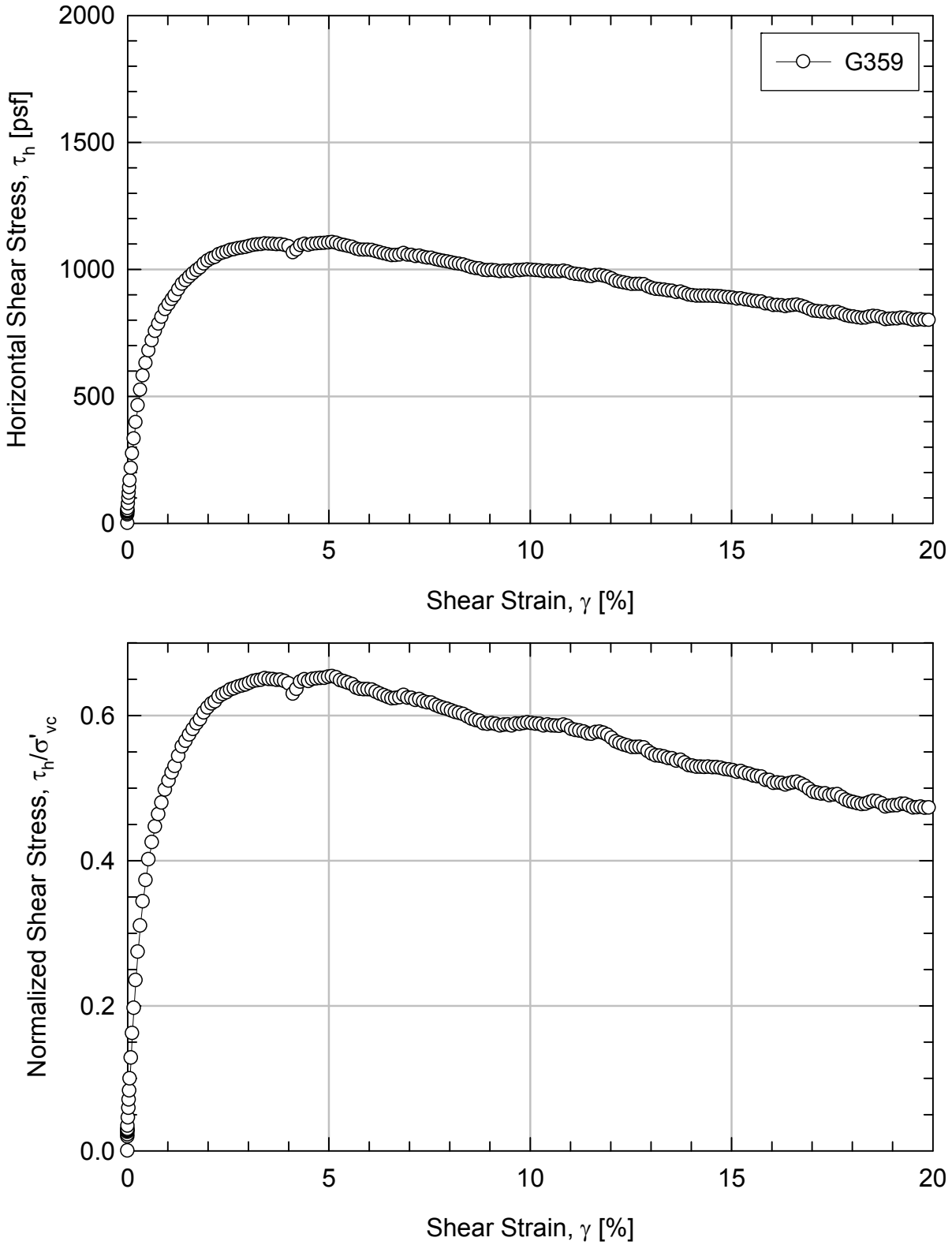


Figure 5.4.52 Horizontal shear stress and normalized shear stress versus shear strain for test DSS G359 on sample 09-408B S7 35.5-38.0 ft

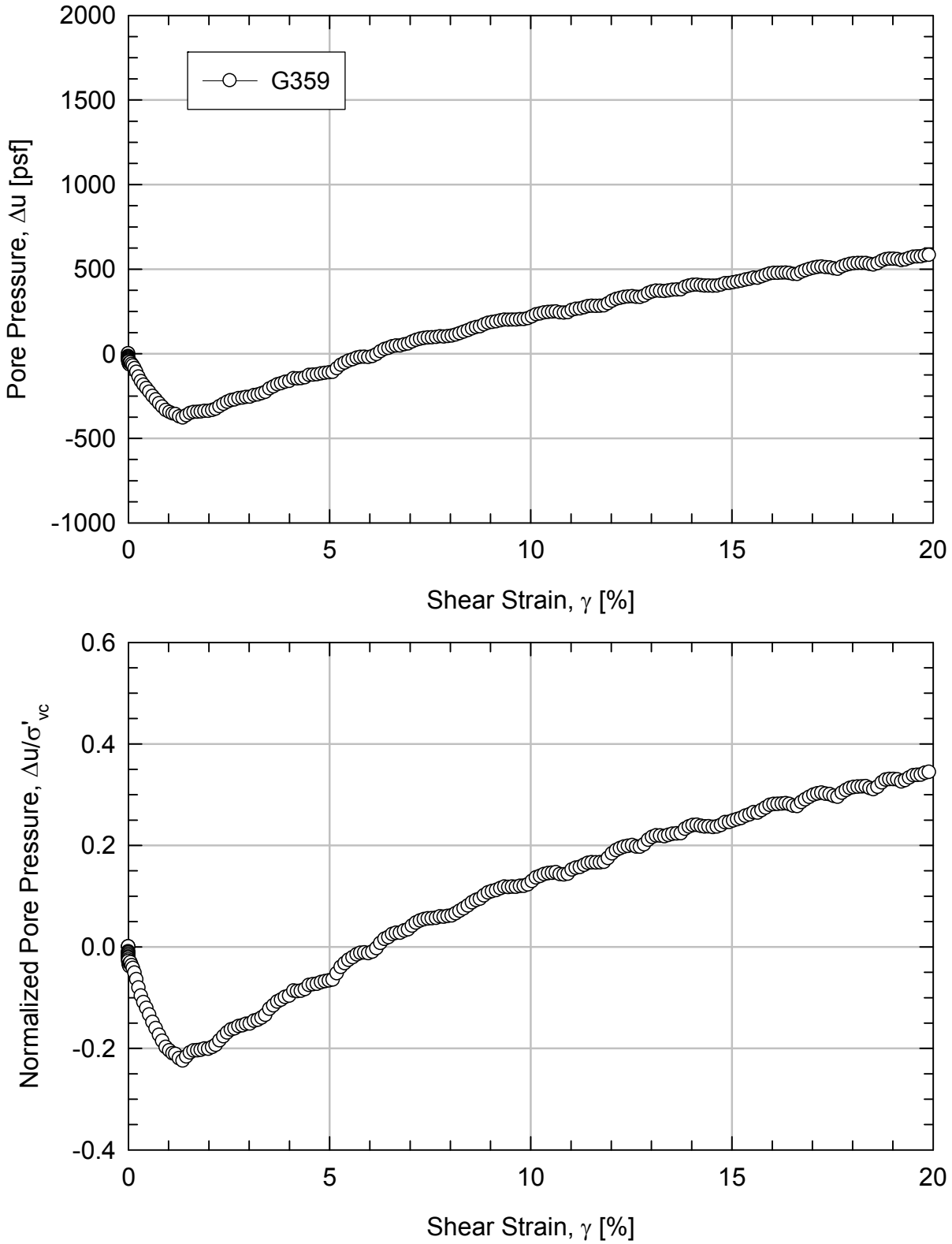


Figure 5.4.53 Pore pressure and normalized pore pressure versus shear strain for test DSS G359 on sample 09-408B S7 35.5-38.0 ft

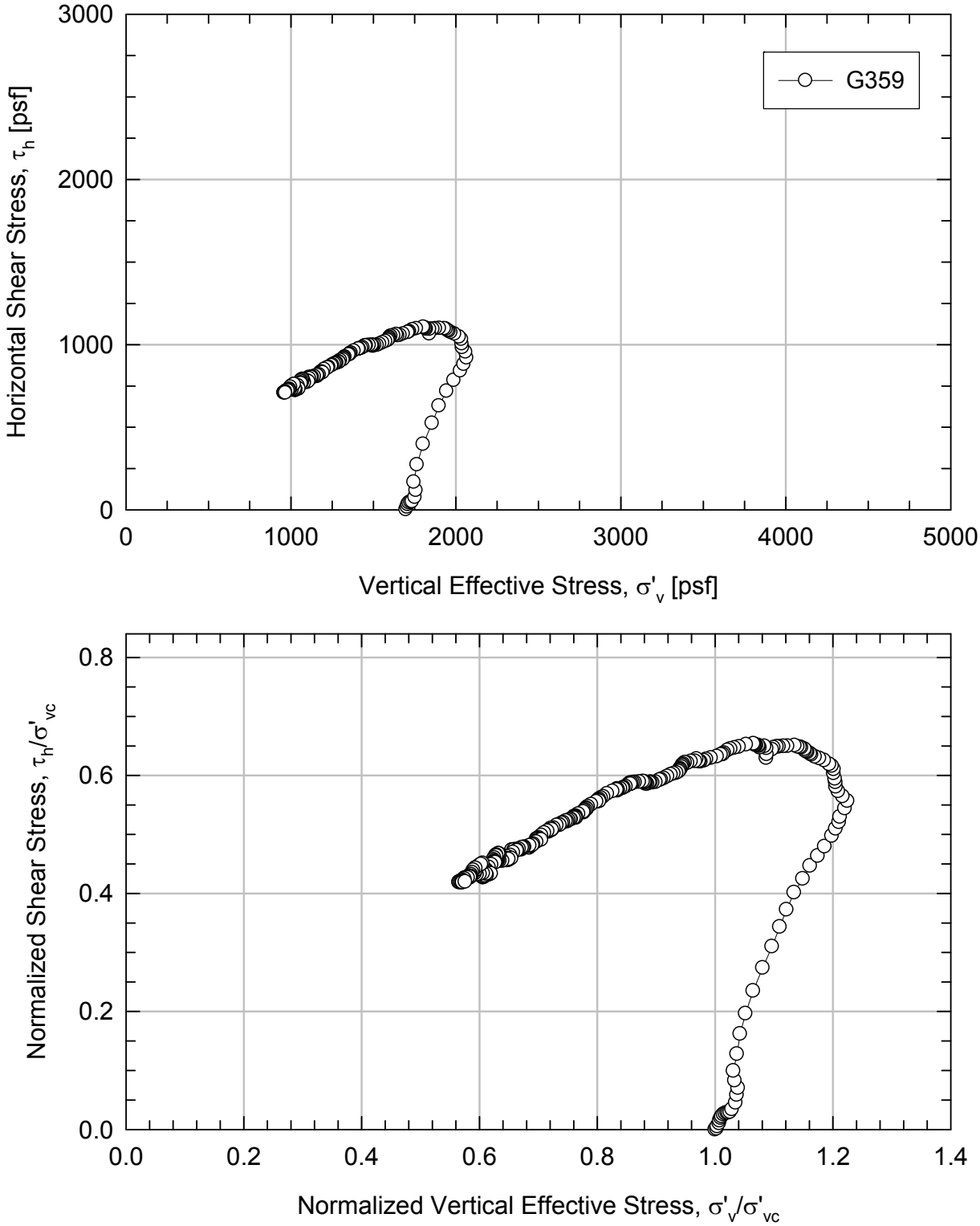


Figure 5.4.54 Horizontal shear stress versus vertical effective stress and normalized shear stress versus normalized vertical effective stress for test DSS G359 on sample 09-408B S7 35.5-38.0 ft

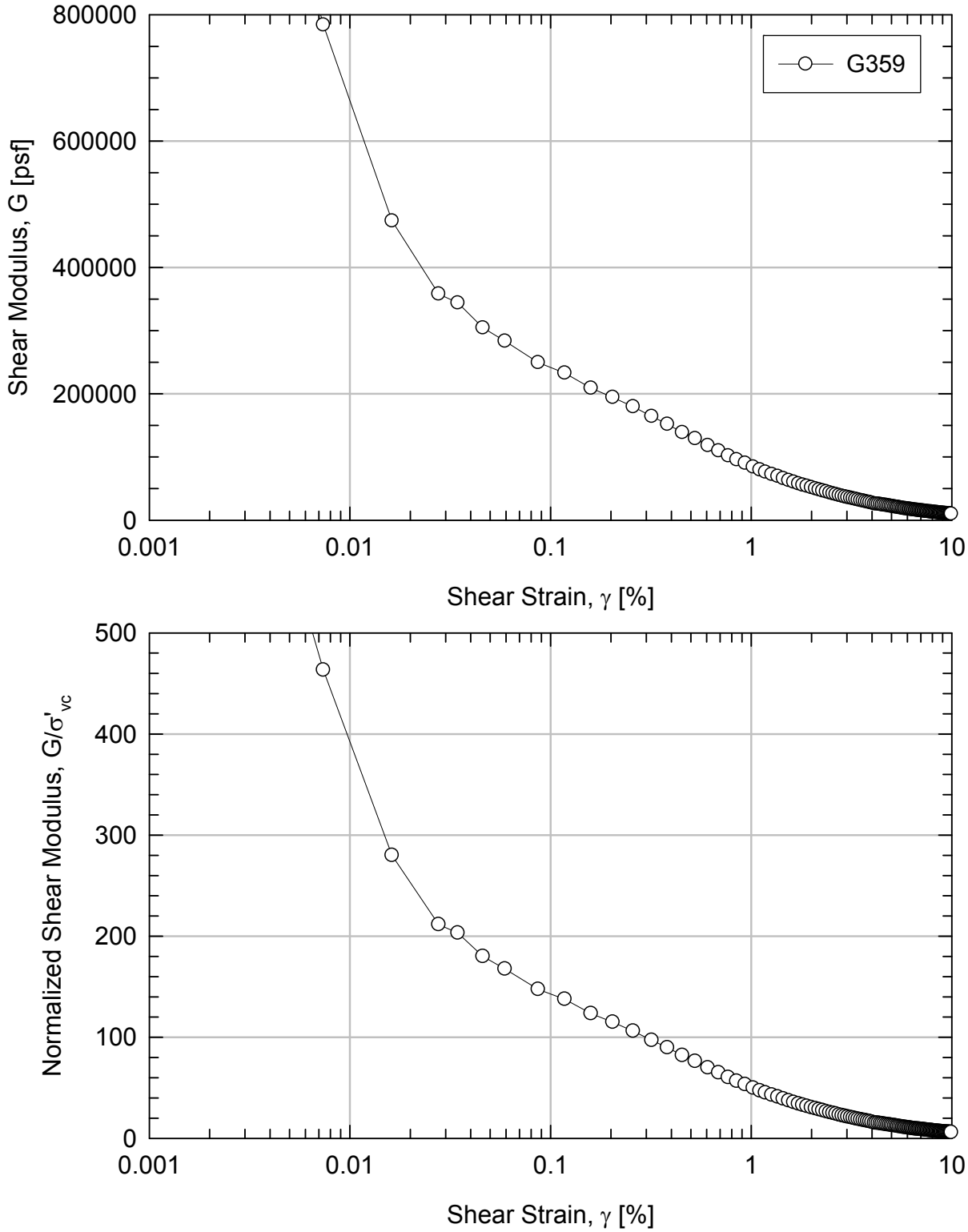


Figure 5.4.55 Shear modulus and normalized shear modulus versus shear strain for test DSS G359 on sample 09-408B S7 35.5-38.0 ft

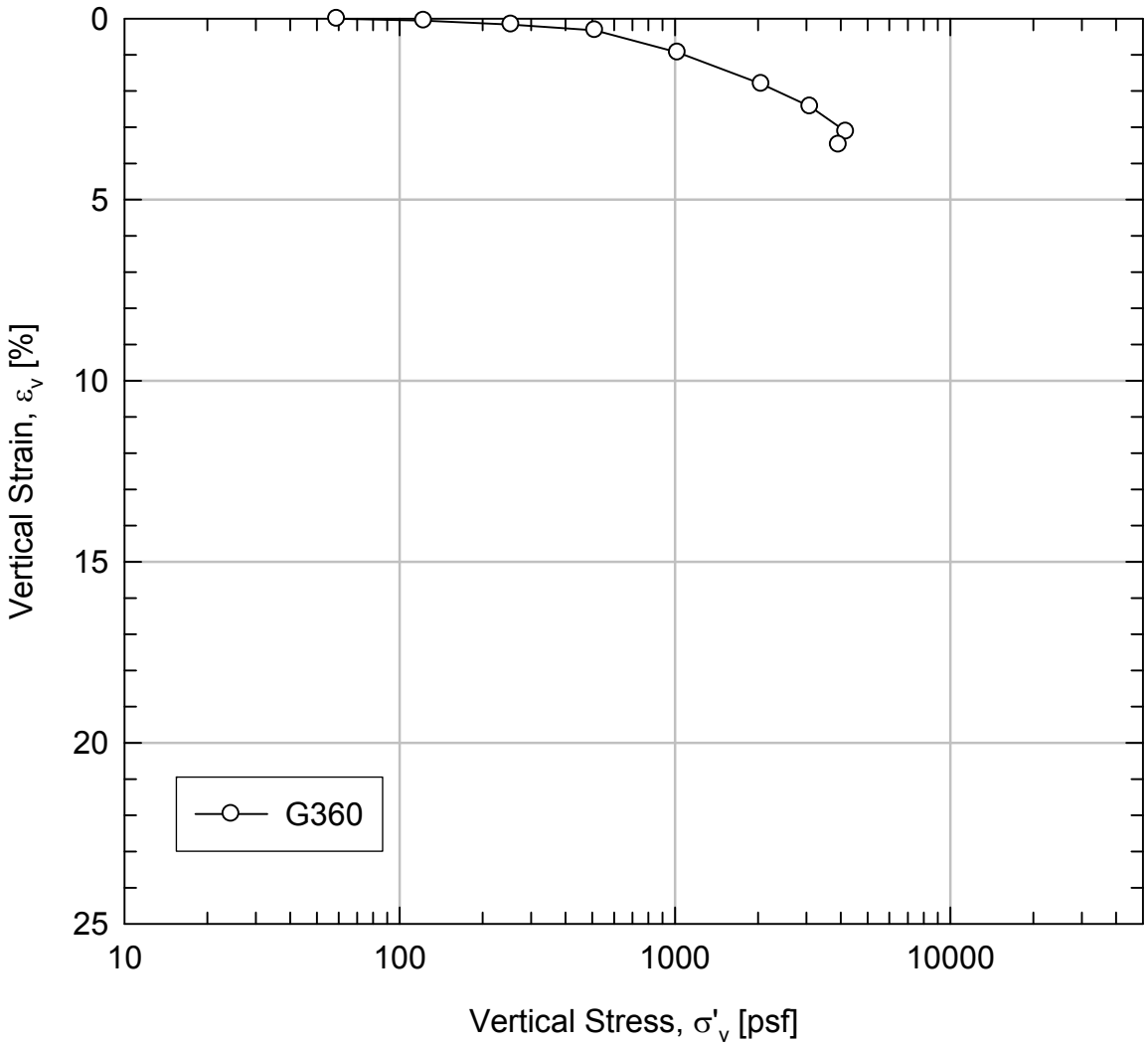


Figure 5.4.56 Vertical strain versus vertical effective stress for test DSS G360 on sample 09-503B S3 35.5-38.0 ft

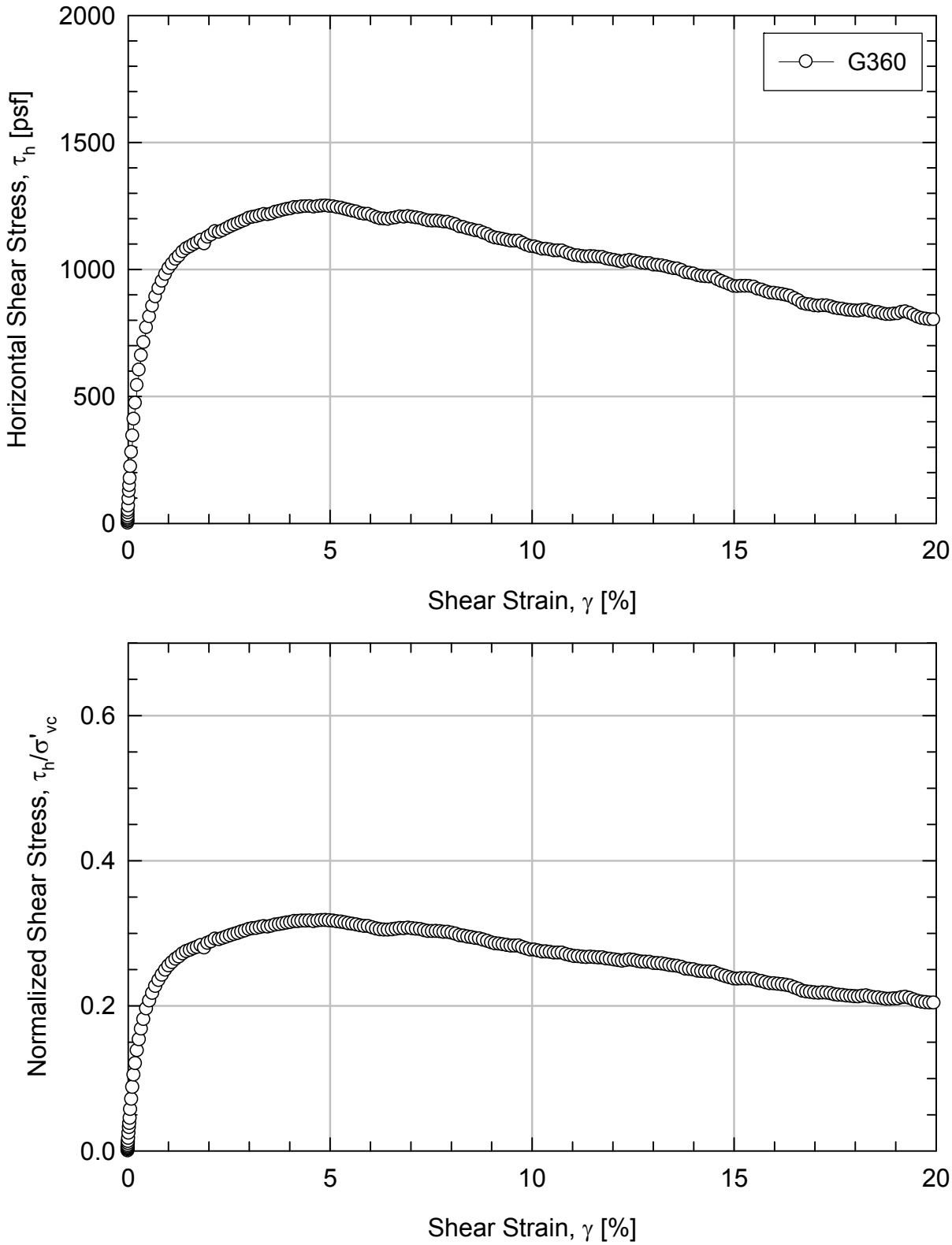


Figure 5.4.57 Horizontal shear stress and normalized shear stress versus shear strain for test DSS G360 on sample 09-503B S3 35.5-38.0 ft

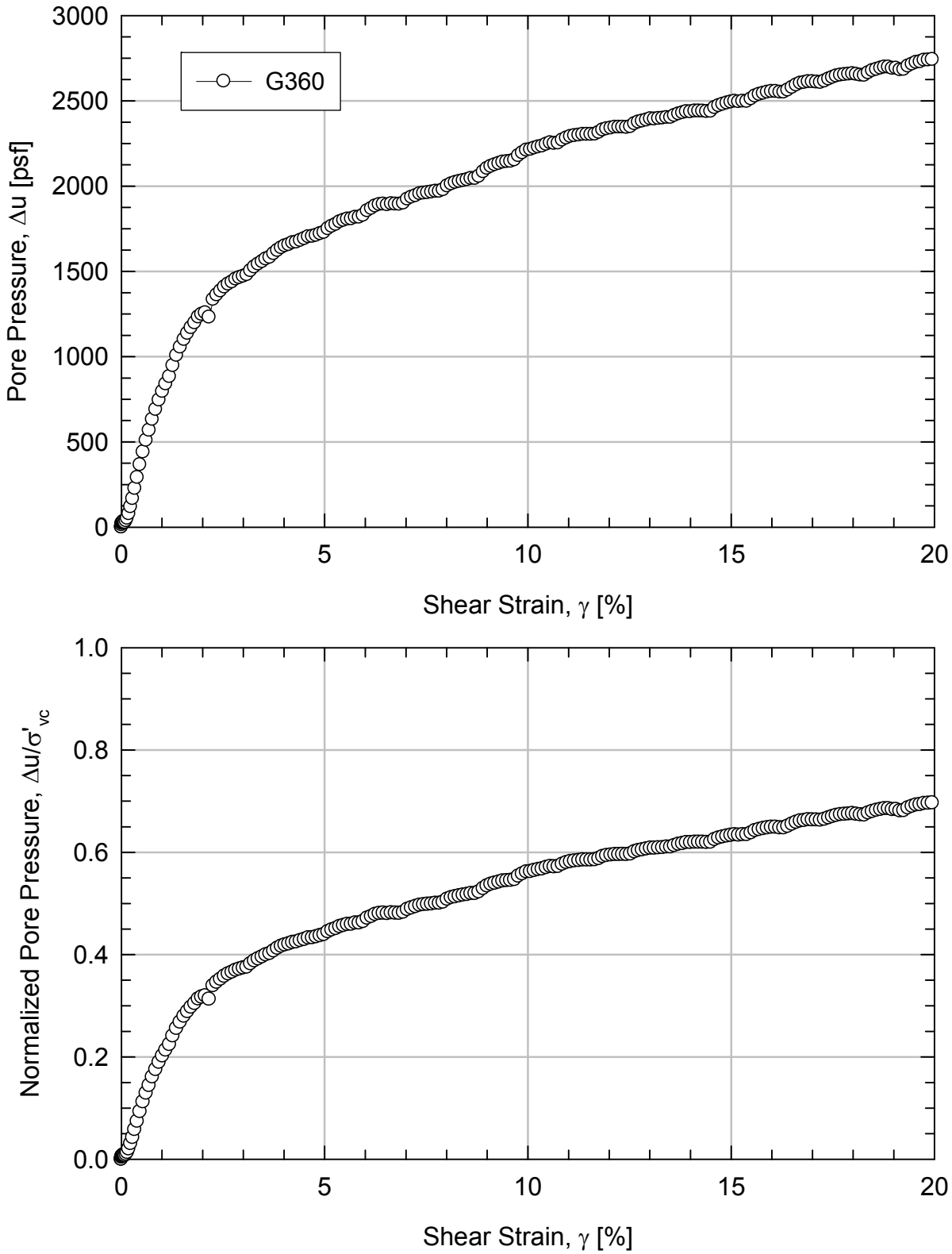


Figure 5.4.58 Pore pressure and normalized pore pressure versus shear strain for test DSS G360 on sample 09-503B S3 35.5-38.0 ft

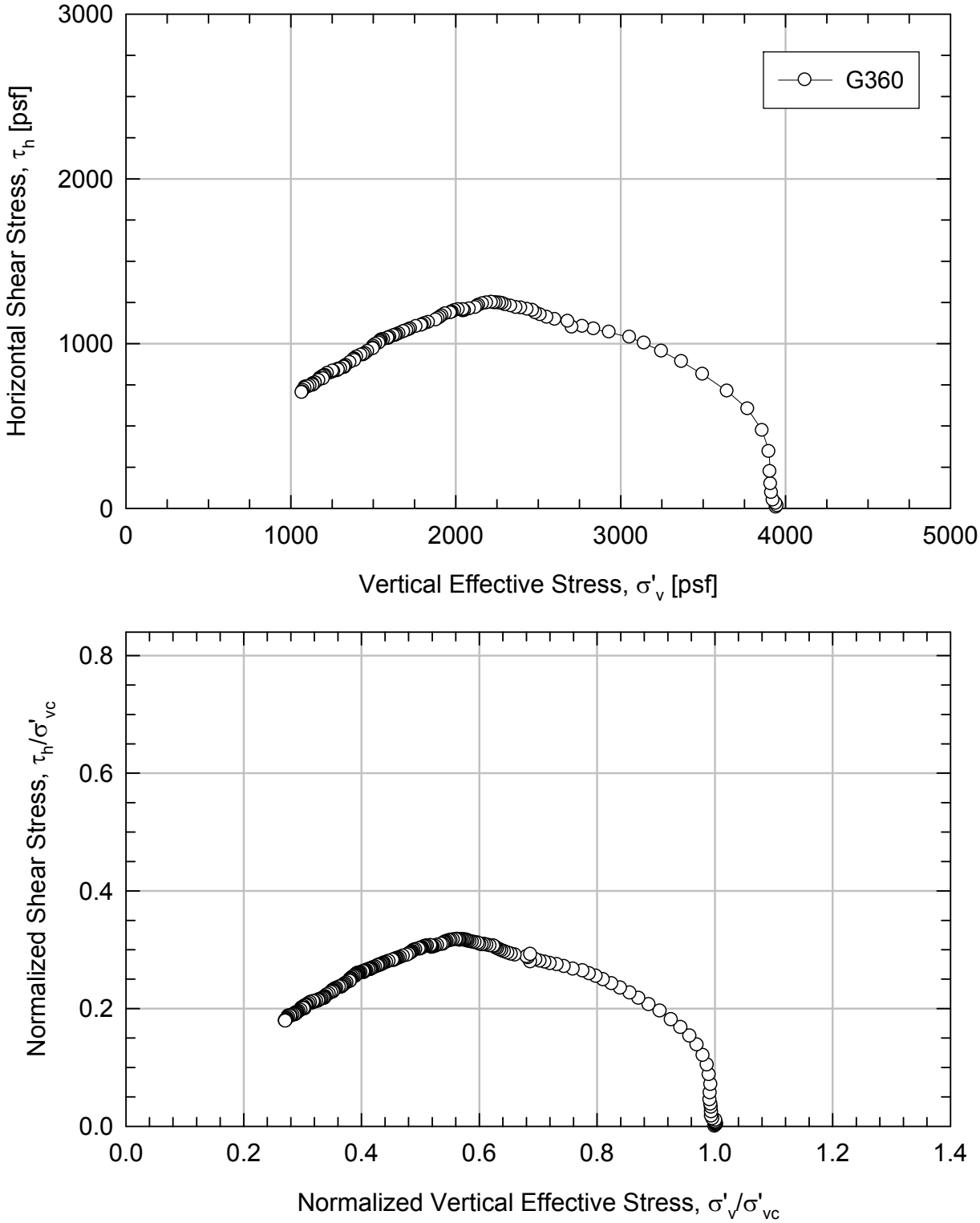


Figure 5.4.59 Horizontal shear stress versus vertical effective stress and normalized shear stress versus normalized vertical effective stress for test DSS G360 on sample 09-503B S3 35.5-38.0 ft

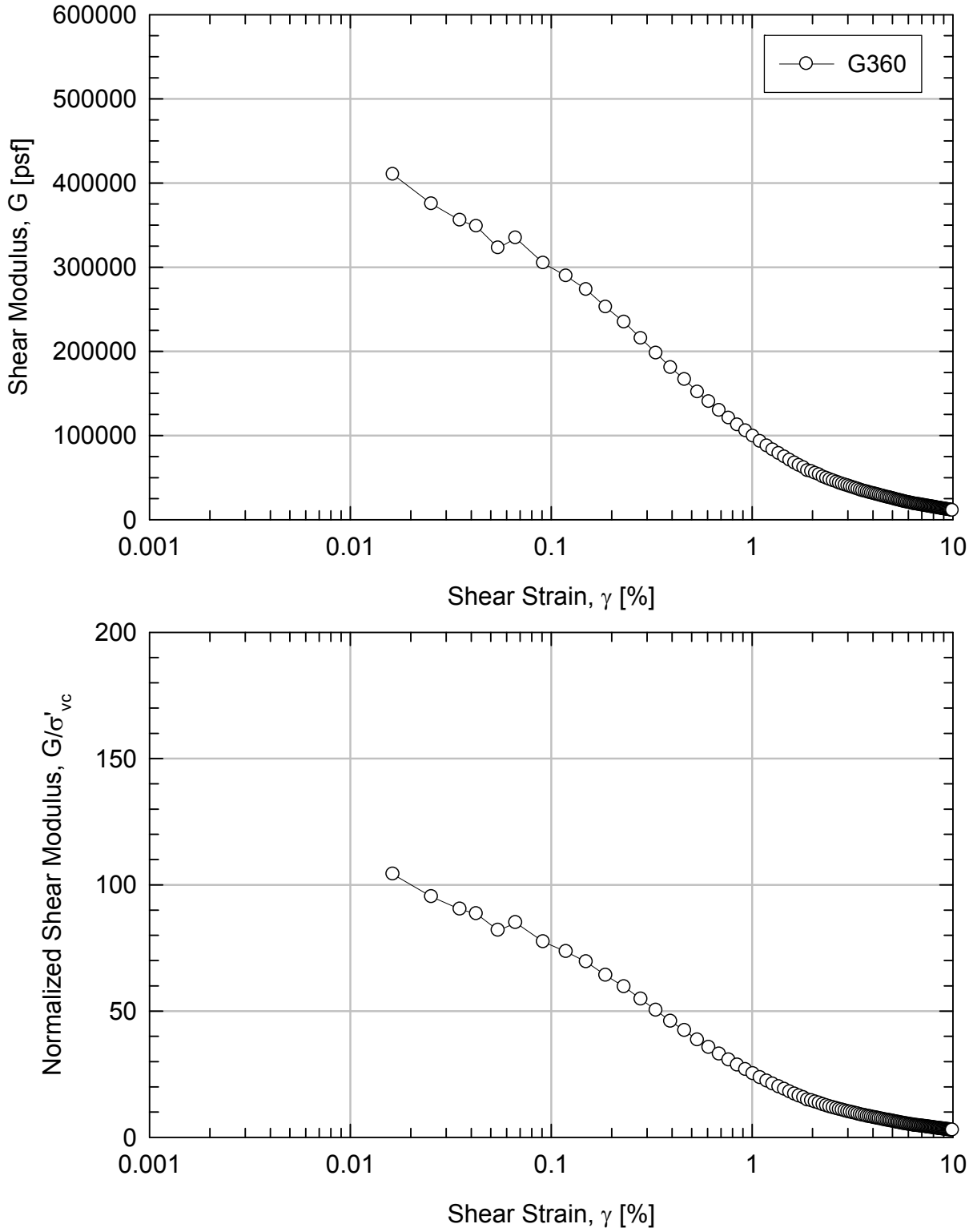


Figure 5.4.60 Shear modulus and normalized shear modulus versus shear strain for test DSS G360 on sample 09-503B S3 35.5-38.0 ft

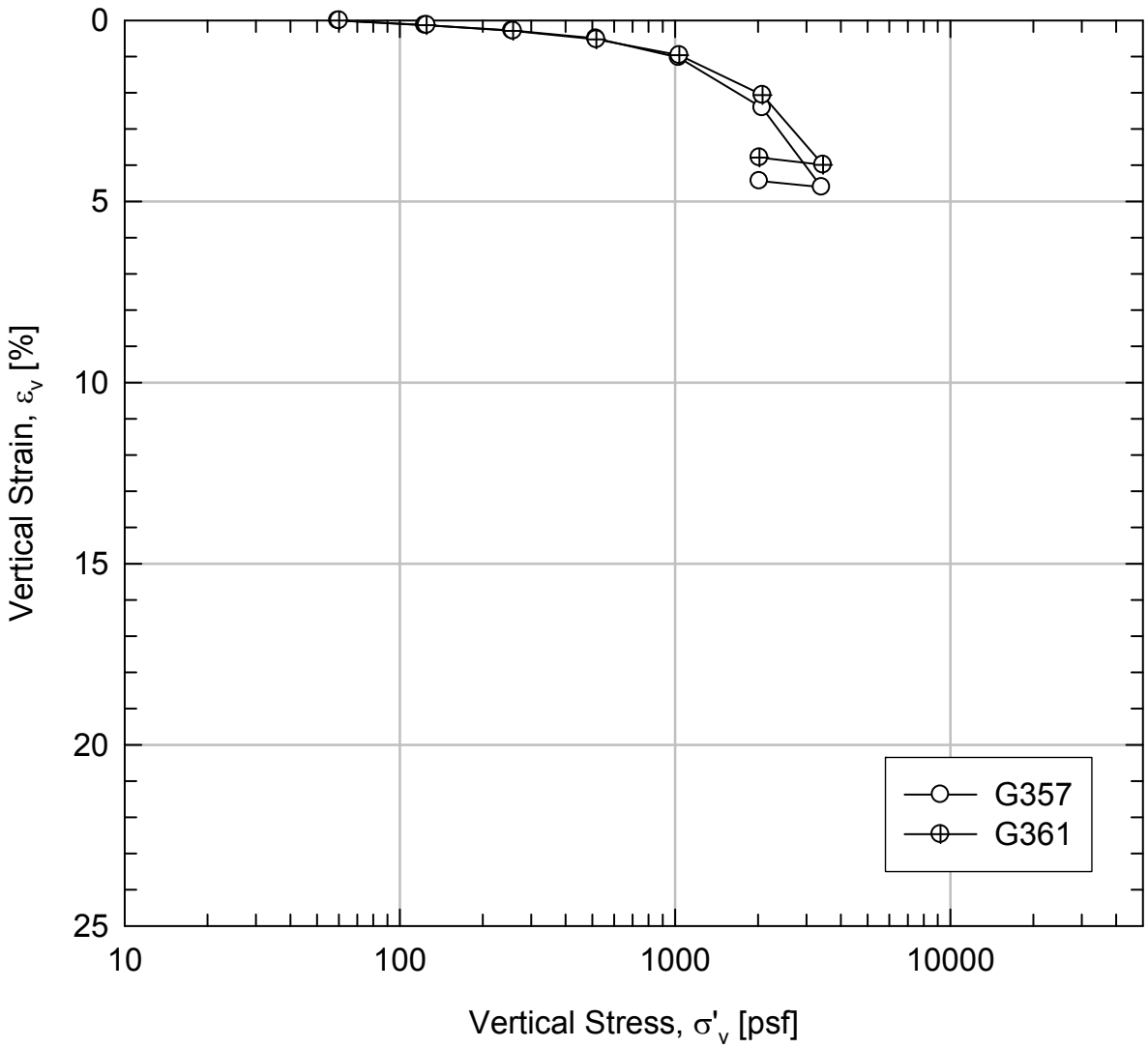


Figure 5.4.61 Vertical strain versus vertical effective stress for test DSS G357 and creep test G361 on sample 09-100B S6 35.5-38.0 ft

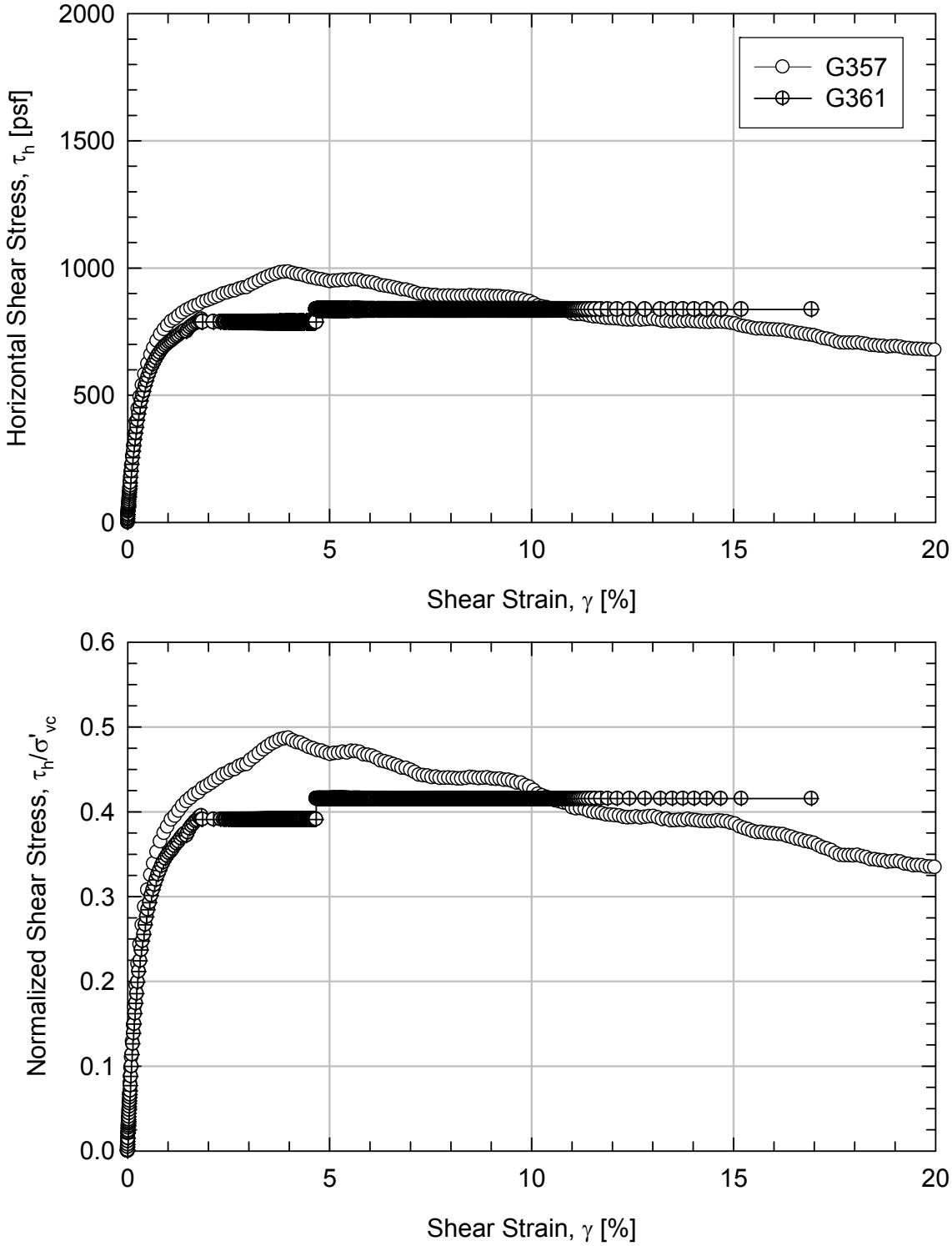


Figure 5.4.62 Horizontal shear stress and normalized shear stress versus shear strain for test DSS G357 and creep test G361 on sample 09-100B S6 35.5-38.0 ft

