Soil-Pipe Interface Friction Coefficients for Buried PE4710 Pipe

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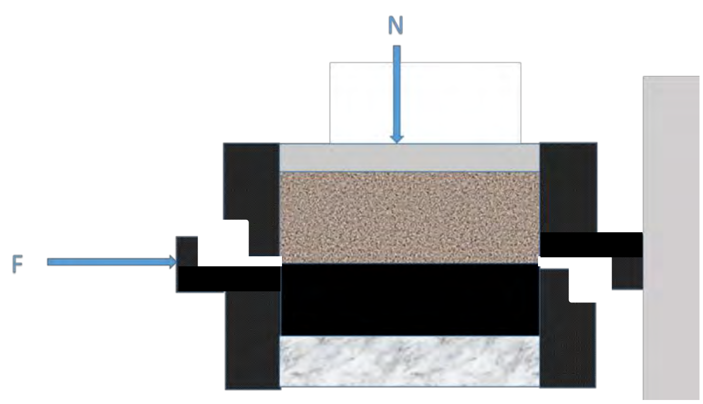

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Executive Summary

This study reports interface friction coefficients that were experimentally determined for various soil types sliding on thick coupons of PE4710 high density polyethylene (HDPE). These values and related discussion are intended to facilitate more accurate assessment of pipe resistance to axial movement. Factors affecting frictional resistance between embedment and pipe experiencing axial movement due to temperature and pressure change were studied.

The interface friction coefficients for soils sliding on 12-inch (305-mm) square, 2-inch (51-mm) thick coupons of PE4710 high density polyethylene were measured and the results are reported herein. Five different soils and twenty-two test coupons were used in testing. Peak and residual friction coefficients were calculated from results of direct shear tests: the latter of these is generally most appropriate for use in buried pipe design. Residual friction coefficients for free-draining coarse-grained soils were between 0.15 and 0.5, whereas values were between 0.05 and 0.20 for fine-grained soil and coarse-grained soil having significant fines content. Peak friction coefficient was observed to increase with normal stress. This trend was minor and sometimes absent or otherwise indiscernible for the residual friction coefficient. Also, it was observed that the planar features of fractured, angular, platy and elongated coarse-grained particles aligned with the horizontal surface of the test coupons during specimen placement. This may explain an unexpected observation that coarse-grained crushed rock with these characteristics and subangular to angular sand exhibited friction coefficients that were less than or equal to the values determined for a tested subrounded-to-subangular pea-gravel. These observations lead to the recommendation that lower values of interface friction coefficient should be selected for buried pipe design when the embedment has fractured, angular, platy and elongated shape, has more than a few percent fines content, or normal stress is low.

Appropriate use of the results of this study requires understanding that pipe axial displacement is required to fully mobilized frictional resistance. Hence, buried pipeline design must consider where the displacement is sufficient to expect fully mobilized frictional resistance. Also, the design process should consider the method used to estimate the average normal stress. A calculation method that is likely to underestimate the average normal stress on buried pipe would be conservative for this purpose since it would result in a lower estimate of pipe frictional resistance. However, underestimating normal stress is inconsistent with the need to overestimate stress for conservative pipe radial deflection calculation. Consequently, thoughtful consideration must be given to the method used to calculate average normal stress on buried pipe for the purpose of estimating frictional resistance to axial movement.

Section I. Introduction

The interface friction coefficients for soil sliding on 12-inch (305-mm) square, 2-inch (51-mm) thick coupons of PE4710 high density polyethylene (HDPE) were measured in the laboratory by a direct shear testing method. Five different soils and twenty-two coupons were used in testing. Peak and residual interface friction coefficients were calculated and are presented and discussed. Considerations important for appropriate application are identified and also discussed. This report is organized as follows:

Section II - Scope
Section III - Test Program
Section IV - Test Procedure
Section V - Interface Friction Coefficients (μ)
Section VI - Observations
Section VI - Application of μ
Section VII - Recommendations
Section VIII - Conclusion

Section II. Scope

Pipe changes axial length in response to changes in temperature, or internal pressure, which may occur independently or in combination. Also, water flowing through a bend or tee in a pipe creates force that causes pipe axial strain. For buried pipe embedded in soil, axial displacements are resisted by friction that develops on the pipe-soil interface. Frictional resistance is mobilized as the pipe moves axially and is fully mobilized at locations along the pipe where the movement is sufficient to cause the pipe to slip through the embedment. The fully mobilized frictional resistance is given by the equation:

$$\tau = \mu\sigma \qquad (1)$$

Where:

$\tau = Fully\ mobilized\ frictional\ resistance$

$\sigma = Average\ Normal\ Stress$

$\mu = Interface\ Coefficient\ of\ Friction$

This paper reports the results of tests performed to determine values of μ for PE4710 HDPE in contact with typical pipe embedment materials. Measurements and observations made during the testing are used to develop recommendations for appropriate selection of μ for buried pipe design. Considerations for appropriate application of this study's results are presented.