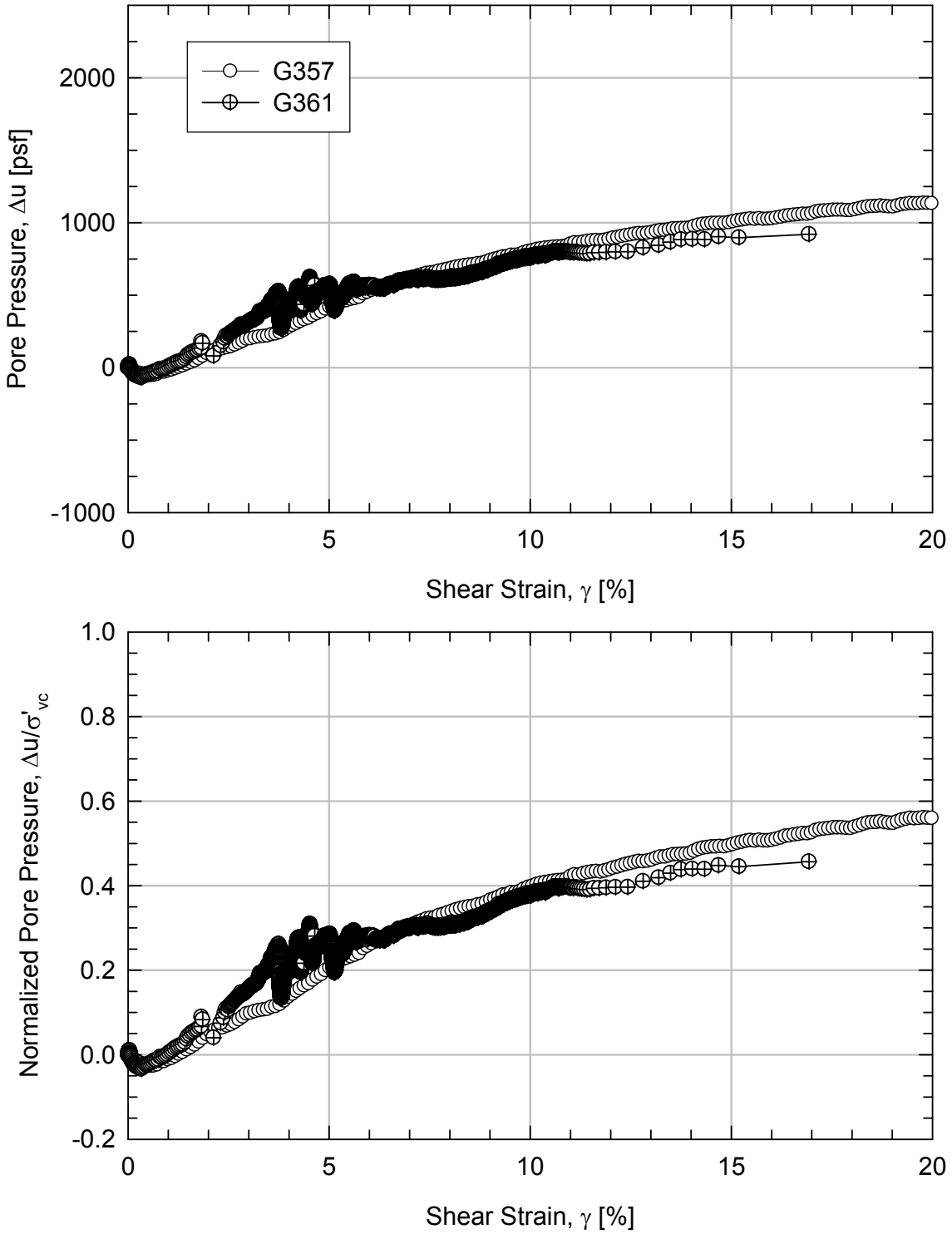


Figure 5.4.63 Pore pressure and normalized pore pressure versus shear strain for test DSS G357 and creep test G361 on sample 09-100B S6 35.5-38.0 ft

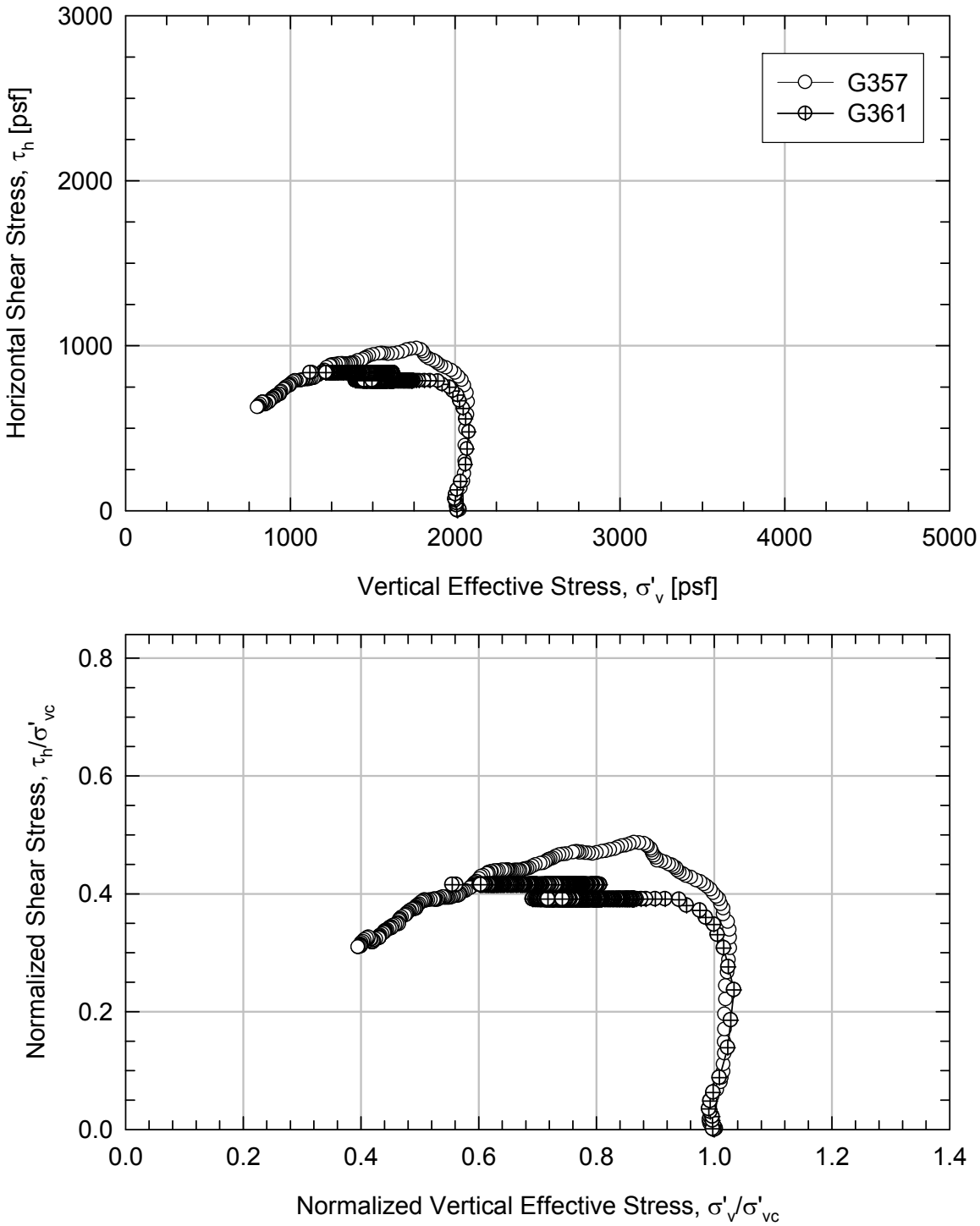


Figure 5.4.64 Horizontal shear stress versus vertical effective stress and normalized shear stress versus normalized vertical effective stress for test DSS G357 and creep test G361 on sample 09-100B S6 35.5-38.0 ft

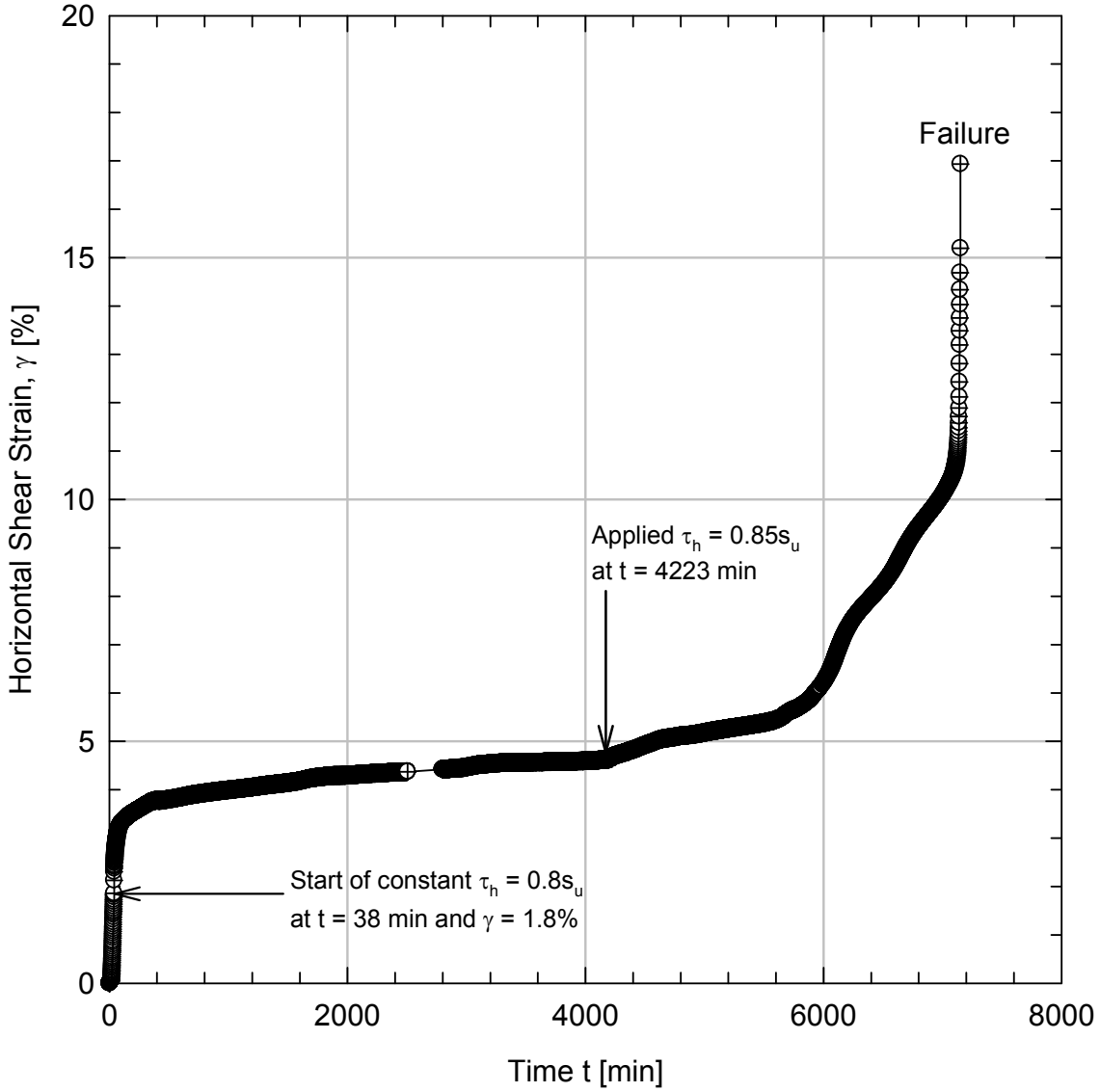


Figure 5.4.65 Horizontal shear strain versus time for DSS creep test G361 on sample 09-100B S6 35.5-38.0 ft

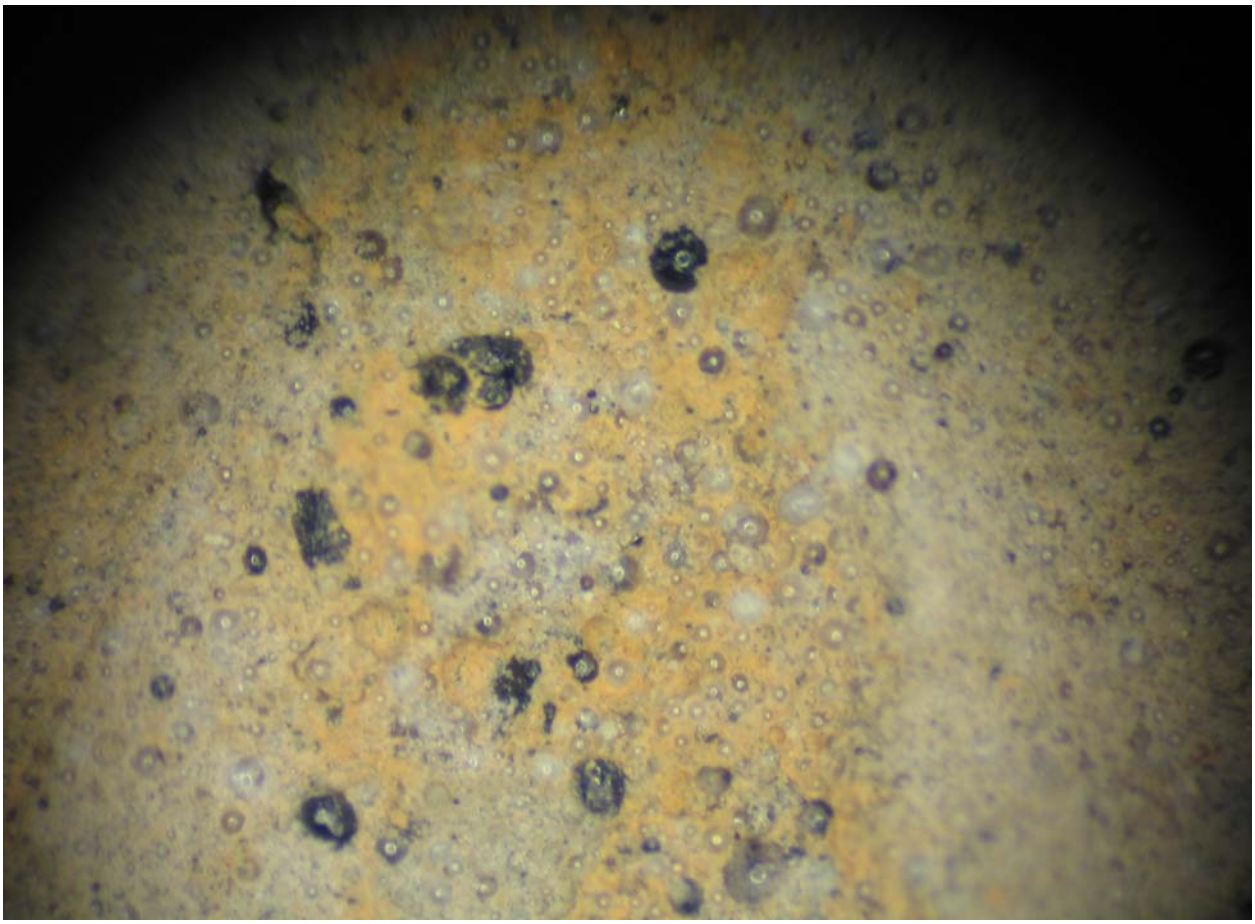


Figure 5.5.1 Photograph through a microscope of split open, oven dried, test specimen CRS167 on 09-100B S5.

Figure 5.5.2 Photograph through a microscope of split open, oven dried, test specimen DSS G357 on 09-100B S6.

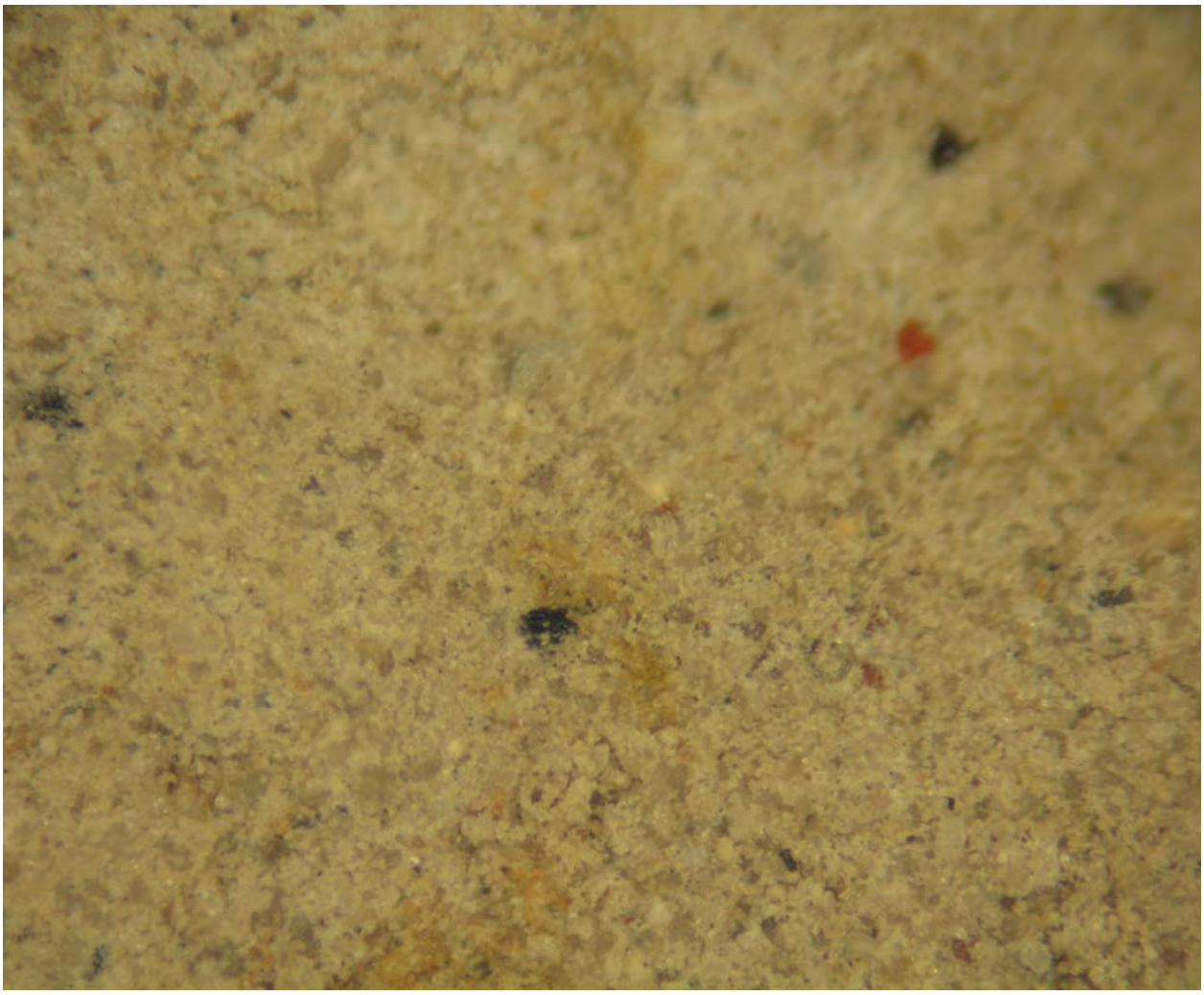


Figure 5.5.3 Photograph through a microscope of split open, oven dried, test specimen DSS G350 on 09-101B S5.

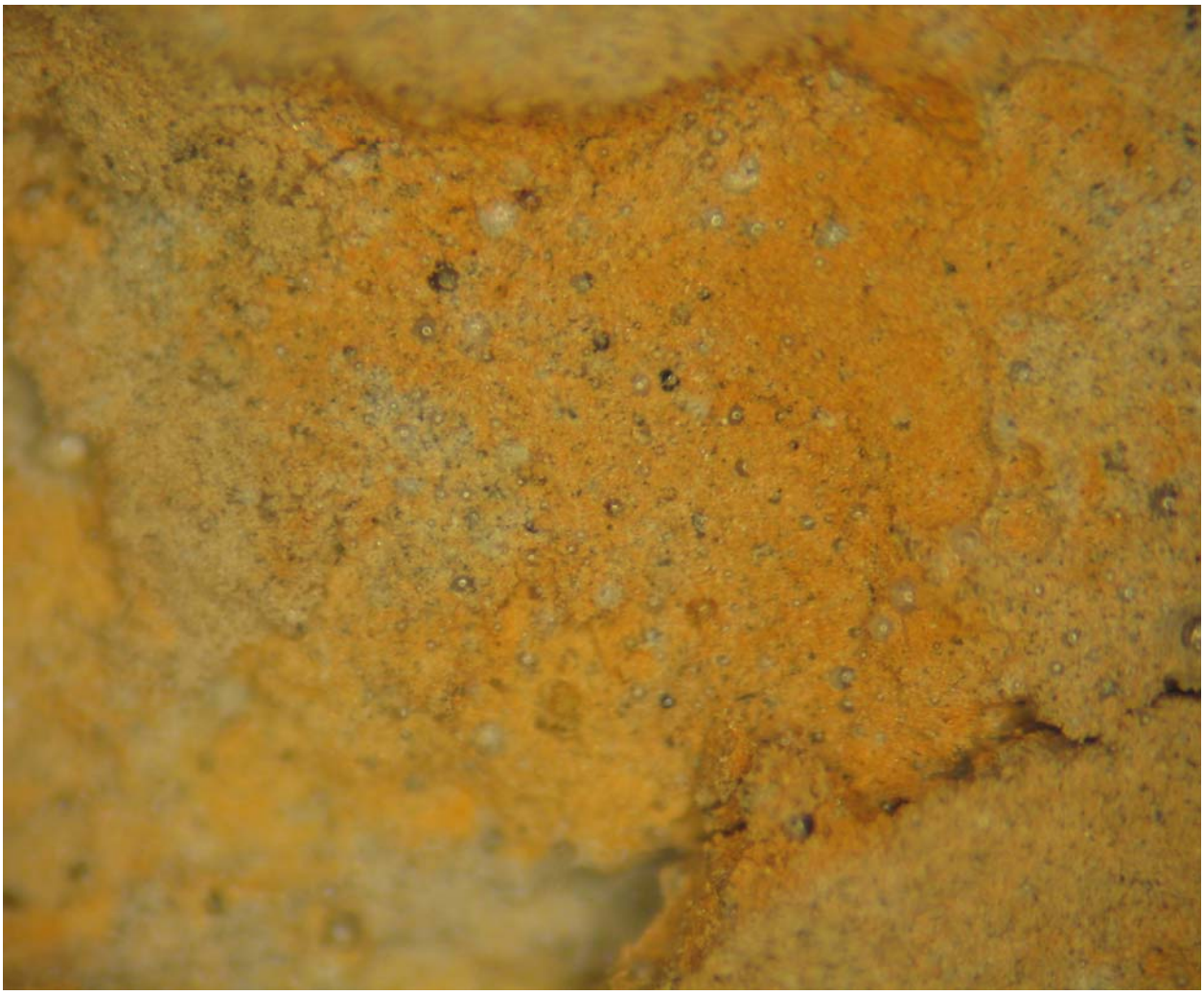


Figure 5.5.4 Photograph through a microscope of split open, oven dried, test specimen DSS G355 on 09-103B S2.

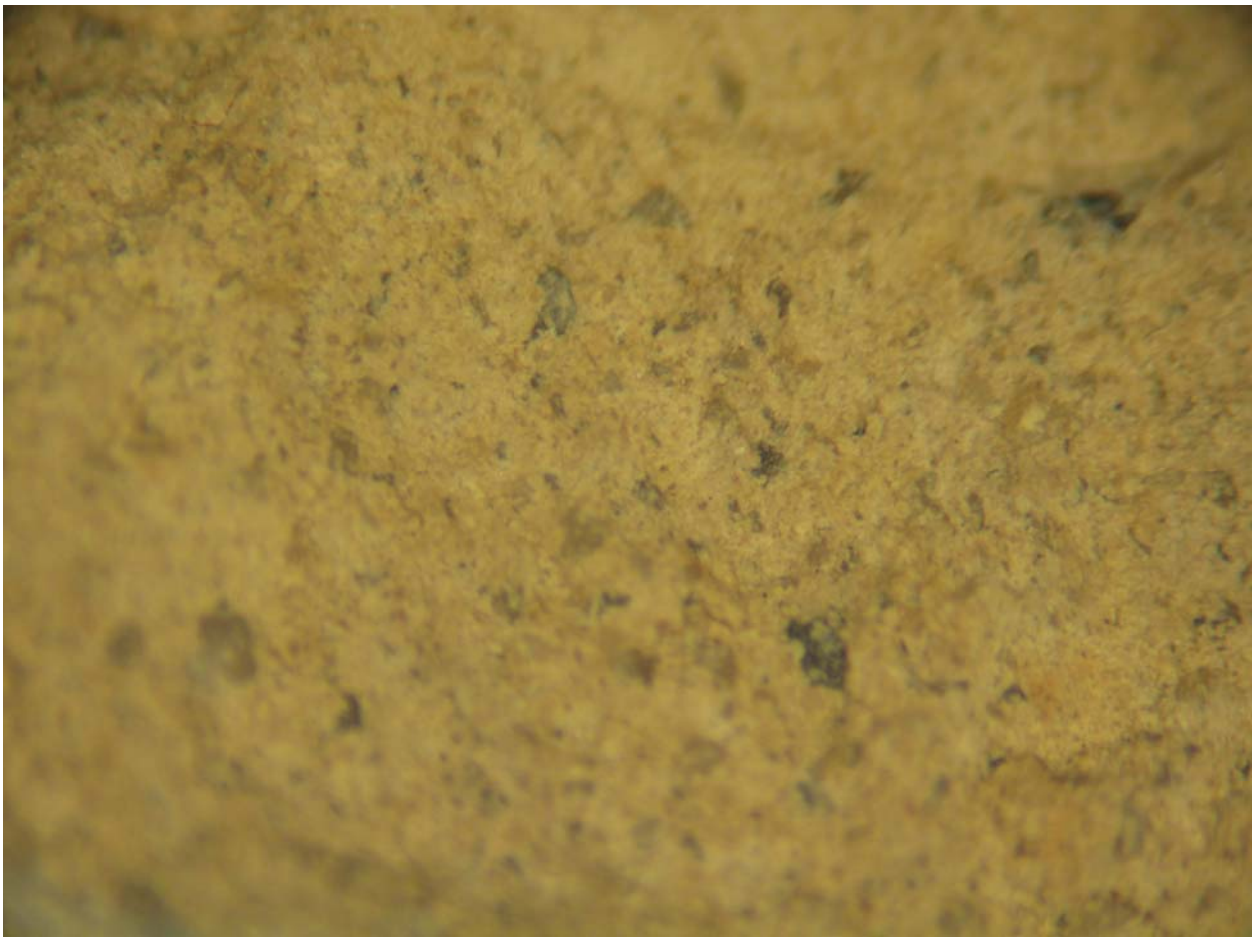


Figure 5.5.5 Photograph through a microscope of split open, oven dried, test specimen DSS G349 on 09-301B S6.

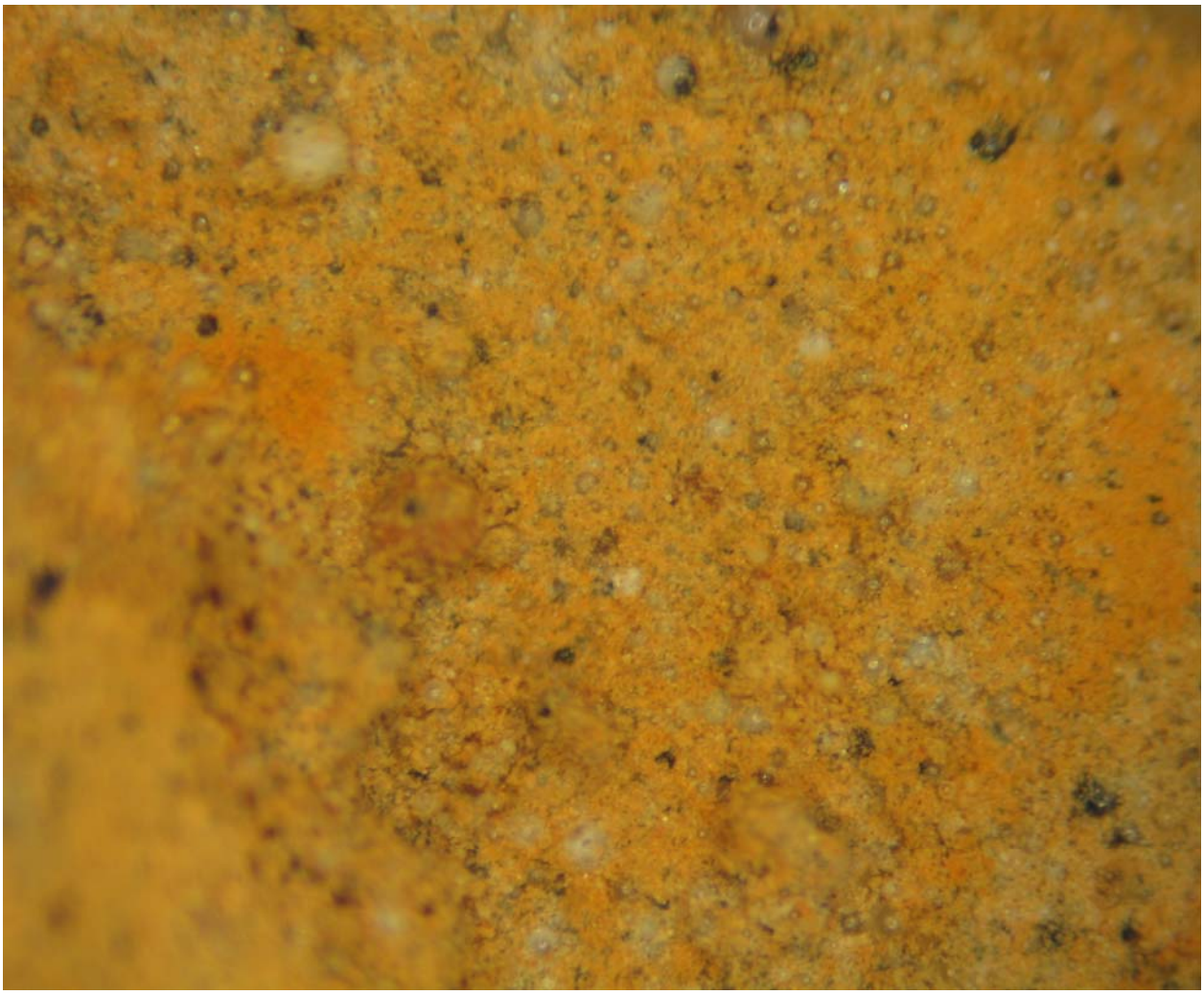
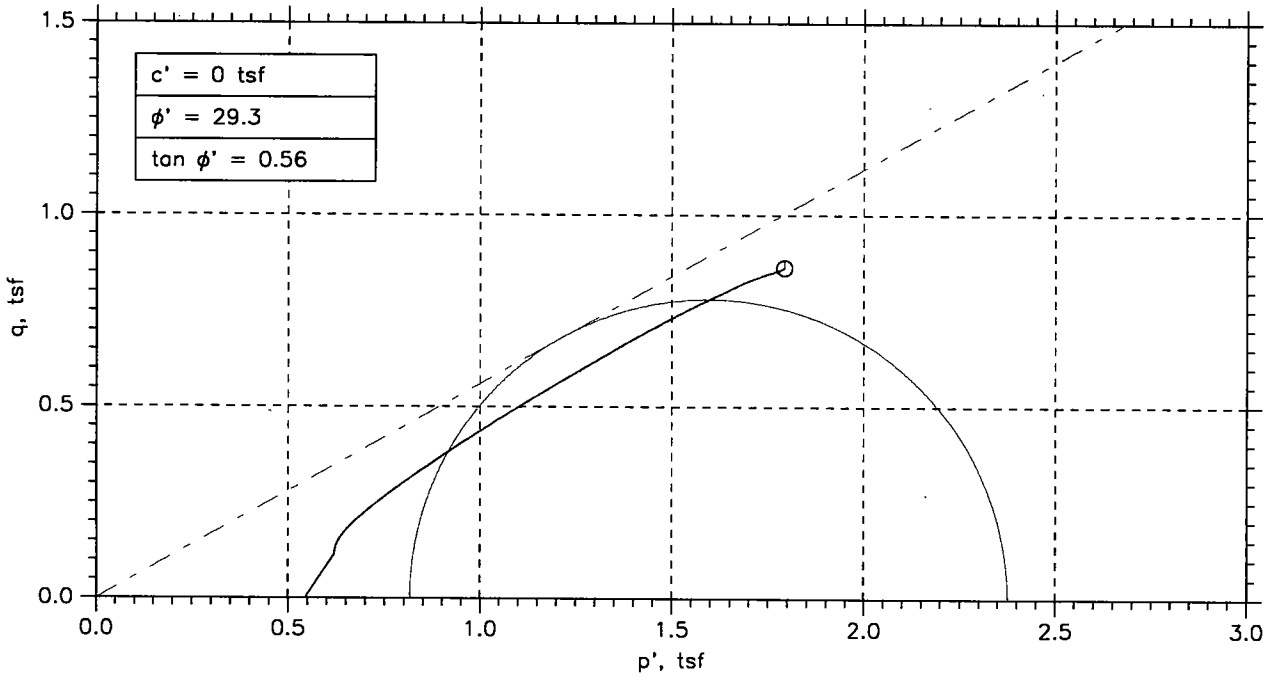


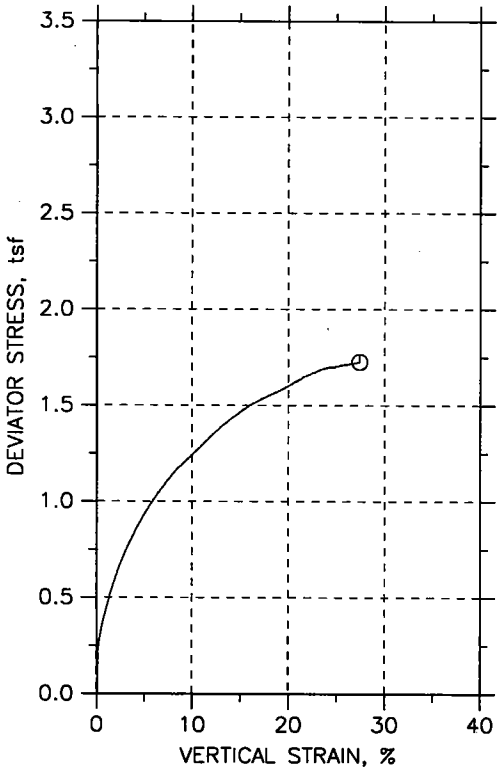
Figure 5.5.6 Photograph through a microscope of split open, oven dried, test specimen DSS G360 on 09-503B S3.

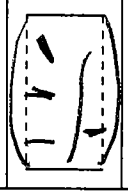
Appendix D

Test Data – Lean Clay Foundation Soil

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⊙			
Test No.		5742102BS4			
Initial	Diameter, in	2.8138			
	Height, in	5.8181			
	Water Content, %	25.27			
	Dry Density, pcf	100.			
	Saturation, %	99.55			
	Void Ratio	0.68533			
Before Shear	Water Content, %	24.99			
	Dry Density, pcf	100.6			
	Saturation, %	100.00			
	Void Ratio	0.67486			
	Back Press., tsf	5.0402			
Minor Prin. Stress, tsf		0.54196			
Max. Dev. Stress, tsf		1.7263			
Time to Failure, min		408.06			
Strain Rate, %/min		0.069			
B-Value		---			
Estimated Specific Gravity		2.70			
Liquid Limit		Pending			
Plastic Limit		Pending			
Plasticity Index		Pending			
Failure Sketch					

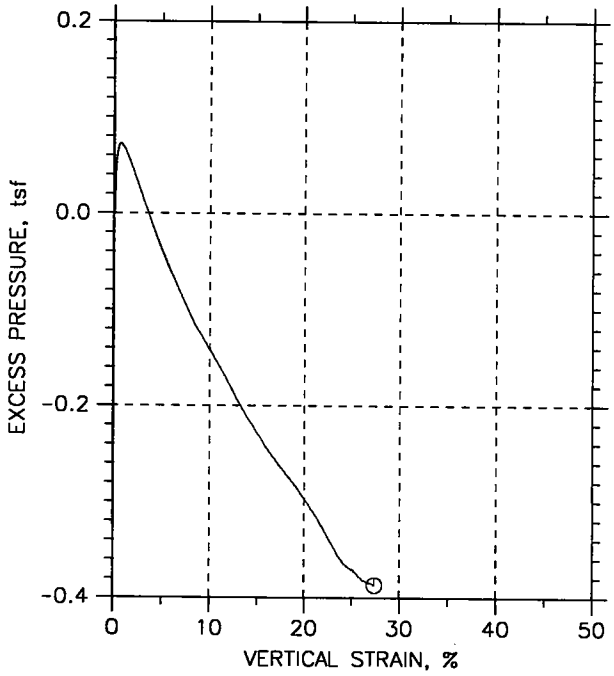
Project: KINGSTON COAL TVA
Location: TN
Project No.: 60095742
Boring No.: 09-102B S-4
Sample Type: 3" OST

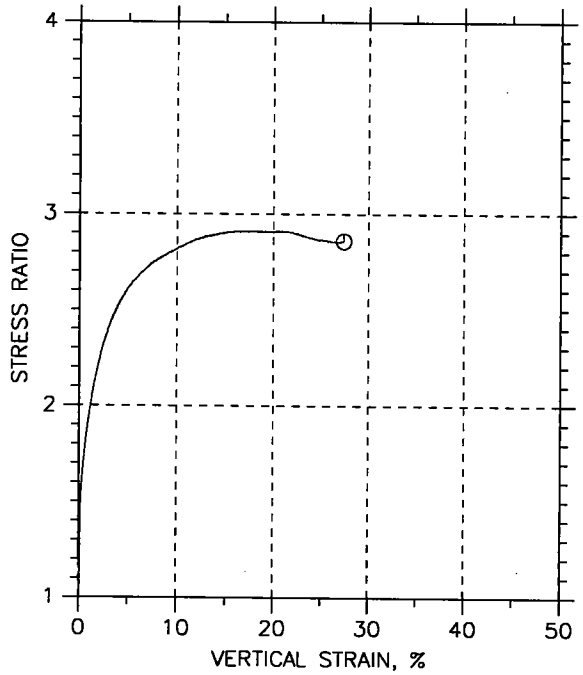
Description: SILTY CLAY AND F-M SAND - GRAY CL SAND AND SILT POCKETS NOTED

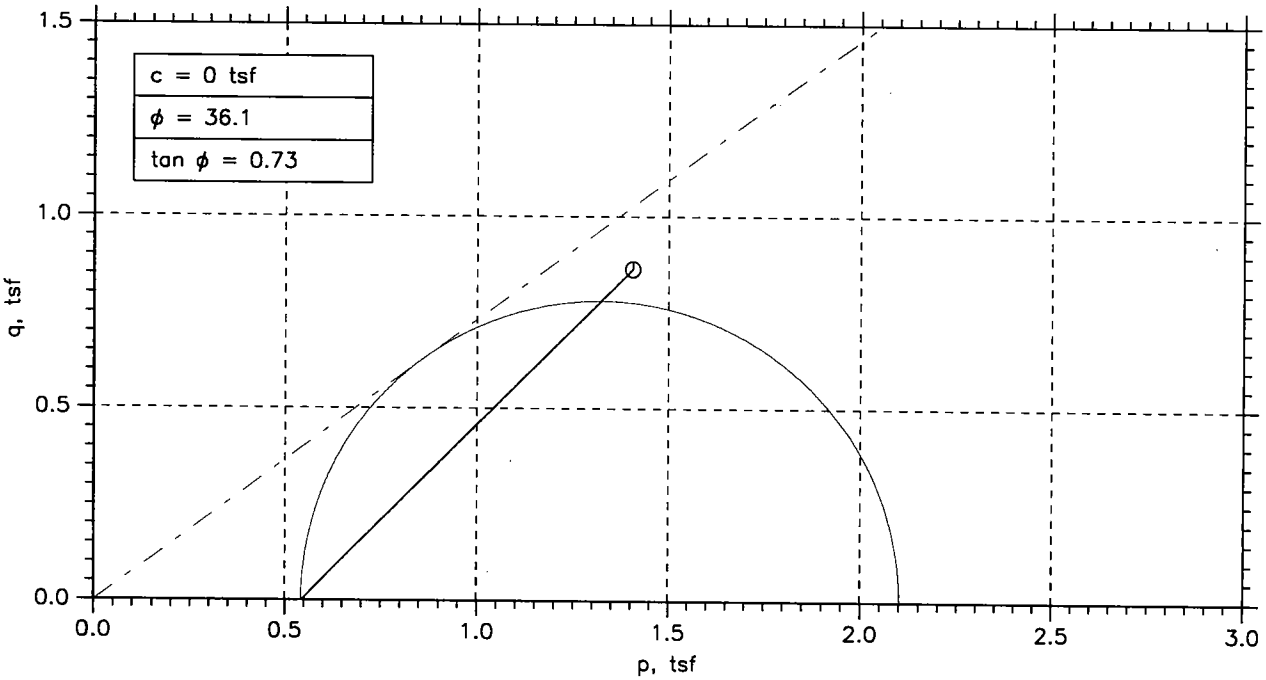
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 0.99

Tue, 31-MAR-2009 11:08:42

TRIAXIAL COMPRESSION TEST REPORT



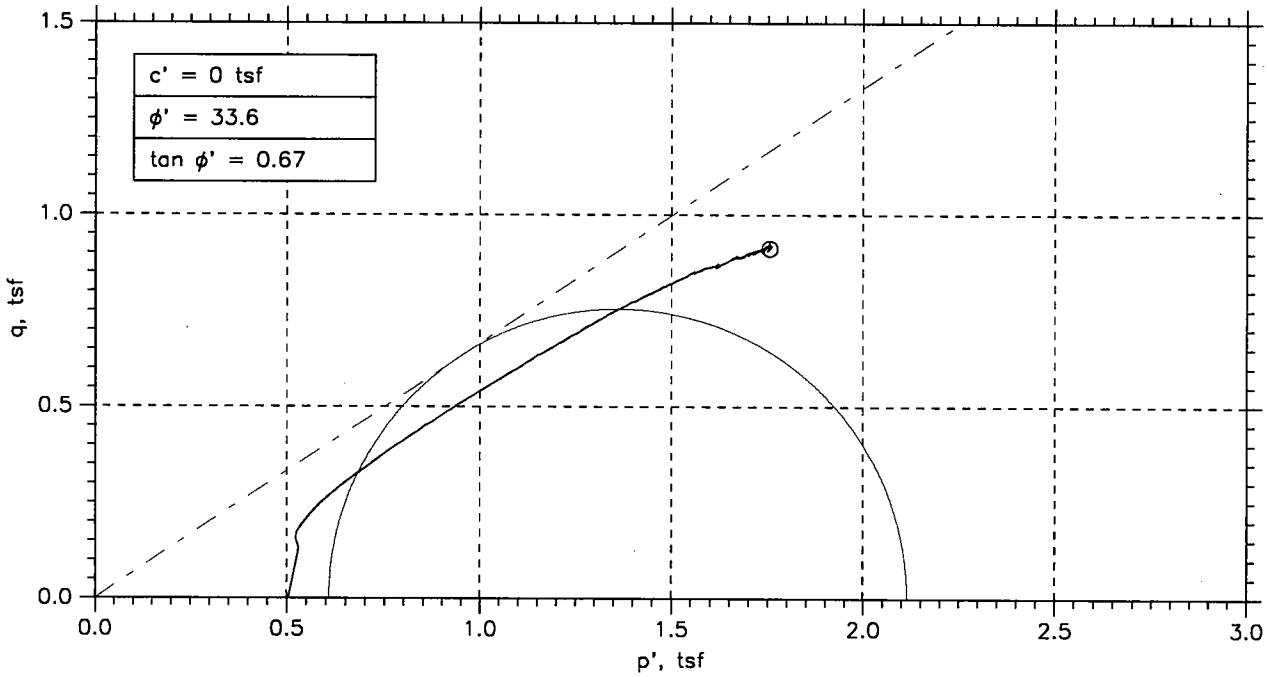


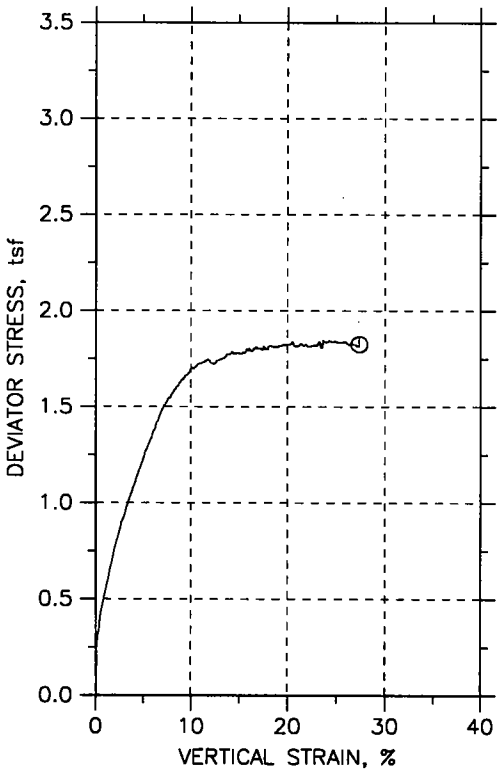


Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: 09-102B S-4	Tested By: BCM	Checked By: WPQ
Sample No.: OST-4	Test Date: 3/18/09	Depth: 45.6'
Test No.: 5742102BS4	Sample Type: 3" OST	Elevation: ----
Description: SILTY CLAY AND F-M SAND - GRAY CL SAND AND SILT POCKETS NOTED		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 0.99		

Tue, 31-MAR-2009 11:09:22

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⊙			
Test No.		5742103BS5			
Initial	Diameter, in	2.8488			
	Height, in	5.7256			
	Water Content, %	25.66			
	Dry Density, pcf	101.5			
	Saturation, %	104.80			
	Void Ratio	0.66109			
Before Shear	Water Content, %	23.81			
	Dry Density, pcf	102.6			
	Saturation, %	100.00			
	Void Ratio	0.64299			
	Back Press., tsf	5.0399			
Minor Prin. Stress, tsf		0.50415			
Max. Dev. Stress, tsf		1.8475			
Time to Failure, min		348.48			
Strain Rate, %/min		0.069			
B-Value		---			
Estimated Specific Gravity		2.70			
Liquid Limit		Pending			
Plastic Limit		Pending			
Plasticity Index		Pending			
Failure Sketch					

Project: KINGSTON COAL TVA

Location: TN

Project No.: 60095742

Boring No.: 09-103B S-5

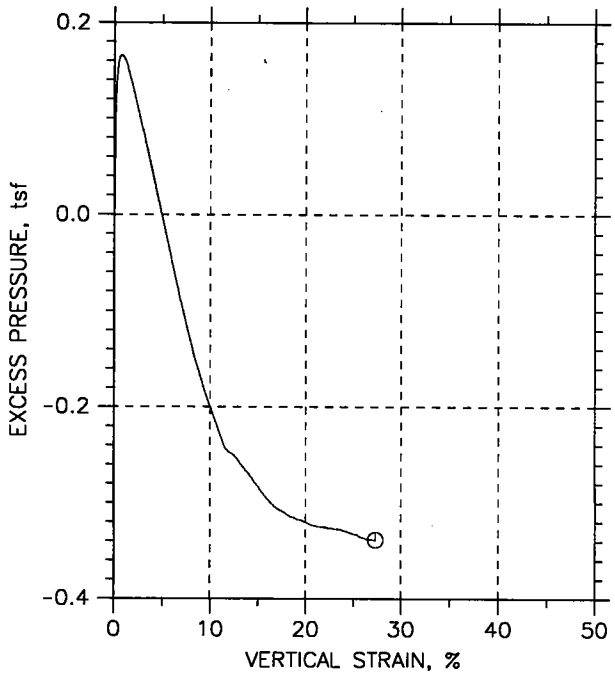
Sample Type: 3"OST

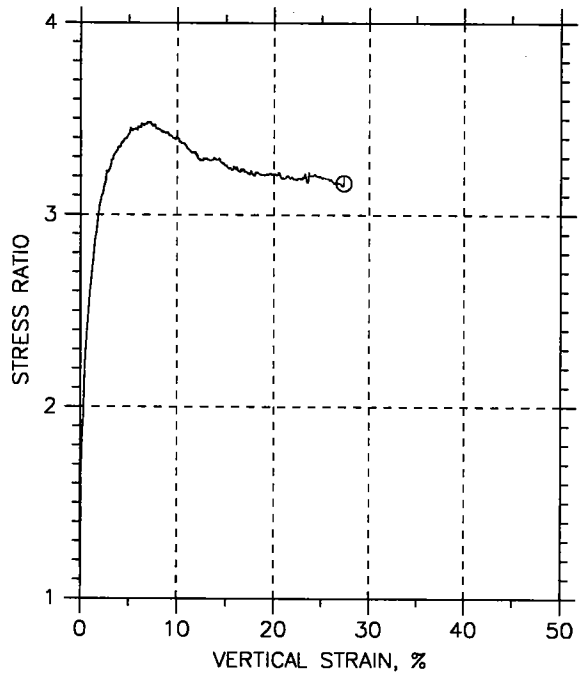
Description: CLAYEY SILT TRACE F-C SAND - BROWN AND GRAY

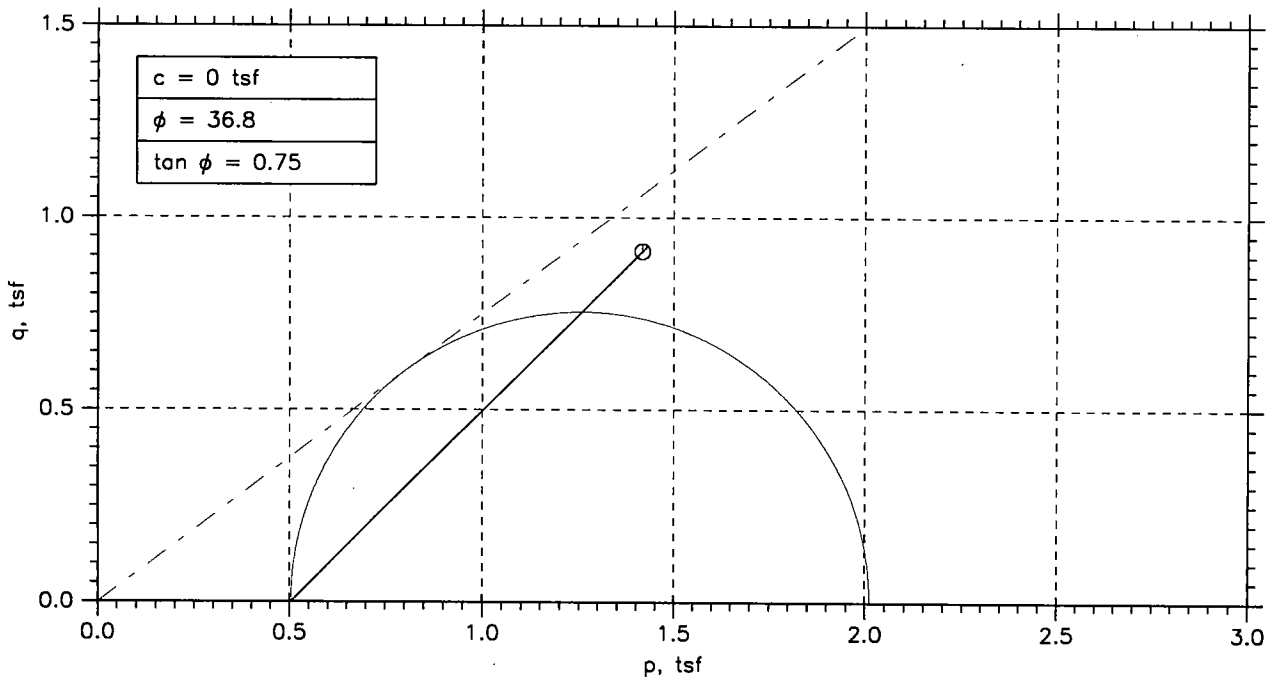
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 0.95

Tue, 31-MAR-2009 11:19:18

TRIAXIAL COMPRESSION TEST REPORT



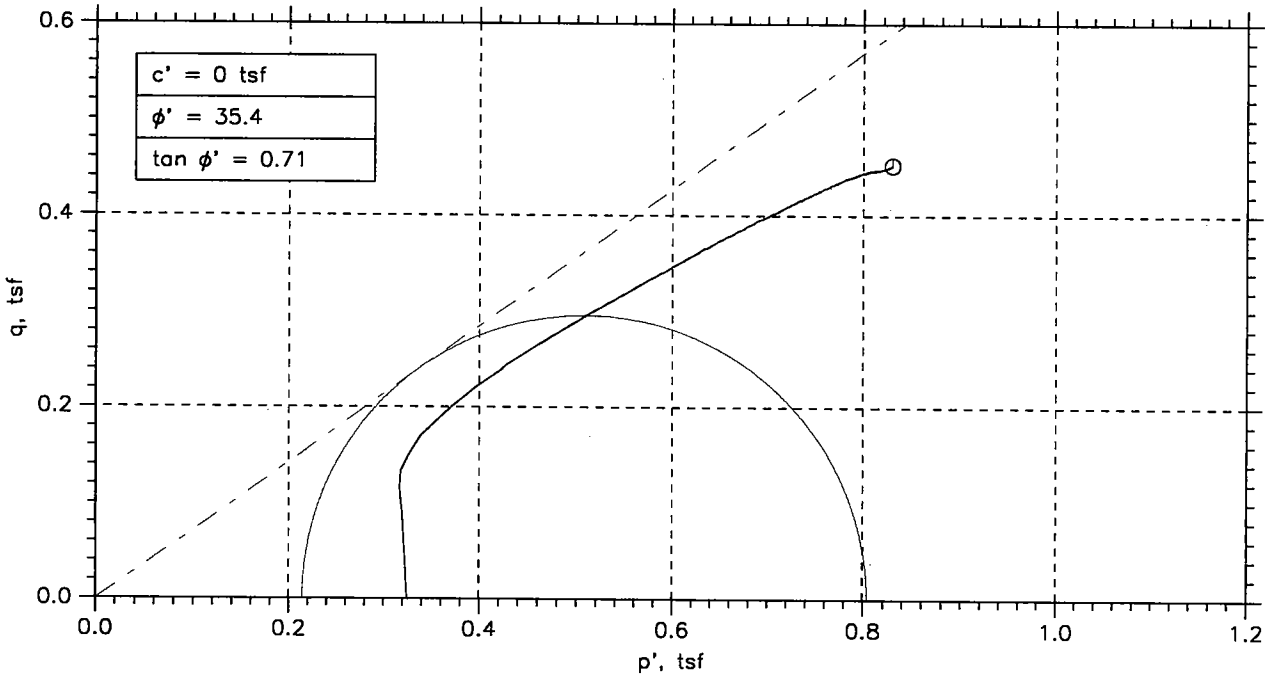


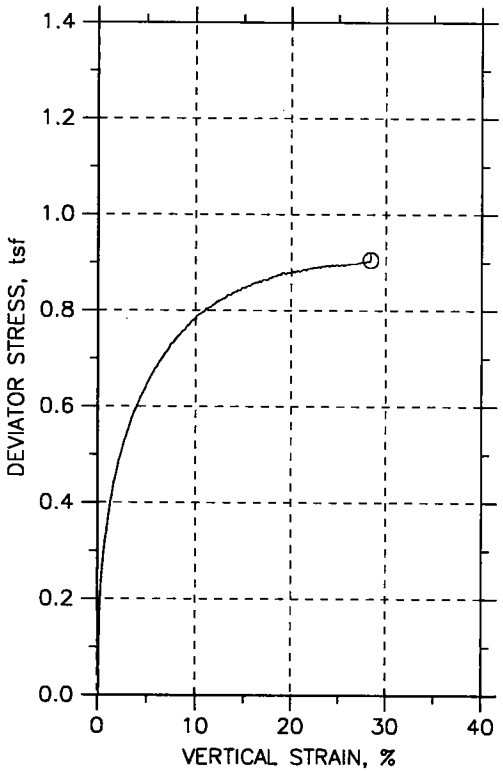


Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: 09-103B S-5	Tested By: BCM	Checked By: WPQ
Sample No.: OST-5	Test Date: 3/18/09	Depth: 42.5'
Test No.: 5742103BS5	Sample Type: 3"OST	Elevation: ----
Description: CLAYEY SILT TRACE F-C SAND - BROWN AND GRAY		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 0.95		

Tue, 31-MAR-2009 11:19:51

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⊙			
Test No.		5742108BS3			
Initial	Diameter, in	2.8244			
	Height, in	6.1685			
	Water Content, %	12.66			
	Dry Density, pcf	112.7			
	Saturation, %	67.96			
	Void Ratio	0.50686			
Before Shear	Water Content, %	18.26			
	Dry Density, pcf	113.4			
	Saturation, %	100.00			
	Void Ratio	0.49679			
	Back Press., tsf	5.0399			
Minor Prin. Stress, tsf		0.32415			
Max. Dev. Stress, tsf		0.90479			
Time to Failure, min		1005			
Strain Rate, %/min		0.029			
B-Value		---			
Estimated Specific Gravity		2.72			
Liquid Limit		Pending			
Plastic Limit		Pending			
Plasticity Index		Pending			
Failure Sketch					

Project: KINGSTON COAL TVA

Location: TN

Project No.: 60095742

Boring No.: 09-108B S-3

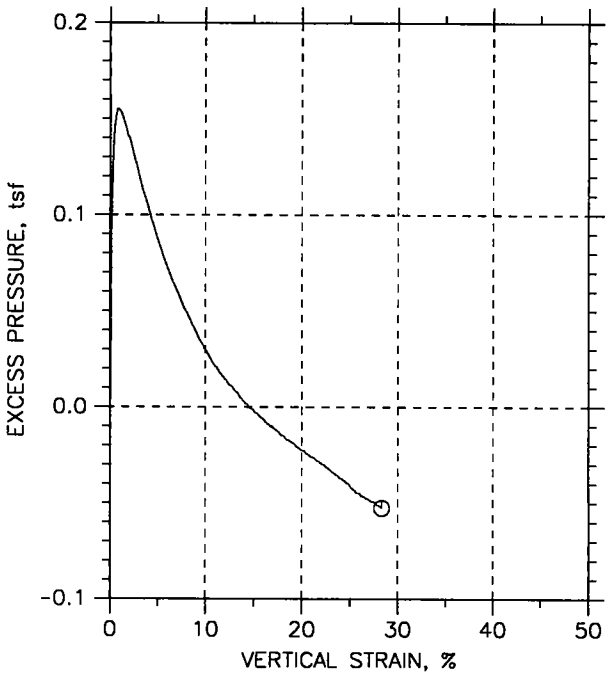
Sample Type: 3" OST

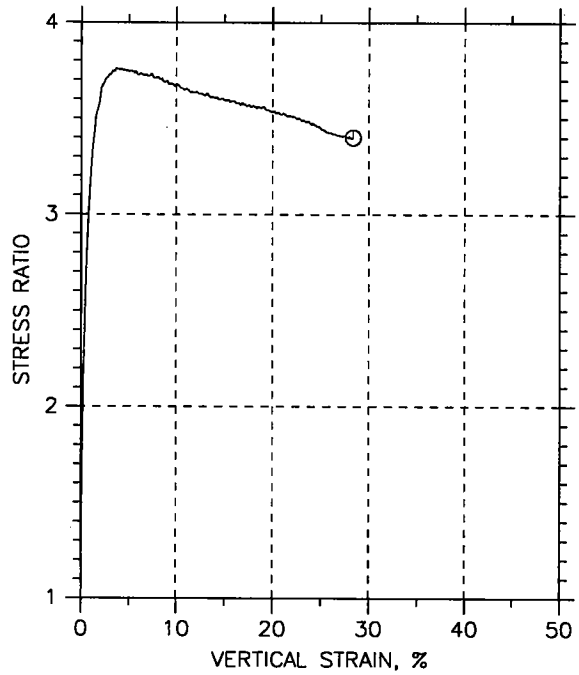
Description: SILTY CLAY SOME F-C SAND (SC) - SL. REDDISH BROWN GRAY. SILT POCKETS NOTED.

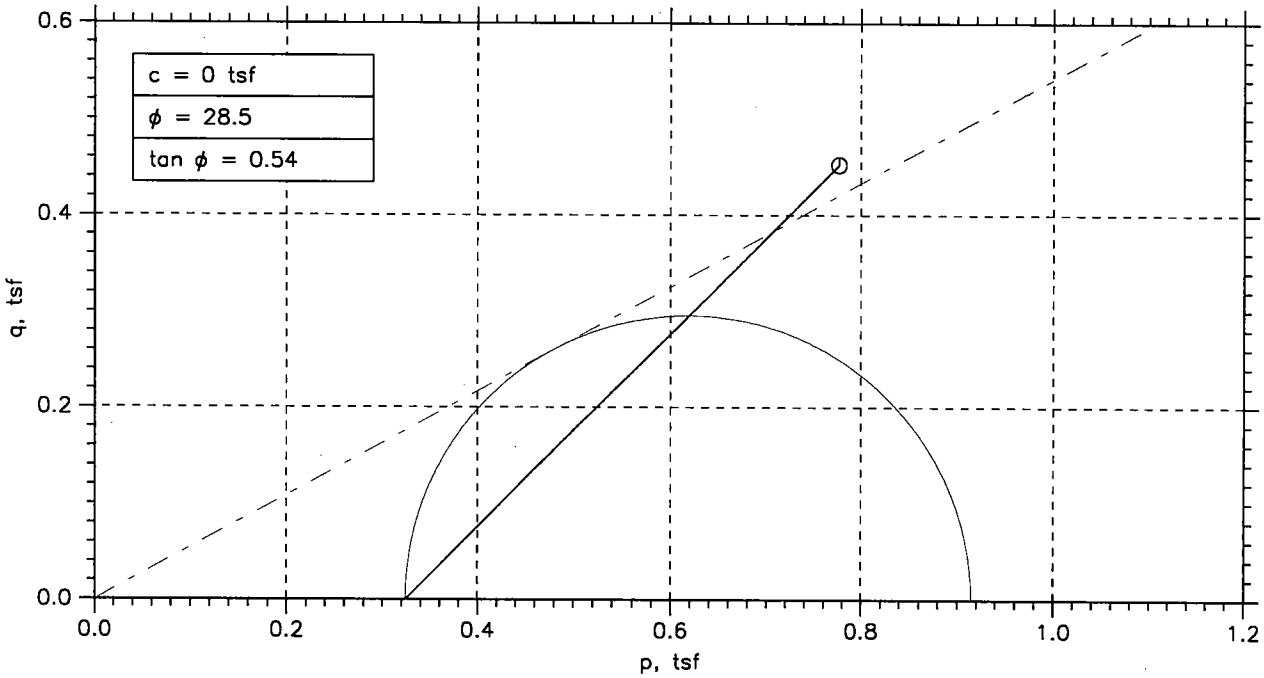
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B Value = 0.99

Tue, 31-MAR-2009 10:07:24

TRIAXIAL COMPRESSION TEST REPORT



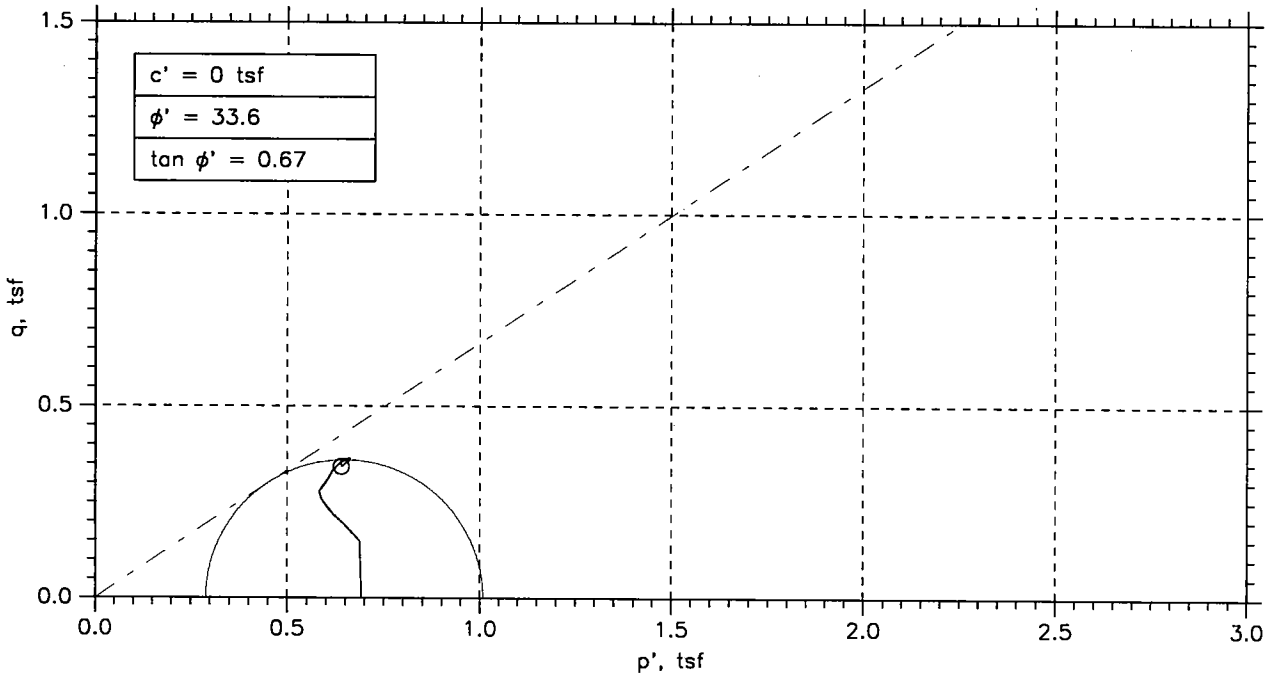


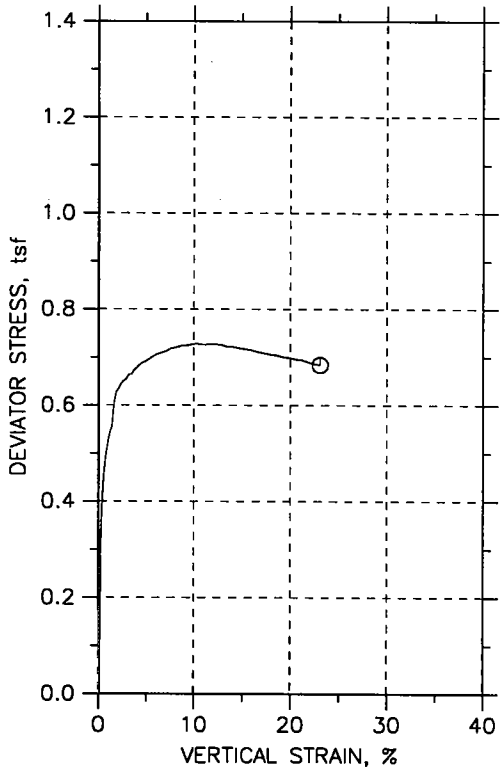


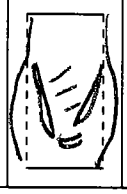
Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: 09-108B S-3	Tested By: BCM	Checked By: WPQ
Sample No.: S-3	Test Date: 2/23/09	Depth: 26.0'-28.5'
Test No.: 5742108BS3	Sample Type: 3" OST	Elevation: -
Description: SILTY CLAY SOME F-C SAND (SC) - SL. REDDISH BROWN GRAY. SILT POCKETS NOTED.		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B Value = 0.99		

Tue, 31-MAR-2009 10:07:47

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⊙			
Test No.		5742109BS4			
Initial	Diameter, in	2.8276			
	Height, in	5.9268			
	Water Content, %	20.13			
	Dry Density, pcf	109.8			
	Saturation, %	101.69			
	Void Ratio	0.53446			
Before Shear	Water Content, %	19.19			
	Dry Density, pcf	111.			
	Saturation, %	100.00			
	Void Ratio	0.51806			
	Back Press., tsf	5.0399			
Minor Prin. Stress, tsf		0.69135			
Max. Dev. Stress, tsf		0.72809			
Time to Failure, min		156			
Strain Rate, %/min		0.067			
B-Value		---			
Estimated Specific Gravity		2.70			
Liquid Limit		Pending			
Plastic Limit		Pending			
Plasticity Index		Pending			
Failure Sketch					

Project: KINGSTON COAL TVA

Location: TN

Project No.: 60095742

Boring No.: 09-109B S-4

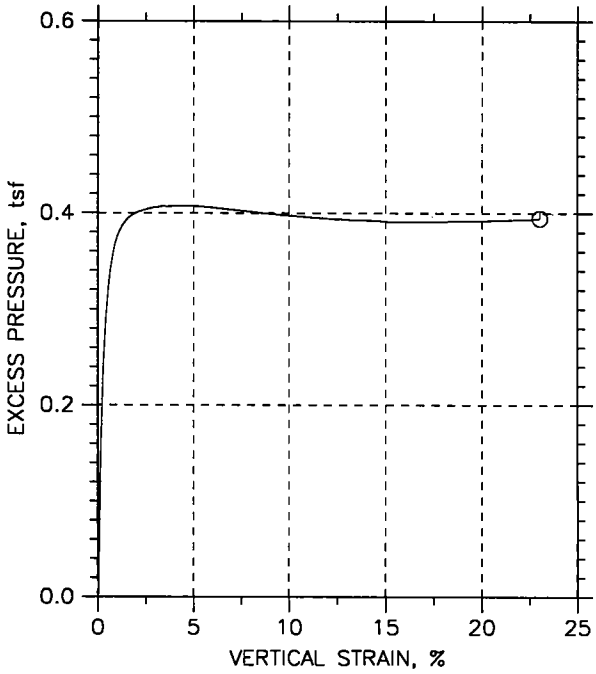
Sample Type: 3" OST

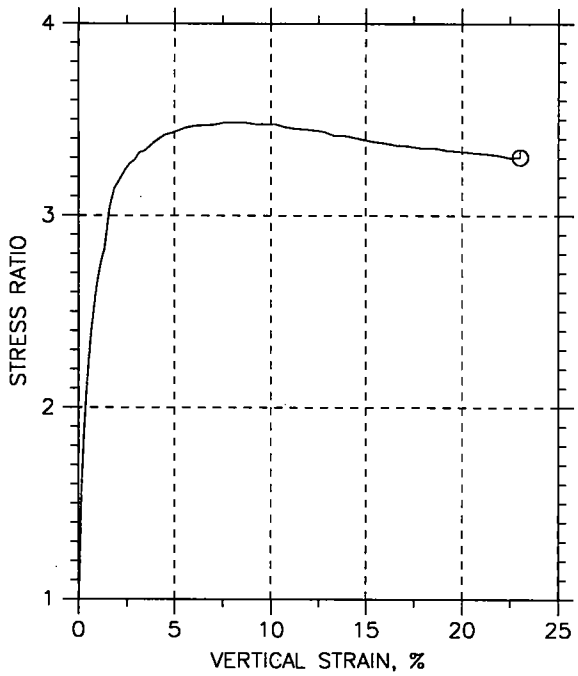
Description: SILTY CLAY TRACE F-C SAND TRACE F GRAVEL - GRAY CL

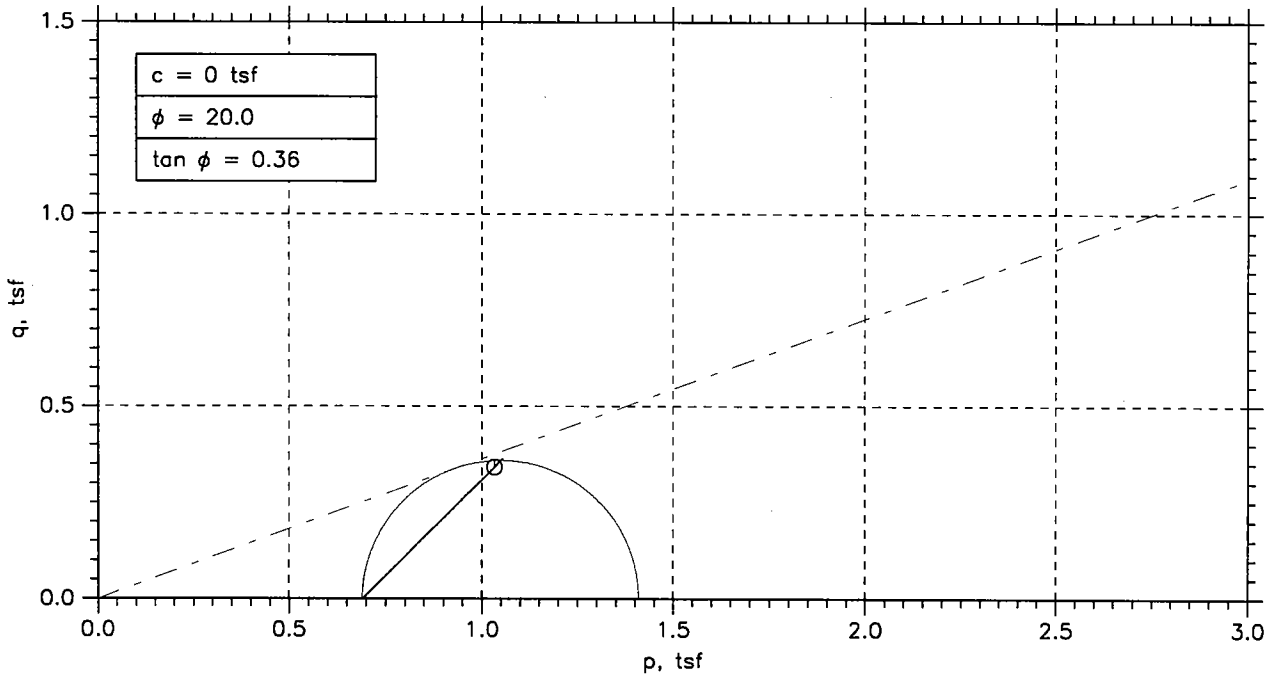
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 0.96

Tue, 31-MAR-2009 12:29:25

TRIAXIAL COMPRESSION TEST REPORT



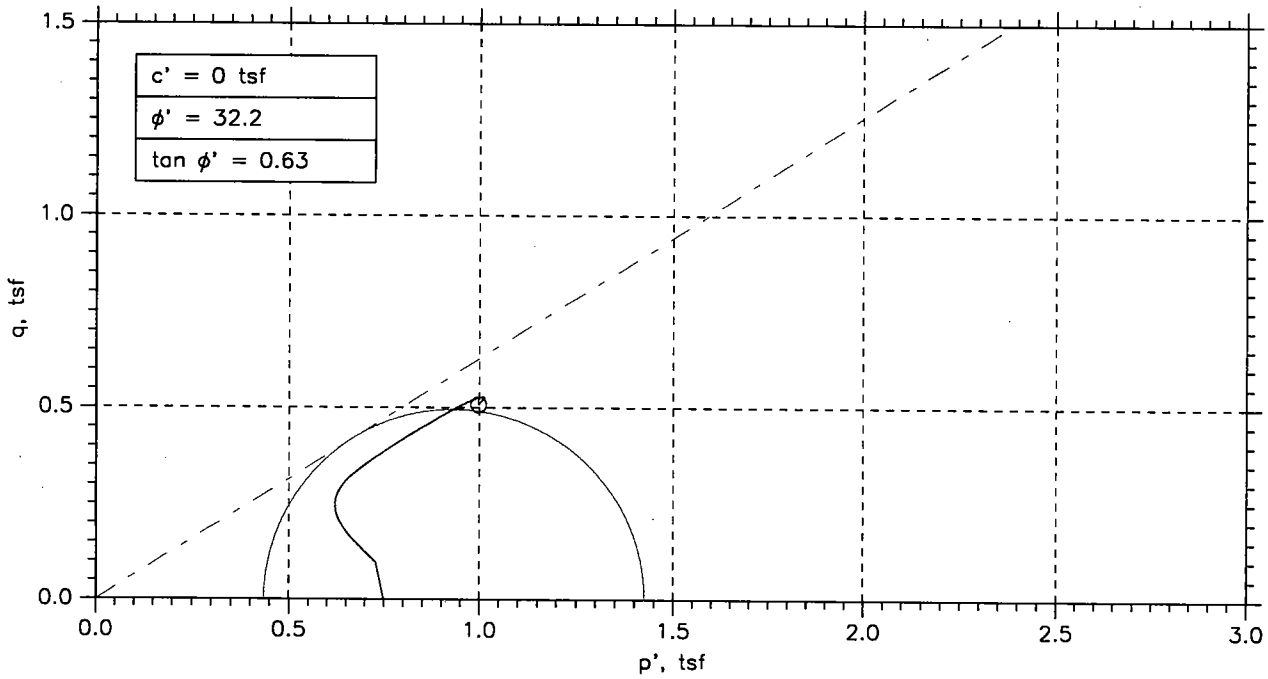


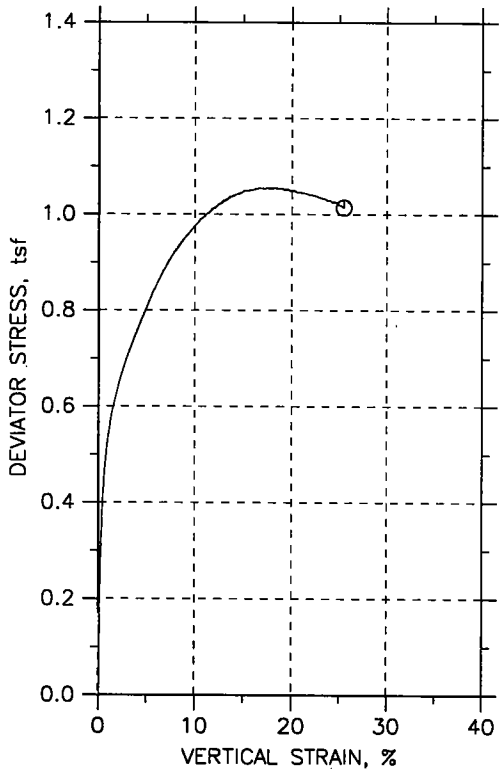


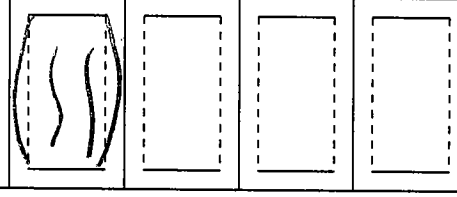
Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: 09-109B S-4	Tested By: BCM	Checked By: WPQ
Sample No.: OST-4	Test Date: 2/26/09	Depth: 28.0'-30.5'
Test No.: 5742109BS4	Sample Type: 3" OST	Elevation: -
Description: SILTY CLAY TRACE F-C SAND TRACE F GRAVEL - GRAY CL		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 0.96		

Tue, 31-MAR-2009 12:29:55

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⊙			
Test No.		5742109BS5			
Initial	Diameter, in	2.8374			
	Height, in	5.8846			
	Water Content, %	21.35			
	Dry Density, pcf	107.8			
	Saturation, %	102.34			
	Void Ratio	0.56317			
Before Shear	Water Content, %	19.67			
	Dry Density, pcf	110.1			
	Saturation, %	100.00			
	Void Ratio	0.53118			
	Back Press., tsf	5.0399			
Minor Prin. Stress, tsf		0.74886			
Max. Dev. Stress, tsf		1.0551			
Time to Failure, min		584			
Strain Rate, %/min		0.03			
B-Value		---			
Estimated Specific Gravity		2.70			
Liquid Limit		Pending			
Plastic Limit		Pending			
Plasticity Index		Pending			
Failure Sketch					

Project: KINGSTON COAL TVA

Location: TN

Project No.: 60095742

Boring No.: 09-109B S-5

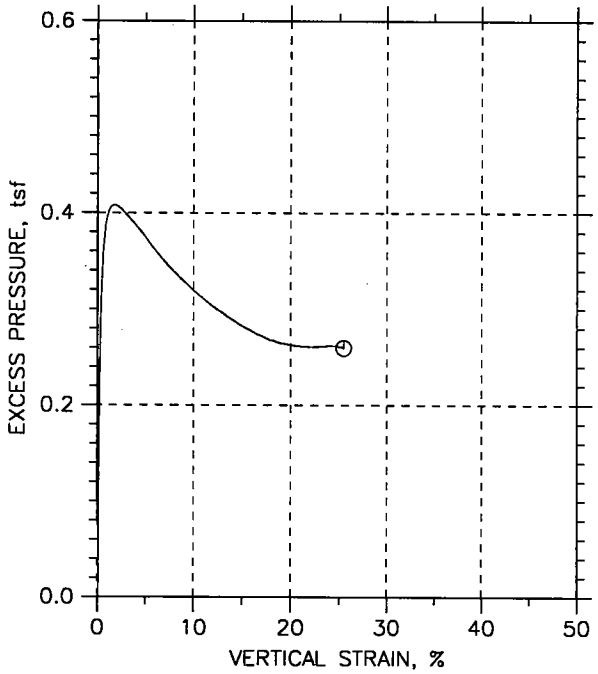
Sample Type: 3" OST

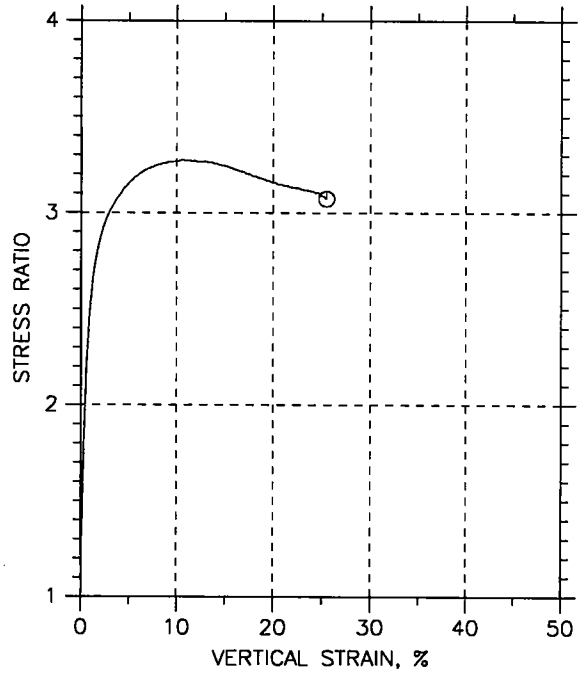
Description: SILTY CLAY SOME F-C SAND - BROWN AND GRAY CL SILT AND SAND POCKETS NOTED.