Section III. Testing Program

Tested PE4710 HDPE coupons were approximately 2-inch thick and 12-inch square. Test coupons had a smooth glossy surface with occasional, minor, random scratches. For examples, pretest photographs of two coupons are shown on Figure 1. The predominant scratch direction, if any, was aligned with the direction of shear displacement during testing. Manufacturer supplied PE4710 properties are summarized in Table 1.

Common names are used to describe soils that were tested: these are, Density Sand, Crushed Rock, Pea Gravel, Silty Sand, and Silty Clay. Table 2 presents tested soil properties, Unified Soil Classification (ASTM D2487), and Uniform Soil Groups - Soil Class (ASTM D2774). Figures 2, 3 and 4 are photographs of the coarse-grained soils. The test program is presented in Table 3. Soil placement conditions, as well as test results that will be discussed in Section V, are likewise presented in Table 3.

Placement of coarse and fine-grained soils in the shear box involved either moderate or no compactive effort. It is possible that the resulting µ are lower than would have been determined had specimens been placed at higher densities. The condition of moderate to no compacted effort was selected with the recognition that the resulting values of interface friction coefficient might be conservative for use in design. In addition, it is speculated that the loose placement of the soil in this research may mimic compacted backfill in close proximity to a buried pipe that may, over time, develop a low density due to radial expansion and contraction of the pipe.

Crushed Rock contained approximately one percent fines (material passing the #200 sieve) and two percent sand size particles (material passing the #4 sieve but retained on the #200 sieve). To evaluate the significance of the finer fraction on the measured friction coefficient, the Crushed Rock was washed over a #4 sieve prior to placement for Test 19. Washing to remove sand and fines produced no measurable effect.

Section IV. Test Procedure

Direct shear tests were performed in general compliance with:

ASTM D5321 *Determining the Shear Strength of Soil-Geosynthetic and Geosynthetic-Geosynthetic Interfaces by Direct Shear*.

ASTM D3080 *Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions*.

The direct shear test apparatus was a Wykeham Farrance Model WF25506 and is shown in Figure 5. The general concept for the direct shear test is depicted on Figure 6. A normal (vertical) load is applied to the soil, which lies above the test coupon in the upper half of a shear box. The lower half of the shear box containing the HDPE test coupon is moved horizontally and the force resisting the horizontal movement of the upper shear box is recorded. The corrected shear stress and normal stress are calculated as the quotient of the respective force and the area of the coupon in contact with the soil. The corrected stresses increase throughout the test because the area of the coupon in contact with soil decreases as the shear box displaces. Horizontal and vertical displacements are recorded throughout the test. The rate of horizontal displacement is controlled.

The upper and lower shear boxes are approximately the same 12-inch by 12-inch (305 mm by 305 mm) size as the test coupons +/- 0.02 inch (0.5 mm). Coupons were cooled to about 32 degrees F prior to placement. This was done to facilitate insertion of the coupon into the lower shear box. The coupons were returned to room temperature prior to testing. The resulting thermal expansion yielded a snug fit of the coupon in the lower shear box.

Both upper and lower shear boxes were approximately 3.9 inch (99 mm) deep. The test coupons were two inches thick and were positioned so that the test surface was level and 0.08 inch (2 mm) above the surface of the lower shear box. Either plaster-of-paris or a gypsum cement material was used to fill the lower shear box below the test coupon and provide a solid bearing surface. A photograph of a test coupon positioned in the lower shear box and placed in the test machine is shown on Figure 7.

Pea Gravel and Crushed Rock were placed with a moderate compactive effort. A moderate compacted effort entailed placing the material in approximately four equal height lifts to a depth of approximately 3.4 inches (86 mm), each lift being hand compacted by tapping the surface repeatedly and uniformly with a 3.3 pound, 3-inch (76 m) diameter steel cylinder for approximately 1 minute.

For tests using Density Sand, the sand was gently placed in the upper shear box on top of the test coupon to attain a near zero relative density. This was done for all tests using Density Sand except test 18. A moderate compacted effort was used to place Density Sand for test 18.

The Silty Sand and Silty Clay were both placed in the upper shear box to a depth of approximately 1.25 inch (32 mm) by spooning and spreading a saturated slurry (material slightly above the approximate liquid limit[1]). Following placement, the soil was consolidated statically under the normal stress to be used for friction coefficient determination.