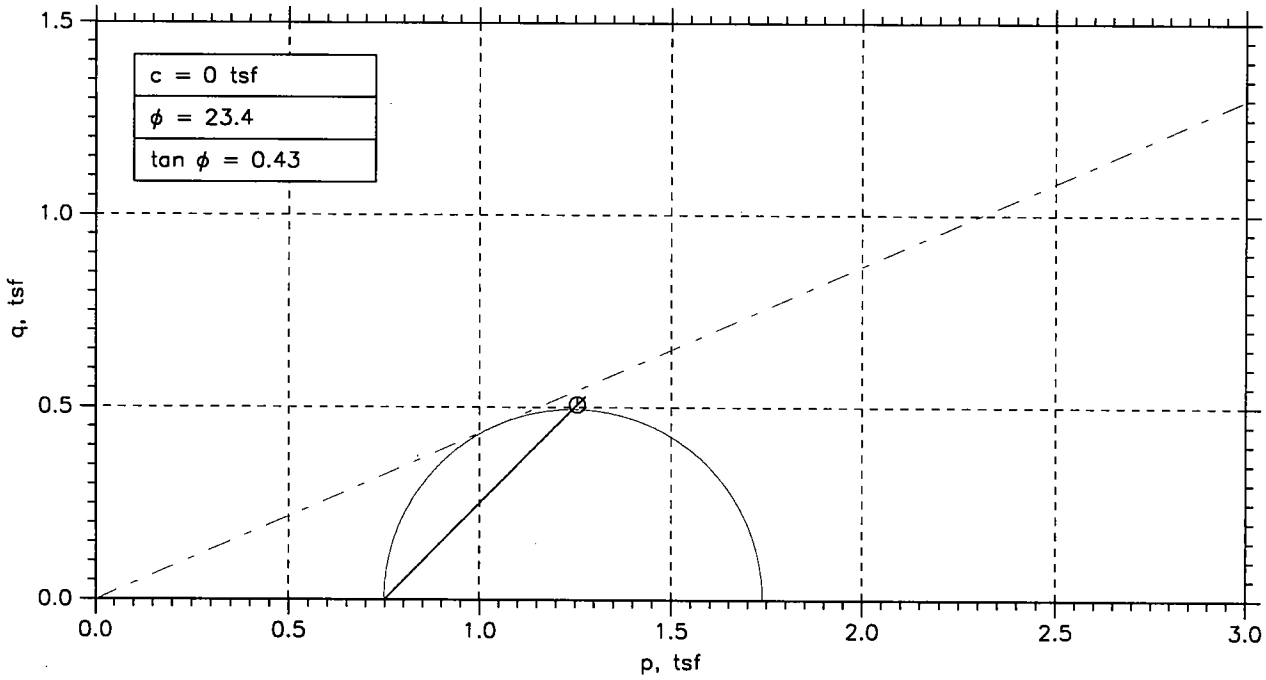
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 1.0

Tue, 31-MAR-2009 12:41:21

TRIAXIAL COMPRESSION TEST REPORT



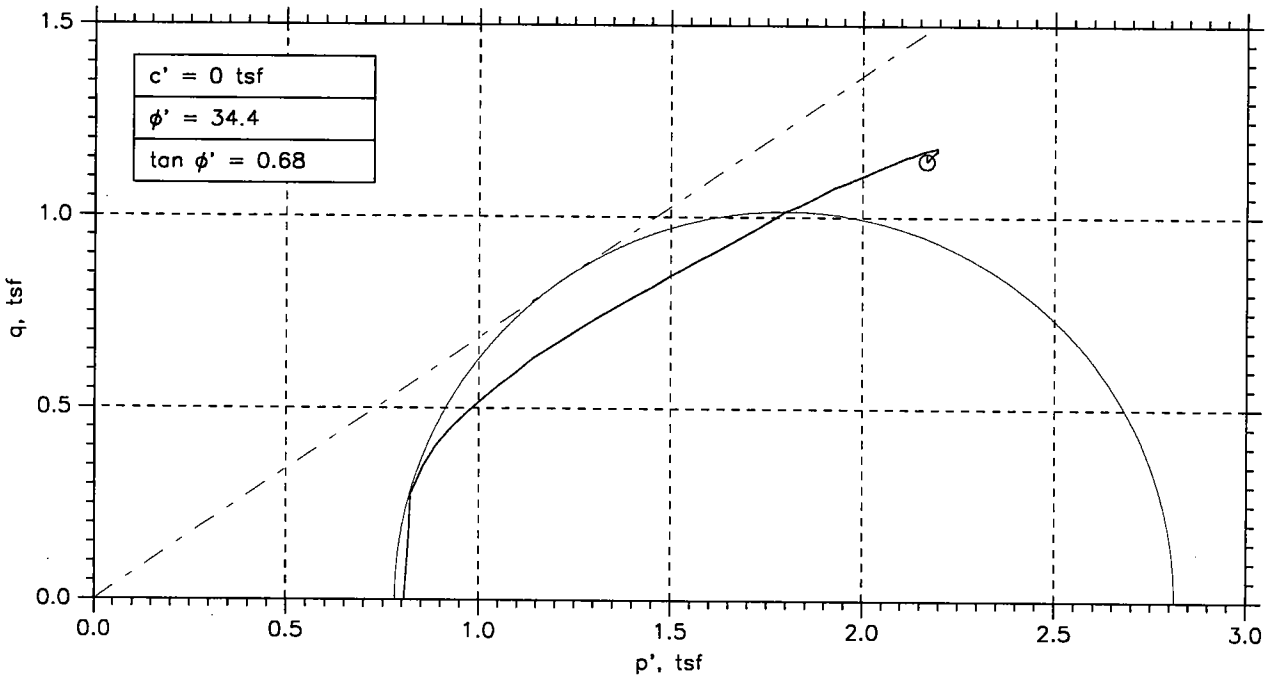


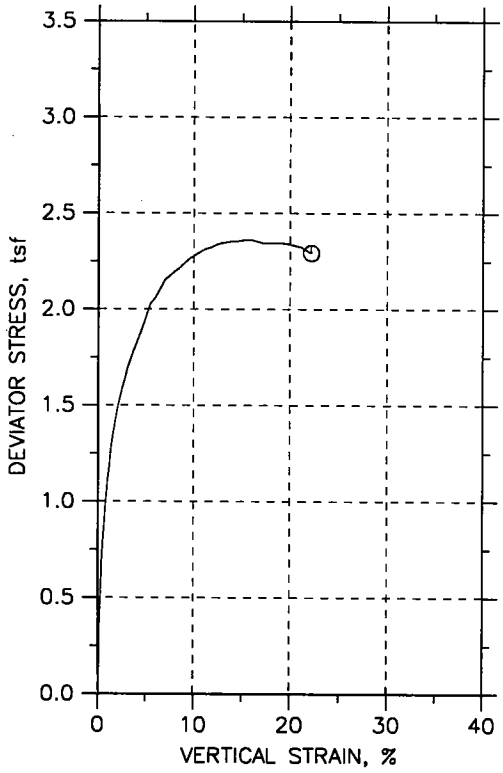


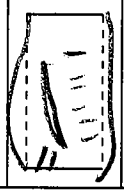
Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: 09-109B S-5	Tested By: BCM	Checked By: WPQ
Sample No.: OST-5	Test Date: 2/26/09	Depth: 30.5'-33.0'
Test No.: 5742109BS5	Sample Type: 3" OST	Elevation: -
Description: SILTY CLAY SOME F-C SAND - BROWN AND GRAY CL SILT AND SAND POCKETS NOTED.		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 1.0		

Tue, 31-MAR-2009 12:41:47

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⊙			
Test No.		5742109BS6			
Initial	Diameter, in	2.848			
	Height, in	5.9528			
	Water Content, %	21.04			
	Dry Density, pcf	107.2			
	Saturation, %	99.24			
	Void Ratio	0.57253			
Before Shear	Water Content, %	19.02			
	Dry Density, pcf	111.4			
	Saturation, %	100.00			
	Void Ratio	0.51343			
	Back Press., tsf	5.0399			
Minor Prin. Stress, tsf		0.80655			
Max. Dev. Stress, tsf		2.3612			
Time to Failure, min		244			
Strain Rate, %/min		0.063			
B-Value		---			
Estimated Specific Gravity		2.70			
Liquid Limit		Pending			
Plastic Limit		Pending			
Plasticity Index		Pending			
Failure Sketch					

Project: KINGSTON COAL TVA

Location: TN

Project No.: 60095742

Boring No.: 09-109B S-6

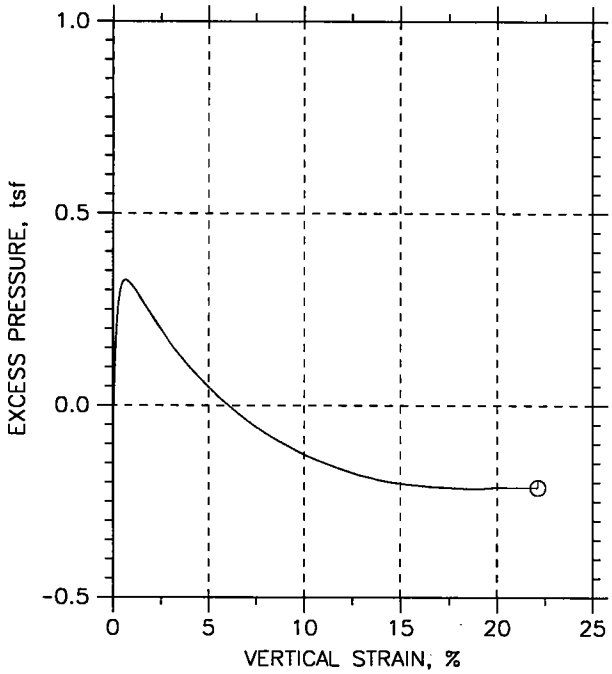
Sample Type: 3" OST

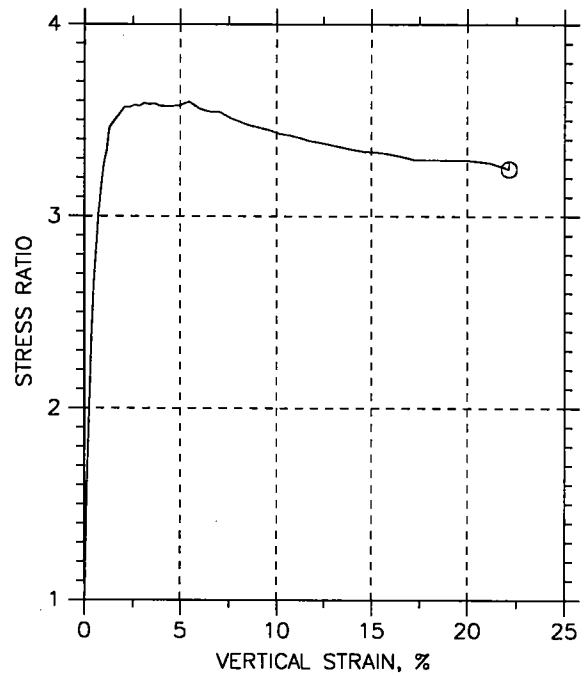
Description: SILTY CLAY SOME F-C SAND - BROWN AND GRAY CL SILT AND SAND POCKETS NOTED.

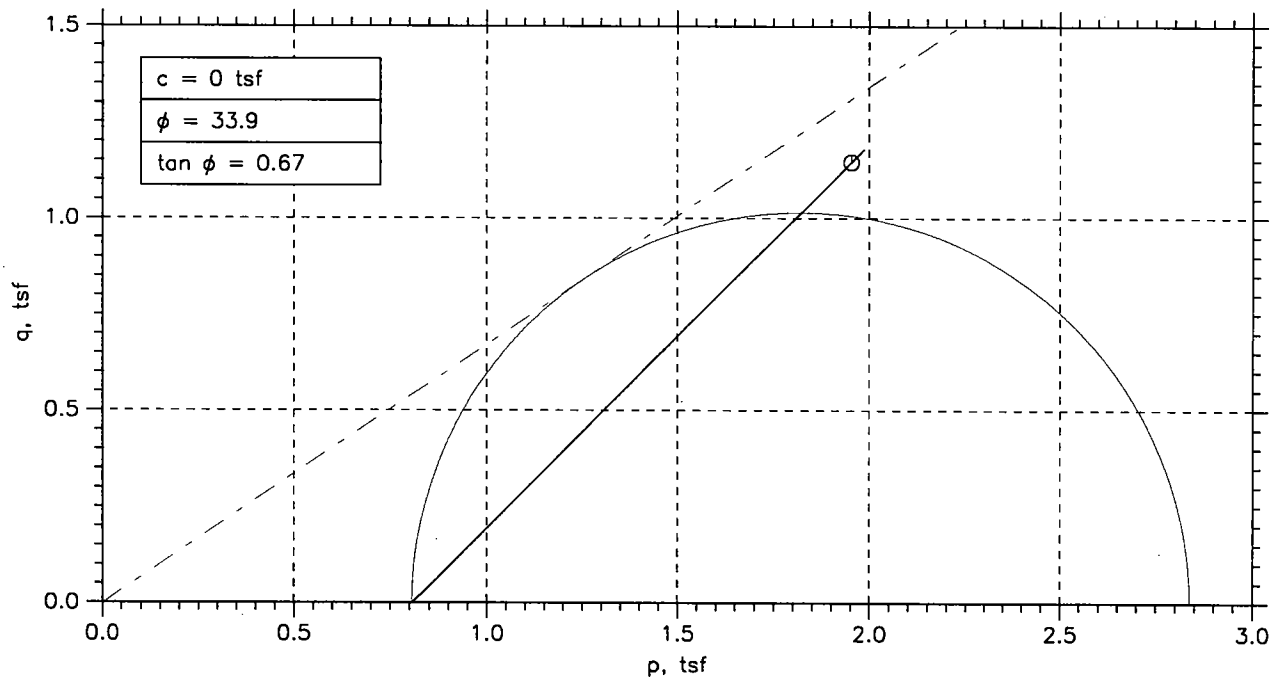
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 1.0

Tue, 31-MAR-2009 12:48:49

TRIAXIAL COMPRESSION TEST REPORT



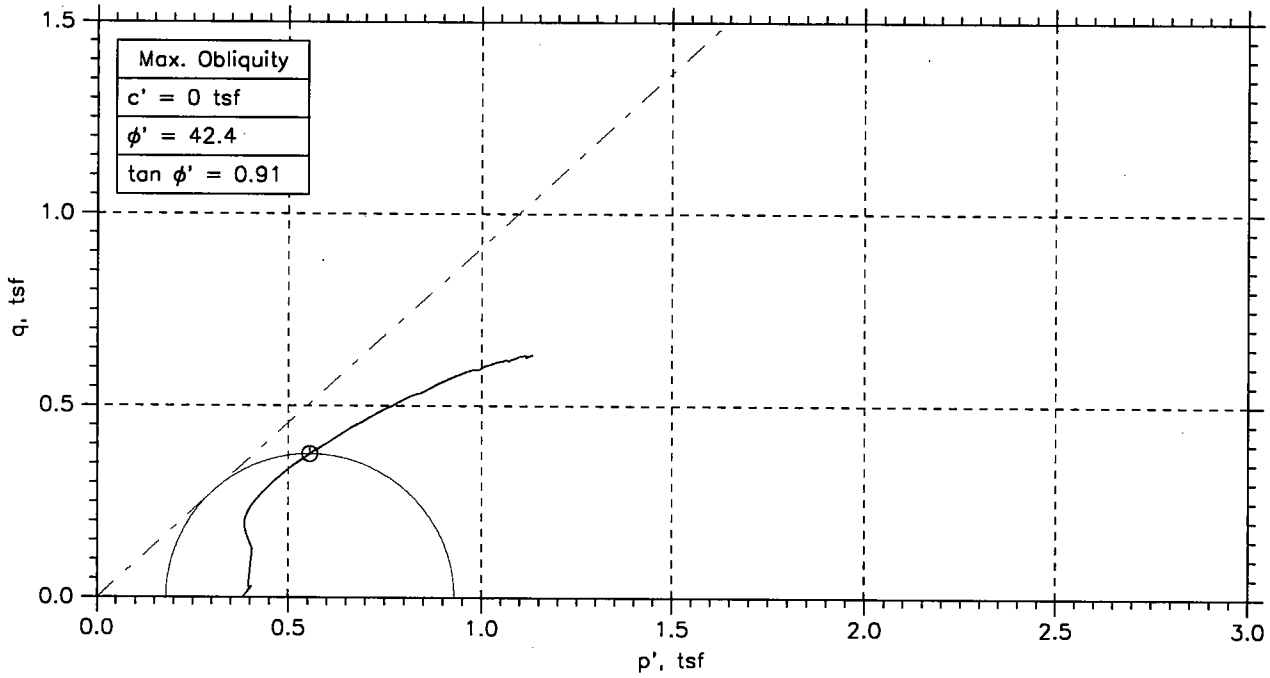


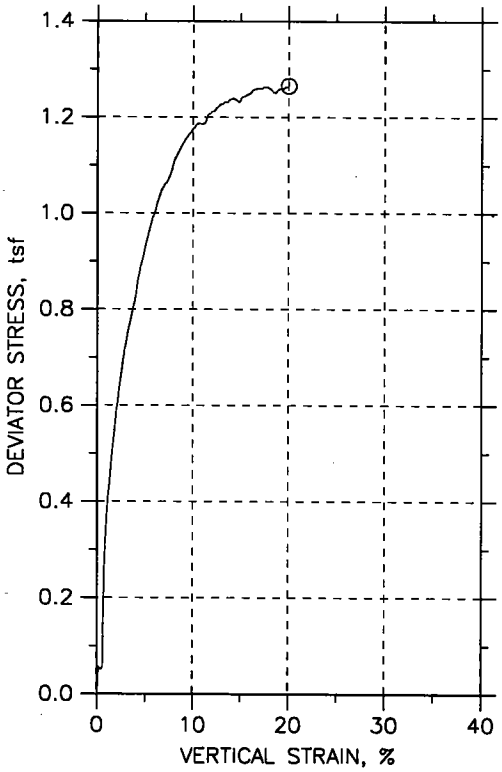


Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: 09-109B S-6	Tested By: BCM	Checked By: WPQ
Sample No.: OST-6	Test Date: 2/26/09	Depth: 33.0'-35.5'
Test No.: 5742109BS6	Sample Type: 3" OST	Elevation: -
Description: SILTY CLAY SOME F-C SAND - BROWN AND GRAY CL SILT AND SAND POCKETS NOTED.		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 1.0		

Tue, 31-MAR-2009 12:49:25

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





Symbol		⊙			
Sample No.		S-3			
Test No.		5742201BS3			
Depth		28.0'-30.5'			
Initial	Diameter, in	2.871			
	Height, in	5.441			
	Water Content, %	23.5			
	Dry Density, pcf	103.3			
	Saturation, %	100.5			
	Void Ratio	0.632			
Before Shear	Water Content, %	22.7			
	Dry Density, pcf	104.4			
	Saturation*, %	100.0			
	Void Ratio	0.614			
	Back Press., tsf	6.984			
Ver. Eff. Cons. Stress, tsf		.3818			
Shear Strength, tsf		.6326			
Strain at Failure, %		20			
Strain Rate, %/min		0.06			
B-Value		0.95			
Estimated Specific Gravity		2.7			
Liquid Limit		Pending			
Plastic Limit		Pending			

Project: KINGSTON COAL

Location: TN

Project No.: 60095742

Boring No.: 09-201B S-3

Sample Type: 3" OST

Description: SILTY CLAY TRACE F-M SAND - BROWN CL

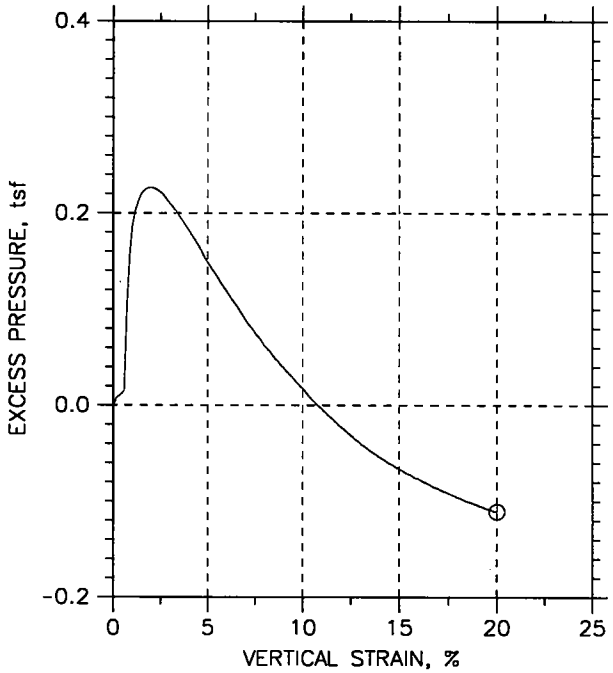
Remarks: TEST PERFORMED AS PER ASTM D 4767

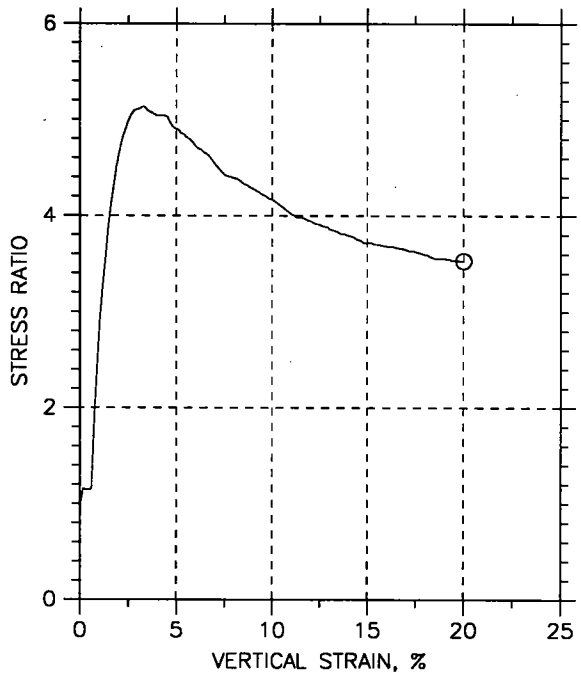
Tue, 31-MAR-2009 13:52:02

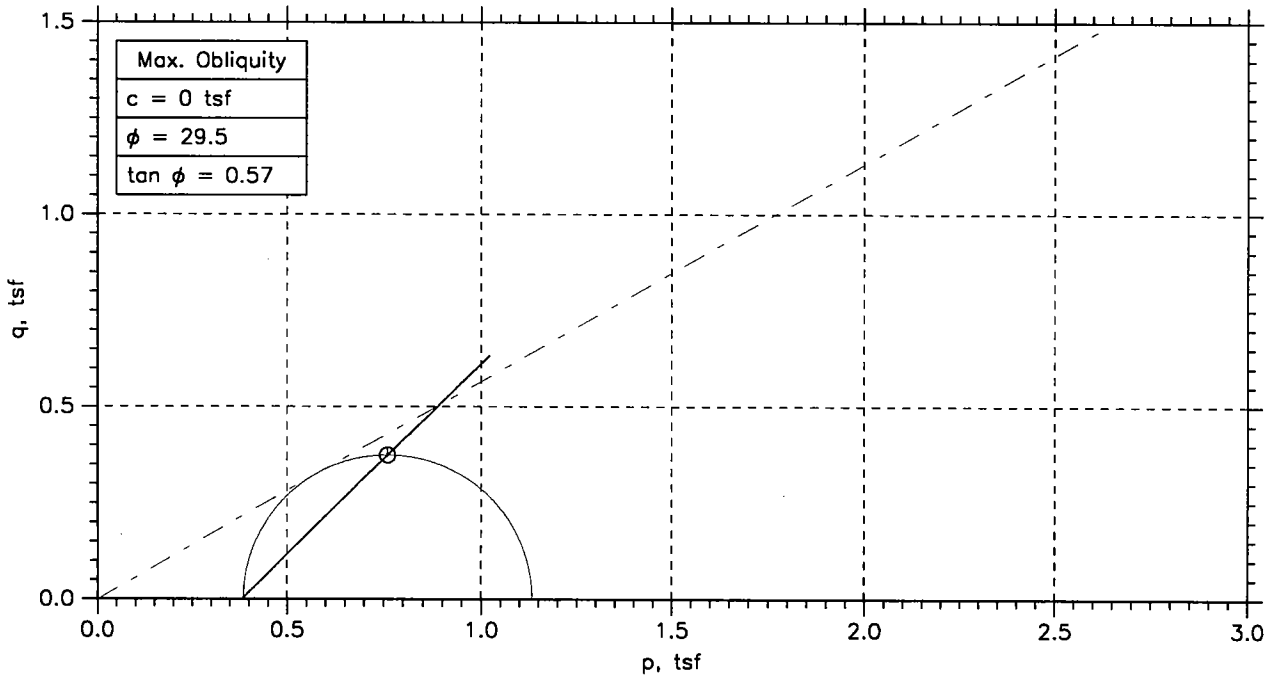
Phase calculations based on start and end of test.

* Saturation is set to 100% for phase calculations.

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



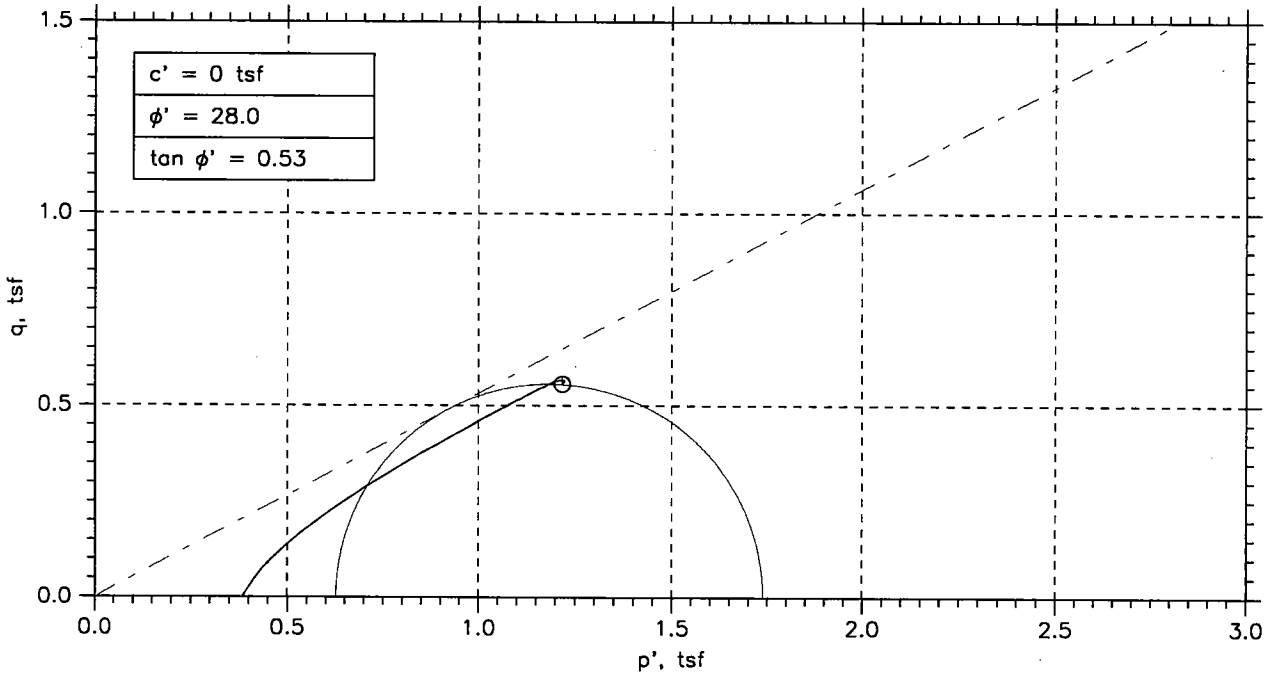


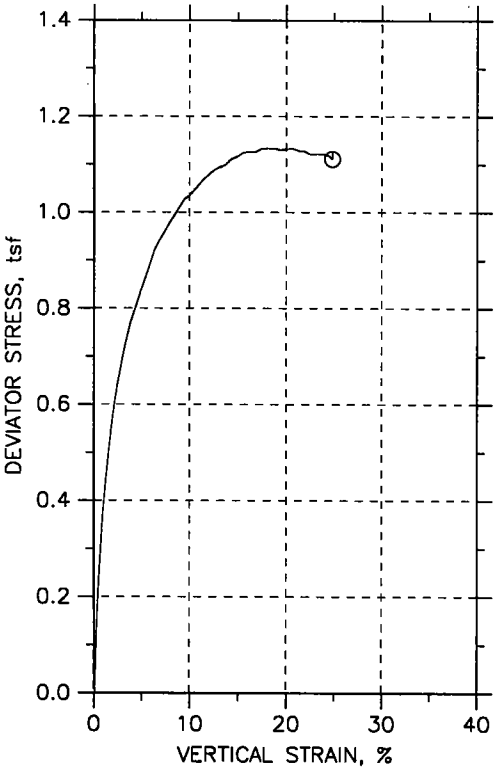


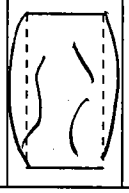
Project: KINGSTON COAL	Location: TN	Project No.: 60095742
Boring No.: 09-201B S-3	Tested By: BCM	Checked By: WPQ
Sample No.: S-3	Test Date: 2/20/09	Depth: 28.0'-30.5'
Test No.: 5742201BS3	Sample Type: 3" OST	Elevation:
Description: SILTY CLAY TRACE F-M SAND - BROWN CL		
Remarks: TEST PERFORMED AS PER ASTM D 4767		

Tue, 31-MAR-2009 14:06:17

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⊙			
Test No.		5742201BS3			
Initial	Diameter, in	2.8366			
	Height, in	6.0945			
	Water Content, %	26.52			
	Dry Density, pcf	98.45			
	Saturation, %	100.57			
	Void Ratio	0.71201			
Before Shear	Water Content, %	26.11			
	Dry Density, pcf	98.86			
	Saturation, %	100.00			
	Void Ratio	0.70497			
	Back Press., tsf	5.0399			
Minor Prin. Stress, tsf		0.38175			
Max. Dev. Stress, tsf		1.1333			
Time to Failure, min		294			
Strain Rate, %/min		0.065			
B-Value		---			
Estimated Specific Gravity		2.70			
Liquid Limit		Pending			
Plastic Limit		Pending			
Plasticity Index		Pending			
Failure Sketch					

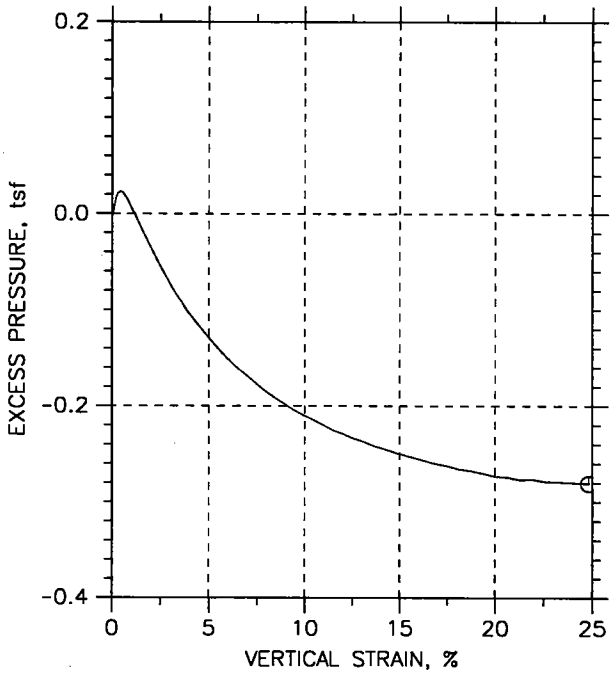
Project: KINGSTON COAL TVA	Failure Sketch
Location: TN	
Project No.: 60095742	
Boring No.: 09-201B S-3	
Sample Type: 3" OST	

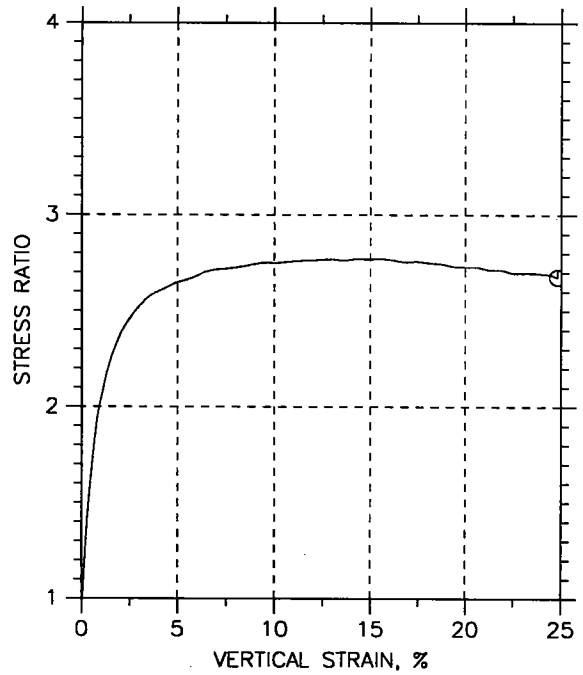
Description: SILTY CLAY LITTLE F-M SAND TRACE F GRAVEL - BROWNISH GRAY CL

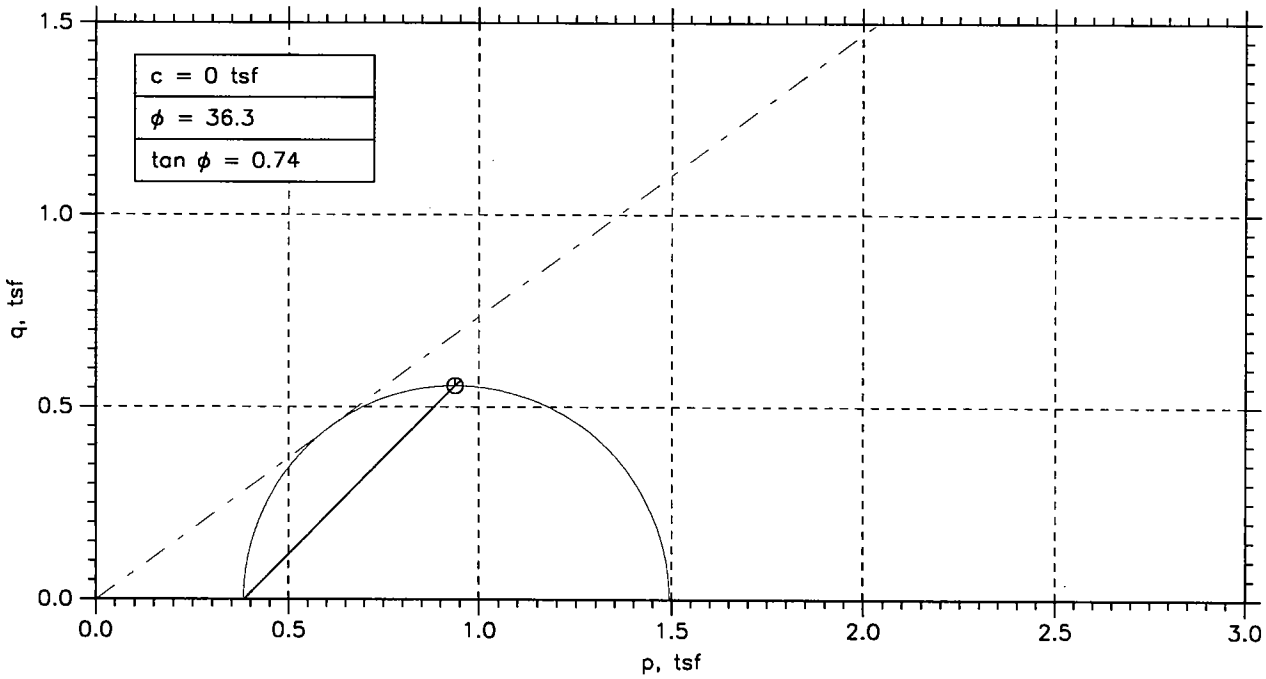
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B Value = 0.98

Tue, 31-MAR-2009 15:08:43

TRIAXIAL COMPRESSION TEST REPORT



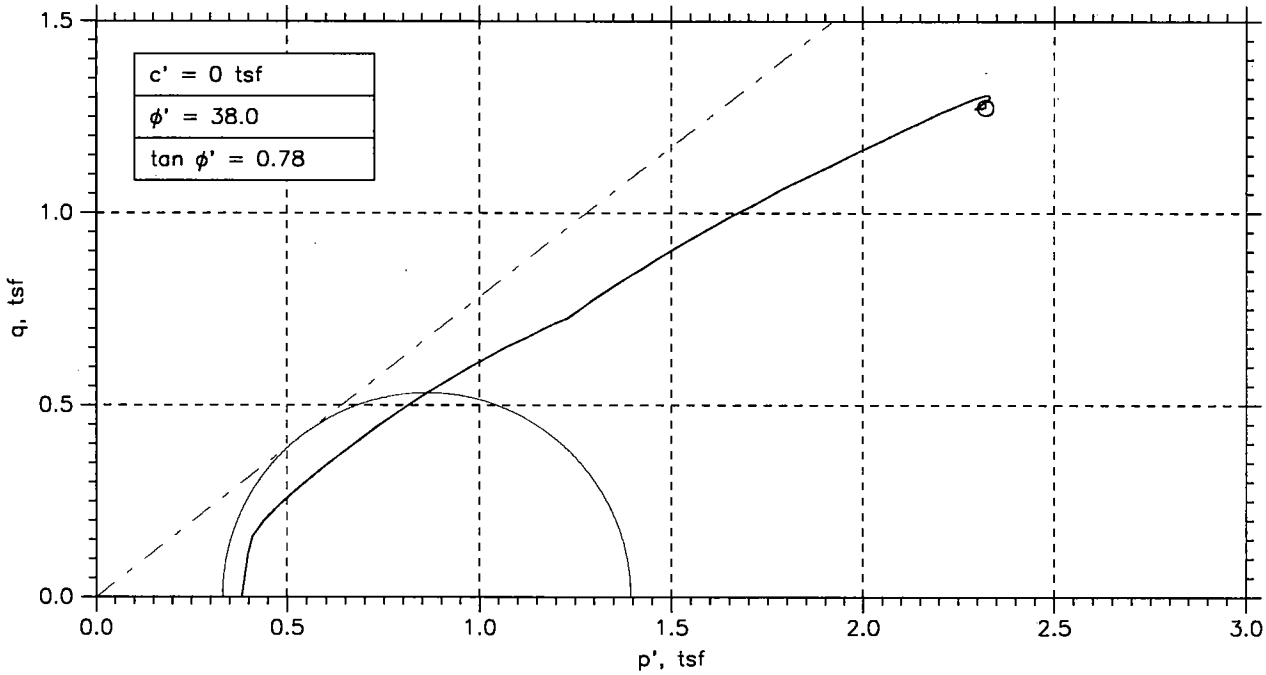


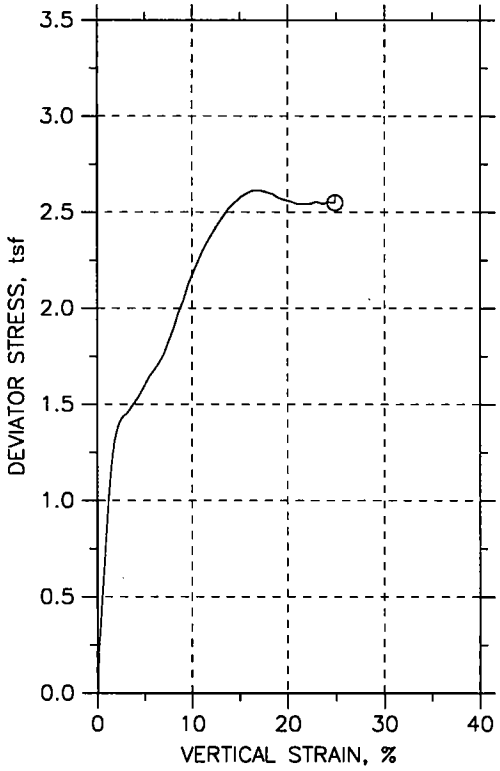


Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: 09-201B S-3	Tested By: BCM	Checked By: WPQ
Sample No.: OST-3	Test Date: 2/23/09	Depth: 28.0'-30.5'
Test No.: 5742201BS3	Sample Type: 3" OST	Elevation:
Description: SILTY CLAY LITTLE F-M SAND TRACE F GRAVEL - BROWNISH GRAY CL		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B Value = 0.98		

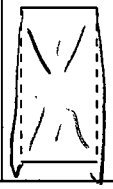
Tue, 31-MAR-2009 15:09:06

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⊙			
Test No.		5742202BS3			
Initial	Diameter, in	2.8386			
	Height, in	6.0059			
	Water Content, %	18.54			
	Dry Density, pcf	111.6			
	Saturation, %	98.04			
	Void Ratio	0.51057			
Before Shear	Water Content, %	18.81			
	Dry Density, pcf	111.8			
	Saturation, %	100.00			
	Void Ratio	0.50784			
	Back Press., tsf	5.0402			
Minor Prin. Stress, tsf		0.3814			
Max. Dev. Stress, tsf		2.6122			
Time to Failure, min		254			
Strain Rate, %/min		0.66			
B-Value		---			
Estimated Specific Gravity		2.70			
Liquid Limit		Pending			
Plastic Limit		Pending			
Plasticity Index		Pending			

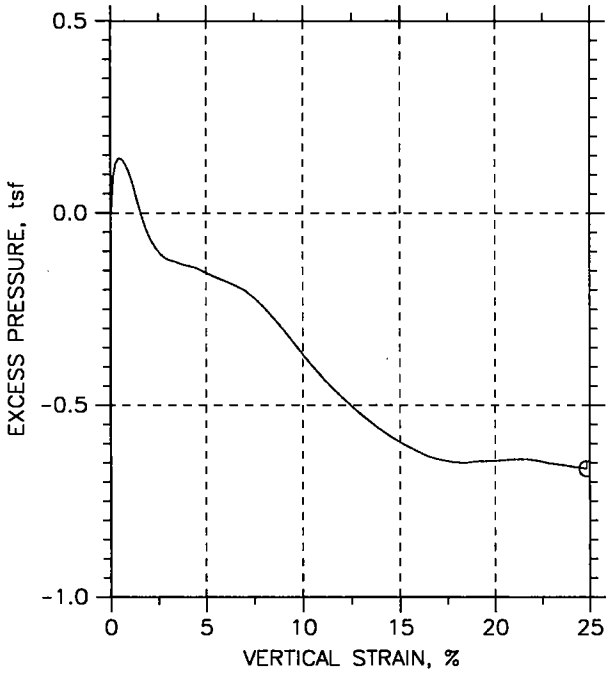
Project: KINGSTON COAL TVA	Failure Sketch			
Location: TN				
Project No.: 60095742				
Boring No.: 09-202B S-3				
Sample Type: 3" OST				

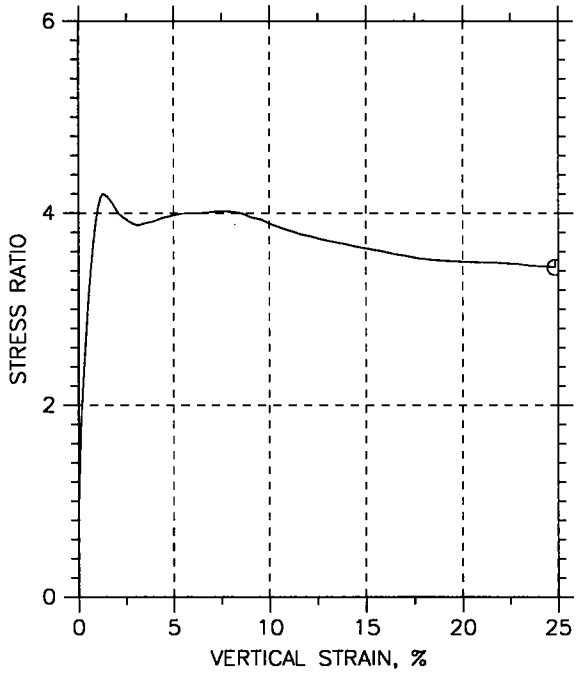
Description: SILTY CLAY SOME F-C SAND (SC) - BROWN AND GRAY. SILT POCKETS NOTED.