For all tests, initially, a seating pressure of approximately 0.7 lb/in^2 was applied to the entire test specimen. This is the pressure created by the static weight of the submerged soil, loading platen and load frame. The shear box and test specimen were then submerged and remained submerged throughout testing. Vertical displacement measurements began following soil consolidation at the seating pressure.

For tests using Silty Clay and Silty Sand, the vertical pressure was typically added in doubling increments beginning with 5 lb/in^2 until the desired test normal stress was achieved. Time was provided for the specimen to fully consolidate under each pressure increment prior to the addition of normal stress or otherwise proceeding with shear displacement. This process typically took several days.

For tests using Pea Gravel, Density Sand and Crushed Rock, the desired test pressure was applied by adding a single increment of load. Time was provided for the specimen to fully compress prior to proceeding. This process typically took approximately one hour.

Following consolidation and prior to the application of shear displacement, four steel screws, “gap screws”, threaded through the upper shear box and resting on the top of the lower shear box, were advanced to separate the upper shear box from the lower shear box and create a gap of 0.16 inch (4

[1] The liquid limit is the water content of the soil above which the soil behaves more like a viscous liquid than a semi-plastic solid (determined per ASTM D4318). For the Nonplastic Silty Sand material, the water content of the saturated slurry was based on judgement and visual observation.

mm). This resulted in the surface of the test coupon being centered in the gap. The gap screws slid on the lower shear box on nylon pads during shear displacement. This maintained the 0.16-inch (4 mm) gap throughout the test. The frictional resistance between the nylon pads and steel of the lower shear box was measured prior to the first test. This measurement was subtracted from subsequent measurements of frictional force.

Shear displacement commenced at a controlled rate and the peak and residual frictional stresses[2] determined. The shear displacement rate was selected to be sufficiently slow to ensure excess soil pore water pressure would not develop during testing. This manner of testing is commonly known as "drained shear." In some instances, the shear rate was changed during testing to evaluate the influence of shear rate on measured frictional resistance. No rate dependence was observed indicating that the test shear rate was sufficiently slow such that excess pore water pressures were not generated. Shear displacement was continued until residual shear stress was achieved as indicated by a steady shear stress with continued shear displacement.

Test 1, 3 and 7 were performed as multistage tests. That is, normal stress was increased in stages as the test progressed. This resulted in several measurements of peak and residual frictional force on a single coupon for each of these tests (the horizontal position was not reset between stages).

Section V. Interface Friction Coefficients

Peak interface friction coefficient (μ_p) and residual interface friction coefficient (μ_r) were calculated for each applied normal stress. μ_r is generally appropriate for use in buried pipe design (McCabe 2014). Summary plots of time, displacements and stresses are presented in Appendix A for each test. Values selected from test measurements to represent the corrected peak and residual shear stresses and respective normal stresses are presented on Table 3 with associated values of μ_p and μ_r. Appendix B present graphs of calculated values of μ as it changed with normal stress for each soil tested.

Pea Gravel, Density Sand and Crushed Rock were free draining and coarse-grained. Calculated values of μ_r for these soils were between 0.15 and 0.5.

The Unified Soil Classification (ASTM D2487) defines silty clay as fine-grained soil and silty sand as a coarse grain soil having fines content greater than 12 percent. The Silty Sand tested had a fines content of about 27 percent. Values of μ_r determined for these soils were between 0.05 and 0.20. μ_p was observed to increase with normal stress. However, this trend was smaller and sometimes absent (or indiscernible) for μ_r.

Both peak and residual, normal and shear stresses resulting from this study are compared in Appendix C with data compiled by Drexel University that represent geosynthetic-to-soil interface friction (Koerner

[2] Often, shear stress increases to a maximum (peak stress) value and then decreases to a constant value (residual stress) with continued shear displacement. Sometimes a peak stress is never observed. When this happens, peak stress is equal to residual stress.

2005). It is observed that the ranges of μ measured in this study are in approximate agreement with the Drexel University data.

PPI TR-21/2001, "Thermal Expansion and Contraction in Plastic Piping Systems", presents that a value of 0.1 is a generally accepted conservative value for μ for the case where smooth surface plastic pipe makes full contact with the embedment material (PPI 2001). The PPI Handbook for Polyethylene Pipe, Chapter 12 – Horizontal Drilling suggests μ for determining the pulling resistance between pipe and ground is typically 0.4 (PPI 2018). These values fall within the range of μ determined by this study.