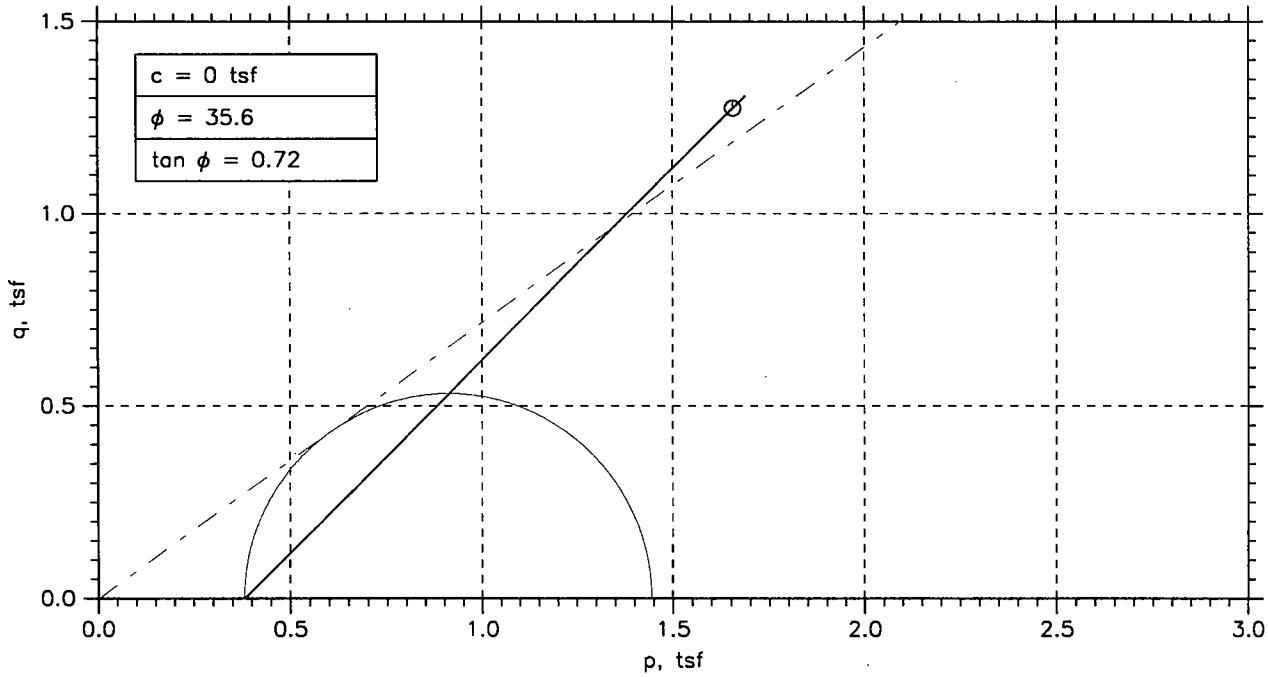
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B Value = 0.95

Tue, 31-MAR-2009 09:50:48

TRIAXIAL COMPRESSION TEST REPORT



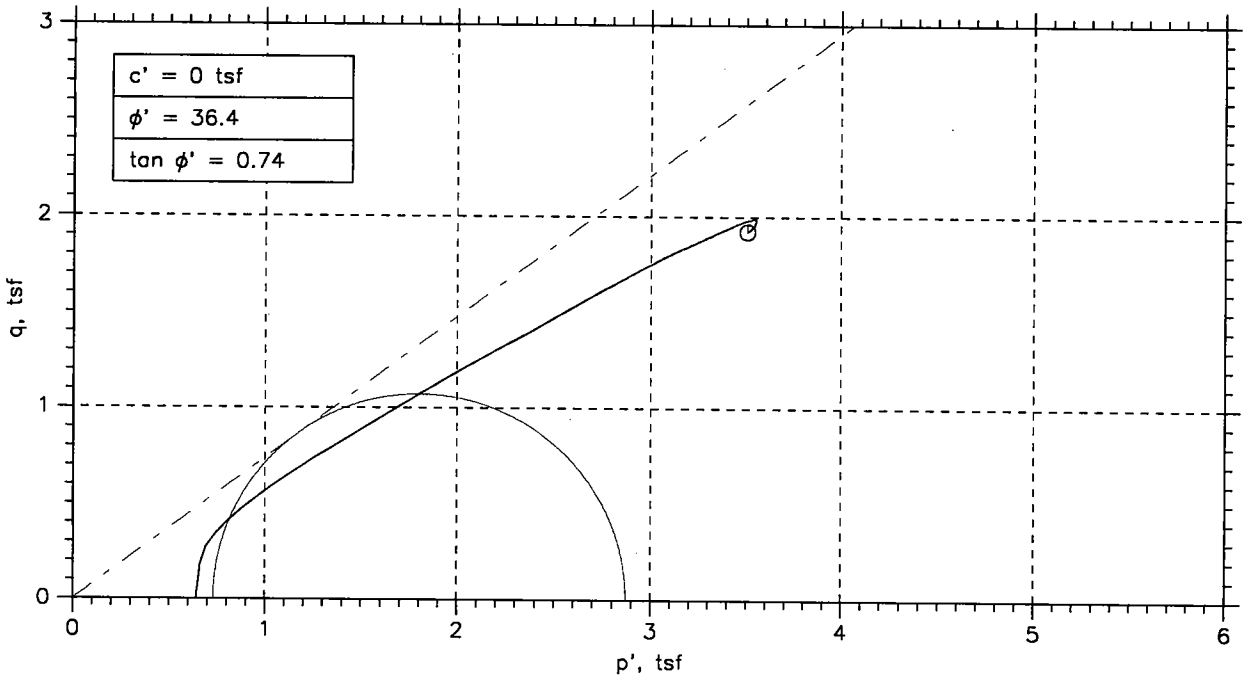


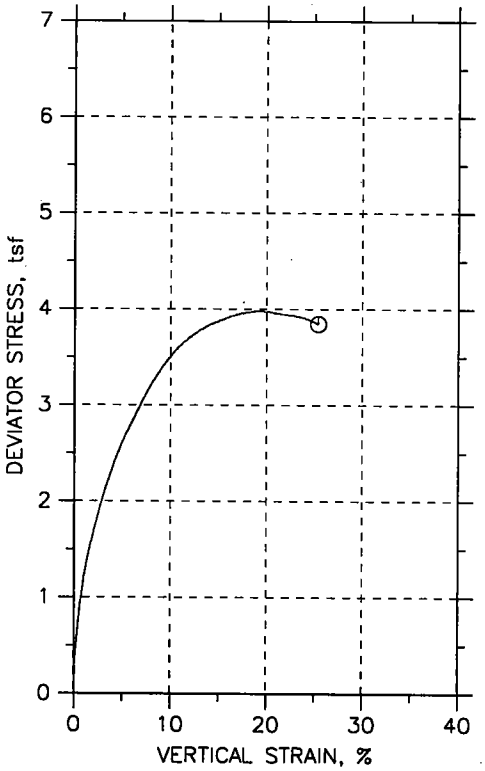


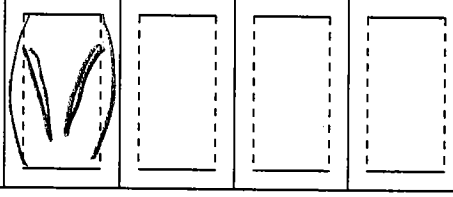
Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: 09-202B S-3	Tested By: BCM/JDW	Checked By: WPQ
Sample No.: S-3	Test Date: 2/19/09	Depth: 31.0'-33.5'
Test No.: 5742202BS3	Sample Type: 3" OST	Elevation: -
Description: SILTY CLAY SOME F-C SAND (SC) - BROWN AND GRAY. SILT POCKETS NOTED.		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B Value = 0.95		

Tue, 31-MAR-2009 09:51:24

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⊙			
Test No.		5742203B2S3			
Initial	Diameter, in	2.8441			
	Height, in	5.898			
	Water Content, %	18.16			
	Dry Density, pcf	112.1			
	Saturation, %	97.30			
	Void Ratio	0.50387			
Before Shear	Water Content, %	18.43			
	Dry Density, pcf	112.5			
	Saturation, %	100.00			
	Void Ratio	0.49768			
	Back Press., tsf	5.0399			
Minor Prin. Stress, tsf		0.64094			
Max. Dev. Stress, tsf		3.9806			
Time to Failure, min		665			
Strain Rate, %/min		0.9			
B-Value		---			
Estimated Specific Gravity		2.70			
Liquid Limit		Pending			
Plastic Limit		Pending			
Plasticity Index		Pending			
Failure Sketch					

Project: KINGSTON COAL TVA

Location: TN

Project No.: 60095742

Boring No.: 09-203B2 S3

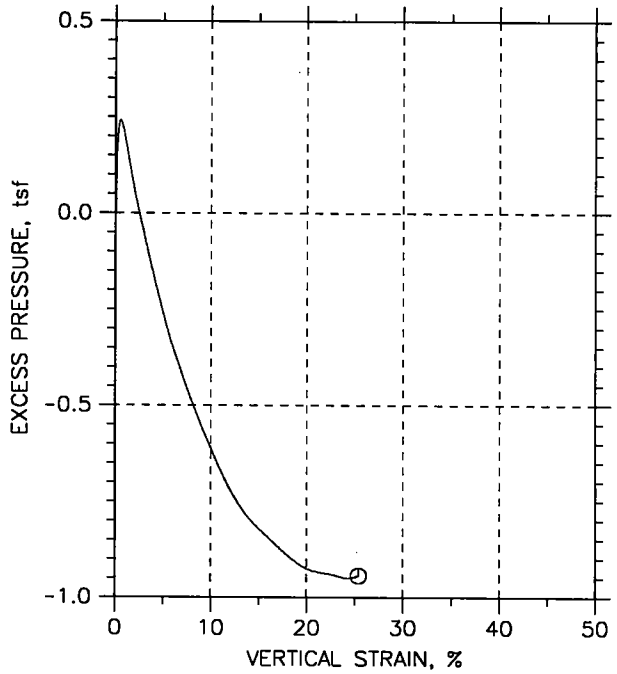
Sample Type: 3" OST

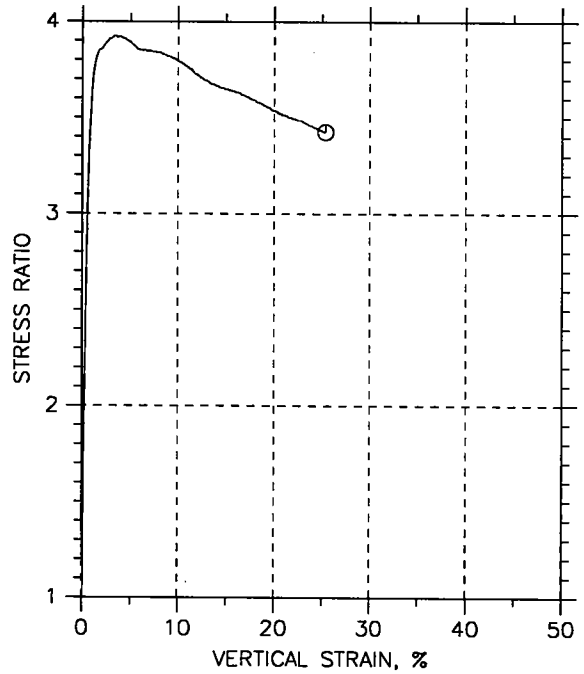
Description: SILTY CLAY SOME F-C SAND - BROWN AND GRAY CL SILT AND SAND POCKETS NOTED.

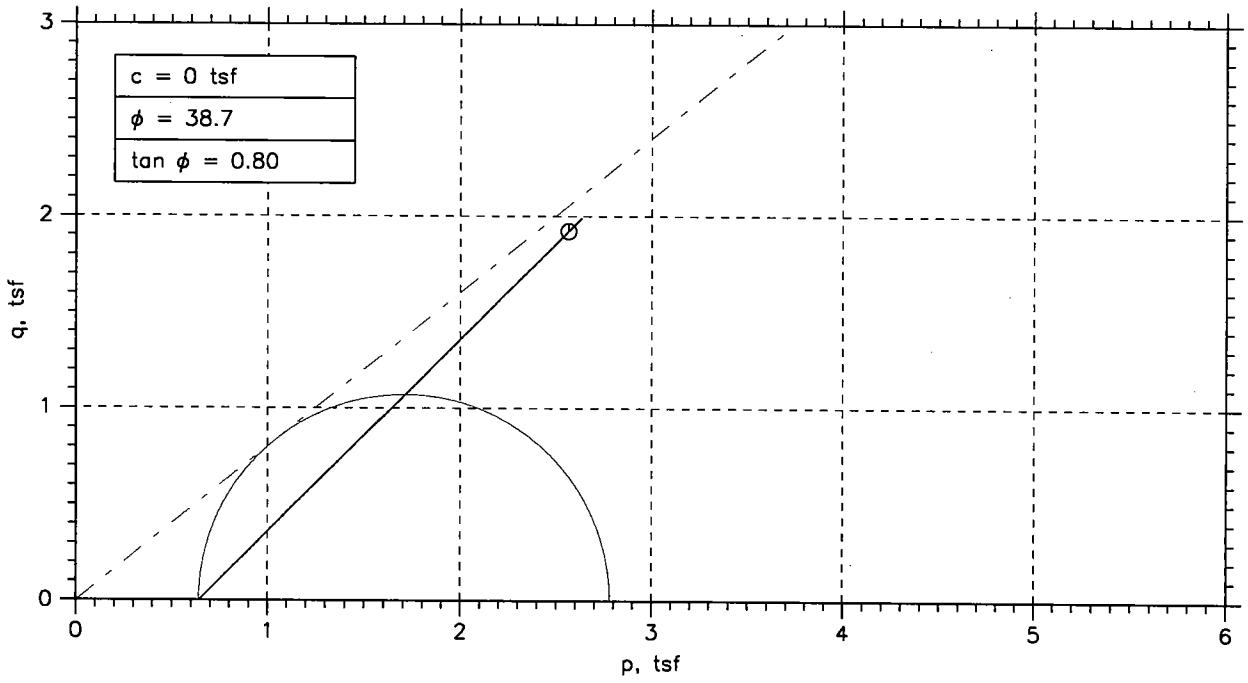
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 0.95

Tue, 31-MAR-2009 12:57:02

TRIAXIAL COMPRESSION TEST REPORT



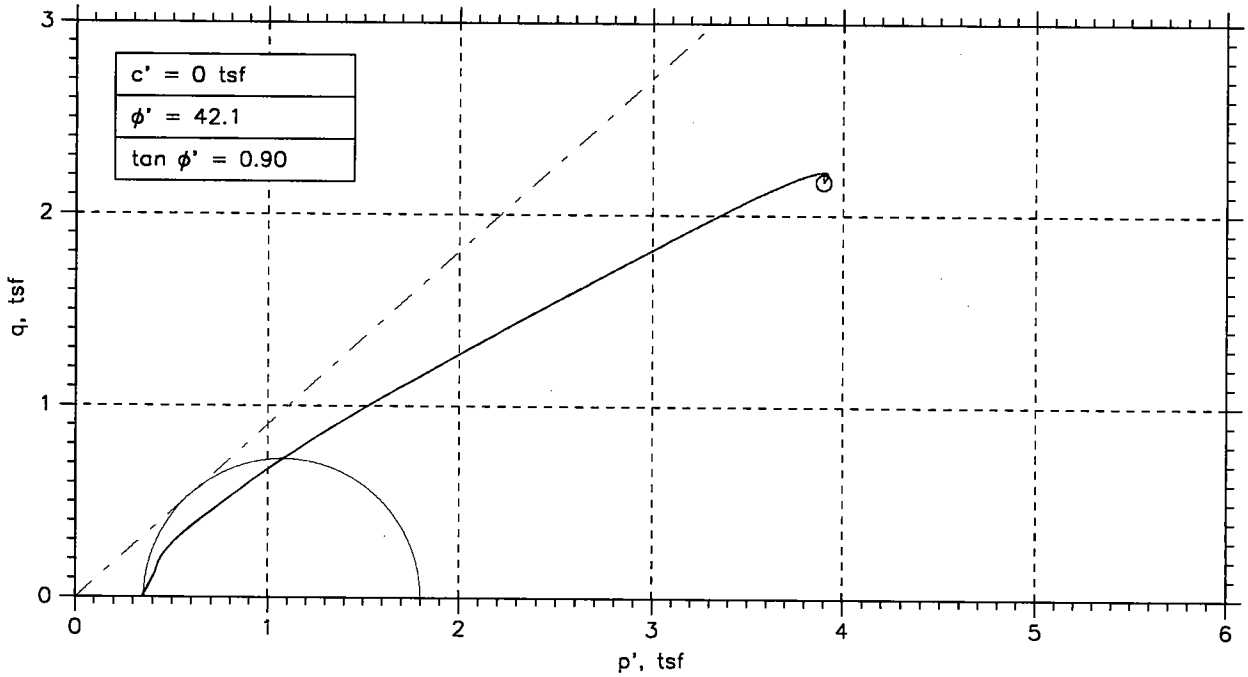


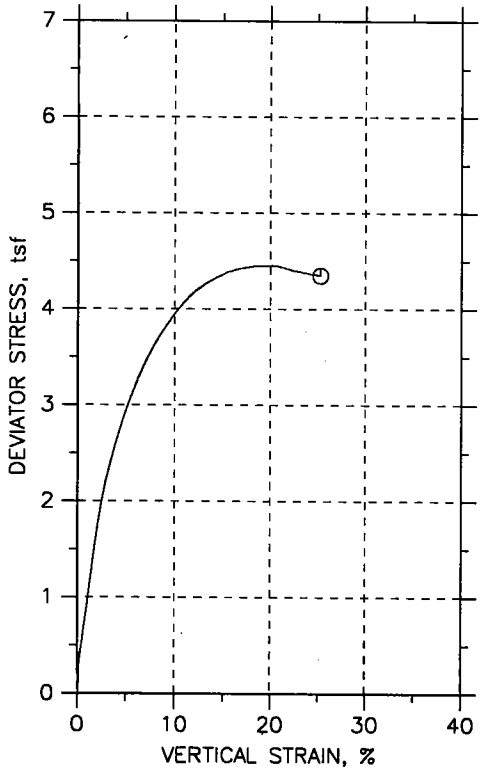


Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: 09-203B2 S3	Tested By: BCM	Checked By: WPQ
Sample No.: OST-3	Test Date: 2/26/09	Depth: 26.0'-28.5'
Test No.: 5742203B2S3	Sample Type: 3" OST	Elevation: -
Description: SILTY CLAY SOME F-C SAND - BROWN AND GRAY CL SILT AND SAND POCKETS NOTED.		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 0.95		

Tue, 31-MAR-2009 12:57:47

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⊙			
Test No.		5742203BS5			
Initial	Diameter, in	2.8331			
	Height, in	5.6008			
	Water Content, %	17.96			
	Dry Density, pcf	114.1			
	Saturation, %	101.58			
	Void Ratio	0.47734			
Before Shear	Water Content, %	17.56			
	Dry Density, pcf	114.3			
	Saturation, %	100.00			
	Void Ratio	0.47417			
	Back Press., tsf	5.0399			
Minor Prin. Stress, tsf		0.34575			
Max. Dev. Stress, tsf		4.4539			
Time to Failure, min		286.56			
Strain Rate, %/min		0.069			
B-Value		---			
Estimated Specific Gravity		2.70			
Liquid Limit		Pending			
Plastic Limit		Pending			
Plasticity Index		Pending			
Failure Sketch					

Project: KINGSTON COAL TVA

Location: TN

Project No.: 60095742

Boring No.: 09-203B S-5

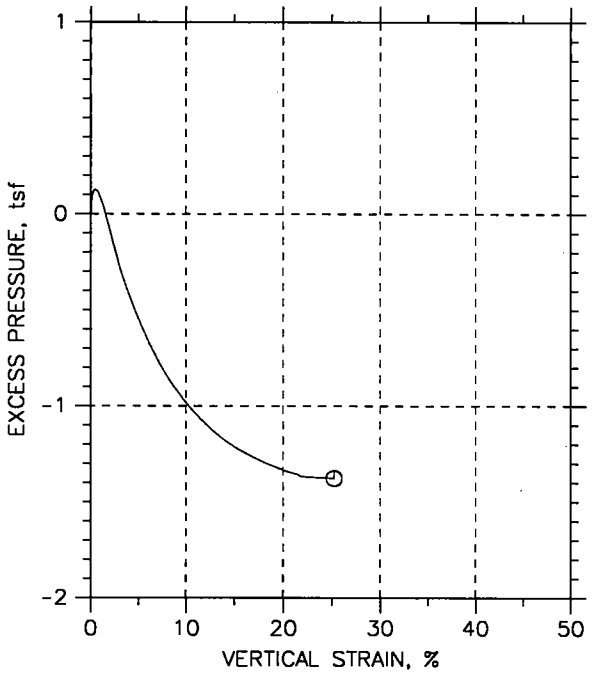
Sample Type: 3" OST

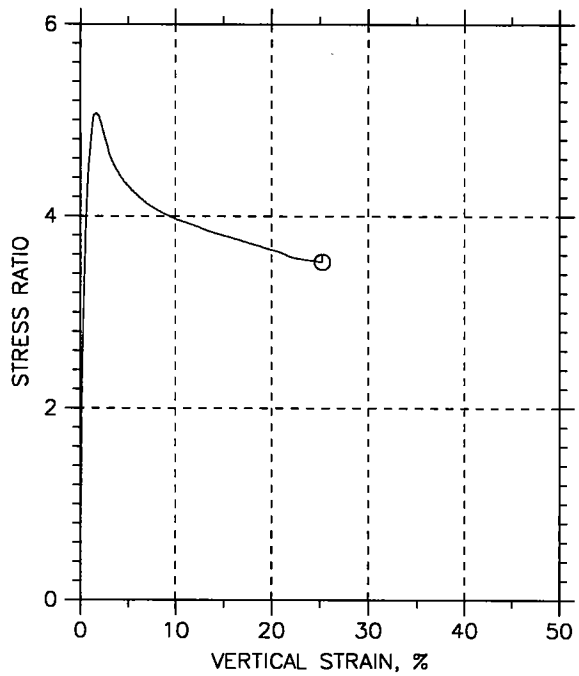
Description: SILTY CLAY AND F-M SAND - BROWN AND GRAY (CL). SILT AND SAND POCKETS NOTED.

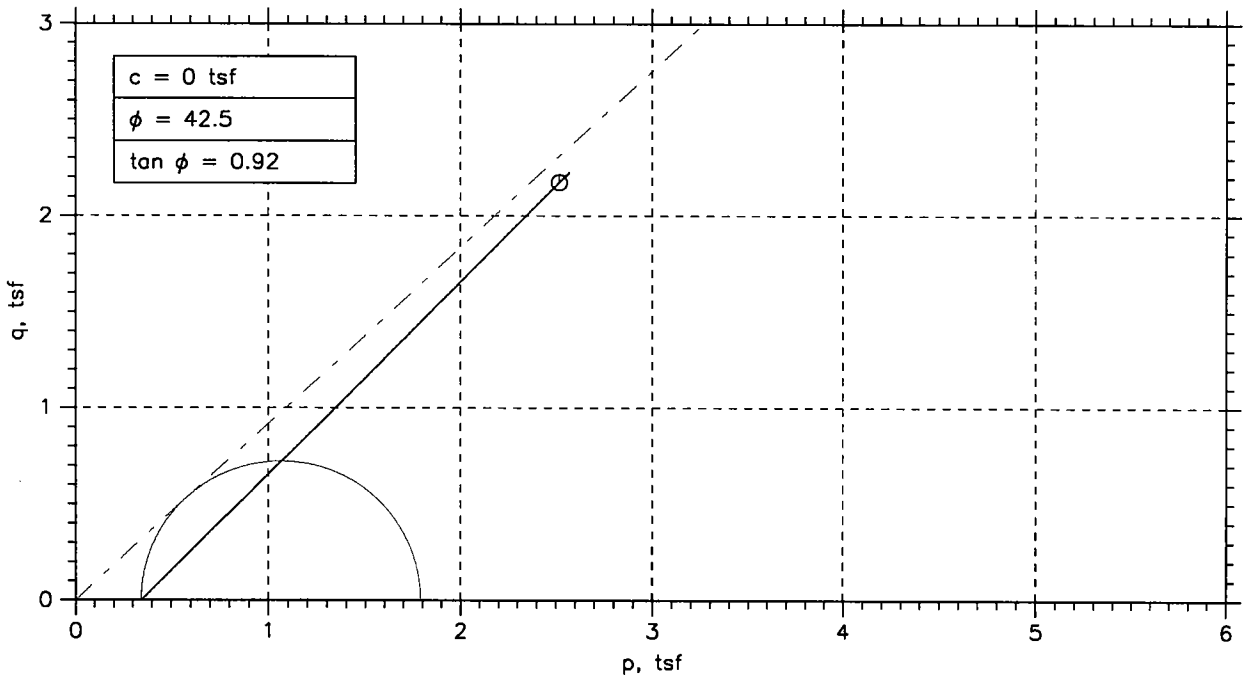
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 1.00

Tue, 31-MAR-2009 11:46:31

TRIAXIAL COMPRESSION TEST REPORT



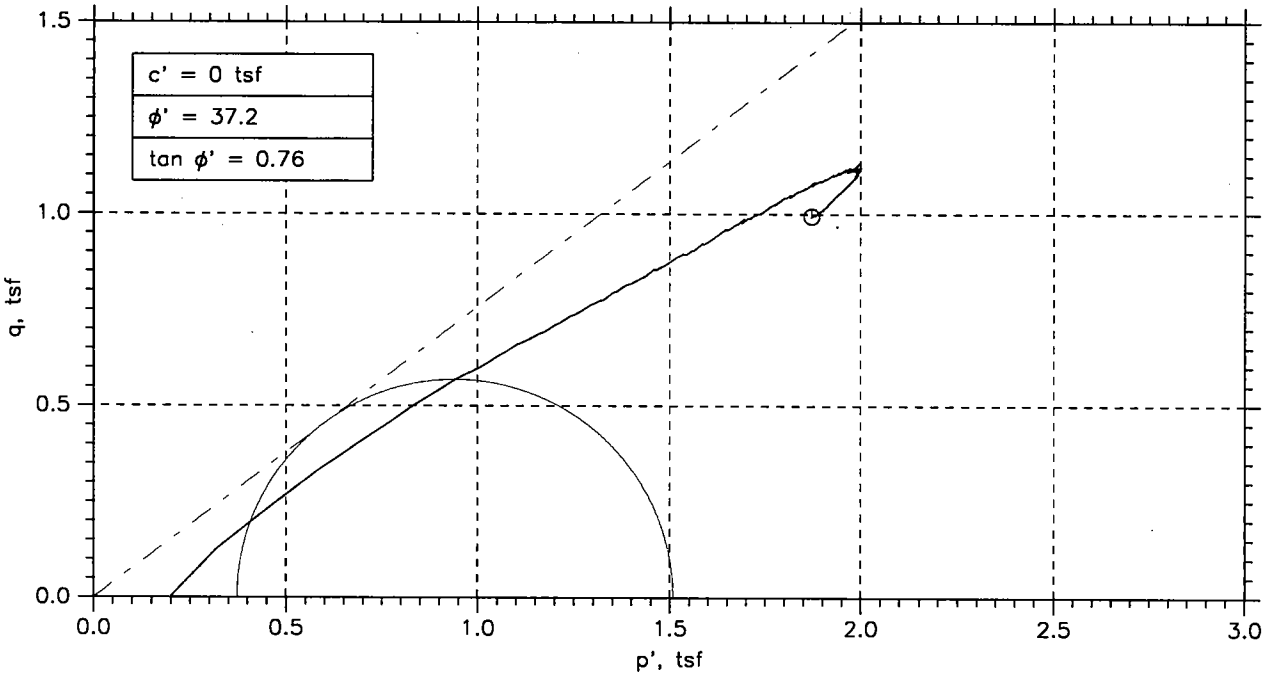


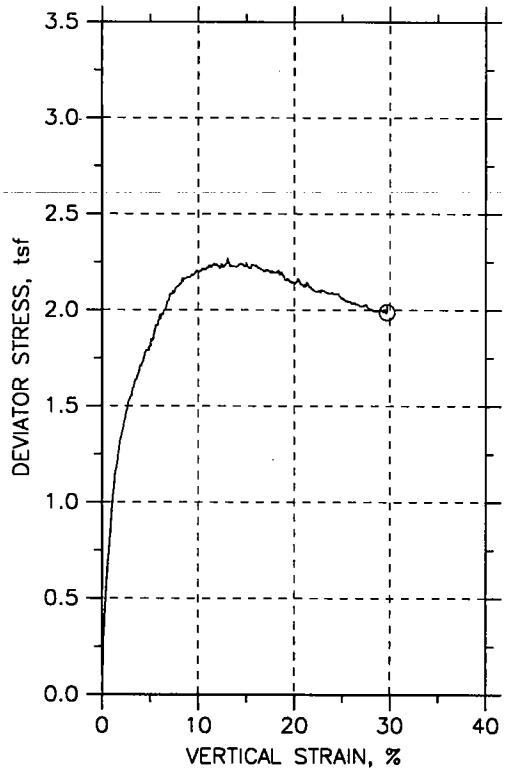


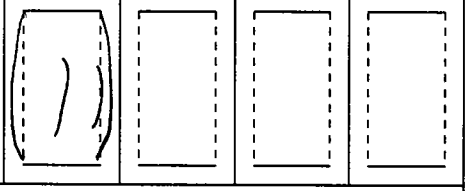
Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: 09-203B S-5	Tested By: BCM	Checked By: WPQ
Sample No.: OST-5	Test Date: 3/20/09	Depth: 29.5'
Test No.: 5742203BS5	Sample Type: 3" OST	Elevation: ----
Description: SILTY CLAY AND F-M SAND - BROWN AND GRAY (CL). sILT AND SAND POCKETS NOTED.		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 1.00		

Tue, 31-MAR-2009 11:47:04

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⊙			
Test No.		5742204BS5			
Initial	Diameter, in	2.8528			
	Height, in	5.9185			
	Water Content, %	21.45			
	Dry Density, pcf	107.			
	Saturation, %	100.77			
	Void Ratio	0.57487			
Before Shear	Water Content, %	21.20			
	Dry Density, pcf	107.2			
	Saturation, %	100.00			
	Void Ratio	0.57237			
	Back Press., tsf	5.0437			
Minor Prin. Stress, tsf		0.19787			
Max. Dev. Stress, tsf		2.2686			
Time to Failure, min		211.68			
Strain Rate, %/min		0.069			
B-Value		---			
Estimated Specific Gravity		2.70			
Liquid Limit		Pending			
Plastic Limit		Pending			
Plasticity Index		Pending			
Failure Sketch					

Project: KINGSTON COAL TVA

Location: TN

Project No.: 60095742

Boring No.: 09-204B S-5

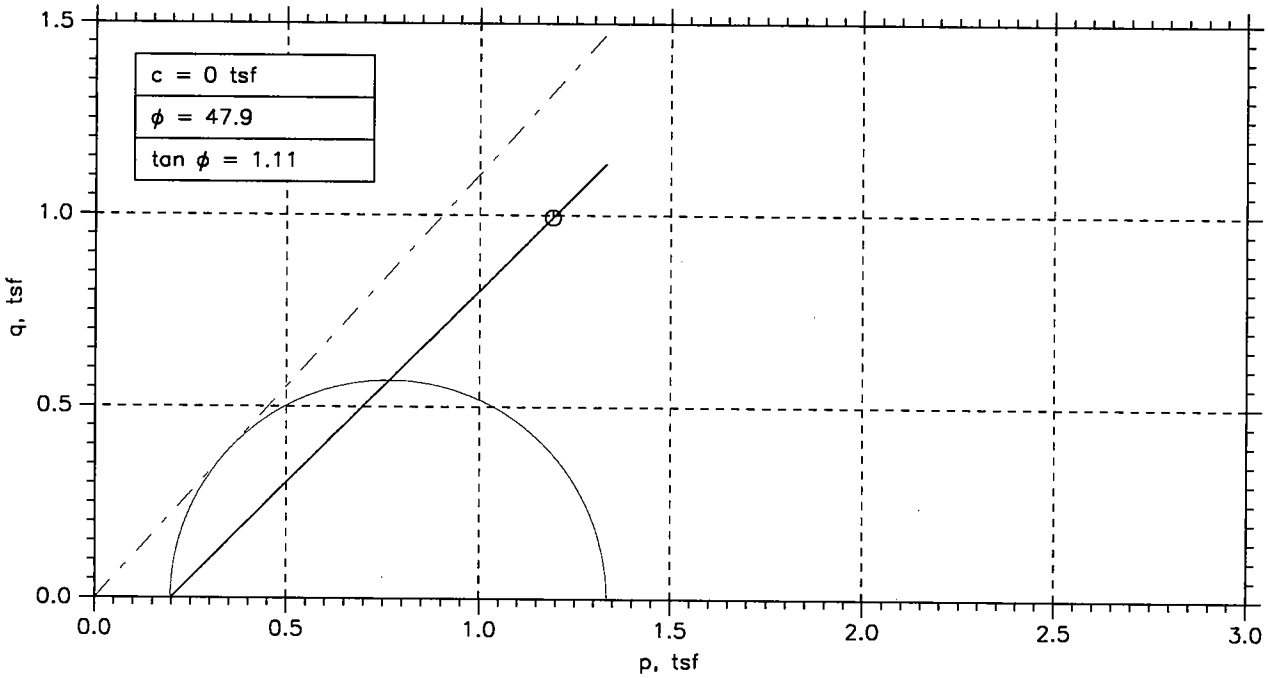
Sample Type: 3" OST

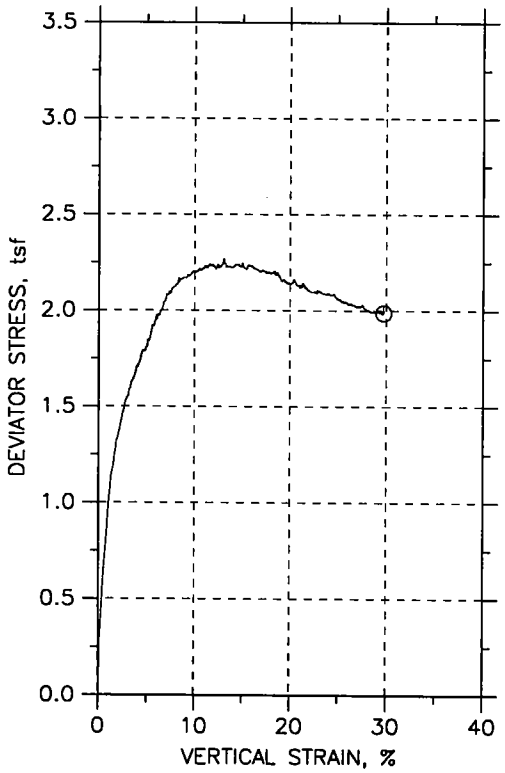
Description: SILTY CLAY AND SAND - BROWN CL SILT AND SAND POCKETS NOTED.

Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = .99

Tue, 31-MAR-2009 16:11:09

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⊙			
Test No.		5742204BS5			
Initial	Diameter, in	2.8528			
	Height, in	5.9185			
	Water Content, %	21.45			
	Dry Density, pcf	107.			
	Saturation, %	100.77			
	Void Ratio	0.57487			
Before Shear	Water Content, %	21.20			
	Dry Density, pcf	107.2			
	Saturation, %	100.00			
	Void Ratio	0.57237			
	Back Press., tsf	5.0437			
Minor Prin. Stress, tsf		0.19787			
Max. Dev. Stress, tsf		2.2686			
Time to Failure, min		211.68			
Strain Rate, %/min		0.069			
B-Value		---			
Estimated Specific Gravity		2.70			
Liquid Limit		0			
Plastic Limit		0			
Plasticity Index		0			
Failure Sketch					

Project: KINGSTON COAL TVA

Location: TN

Project No.: 60095742

Boring No.: 09-204B S-5

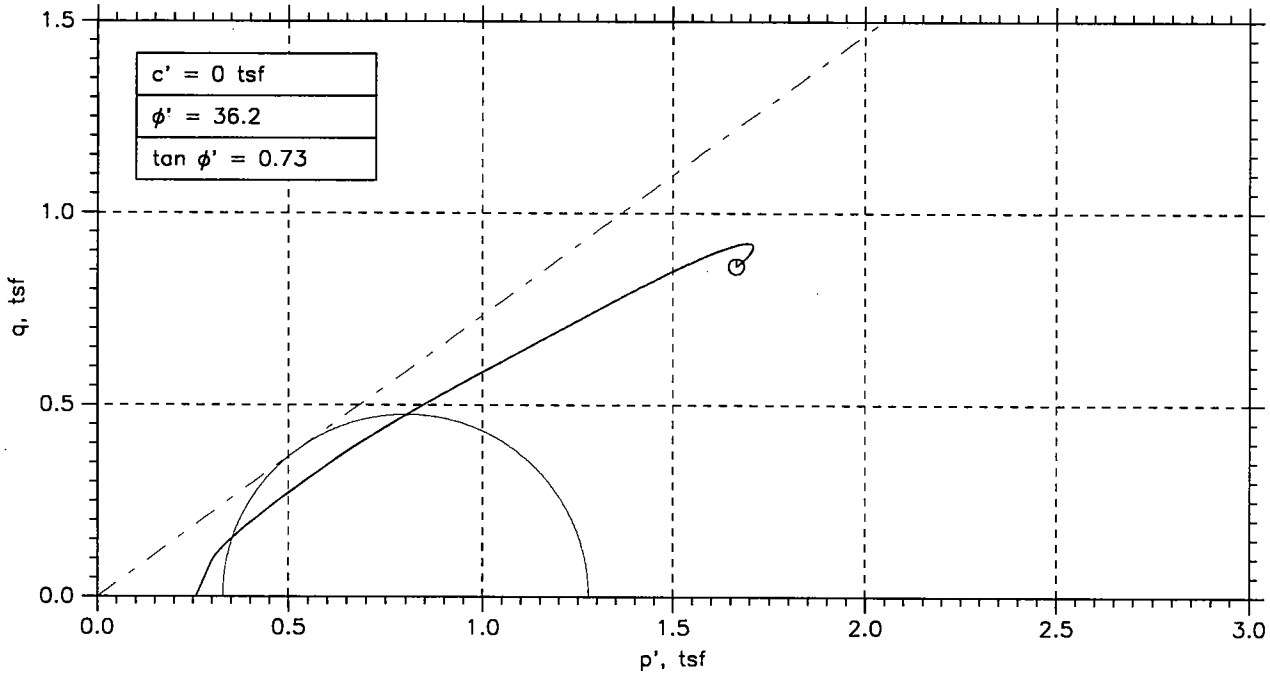
Sample Type: 3" OST

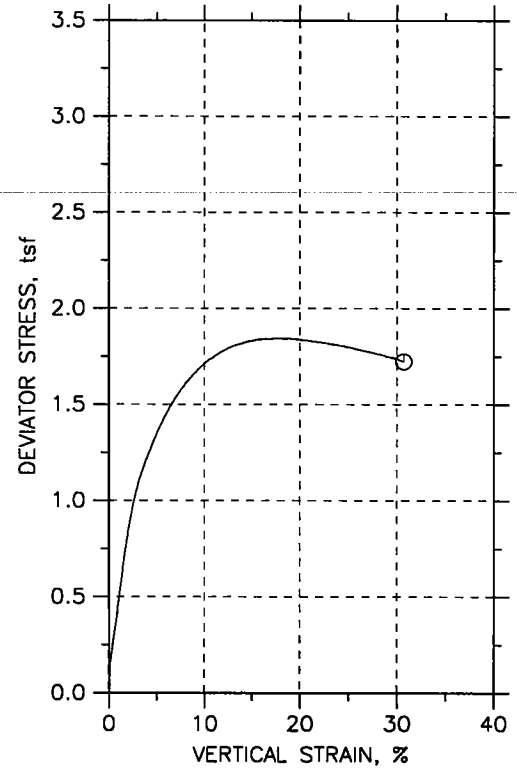
Description: SILTY CLAY AND SAND - BROWN CL SILT AND SAND POCKETS NOTED.

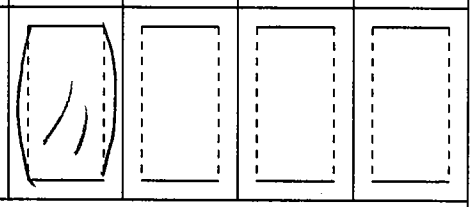
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = .99

Tue, 31-MAR-2009 16:11:36

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⊙			
Test No.		5742204BS6			
Initial	Diameter, in	2.8591			
	Height, in	5.9331			
	Water Content, %	20.35			
	Dry Density, pcf	106.			
	Saturation, %	93.05			
	Void Ratio	0.59036			
Before Shear	Water Content, %	21.83			
	Dry Density, pcf	106.1			
	Saturation, %	100.00			
	Void Ratio	0.58933			
	Back Press., tsf	5.0127			
Minor Prin. Stress, tsf		0.25768			
Max. Dev. Stress, tsf		1.8448			
Time to Failure, min		263.52			
Strain Rate, %/min		0.069			
B-Value		---			
Estimated Specific Gravity		2.70			
Liquid Limit		Pending			
Plastic Limit		Pending			
Plasticity Index		Pending			
Failure Sketch					

Project: KINGSTON COAL TVA

Location: TN

Project No.: 60095742

Boring No.: 09-204B S-6

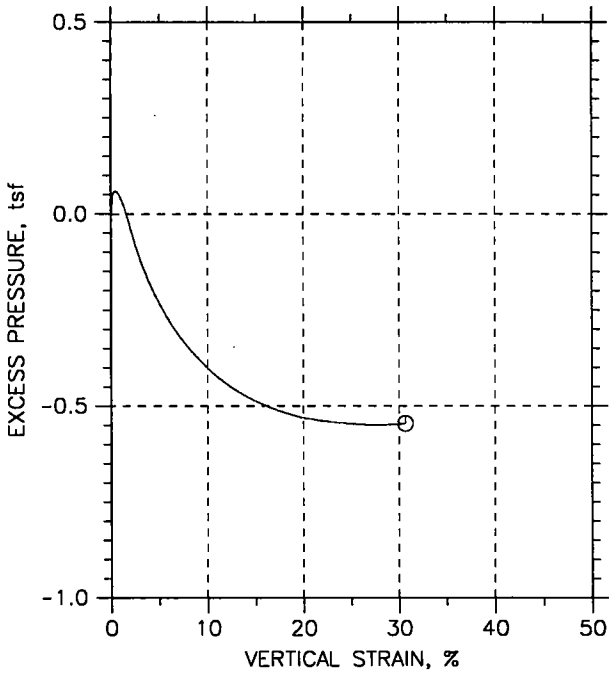
Sample Type: 3" OST

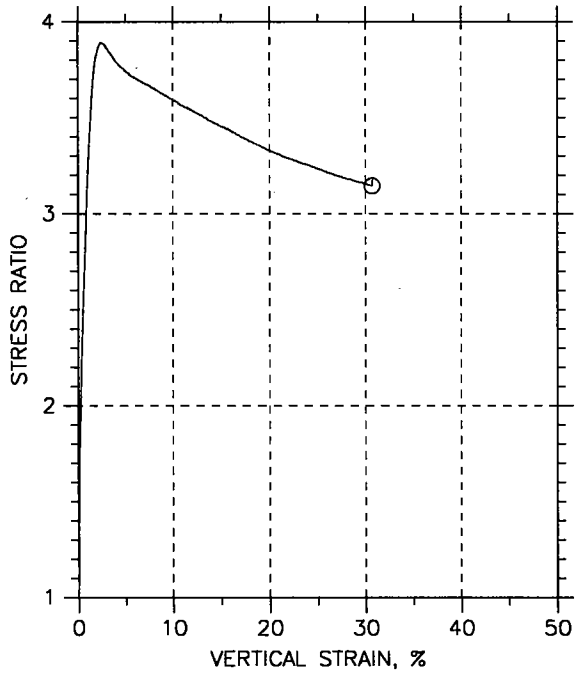
Description: SILTY CLAY AND SAND - BROWN AND GRAY CL SILT AND SAND POCKETS NOTED.

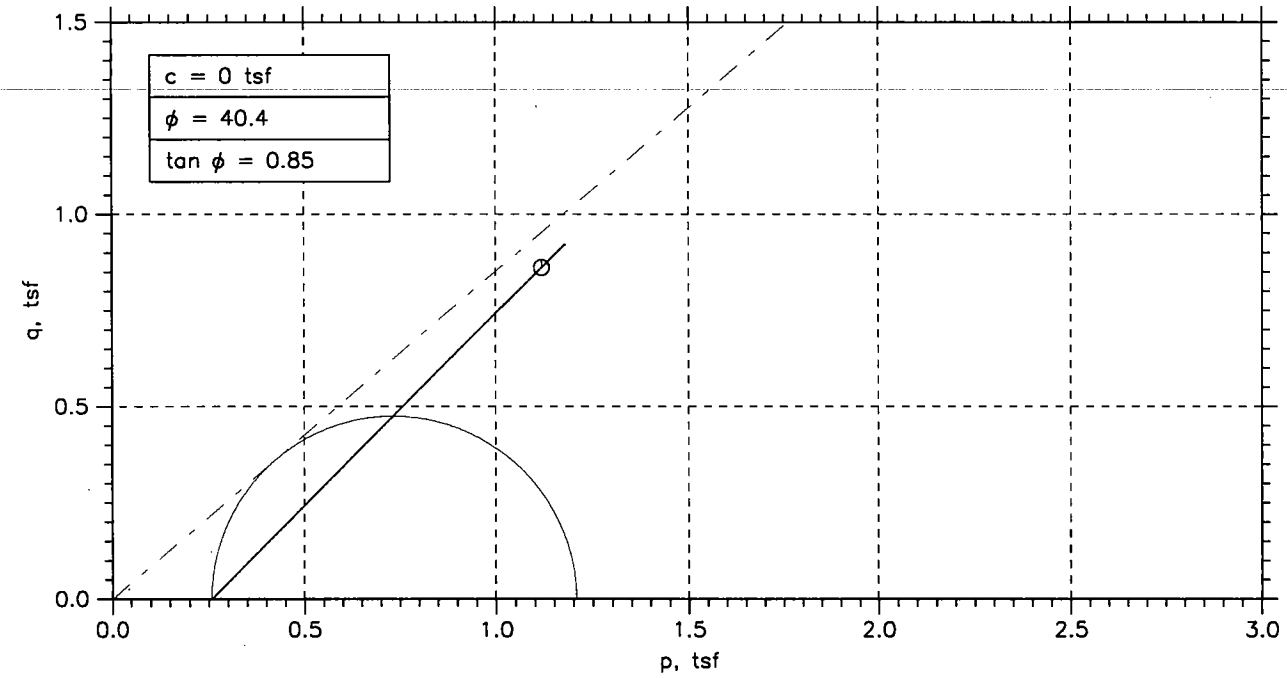
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 0.99

Tue, 31-MAR-2009 15:47:30

TRIAXIAL COMPRESSION TEST REPORT



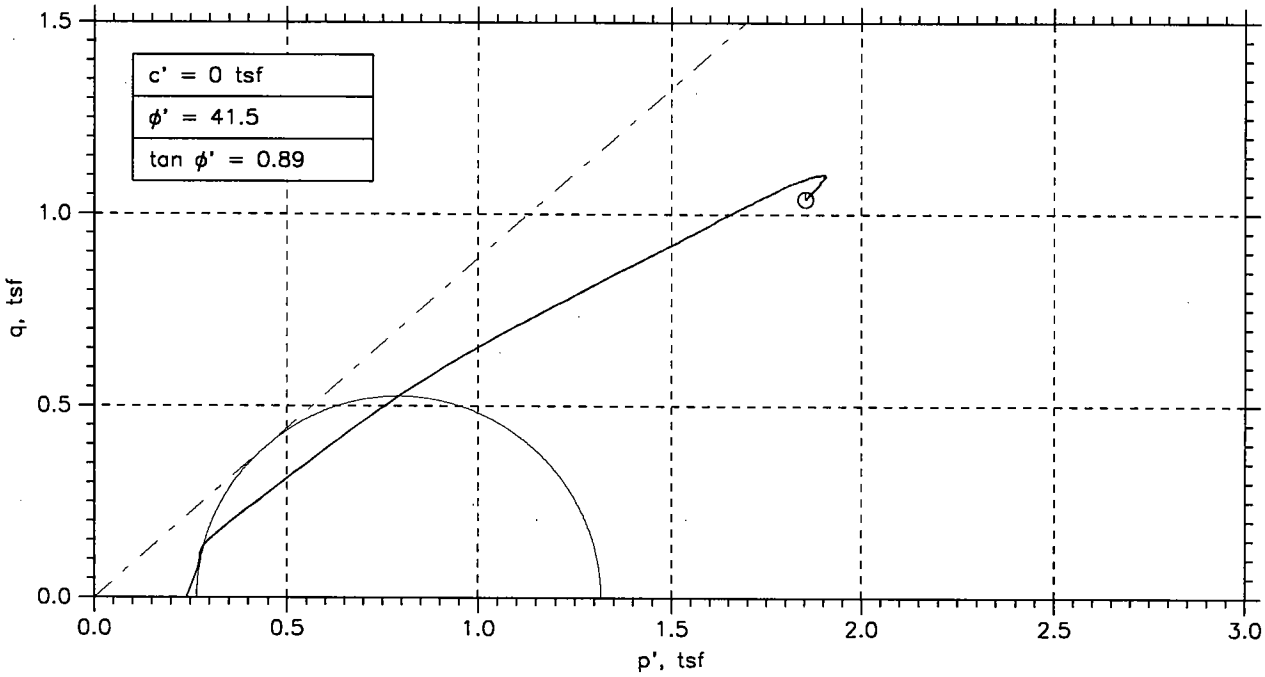


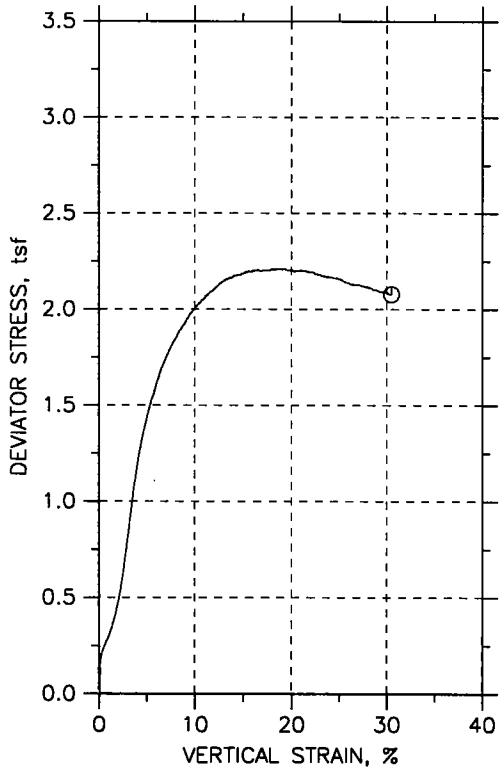


Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: 09-204B S-6	Tested By: BCM	Checked By: WPQ
Sample No.: OST-6	Test Date: 3/26/09	Depth: 18.0'-20.5'
Test No.: 5742204BS6	Sample Type: 3" OST	Elevation: -----
Description: SILTY CLAY AND SAND - BROWN AND GRAY CL SILT AND SAND POCKETS NOTED.		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 0.99		

Tue, 31-MAR-2009 15:48:31

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⏲			
Test No.		5742205BS3			
Initial	Diameter, in	2.7976			
	Height, in	5.9642			
	Water Content, %	19.51			
	Dry Density, pcf	108.6			
	Saturation, %	95.54			
	Void Ratio	0.5514			
Before Shear	Water Content, %	20.40			
	Dry Density, pcf	108.7			
	Saturation, %	100.00			
	Void Ratio	0.55079			
	Back Press., tsf	5.0377			
Minor Prin. Stress, tsf		0.23992			
Max. Dev. Stress, tsf		2.2099			
Time to Failure, min		280.8			
Strain Rate, %/min		0.069			
B-Value		---			
Estimated Specific Gravity		2.70			
Liquid Limit		Pending			
Plastic Limit		Pending			
Plasticity Index		Pending			
Failure Sketch					

Project: KINGSTON COAL TVA

Location: TN

Project No.: 60095742

Boring No.: 09-205B S-3

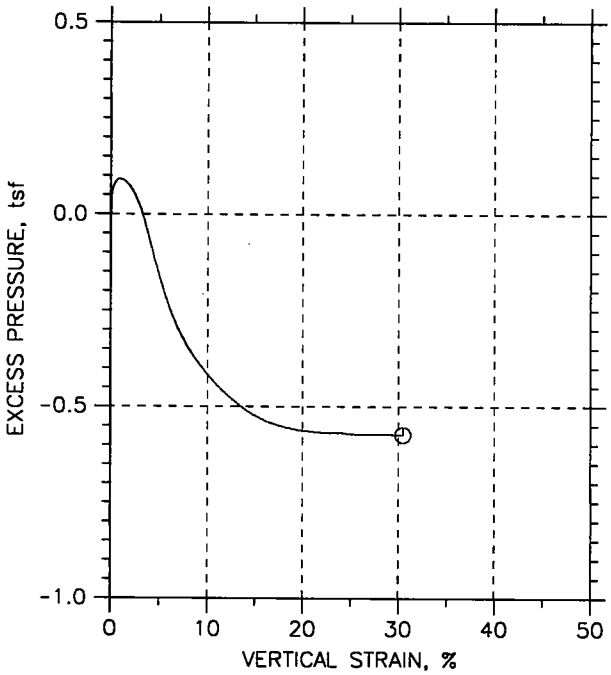
Sample Type: 3" OST

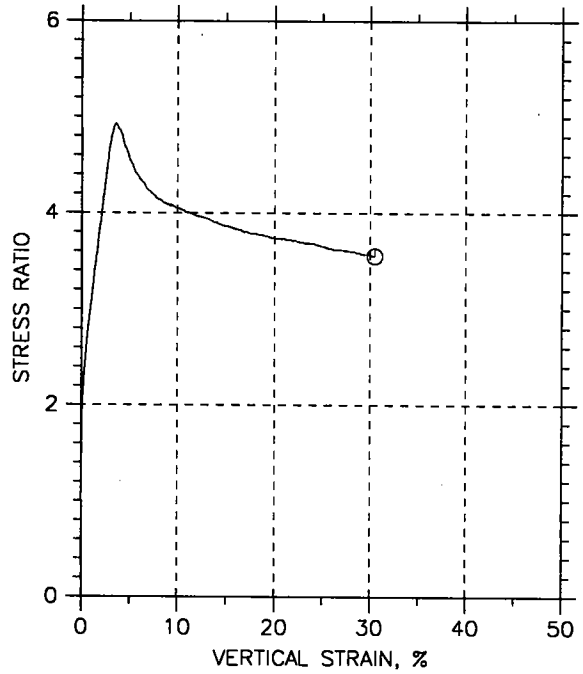
Description: SILTY CLAY LITTLE F-C SAND - BROWN AND GRAY CL

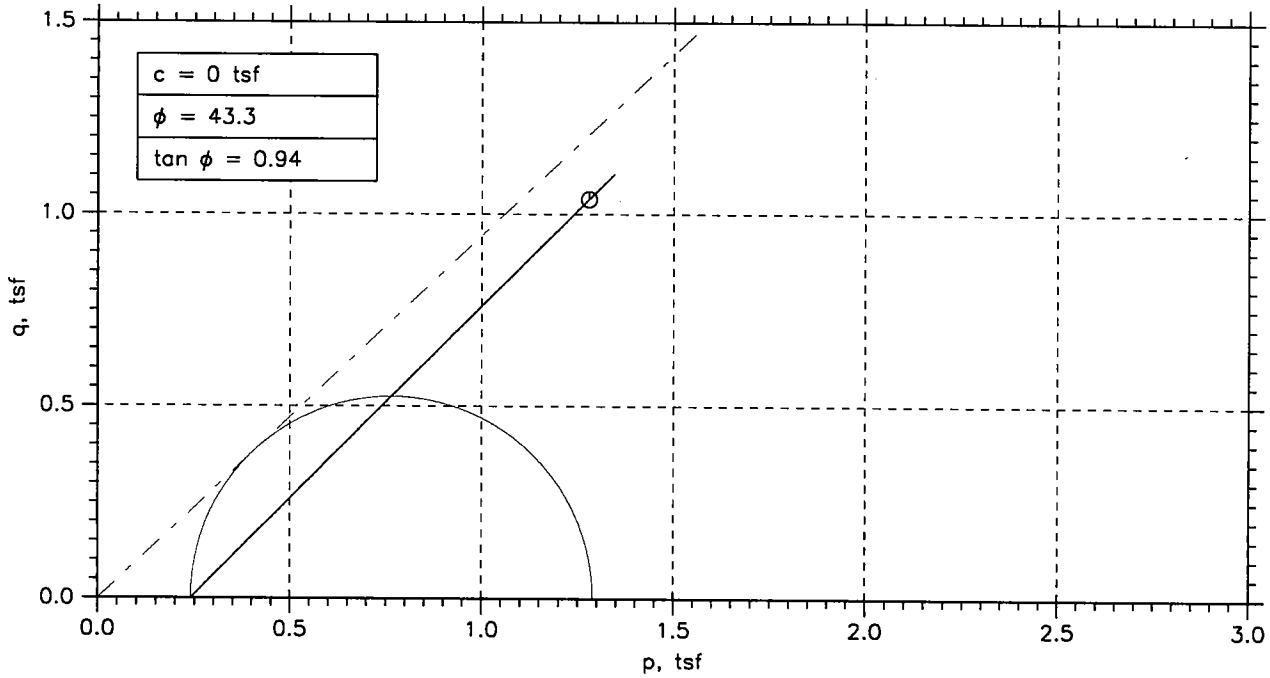
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 0.99

Tue, 31-MAR-2009 15:40:22

TRIAXIAL COMPRESSION TEST REPORT



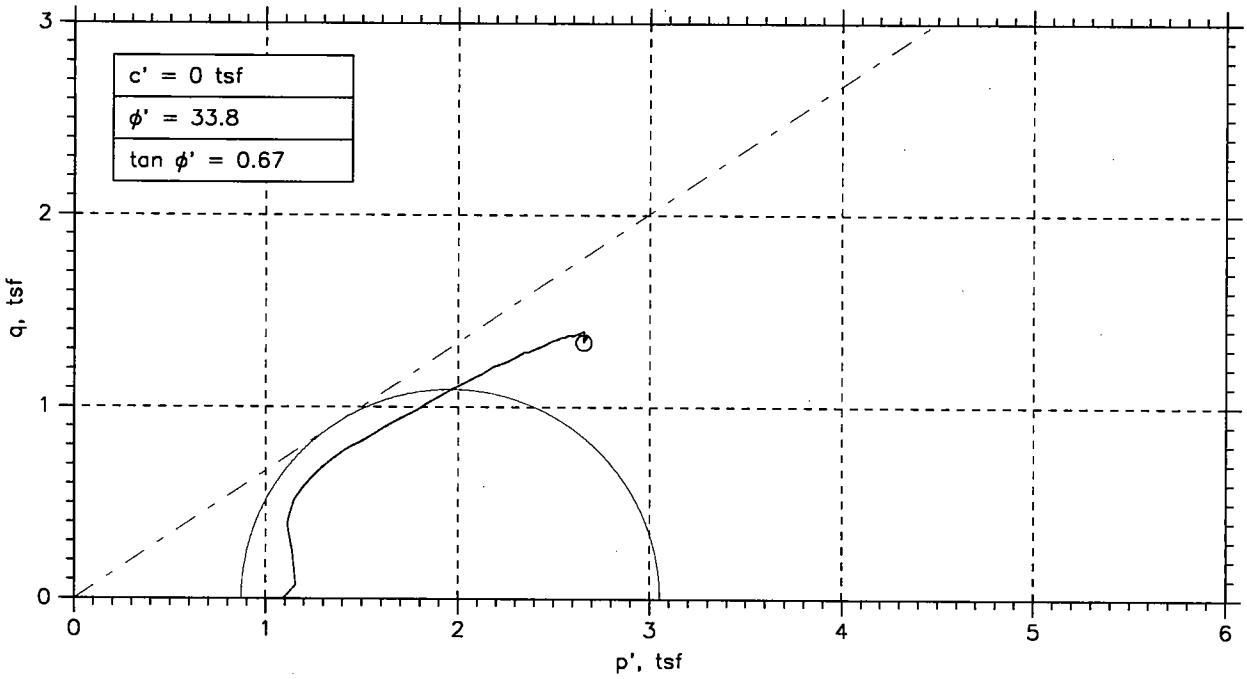


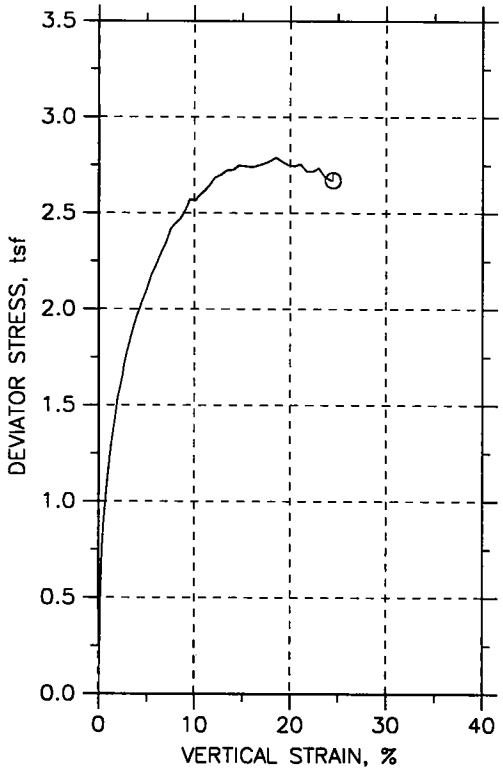


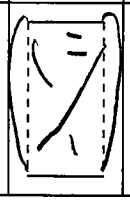
Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: 09-205B S-3	Tested By: BCM	Checked By: WPQ
Sample No.: OST-3	Test Date: 3/25/09	Depth: 18.5'-21.0'
Test No.: 5742205BS3	Sample Type: 3" OST	Elevation: ----
Description: SILTY CLAY LITTLE F-C SAND - BROWN AND GRAY CL		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 0.99		

Tue, 31-MAR-2009 15:49:52

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⊙			
Test No.		5742303BS10			
Initial	Diameter, in	2.8594			
	Height, in	6.1457			
	Water Content, %	21.31			
	Dry Density, pcf	106.2			
	Saturation, %	98.07			
	Void Ratio	0.58668			
Before Shear	Water Content, %	21.21			
	Dry Density, pcf	107.2			
	Saturation, %	100.00			
	Void Ratio	0.57263			
	Back Press., tsf	5.0399			
Minor Prin. Stress, tsf		1.0873			
Max. Dev. Stress, tsf		2.7884			
Time to Failure, min		304			
Strain Rate, %/min		0.064			
B-Value		---			
Estimated Specific Gravity		2.70			
Liquid Limit		Pending			
Plastic Limit		Pending			
Plasticity Index		Pending			
Failure Sketch					

Project: KINGSTON COAL TVA

Location: TN

Project No.: 60095742

Boring No.: 09-303B S10

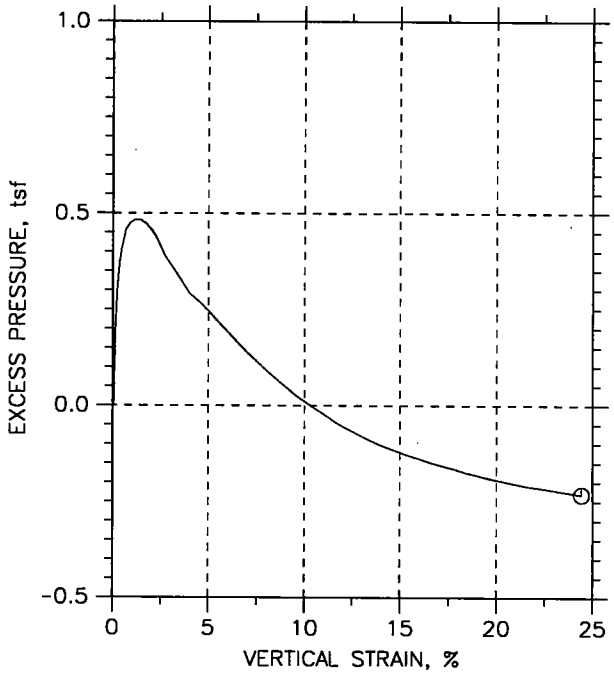
Sample Type: 3" OST

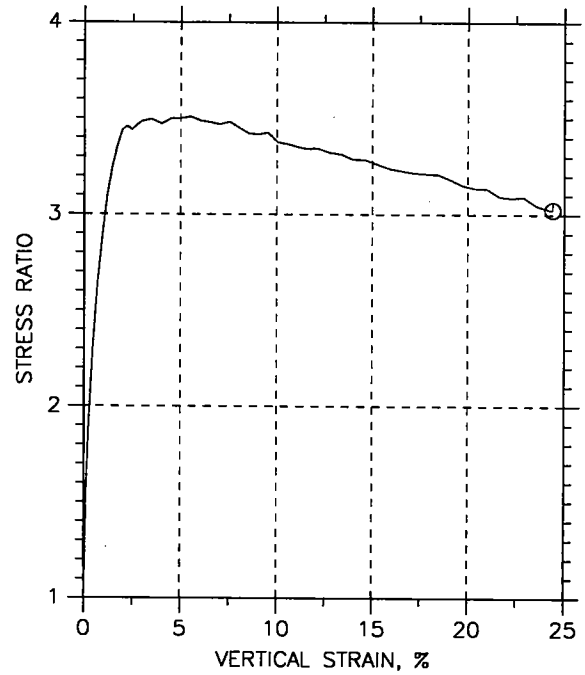
Description: SILTY CLAY SOME F-C SAND (SC) - GRAY AND BROWN. SILT POCKETS NOTED. VERTICAL SAND SEAM NOTED.

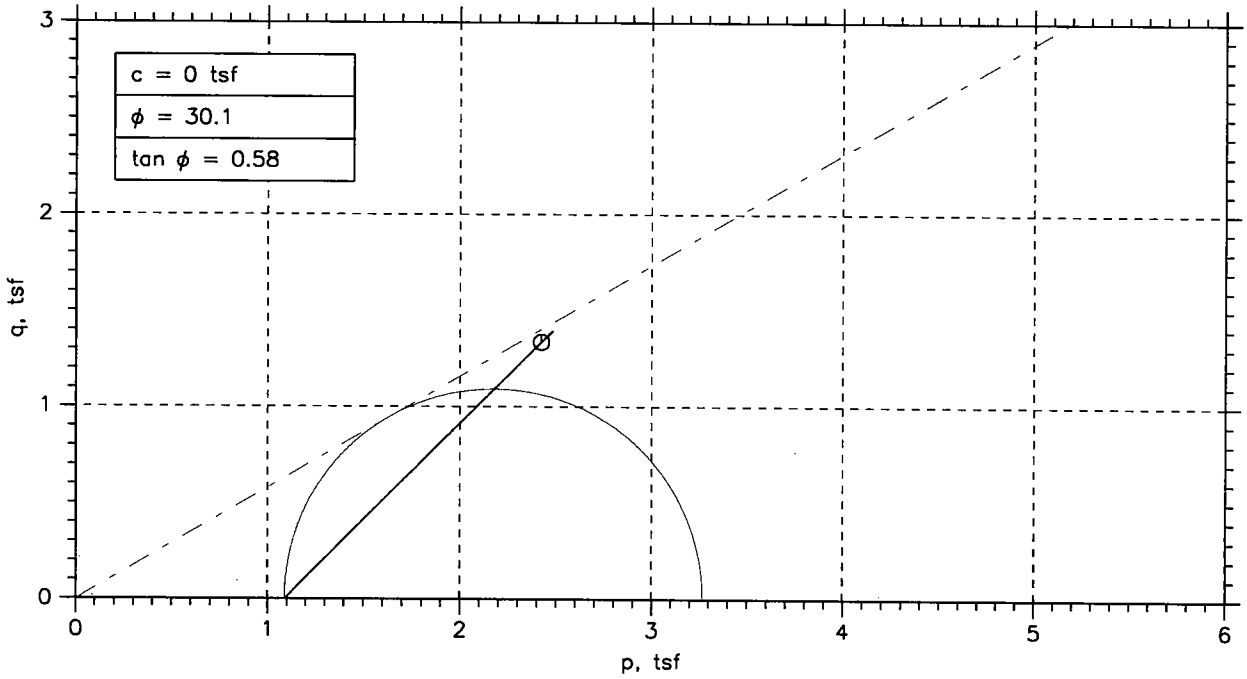
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B Value = 0.95

Tue, 31-MAR-2009 10:37:33

TRIAXIAL COMPRESSION TEST REPORT



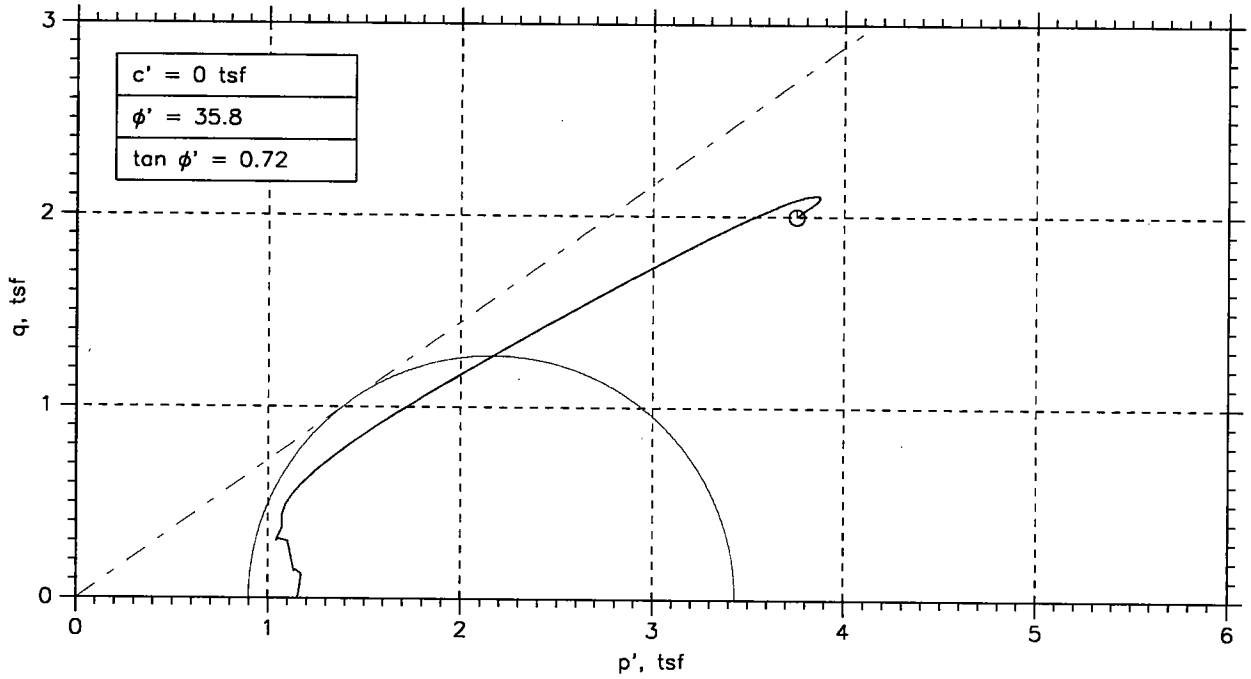


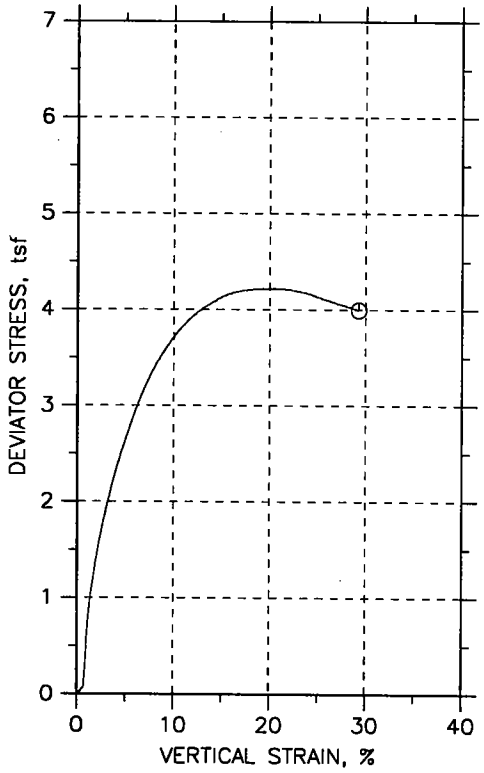


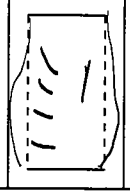
Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: 09-303B S10	Tested By: BCN/JDW	Checked By: WPQ
Sample No.: S-10	Test Date: 2/23/2009	Depth: 90.0'-92.5'
Test No.: 5742303BS10	Sample Type: 3" OST	Elevation:
Description: SILTY CLAY SOME F-C SAND (SC) - GRAY AND BROWN. SILT POCKETS NOTED. VERTICAL SAND SEAM NOTED.		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B Value = 0.95		

Tue, 31-MAR-2009 10:37:56

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⏲			
Test No.		5742303BS12			
Initial	Diameter, in	2.9008			
	Height, in	6.0181			
	Water Content, %	28.91			
	Dry Density, pcf	100.6			
	Saturation, %	115.53			
	Void Ratio	0.67563			
Before Shear	Water Content, %	24.29			
	Dry Density, pcf	101.8			
	Saturation, %	100.00			
	Void Ratio	0.65585			
	Back Press., tsf	5.0399			
Minor Prin. Stress, tsf		1.1449			
Max. Dev. Stress, tsf		4.2176			
Time to Failure, min		735			
Strain Rate, %/min		0			
B-Value		---			
Estimated Specific Gravity		2.70			
Liquid Limit		Pending			
Plastic Limit		Pending			
Plasticity Index		Pending			
Failure Sketch					

Project: KINGSTON COAL TVA

Location: TN

Project No.: 60095742

Boring No.: 09-303B S12

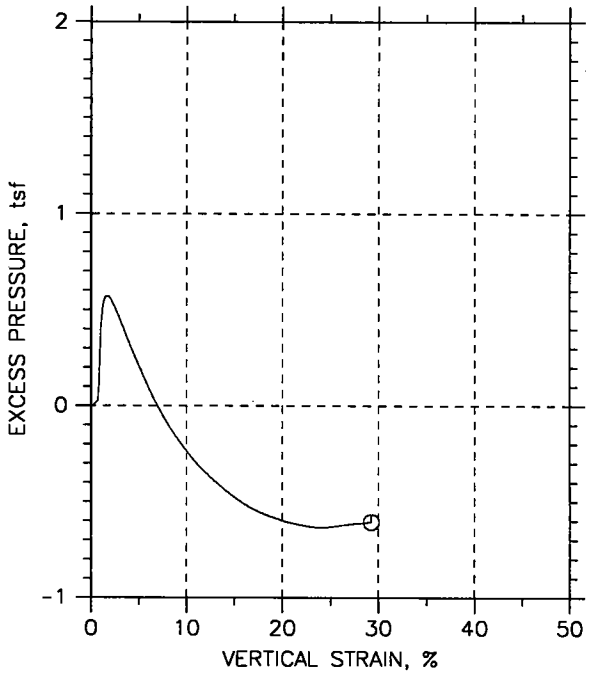
Sample Type: 3" OST

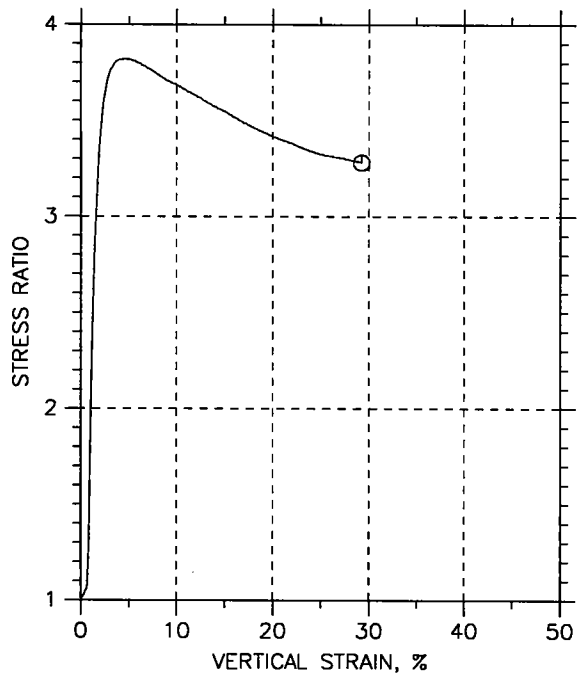
Description: SILTY CLAY SOME F-C SAND (SC) - BROWN AND GRAY. SILT AND SAND POCKETS NOTED.

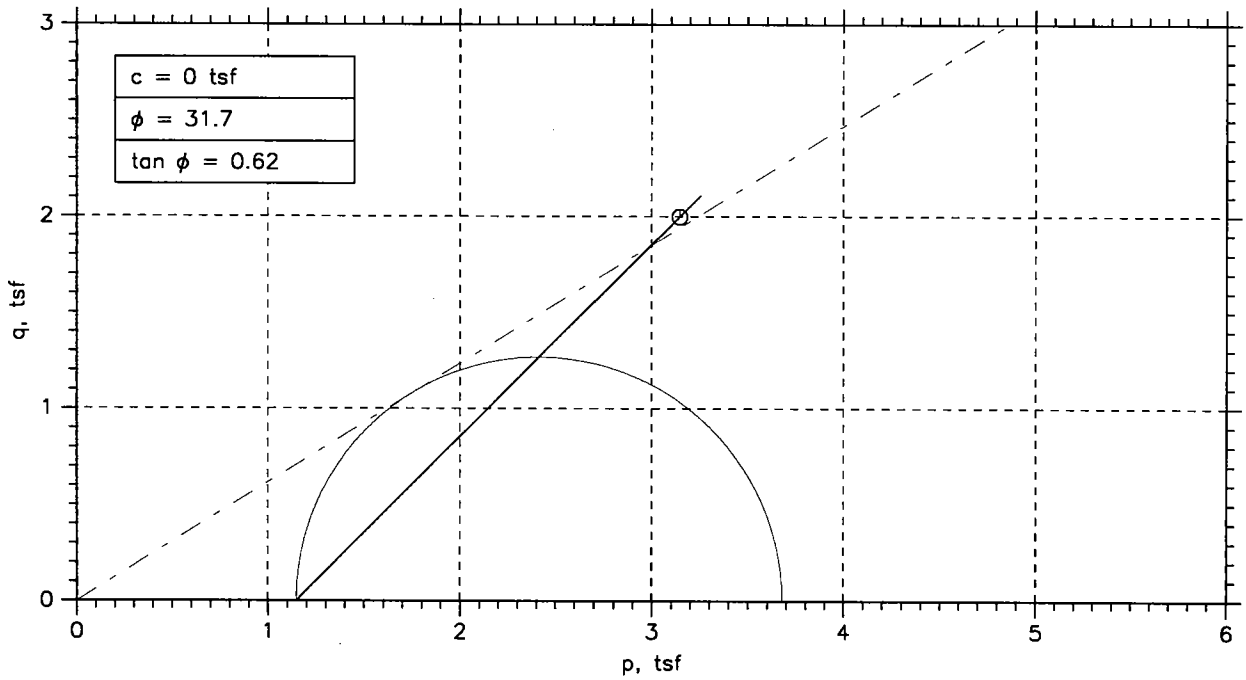
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B Value = 0.98

Tue, 31-MAR-2009 10:53:57

TRIAXIAL COMPRESSION TEST REPORT



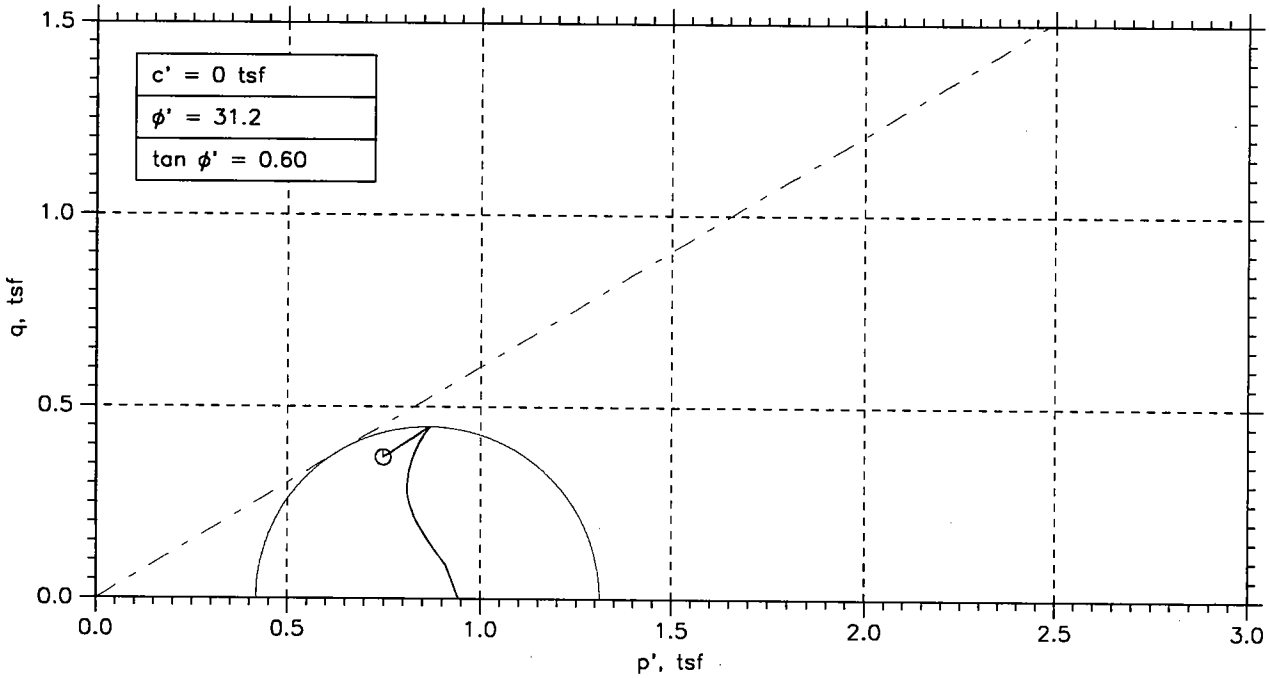


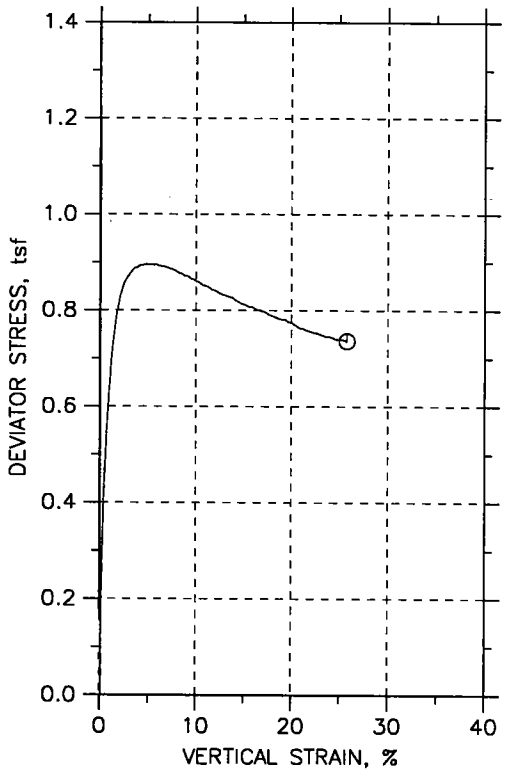


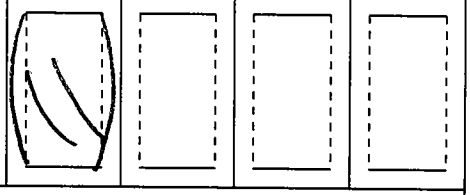
Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: 09-303B S12	Tested By: BCM	Checked By: WPQ
Sample No.: S-12	Test Date: 2/23/09	Depth: 95.0'-97.5'
Test No.: 5742303BS12	Sample Type: 3" OST	Elevation: -
Description: SILTY CLAY SOME F-C SAND (SC) - BROWN AND GRAY. SILT AND SAND POCKETS NOTED.		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B Value = 0.98		