Table 4 compares the extreme values of μ determined by this study with the extreme values scaled from plots of normal and shear stress representing the Drexel database and reproduced in Appendix C, as well as values recently recommended by the ASCE Task Committee on Thrust Restraint Design of Buried Pipelines (Koerner 2005; McCabe 2014). Additionally, values from pullout tests performed on PE tape-wrapped steel pipe are presented (Alam S, et al. 2014). The range of μ's determined by this study are approximately within these ranges except that the values determined by Alam for silty clay are significantly higher.

Alam presents results of 13 pullout tests of PE tape-wrapped steel pipe having cover depths of 4 feet to 16 feet (Alam et.al. 2013). Pipe were buried and tested in a moist compacted state, except one sand test which was flooded. The friction factors[3] presented show a trend toward decreasing friction factor with increasing burial depth. Alam writes: *"It is important to note that the friction factor was back-calculated from Equation 13-6 in AWWA M11 (2004). As a result, the friction factor 'declines' with depth. This is attributed to the fact that the relative contribution of the weight of the pipe and its content decline as the weight of the soil prism above the pipe crown increases."* It is noteworthy that friction coefficients determined in this study are constant or increase with increasing normal stress, suggesting the friction coefficient is constant or increases with cover depth. This is discussed in greater detail in Section VI. The need to thoughtfully evaluate the method used to calculate average normal stress on a buried pipe is discussed in Section VII.

Section VI. Observations

Traditionally, the friction coefficients for two planar surfaces sliding past one-another is explained as being independent of normal stress, i.e. μ is a constant. However, this model does not fit all materials. For example, μ typically decreases with normal stress for soils sliding on an internally developed shear plane. This stress dependent behavior results from shear resistance being governed by a unique physical mechanism. The behavior of μ describing friction between soil and PE4710 HDPE appears to be governed by a mechanism that is different than that governing internal shear of typical soils. This is evident by the observation of graphs in Appendix B that show both μ_p and μ_r to have a tendency to increase with normal stress.

The observation that μ increases with normal stress is hypothesized herein to be attributable to individual particles gouging into and subsequently plowing through the HDPE surface at increasingly

[3] The term "friction factor" is synonymous with "friction coefficient" used herein.

greater depth as normal stress is increased. The resulting gouges are evident on the surface of tested HDPE coupons. For example, Figure 8 shows before and after images that highlight scratch patterns observed on the Test 7 coupon, which was tested at a normal stress of 40 lb/in^2 using Pea Gravel. The shear force required for a particle to plow through plastic expectedly increases with increasing gouge depth and with increasing number of particles involved. The mechanism is complex and further development is beyond the scope of this research. However, subsequent discussion presents observations that seemingly support this mechanism.

The following observations are discussed in the context of the hypothesized mechanism for friction presented in the previous paragraph:

1. Gouging was more evident for subrounded-to-subangular Pea Gravel than any other material.
2. Gouging was less extreme for tested materials having a predominance of planar features, i.e. platy, elongated, fractured faces, and predominantly angular shapes.
3. Gouging was not visibly discernable on the surface of coupons tested using Silty Clay or Silty Sand, although a less light reflective surface was discernable.
4. The friction coefficient increased as the extent of gouging increased.

When placing soil on the HDPE test coupons, it was observed that planar features of fractured, angular, platy and elongated particles aligned with the plane of the test coupons. That is, flat sides of particles rested flat on the coupon surface. Herein, it is hypothesized that particles lying flat on the HDPE surface result in lower particle contact stress with the coupon and consequently less gouging of the coupon surface. This may explain why both Crushed Rock having fractured, angular, platy and elongated characteristics and the subangular Density Sand exhibited friction coefficients that are less than or equal to the values determined for subrounded-to-subangular Pea Gravel.

Table 5 presents a summary of the friction ranges observed for the five soils tested. The low μ associated with Silty Sand is noteworthy. A silty sand is considered a coarse-grained soil by the Unified Soil Classification System and has a demonstrated higher coefficient of internal friction than fine-grained soils. However, this relationship does not hold for the soil-HDPE interface: μ_r results for both Silty Sand and Silty Clay are about the same. The association of low frictional resistance with an absence of visible gouging on the surface of coupons tested using Silty Sand and Silty Clay suggests that frictional resistance increases with the soils tendency for gouging and that tendency is diminished in coarse-grained soil by the inclusion of a small percentage of fines.

Section VII. Recommendations

The results and observations discussed in the previous sections lead to the following recommendations for selection of interface friction coefficient:

1. The appropriate value of μ for buried pipe design applications for Class I and Class II embedment ((ASTM D2774, crushed rock and clean coarse-grained soil) is likely in the range 0.15 and 0.50.

2. The appropriate value of μ for buried pipe design applications for any class of soil other than Class I and Class II embedment is likely in the range 0.05 and 0.20.
3. The lower values in these ranges are recommended when the embedment material is expected to include significant fractured, angular, platy or elongated shaped particles.
4. The lower values in these ranges are recommended when normal stress is low.

Section VII. Application of μ

Buried pipeline designs sometimes rely on interface friction to resist axial displacement due to temperature or pressure change and to control axial pipe movement associated with changes in flow. Shear force that resists axial displacement of the pipe is governed by either the embedment shear stress - shear strain relationship or the fully mobilized frictional resistance. The magnitude of pipe axial displacement and both embedment type and condition determine if soil properties or pipe interface properties control. The fully mobilized frictional resistance is given by equation 1. It must not be assumed that the fully mobilized frictional resistance is instantaneously available to resist axial displacement. The ASCE Task Committee on Thrust Restraint Design of Buried Pipelines presents that experimental data for both pile foundations and pipes show that peak frictional resistance develops at about 3 to 10 mm of displacement depending on soil type (ASCE 2014).

Embedment adjacent to a pipe that is experiencing axial elongation or contraction will undergo shear strain and consequently move with the pipe until the shear stress at the interface exceeds the available frictional resistance. For example, consider a long, buried pipe that is free to move axially at both ends and experiences a pressure increase or a temperature decrease. The pipe contracts axially. Figure 9 shows a graph depicting pipe displacement and interface frictional resistance. Note that no frictional resistance to movement has developed at the center of the pipe because it has not displaced. Frictional resistance increases with increasing distance from pipe centerline. Initially, the frictional resistance is controlled by the embedment shear stress-shear strain properties. Accumulative displacement near the end of the pipe is sufficient to fully mobilize frictional resistance. Once fully mobilize the frictional resistance remains constant and the pipe in this region slips through the embedment. Notice that axial displacement of the pipe increases with increasing distance from centerline. Both the axial displacement and the rate of change of axial displacement are greatest near the ends of the pipe where the cumulative force from frictional restraint decreases to zero. This example is intended to dispel the belief that fully mobilized friction is available instantaneously upon the onset of axial elongation or contraction. Care should be taken during design to determine where displacement is sufficient to expect frictional resistance to be fully mobilized and to realize that axial displacement occurs within this region.

The fully mobilized frictional resistance is directly proportional to the average normal stress acting on the pipe interface. Design engineers commonly overestimate the stress on pipe for the purpose of calculating a conservative estimate of pipe radial deflection. However, overestimating the stress would lead to an unconservative estimate of fully mobilized frictional resistance to axial displacement. Care should be taken to develop a conservative lower bound estimate of the average normal stress that may exist during the life of the pipeline at locations where fully mobilize frictional resistance will be needed.

Cycles of axial elongation and contraction occur throughout the useful life of most pipelines. This concern is not specifically addressed by this research. However, it is worth noting that applying cyclic shear stresses to the embedment is expected to reduce the embedment shear modulus, which in turn will result in larger shear displacement of the embedment soil being required to fully mobilize friction.

Section VIII. Conclusions

The interface friction coefficients for soils sliding on approximately 12-inch (305-mm) square, 2-inch (51-mm) thick coupons of PE4710 HDPE were measured. Twenty-two test coupons were used in testing. Soils were selected to represent five different potential pipe embedment materials. Commonly used names that describe the soil tested are Density Sand, Pea Gravel, Crushed Rock, Silty Sand and Silty Clay. These soils were placed with either no compactive effort or moderate compactive effort. Gouging of the HDPE was observed and appears to be a key factor in the mechanism governing shear resistance.

Peak and residual friction coefficients were calculated from results of direct shear tests: the latter of these is generally most appropriate for use in buried pipe design. Residual friction coefficients for free-draining coarse-grained soils were between 0.15 and 0.5, whereas values were between 0.05 and 0.20 for fine-grained soil and a coarse-grained soil having approximately 27 percent fines.

The tested crushed rock had a high percentage of flat and elongated particles which, during placement in the test apparatus, were observed to preferentially lay flat on the surface of HDPE test coupons. The Density Sand was angular-to-subangular in shape and the Pea Gravel was subrounded-to-subangular. Less HDPE gouging was observed when testing the Pea Gravel than either the Density Sand or Crushed Rock. Furthermore, the friction coefficients for the Pea Gravel were as great as or greater than those determined for Density Sand and Crushed rock. It is hypothesized that these unexpected results are because the flat surface of the latter materials preferentially aligned with the plane of the horizontal surface of the test coupon. This observation and hypothesis supports a recommendation that the lower values of measured friction coefficients be used for buried pipeline design when embedment material has a significant quantity of flat surfaces.