Tue, 31-MAR-2009 10:54:36

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⊙			
Test No.		5742402BS5			
Initial	Diameter, in	2.8252			
	Height, in	5.6949			
	Water Content, %	29.51			
	Dry Density, pcf	94.16			
	Saturation, %	100.85			
	Void Ratio	0.79008			
Before Shear	Water Content, %	27.89			
	Dry Density, pcf	96.15			
	Saturation, %	100.00			
	Void Ratio	0.75303			
	Back Press., tsf	5.0402			
Minor Prin. Stress, tsf		0.94302			
Max. Dev. Stress, tsf		0.89571			
Time to Failure, min		156			
Strain Rate, %/min		0.031			
B-Value		---			
Estimated Specific Gravity		2.70			
Liquid Limit		Pending			
Plastic Limit		Pending			
Plasticity Index		Pending			
Failure Sketch					

Project: KINGSTON COAL TVA

Location: TN

Project No.: 60095742

Boring No.: 09-402B S-5

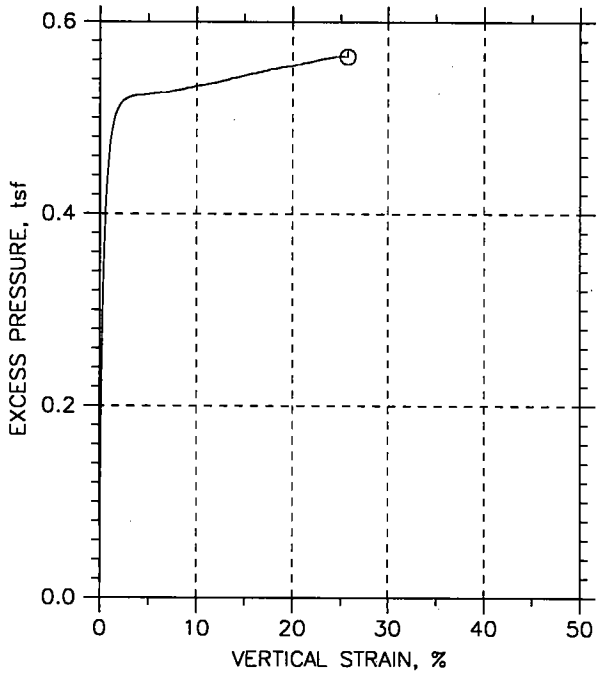
Sample Type: 3" OST

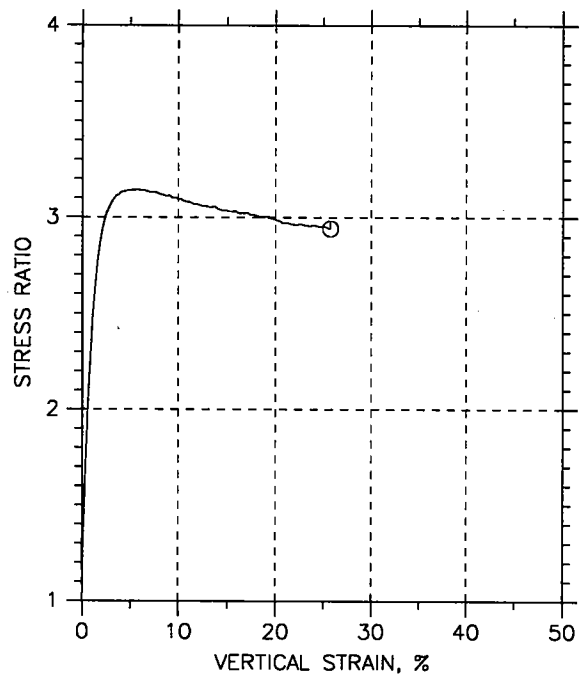
Description: SILTY CLAY TRACE F-C SAND TRACE F GRAVEL - GRAY CL

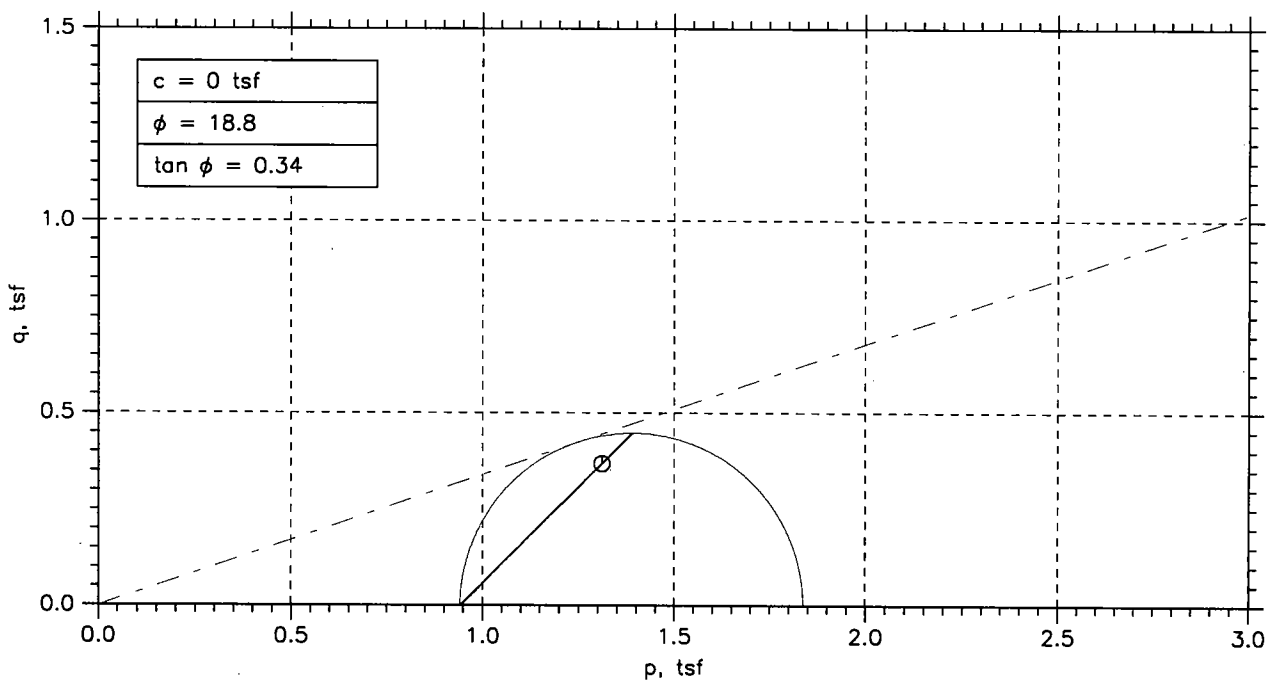
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 1.0

Tue, 31-MAR-2009 13:19:48

TRIAXIAL COMPRESSION TEST REPORT



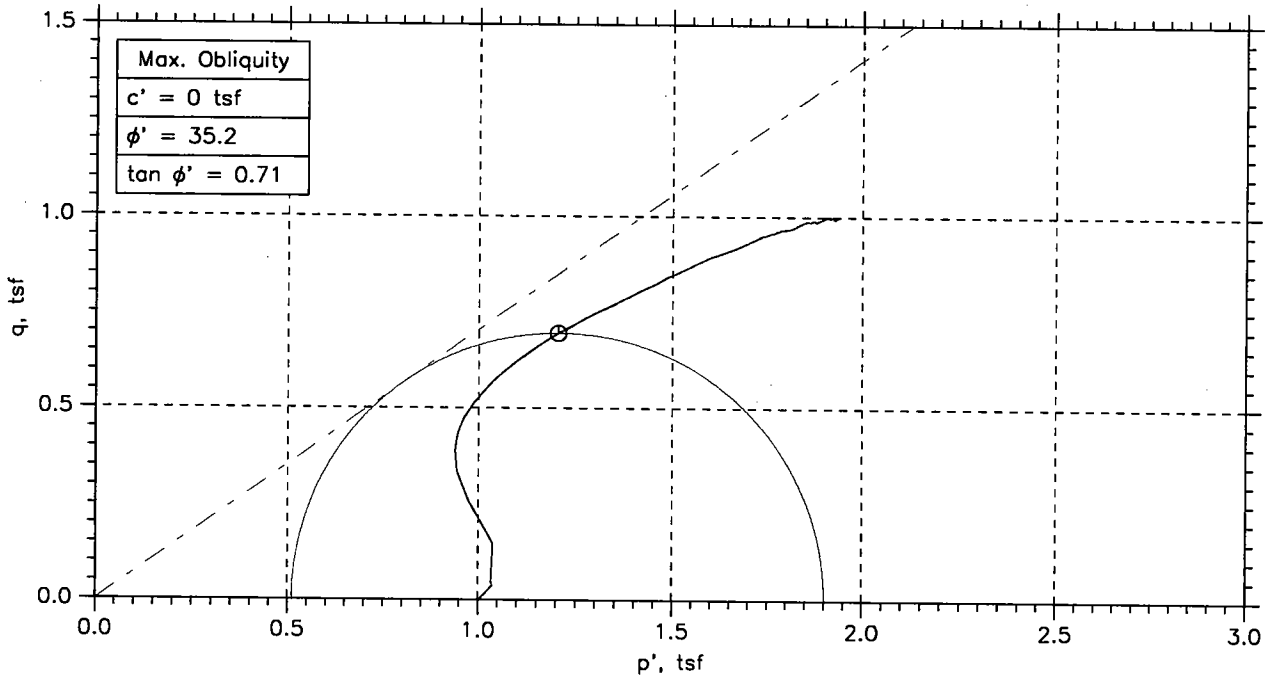


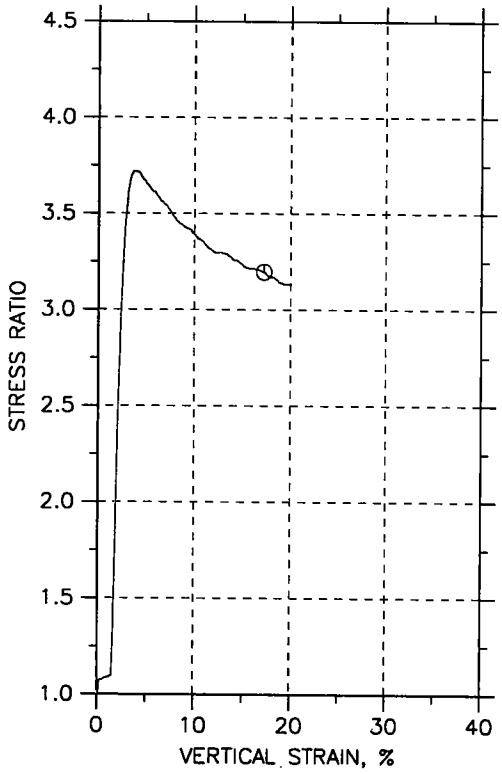


Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: 09-402B S-5	Tested By: BCM	Checked By: WPQ
Sample No.: OST-5	Test Date: 2/26/09	Depth: 38.5'-41.0'
Test No.: 5742402BS5	Sample Type: 3" OST	Elevation: -
Description: SILTY CLAY TRACE F-C SAND TRACE F GRAVEL - GRAY CL		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 1.0		

Tue, 31-MAR-2009 13:20:10

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767





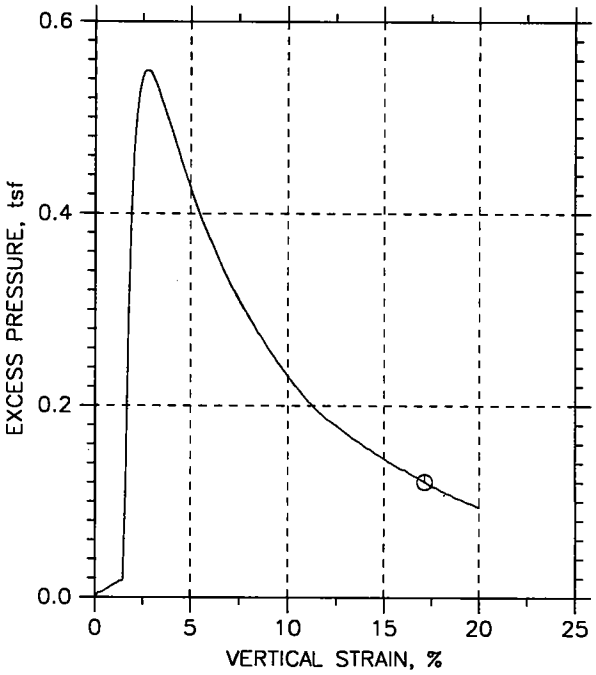
Symbol		⊙			
Sample No.		OST-6			
Test No.		574402BS6			
Depth		41.0'-43.0'			
Initial	Diameter, in	2.871			
	Height, in	5.441			
	Water Content, %	10.6			
	Dry Density, pcf	115.4			
	Saturation, %	62.1			
	Void Ratio	0.461			
Before Shear	Water Content, %	15.9			
	Dry Density, pcf	118.			
	Saturation*, %	100.0			
	Void Ratio	0.429			
	Back Press., tsf	7.344			
Ver. Eff. Cons. Stress, tsf		1.001			
Shear Strength, tsf		1.			
Strain at Failure, %		17.1			
Strain Rate, %/min		0.08			
B-Value		0.95			
Estimated Specific Gravity		2.7			
Liquid Limit		Pending			
Plastic Limit		Pending			

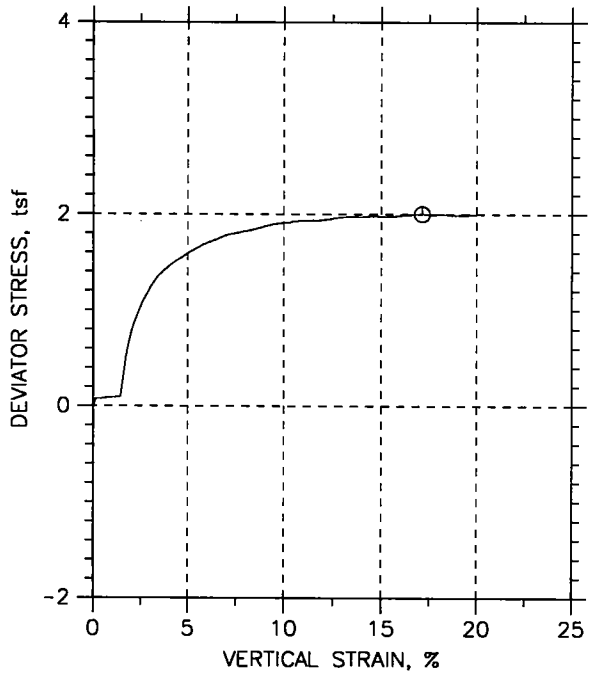
Project: KINGSTON COAL
Location: TN
Project No.: 60095742
Boring No.: 09-402B S-6
Sample Type: 3" OST
Description: SILTY CLAY AND F-M SAND - BROWN AND GRAY
Remarks: TEST PERFORMED AS PER ASTM D 4767

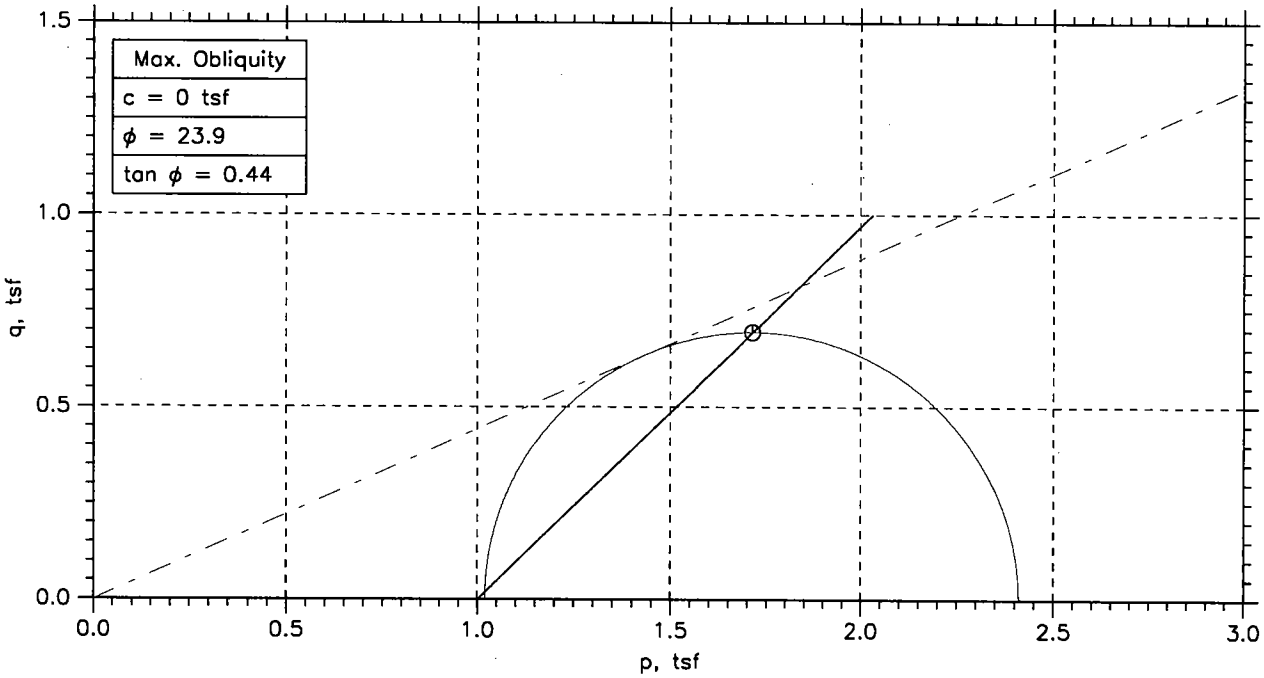
Tue, 31-MAR-2009 14:19:59

Phase calculations based on start and end of test.
* Saturation is set to 100% for phase calculations.

CONSOLIDATED UNDRAINED TRIAXIAL TEST by ASTM D4767



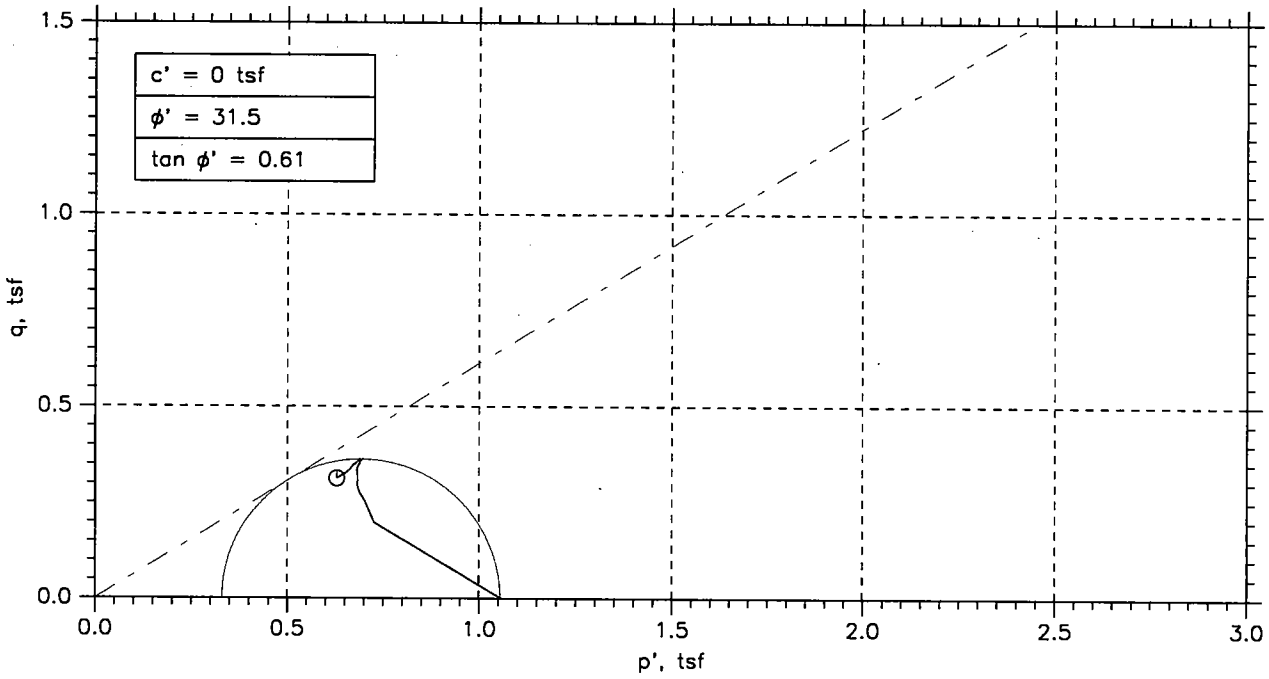


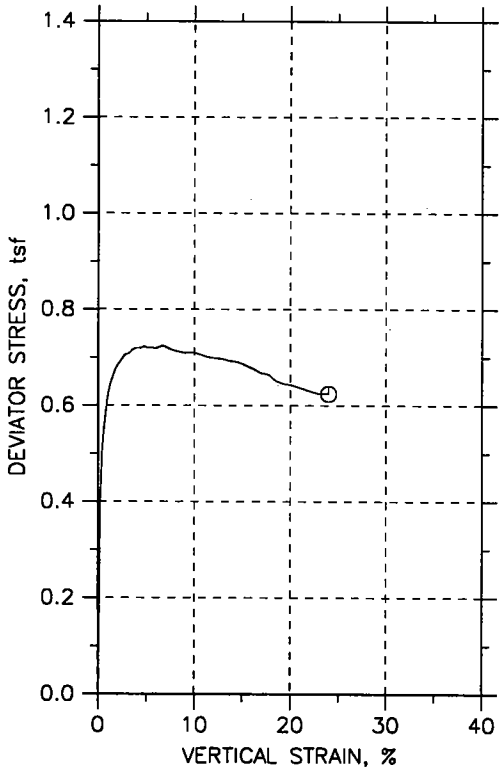


Project: KINGSTON COAL	Location: TN	Project No.: 60095742
Boring No.: 09-402B S-6	Tested By: BCM	Checked By: WPQ
Sample No.: OST-6	Test Date: 2/20/09	Depth: 41.0'-43.0'
Test No.: 574402BS6	Sample Type: 3" OST	Elevation: ----
Description: SILTY CLAY AND F-M SAND - BROWN AND GRAY		
Remarks: TEST PERFORMED AS PER ASTM D 4767		

Tue, 31-MAR-2009 14:24:19

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⦶			
Test No.		5742402BS7			
Initial	Diameter, in	2.8059			
	Height, in	5.926			
	Water Content, %	22.67			
	Dry Density, pcf	109.			
	Saturation, %	112.02			
	Void Ratio	0.54644			
Before Shear	Water Content, %	19.99			
	Dry Density, pcf	109.5			
	Saturation, %	100.00			
	Void Ratio	0.53979			
	Back Press., tsf	5.0399			
Minor Prin. Stress, tsf		1.0585			
Max. Dev. Stress, tsf		0.72407			
Time to Failure, min		68.004			
Strain Rate, %/min		0.15			
B-Value		---			
Estimated Specific Gravity		2.70			
Liquid Limit		Pending			
Plastic Limit		Pending			
Plasticity Index		Pending			
Failure Sketch					

Project: KINGSTON COAL TVA

Location: TN

Project No.: 60095742

Boring No.: 09-402B S7

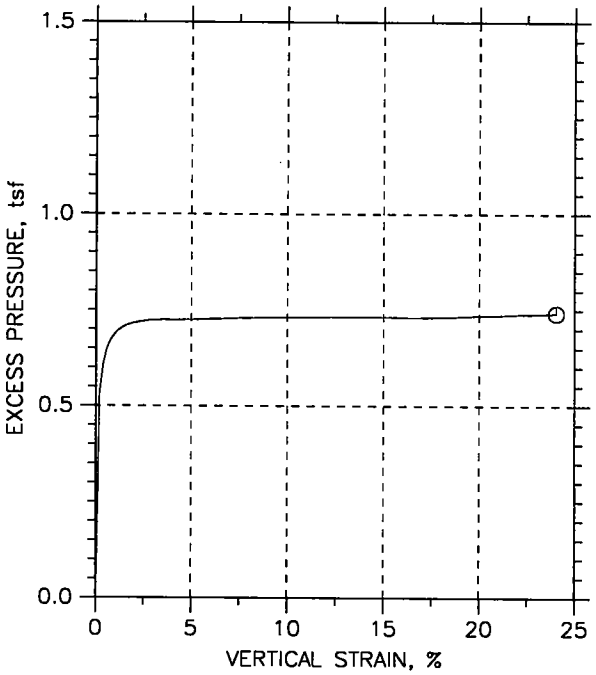
Sample Type: 3"OST

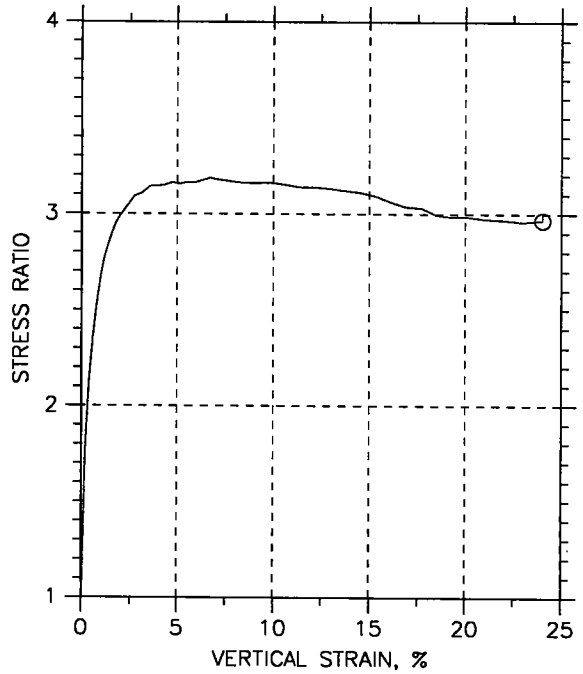
Description: SILTY CLAY SOME F-C SAND - BROWNISH GRAY CL SILT AND SAND POCKETS NOTED.

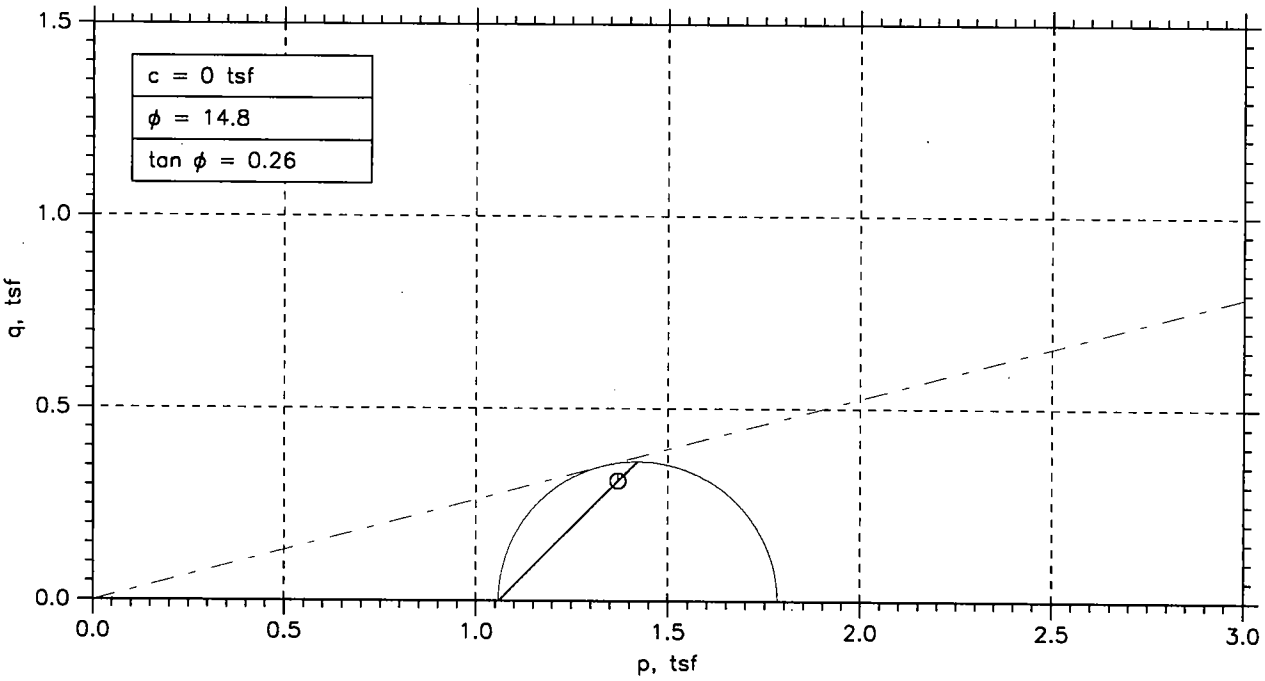
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 0.96

Tue, 31-MAR-2009 13:28:00

TRIAXIAL COMPRESSION TEST REPORT



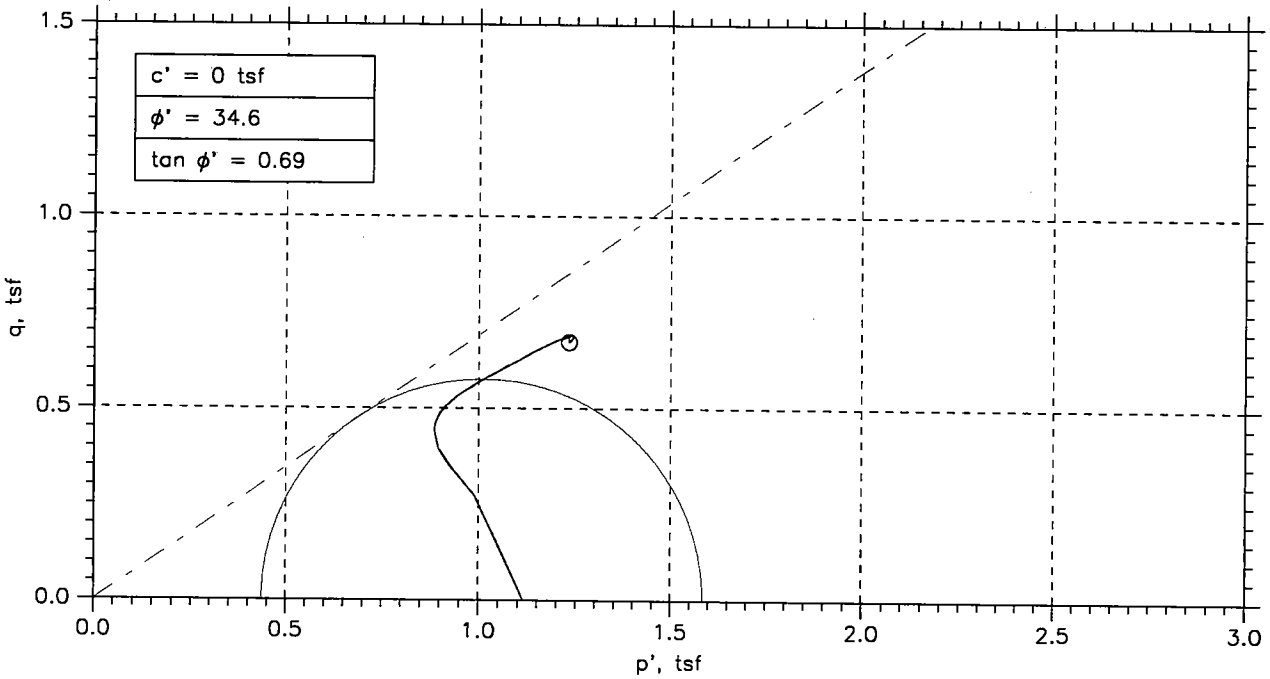


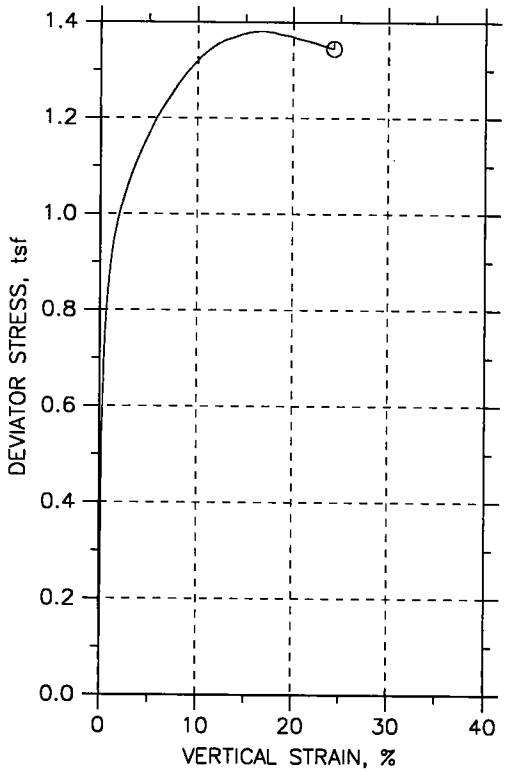


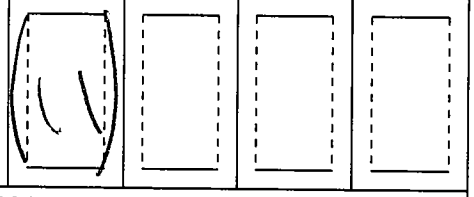
Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: 09-402B S7	Tested By: BCM	Checked By: WPQ
Sample No.: OST-7	Test Date: 3/11/09	Depth: 43.5'-46.0'
Test No.: 5742402BS7	Sample Type: 3"OST	Elevation: ----
Description: SILTY CLAY SOME F-C SAND - BROWNISH GRAY CL SILT AND SAND POCKETS NOTED.		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 0.96		

Tue, 31-MAR-2009 13:28:26

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⊙			
Test No.		5742402BS8			
Initial	Diameter, in	2.8209			
	Height, in	5.6189			
	Water Content, %	23.52			
	Dry Density, pcf	107.			
	Saturation, %	108.94			
	Void Ratio	0.58729			
Before Shear	Water Content, %	20.58			
	Dry Density, pcf	108.9			
	Saturation, %	100.00			
	Void Ratio	0.55967			
	Back Press., tsf	5.0402			
Minor Prin. Stress, tsf		1.1158			
Max. Dev. Stress, tsf		1.3803			
Time to Failure, min		164			
Strain Rate, %/min		0.15			
B-Value		---			
Estimated Specific Gravity		2.72			
Liquid Limit		Pending			
Plastic Limit		Pending			
Plasticity Index		Pending			
Failure Sketch					

Project: KINGSTON COAL TVA

Location: TN

Project No.: 60095742

Boring No.: 09-402B S-8

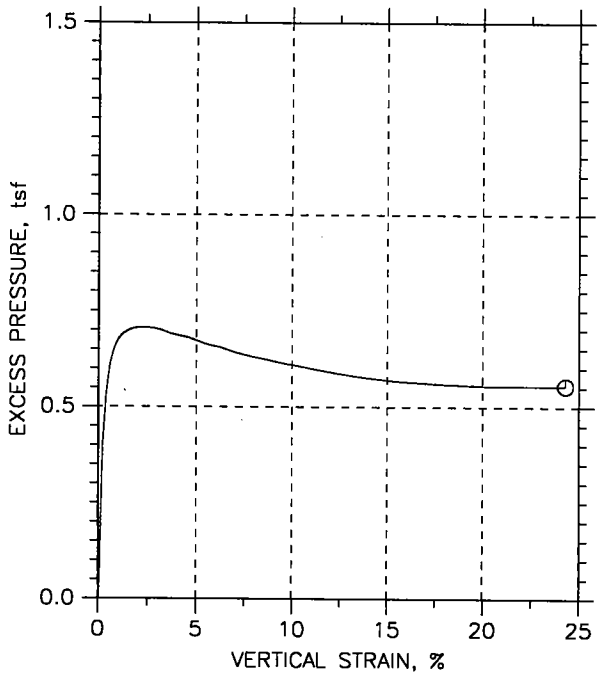
Sample Type: 3" OST

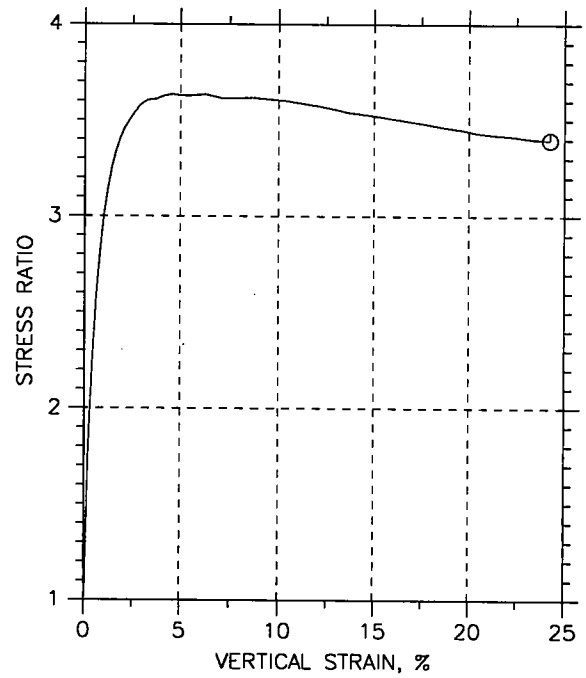
Description: SILTY CLAY SOME F-C SAND - BROWNISH GRAY CL SILT AND SAND POCKETS NOTED.