Peak friction coefficient was observed to increase with normal stress. This trend was minor and sometimes absent or otherwise indiscernible for the residual friction coefficient. These observations support a recommendation that lower values of interface friction coefficient be selected for buried pipe design when normal stress is expected to be low.

It was observed that the friction coefficients measured for Silty Sand having an approximately 27 percent fines content was about the same as that for fine grained Silty Clay. Silty Sand is commonly considered a coarse-grained soil and expectedly would have higher coefficients of friction for internal sliding than the fine-grained Silty Clay. This result, coupled with the observation of no gouging of HDPE by either of these materials, leads to recommendation that coarse-grained soils having 12 percent or more fines content be grouped with fine grained soils for the purpose of interface friction coefficient selection.

The above results and observations lead to the following recommendations for selection of interface friction coefficient.

1. The appropriate value of μ for buried pipe design applications for Class I and Class II embedment (ASTM D2774, crushed rock and clean coarse-grained soil) is likely in the range 0.15 and 0.50.
2. The appropriate value of μ for buried pipe design applications for any class of soil other than Class I and Class II embedment is likely in the range 0.05 and 0.20.
3. The lower values in these ranges are recommended when the embedment material is expected to include significant fractured, angular, platy or elongated shaped particles.
4. The lower values in these ranges are recommended when normal stress is low.

Section IX. References

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Table 1. Manufacturer supplied properties for tested PE4710 Coupons.

Test	Nominal value	Units	Test method
Hydrostatic design basis, 73°F (23°C)	1600	psi	ASTM D 2837
Hydrostatic design basis140°F (60°C	1000	psi	ASTM D 2837
Minimum required strength, 68°F (20°C)	10	Mpa	ISO 12162
Creep rupture strength, 20°C, 12.4 MPa	> 200	hours	ASTM 1598
Resistance to rapid crack propagation, P_c @ 32°F[1]	>10	bar	ISO 13477
Resistance to rapid crack propagation, T_c @ 5 bar[1]	<20	ºF	ISO 13477
Notched pipe test, 80°C, 4.6 MPa[1]	>500	hours	ISO 13477
Hi Load Melt Index	7.0	g/10 min	ASTM D 1238
Melt Index	0.04	g/10 min	ASTM D 1238
Density	0.949	g/cc	ASTM D 1505
DSC Induction Temperature	250	°C	ASTM D 3350
2% Secant Modulus	146,000	psi	ASTM D 790
Tensile stress @ yield	3500	psi	ASTM D 638
Tensile stress @ break	5100	psi	ASTM D 638
Elongation @ break	800	%	ASTM D 638
Brittleness Temperature	<-76	°C	ASTM D 746
P ENT at 2.4 MPa and 80°C	>2000	hours	ASTM F 1473

1. Pipe diameter of 4 inches and SDR
2. Values were determined on natural resin.

Table 2. Soil Material Properties.

Pipe Soil Class	Common Name	Unified Soil Classification	Percent Fines	Percent Sand	Percent Gravel	Liquid Limit	Plastic Limit	Specific Gravity	Angularity	Particle Shape	Maximym Dry Unit Weight ASTM D698 (lb/ft^3)	Maximum Dry Unit Weight ASTM D7382 (lb/ft^3)	Minimum Dry Unit Weight ASTM D4254 (lb/ft^3)
I	Crushed Rock	Poorly Graded Gravel (GP)	1	2	97			2.53	Angular	20 percent flat;20 percent elongated	N/A	70.1	55.1
I	Crushed Rock Washed	Poorly Graded Gravel (GP)	0	0	100				Angular	20 percent flat;20 percent elongated	N/A	See Note 1 Below	
II	Density Sand	Poorly Graded Sand (SP)	0	100	0			2.62	Subangular to Angular	Approximately Equidimensional	N/A	124.8	92.2
II	Pea Gravel	Poorly Graded Gravel (GP)	0	trace	100			2.56	Subrounded to Subangular	Approximately Equidimensional	N/A	105.3	89.5
III	Silty Sand	Silty Sand (SM)	27	73	0	Nonplastic		2.60	N/A	N/A	114.8	102.0	82.3
IV	Silty Clay	Silty Clay (CL-ML)	97	3	0	22	16	2.71	N/A	N/A	105.7	N/A	N/A

Note 1: The minimum and maximum unit weights were not tested. The values of 70.1 lb/ft3 and 55.1 lb/ft3 measured for unwashed crushed rock are used to calculate percent compaction ar

Note 2: The values presented for Crushed Rock, Crushed Rock Washed, and Pea Gravel represent Saturated Surface Dry - Bulk Specific Gravity.

Table 3. Test conditions and results.

						Dry Unit Weight (lb/ft³)		Relative Density (ASTM D4254)		Percent Compaction		Water Content			Void Ratio			Peak Shear (lb/in²)				Residual Shear (lb/in²)			
Test No.	Stage (See Note 1)	Applied Normal Stress (lb/in²)	PE Coupon Number (See Note 4)	Soil Tested (See Note 2)	Compaction Effort (See Note 3)	Placement	Start of Shear	Placement	Start of Shear	Placement	Start of Shear	Placement	Start of Shear	End of Shear	Placement	Start of Shear	Shear Displacement Rate (in/min)	Corrected Shear Stress	Corrected Normal Stress	Displacement	Coefficient of Friction	Corrected Shear Stress	Corrected Normal Stress	Displacement	Coefficient of Friction
1	1	5	1	Density Sand	Minimal	93.0	94.0	3.3	7.2	74.5	75.3	0.29	0.28		0.76	0.74	0.002	1.5	5.2	0.23	0.28	1.5	5.2	0.27	0.28
	2	10					95.1						0.28			0.72	0.002	4.4	10.4	0.31	0.43	4.3	10.4	0.37	0.41
	3	20					95.5						0.27			0.71	0.002	9.7	20.8	0.31	0.46	9.3	21.0	0.49	0.44
	4	40					95.9						0.27	0.27		0.71	0.002	20.1	42.0	0.55	0.48	19.7	42.1	0.56	0.47
2		5	2	Density Sand	Minimal	94.2	94.7	8.0	10.0	75.5	75.9	0.28	0.28	0.27	0.74	0.73	0.002	1.2	5.2	0.28	0.23	1.2	5.2	0.28	0.23
3		10	3	Density Sand	Minimal	98.3	99.3	23.7	27.4	78.8	79.6	0.25	0.25		0.66	0.65	0.004	4.0	10.3	0.17	0.39	4.0	10.3	0.17	0.39
	1	20					99.7						0.24			0.64	0.004	9.5	20.5	0.24	0.46	9.3	20.6	0.27	0.45
	2	40					99.9						0.24	0.24		0.64	0.004	20.4	41.3	0.34	0.49	18.1	43.1	0.84	0.42
4		40	14	Density Sand	Minimal	97.8	99.8	22.1	29.0	78.4	79.9	0.26	0.24	0.24	0.67	0.64	0.004	17.9	40.8	0.20	0.44	17.8	41.1	0.28	0.43
18		5	14	Density Sand	Moderate	101.1	101.7	33.6	35.7	81.0	81.5	0.24	0.23	0.23	0.62	0.61	0.0015	1.0	5.1	0.05	0.20	1.1	5.4	0.61	0.20
5		20	5	Pea Gravel	Moderate	98.9	104.9	63.3	97.8	93.9	99.6	0.25	0.21	0.21	0.65	0.56	0.004	7.6	20.3	0.08	0.38	7.3	20.3	0.10	0.36
6		5	4	Pea Gravel	Moderate	101.4	101.9	78.3	81.2	96.3	96.8	0.23	0.23	0.23	0.61	0.60	0.008	2.4	5.2	0.20	0.45	2.4	5.7	1.17	0.42
7	1	1	7	Pea Gravel	Moderate	101.8	101.8	80.3	80.3	96.6	96.6	0.23	0.23	0.23	0.61	0.61	0.002	0.4	0.8	0.13	0.51	0.4	0.8	0.17	0.49
	2	40					102.9						0.23	0.22		0.59	0.04	22.7	41.1	0.28	0.55	19.2	41.8	0.48	0.46
9		10	6	Pea Gravel	Moderate	101.1	101.5	76.3	79.0	96.0	96.4	0.24	0.23	0.23	0.62	0.61	0.04	3.9	10.2	0.05	0.38	3.9	10.7	0.63	0.36
10		10	9	Silty Clay	Static Loading	87.1	98.8			82.4	93.5	0.37	0.27	0.26	0.93	0.70	0.00027	1.4	10.2	0.14	0.13	1.2	10.4	0.46	0.12
11		5	10	Silty Clay	Static Loading	80.8	88.2			76.5	83.4	0.37	0.28	0.27	1.08	0.91	0.00027	0.9	5.2	0.24	0.18	0.8	5.2	0.45	0.16
12		40	11	Silty Clay	Static Loading	85.3	100.3			80.7	94.8	0.36	0.25	0.24	0.98	0.68	0.00027	8.3	40.7	0.20	0.20	7.5	43.2	0.87	0.17
13		20	12	Silty Clay	Static Loading	85.3	98.0			80.7	92.7	0.37	0.27	0.25	0.98	0.72	0.00027	3.4	20.3	0.17	0.17	2.9	22.0	1.05	0.13
14		20	13	Crushed Rock	Moderate	67.8	70.0	87.4	99.4	96.7	99.8	0.53	0.50	0.49	1.33	1.26	0.0015	4.4	20.3	0.13	0.22	3.8	23.7	1.82	0.16
15		5	15	Crushed Rock	Moderate	66.5	67.5	80.3	86.1	94.9	96.3	0.54	0.53	0.53	1.37	1.34	0.0015	1.4	5.1	0.04	0.28	1.2	5.9	1.63	0.20
16		10	16	Crushed Rock	Moderate	65.7	65.7	75.6	75.6	93.8	93.8	0.55	0.55	0.55	1.40	1.40	0.0015	2.9	10.1	0.07	0.29	2.1	11.9	1.82	0.18
17		40	17	Crushed Rock	Moderate	67.0	69.3	83.1	95.5	95.6	98.8	0.54	0.51	0.48	1.36	1.28	0.0015	13.1	40.6	0.17	0.32	10.0	48.1	2.00	0.21
19		10	19	Washed Crushed Rock	Moderate	67.8	68.1	87.8	89.3	96.8	97.2	0.52	0.52	0.52	1.33	1.32	0.0015	2.4	10.1	0.08	0.23	1.7	10.7	0.71	0.16
20		20	20	Silty Sand	Static Loading	96.0	107.3			83.7	93.5	0.22	0.21	0.20	0.75	0.57	0.0015	3.0	20.2	0.08	0.15	2.3	23.4	1.71	0.10
21		10	21	Silty Sand	Static Loading	102.1	107.5			88.9	93.6	0.24	0.21	0.20	0.65	0.57	0.0015	1.3	10.1	0.08	0.13	0.5	11.6	1.60	0.05
22		5	22	Silty Sand	Static Loading	92.1	101.8			83.5	88.7	0.18	0.24	0.23	0.83	0.57	0.0015	0.7	5.1	0.07	0.14	0.3	5.8	1.63	0.06
23		40	23	Silty Sand	Static Loading	96.1	110.1			83.5	95.9	0.21	0.13	0.13	0.57	0.35	0.0012	7.1	40.4	0.12	0.17	6.3	45.5	1.44	0.14