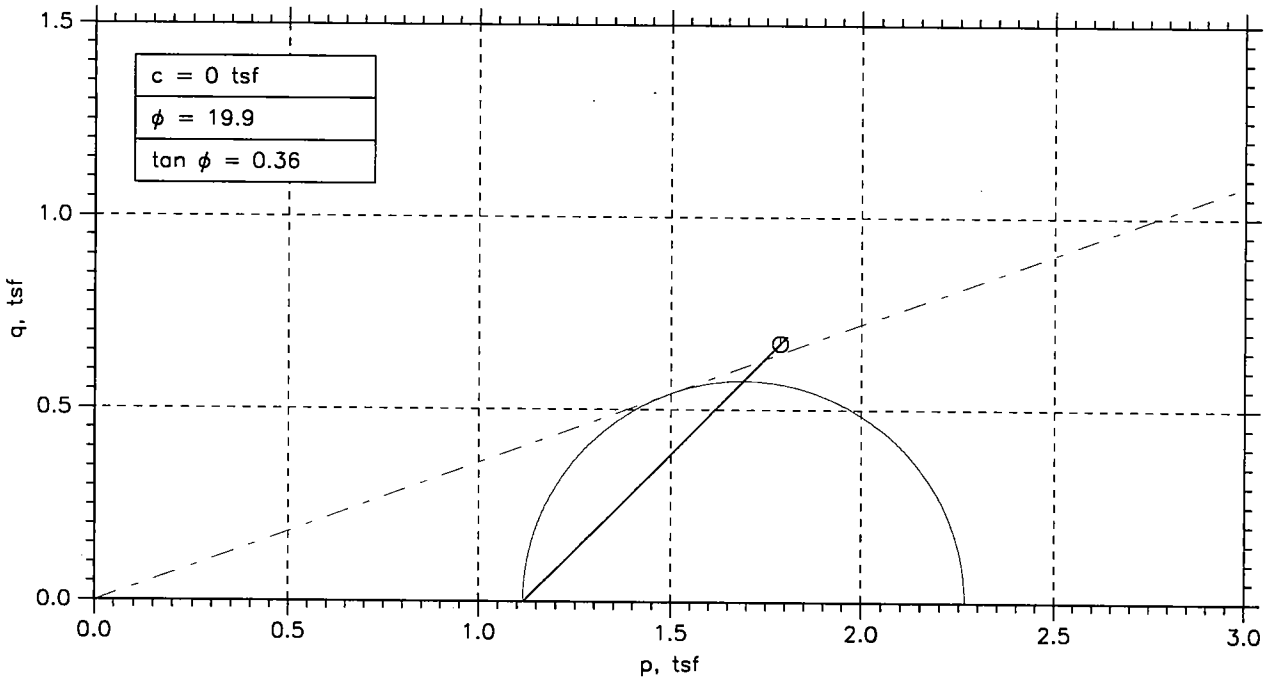
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 1.00

Tue, 31-MAR-2009 13:39:12

TRIAXIAL COMPRESSION TEST REPORT







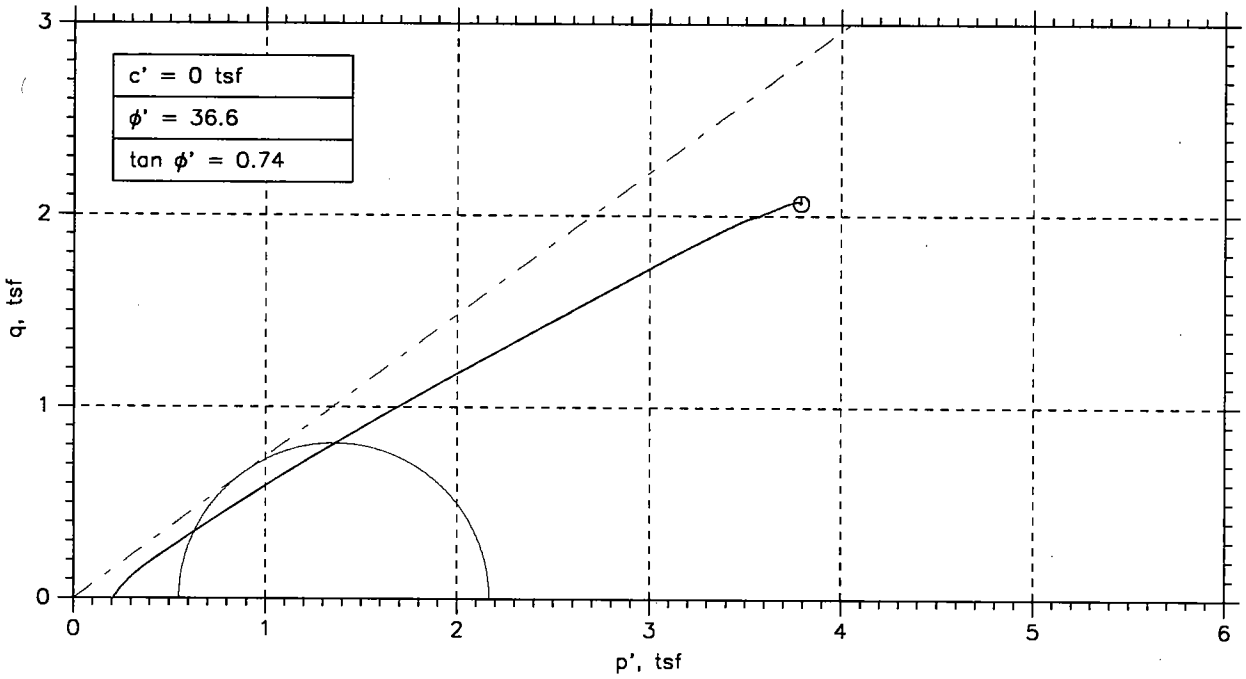
Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: 09-402B S-8	Tested By: BCM	Checked By: WPQ
Sample No.: OST-8	Test Date: 3/11/09	Depth: 46.0'-48.5'
Test No.: 5742402BS8	Sample Type: 3" OST	Elevation: ----
Description: SILTY CLAY SOME F-C SAND - BROWNISH GRAY CL SILT AND SAND POCKETS NOTED.		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 1.00		

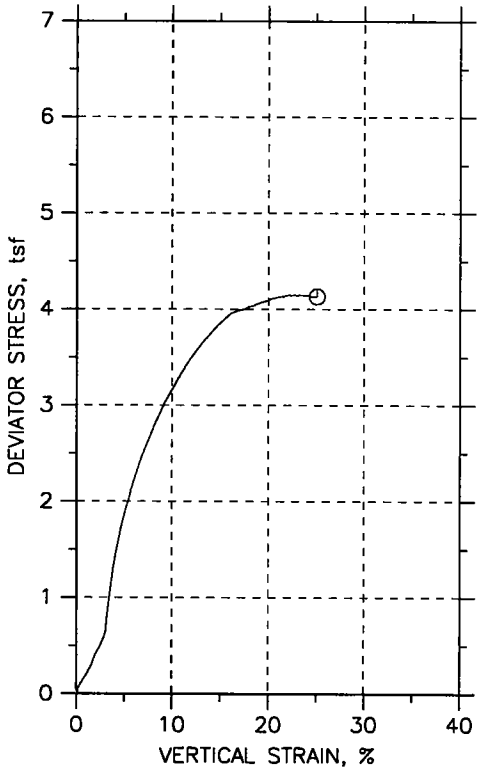
Tue, 31-MAR-2009 13:39:32

Appendix E

Test Data – Sandy Silt to Silty Sand

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⊙			
Test No.		5742105BS2			
Initial	Diameter, in	2.837			
	Height, in	5.7661			
	Water Content, %	21.04			
	Dry Density, pcf	107.5			
	Saturation, %	102.13			
	Void Ratio	0.55016			
Before Shear	Water Content, %	20.59			
	Dry Density, pcf	107.6			
	Saturation, %	100.00			
	Void Ratio	0.54968			
	Back Press., tsf	5.0402			
Minor Prin. Stress, tsf		0.20214			
Max. Dev. Stress, tsf		4.1477			
Time to Failure, min		335.52			
Strain Rate, %/min		0			
B-Value		---			
Estimated Specific Gravity		2.67			
Liquid Limit		Pending			
Plastic Limit		Pending			
Plasticity Index		Pending			
Failure Sketch					

Project: KINGSTON COAL TVA

Location: TN

Project No.: 60095742

Boring No.: 09-105B S-2

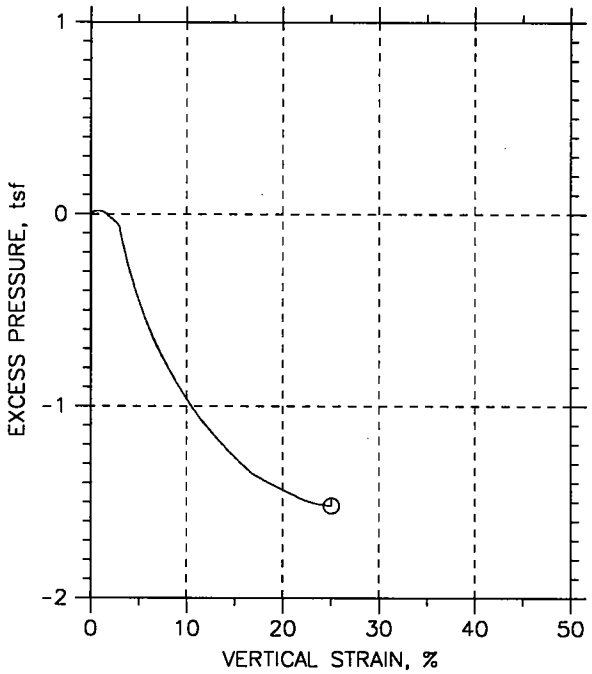
Sample Type: 3" OST

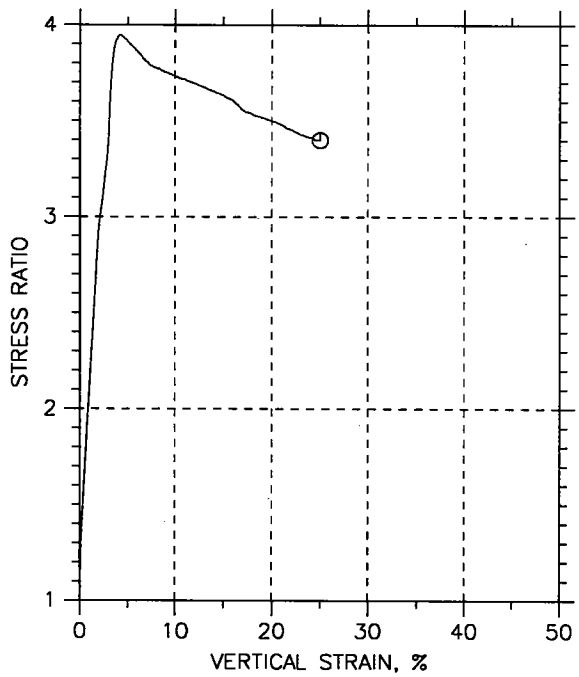
Description: SILTY SAND TRACE CLAY - BROWN (SM)

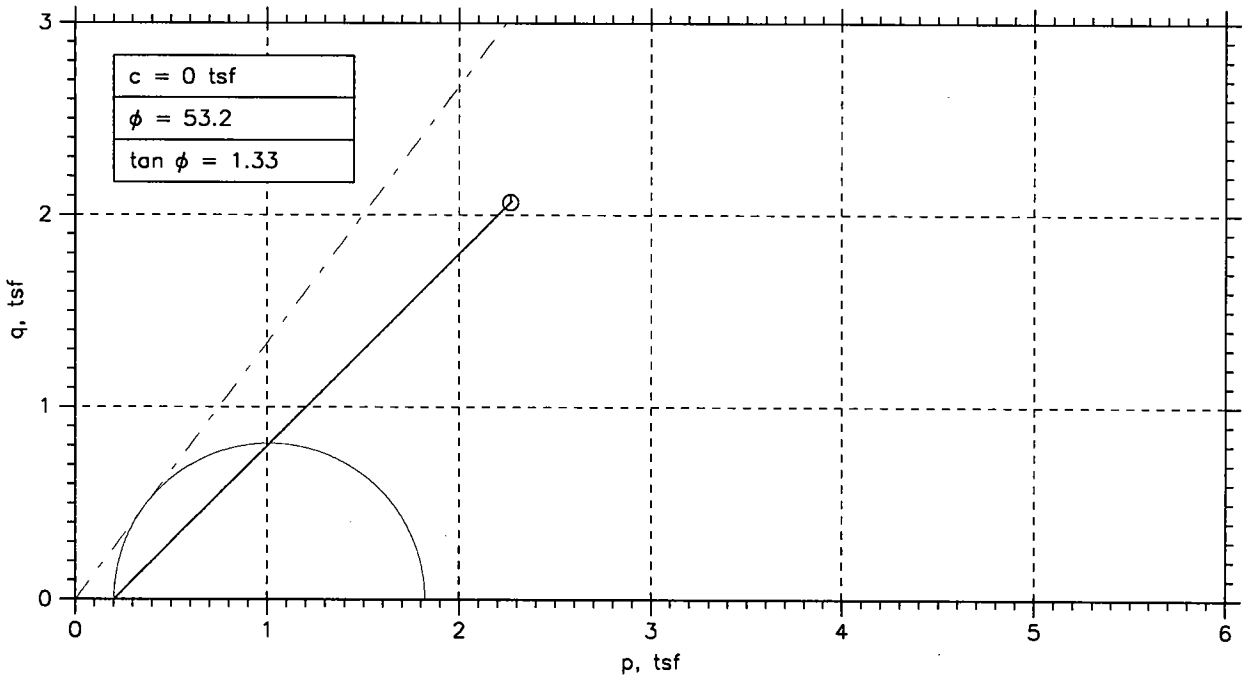
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 1.00

Tue, 31-MAR-2009 11:34:01

TRIAXIAL COMPRESSION TEST REPORT



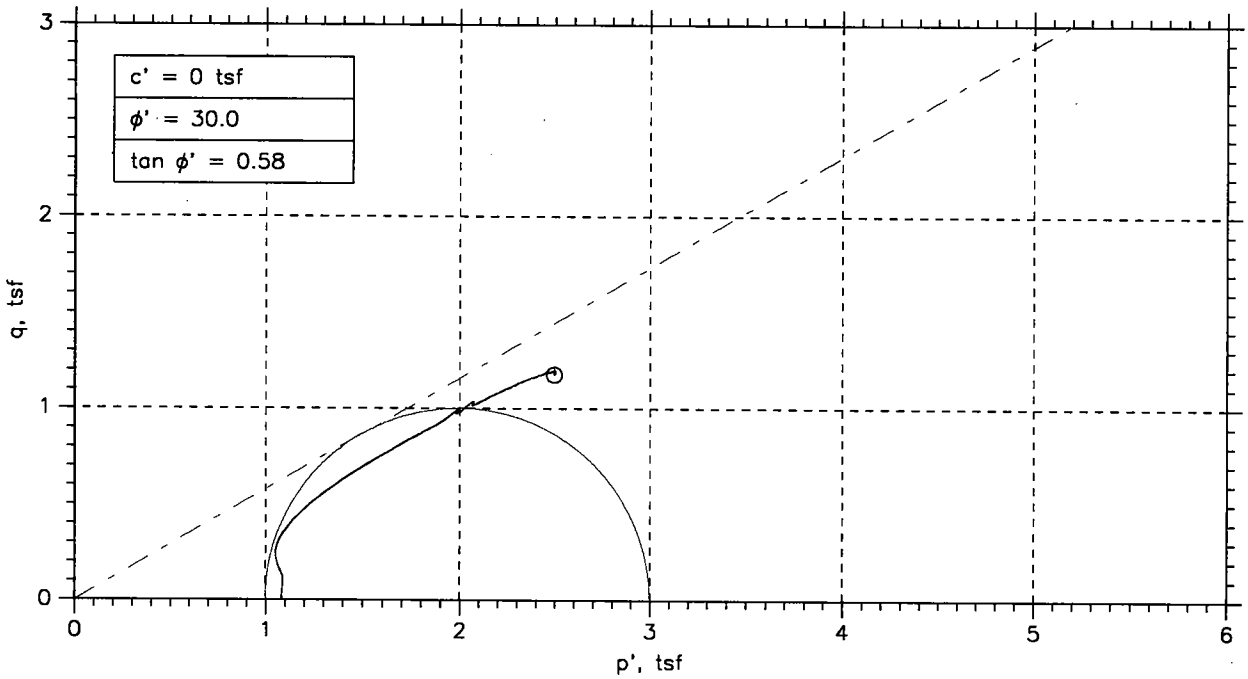


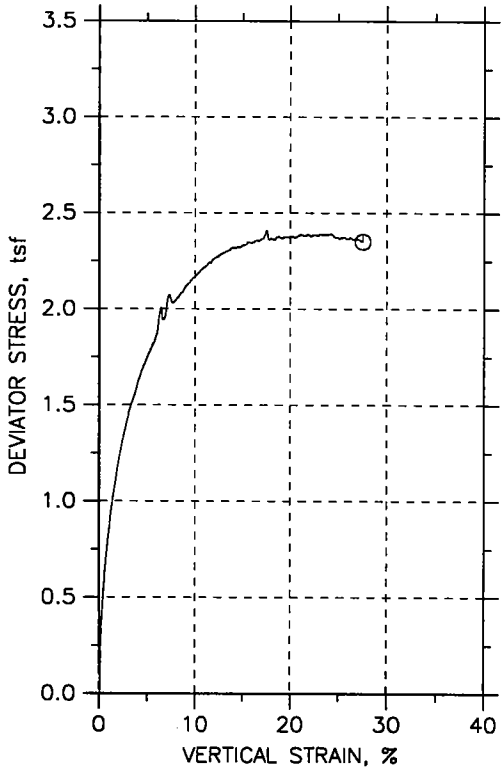


Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: 09-105B S-2	Tested By: BCM	Checked By: WPQ
Sample No.: OST-2	Test Date: 3/20/09	Depth: 17.0'
Test No.: 5742105BS2	Sample Type: 3" OST	Elevation: ----
Description: SILTY SAND TRACE CLAY - BROWN (SM)		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 1.00		

Tue, 31-MAR-2009 11:34:22

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⊙			
Test No.		5742301BS7			
Initial	Diameter, in	2.8524			
	Height, in	5.8098			
	Water Content, %	19.81			
	Dry Density, pcf	110.3			
	Saturation, %	102.56			
	Void Ratio	0.51752			
Before Shear	Water Content, %	18.57			
	Dry Density, pcf	111.7			
	Saturation, %	100.00			
	Void Ratio	0.49764			
	Back Press., tsf	5.0399			
Minor Prin. Stress, tsf		1.0801			
Max. Dev. Stress, tsf		2.4084			
Time to Failure, min		263.52			
Strain Rate, %/min		0.069			
B-Value		---			
Estimated Specific Gravity		2.68			
Liquid Limit		Pending			
Plastic Limit		Pending			
Plasticity Index		Pending			
Failure Sketch					

Project: KINGSTON COAL TVA

Location: TN

Project No.: 60095742

Boring No.: 09-301B S7

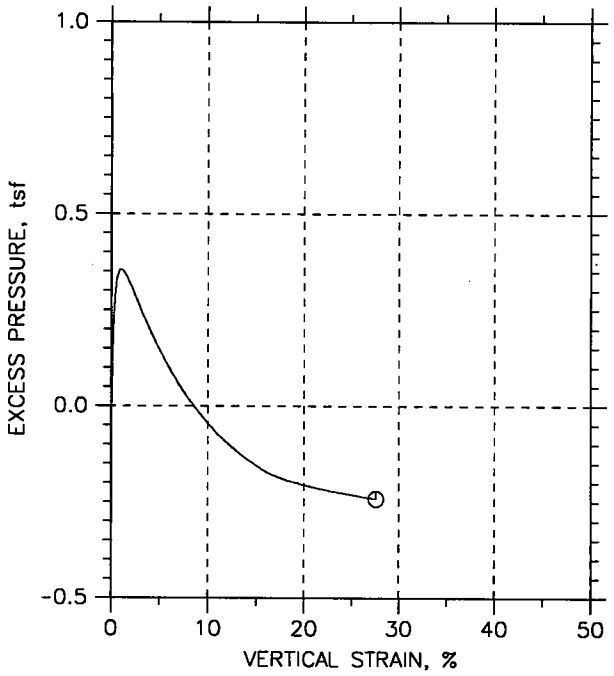
Sample Type: 3" OST

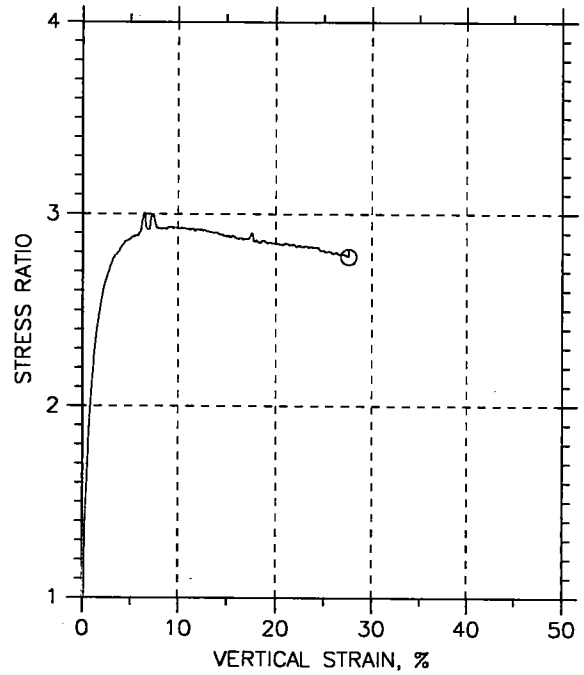
Description: CLAYEY SILT LITTLE F-M SAND - GRAY AND BROWN (CL-ML)

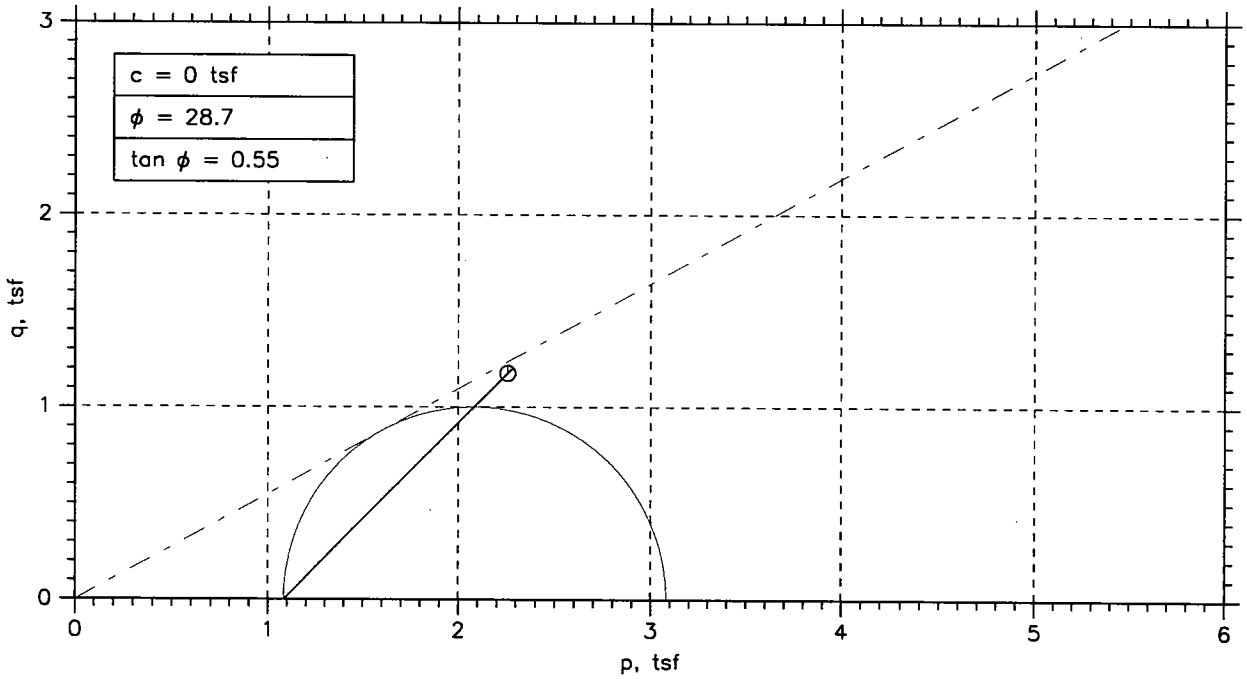
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATION TEST PERFORMED AS PER ASTM D 4767

Tue, 31-MAR-2009 11:56:34

TRIAXIAL COMPRESSION TEST REPORT



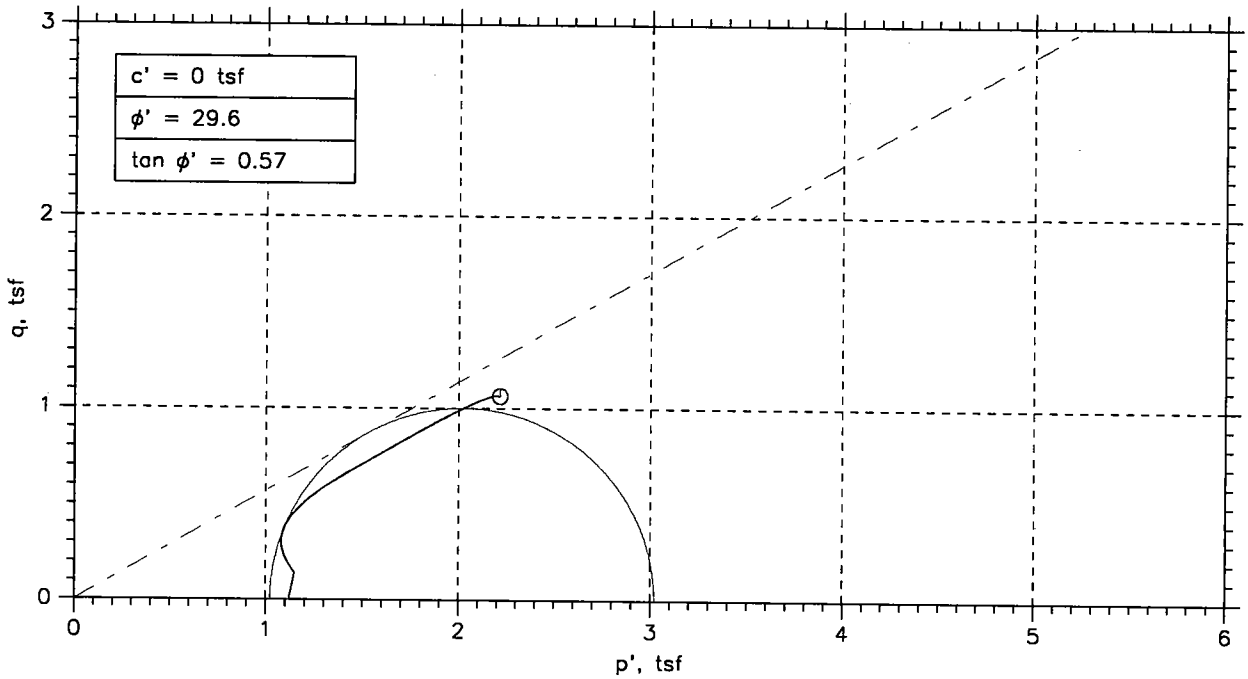


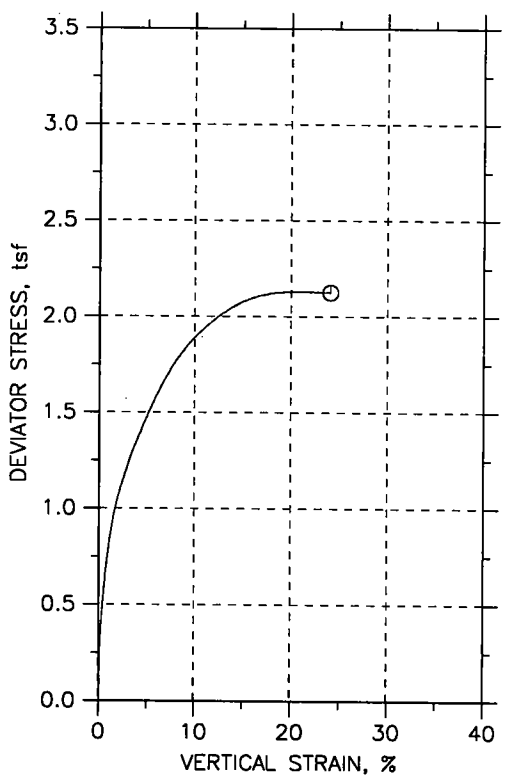


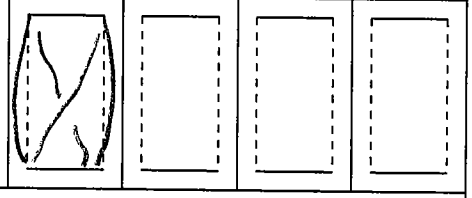
Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: 09-301B S7	Tested By: BCM	Checked By: WPQ
Sample No.: OST-7	Test Date: 3/17/09	Depth: 91.0'
Test No.: 5742301BS7	Sample Type: 3" OST	Elevation: -
Description: CLAYEY SILT LITTLE F-M SAND - GRAY AND BROWN (CL-ML)		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATION TEST PERFORMED AS PER ASTM D 4767		

Tue, 31-MAR-2009 11:56:56

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⊙			
Test No.		5742301BS8			
Initial	Diameter, in	2.8268			
	Height, in	6.0394			
	Water Content, %	21.42			
	Dry Density, pcf	104.5			
	Saturation, %	95.01			
	Void Ratio	0.60635			
Before Shear	Water Content, %	22.48			
	Dry Density, pcf	104.7			
	Saturation, %	100.00			
	Void Ratio	0.60467			
	Back Press., tsf	5.0399			
Minor Prin. Stress, tsf		1.1161			
Max. Dev. Stress, tsf		2.1307			
Time to Failure, min		332.64			
Strain Rate, %/min		0.069			
B-Value		---			
Estimated Specific Gravity		2.69			
Liquid Limit		Pending			
Plastic Limit		Pending			
Plasticity Index		Pending			
Failure Sketch					

Project: KINGSTON COAL TVA

Location: TN

Project No.: 60095742

Boring No.: 09-301B S-8

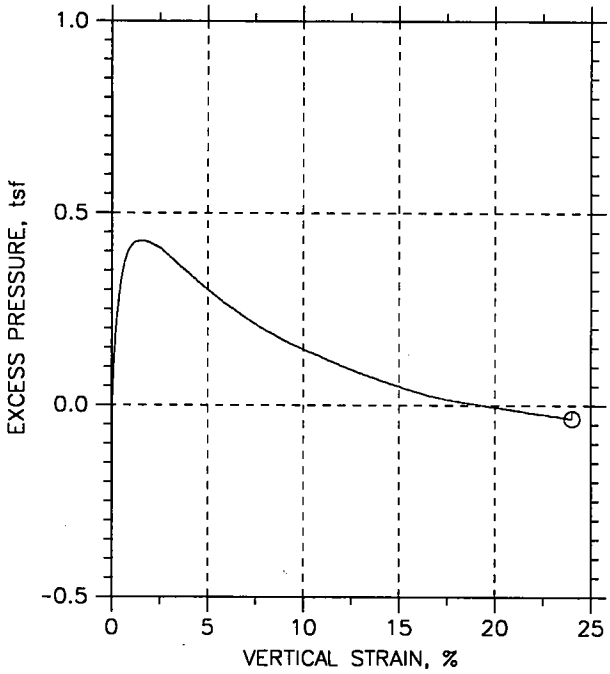
Sample Type: 3" OST

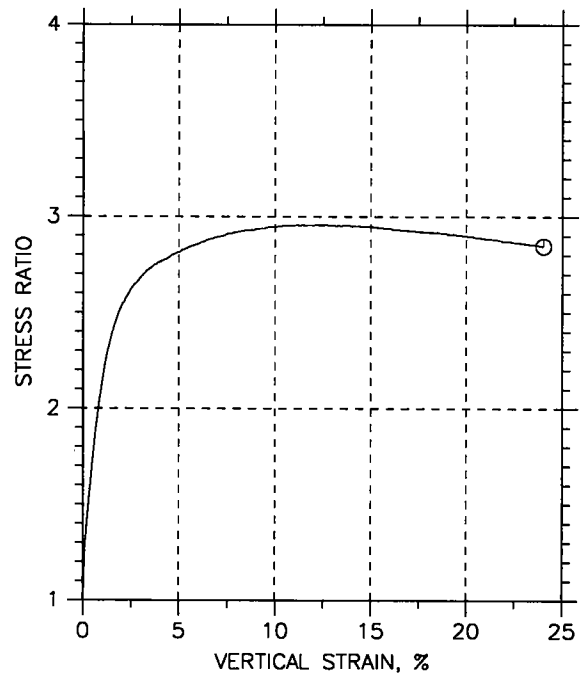
Description: CLAYEY SILT SOME F-C SAND - BROWN AND GRAY (CL-ML)

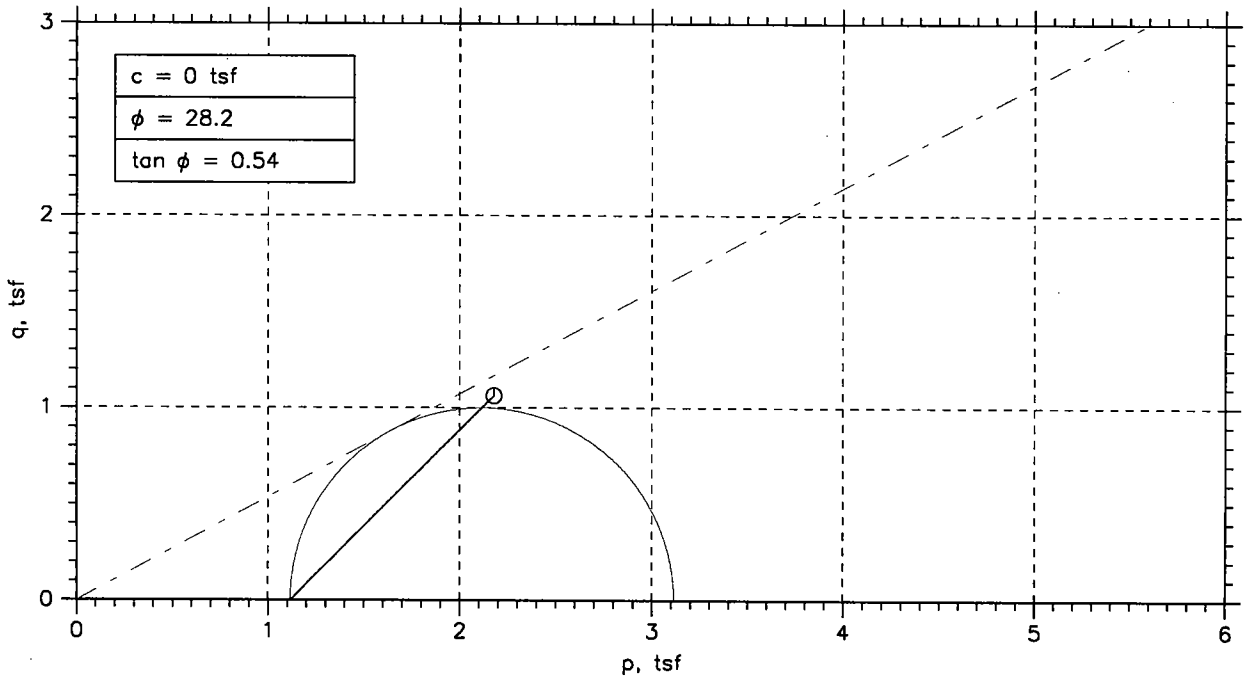
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 0.97

Tue, 31-MAR-2009 12:06:51

TRIAXIAL COMPRESSION TEST REPORT



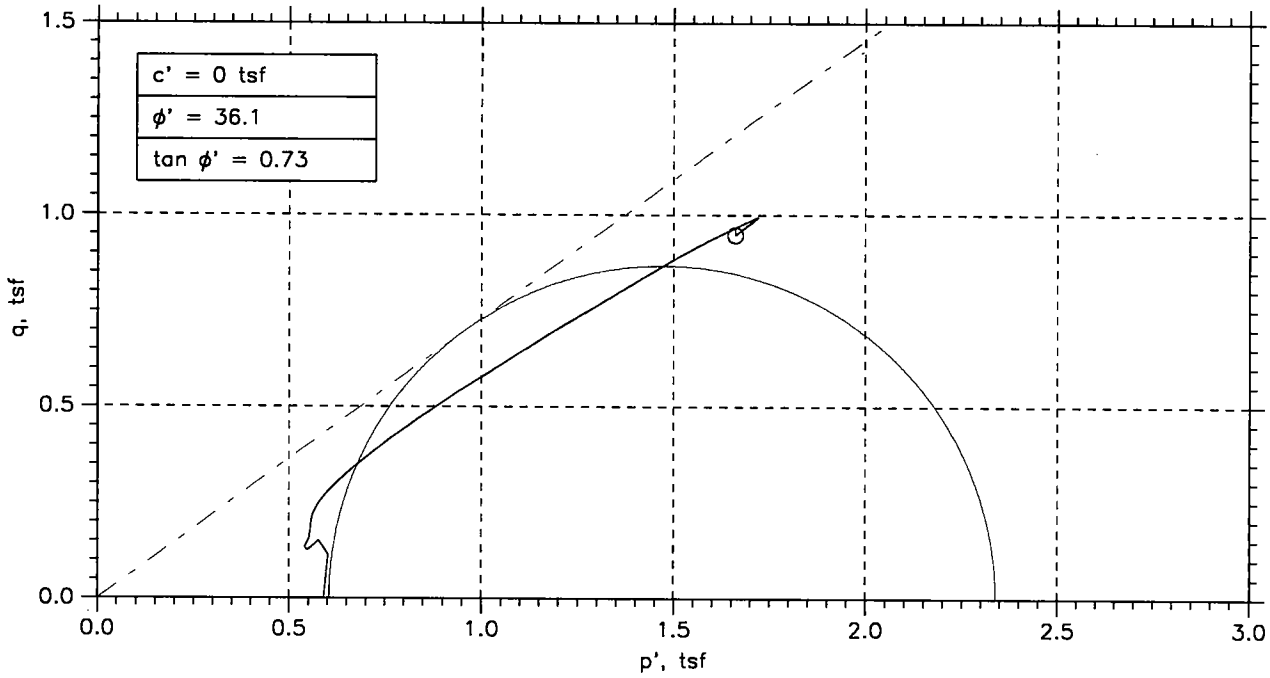


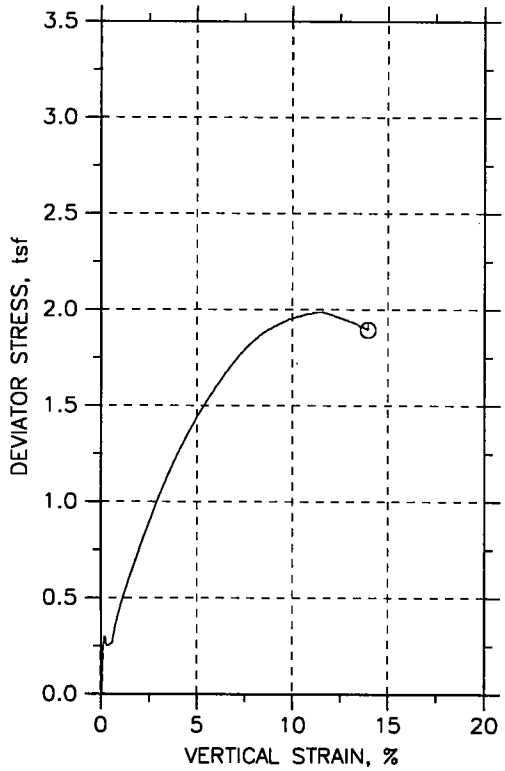


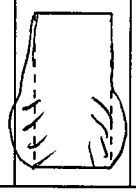
Project: KINGSTON COAL TVA	Location: TN	Project No.: 60095742
Boring No.: 09-301B S-8	Tested By: BCM	Checked By: WPQ
Sample No.: OST-8	Test Date: 3/18/09	Depth: 94.0'
Test No.: 5742301BS8	Sample Type: 3" OST	Elevation: ----
Description: CLAYEY SILT SOME F-C SAND - BROWN AND GRAY (CL-ML)		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 0.97		

Tue, 31-MAR-2009 12:09:14

TRIAXIAL COMPRESSION TEST REPORT





Symbol		⦶			
Test No.		5742402BS9			
Initial	Diameter, in	2.8461			
	Height, in	5.6953			
	Water Content, %	27.08			
	Dry Density, pcf	102.7			
	Saturation, %	115.31			
	Void Ratio	0.62934			
Before Shear	Water Content, %	22.15			
	Dry Density, pcf	105.			
	Saturation, %	100.00			
	Void Ratio	0.59357			
	Back Press., tsf	5.0399			
Minor Prin. Stress, tsf		0.59055			
Max. Dev. Stress, tsf		1.9867			
Time to Failure, min		171.36			
Strain Rate, %/min		0.069			
B-Value		---			
Estimated Specific Gravity		2.68			
Liquid Limit		Pending			
Plastic Limit		Pending			
Plasticity Index		Pending			
Failure Sketch					

Project: KINGSTON COAL TVA

Location: TN

Project No.: 6005742

Boring No.: 09-402 S-9

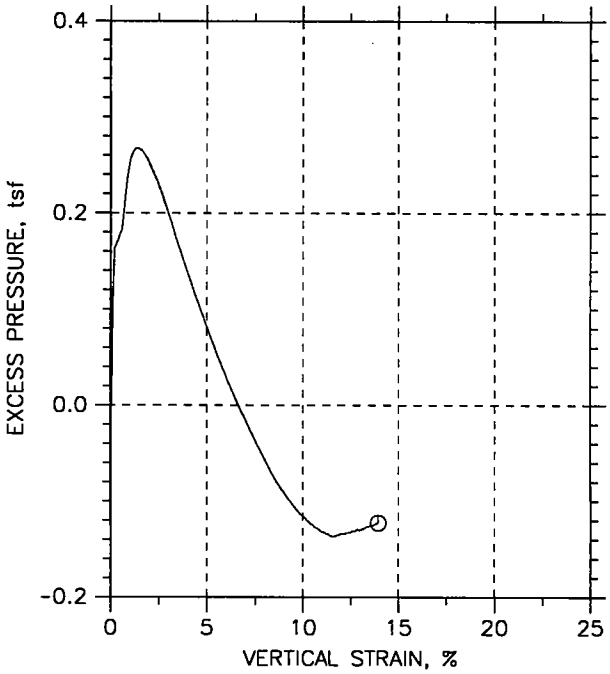
Sample Type: 3" OST

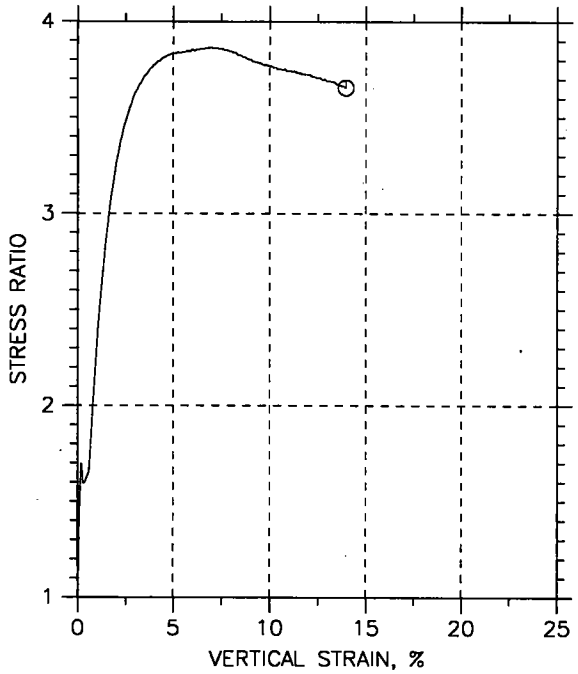
Description: CLAYEY SILT SOME F-M SAND - GRAY (CL-ML)

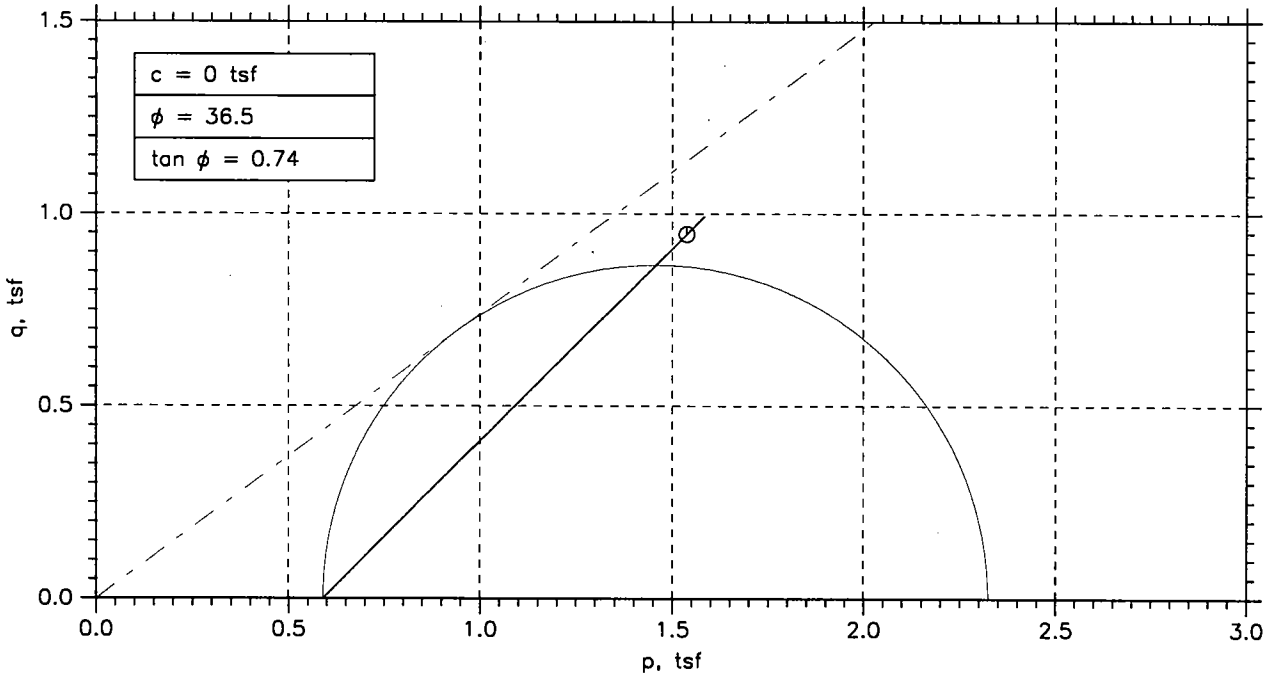
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 0.98

Tue, 31-MAR-2009 15:03:41

TRIAXIAL COMPRESSION TEST REPORT







Project: KINGSTON COAL TVA	Location: TN	Project No.: 6005742
Boring No.: 09-402 S-9	Tested By: BCM	Checked By: WPQ
Sample No.: OST-9	Test Date: 3/17/09	Depth: 50.0'
Test No.: 5742402BS9	Sample Type: 3" OST	Elevation: ----
Description: CLAYEY SILT SOME F-M SAND - GRAY (CL-ML)		
Remarks: FAILURE CRITERIA = MAXIMUM EFFECTIVE STRESS RATIO TEST PERFORMED AS PER ASTM D 4767 B VALUE = 0.98		

Tue, 31-MAR-2009 15:04:03