Note 1. In some instances the normal stress was increased in a stepwise fashion during the test. Each applied normal stress is assigned a number indicating the order of the staged process.

Note 2. See Table 1 for materal descriptions.

Note 3. "Static Loading" is a compaction method that entails placing the test specimen at a moisture content near the liquid limit the applying vertical pressure that compacts the specimen. Pressure was applied in incements that doubled beginning with 5 psi.

Note 4. Each PE coupon tested was assigned a unique number.

Table 4. Comparison of interface friction coefficients.

				This Study				Koerner, 2005 (adhesion=0, scaled from plots)				ASCE 2014 See Note 1	Alam et.al. - 2013 (See Note 2)	
ASTM D2774	ASTM D2487		Common Name	μ_p		μ_r		μ_p		μ_r		μ_r	μ (Peak)	
Soil Class	Symbol	Group Name	Material Tested	High	Low	High	Low	High	Low	High	Low		High	Low
I	GP	Poorly Graded Gravel	Crushed Rock w trace of clay fines	0.35	0.22	0.20	0.16	0.70	0.24	0.52	0.15	N/A	N/A	N/A
II	GP	Poorly Graded Gravel	Pea Gravel	0.55	0.30	0.43	0.31					0.25	0.53	0.36
	SP	Poorly Graded Sand	Density Sand 16-30	0.49	0.22	0.47	0.16					0.25	0.53	0.25
III	SM	Silty Sand	Silty Sand	0.18	0.13	0.14	0.05					0.20	N/A	N/A
IV	CL-ML	Silty Clay	Silty Clay	0.20	0.13	0.18	0.13	0.72	0.19	0.33	0.09	0.10	0.39	0.49
Note 1. ASCE Recommended Values (McCabe 2014). Data was not provided in reference.														
Note 2. Results of 13 pullout tests on PE wrapped steel pipe. Values back calculated using AWWA M-11 equation 13-6.														

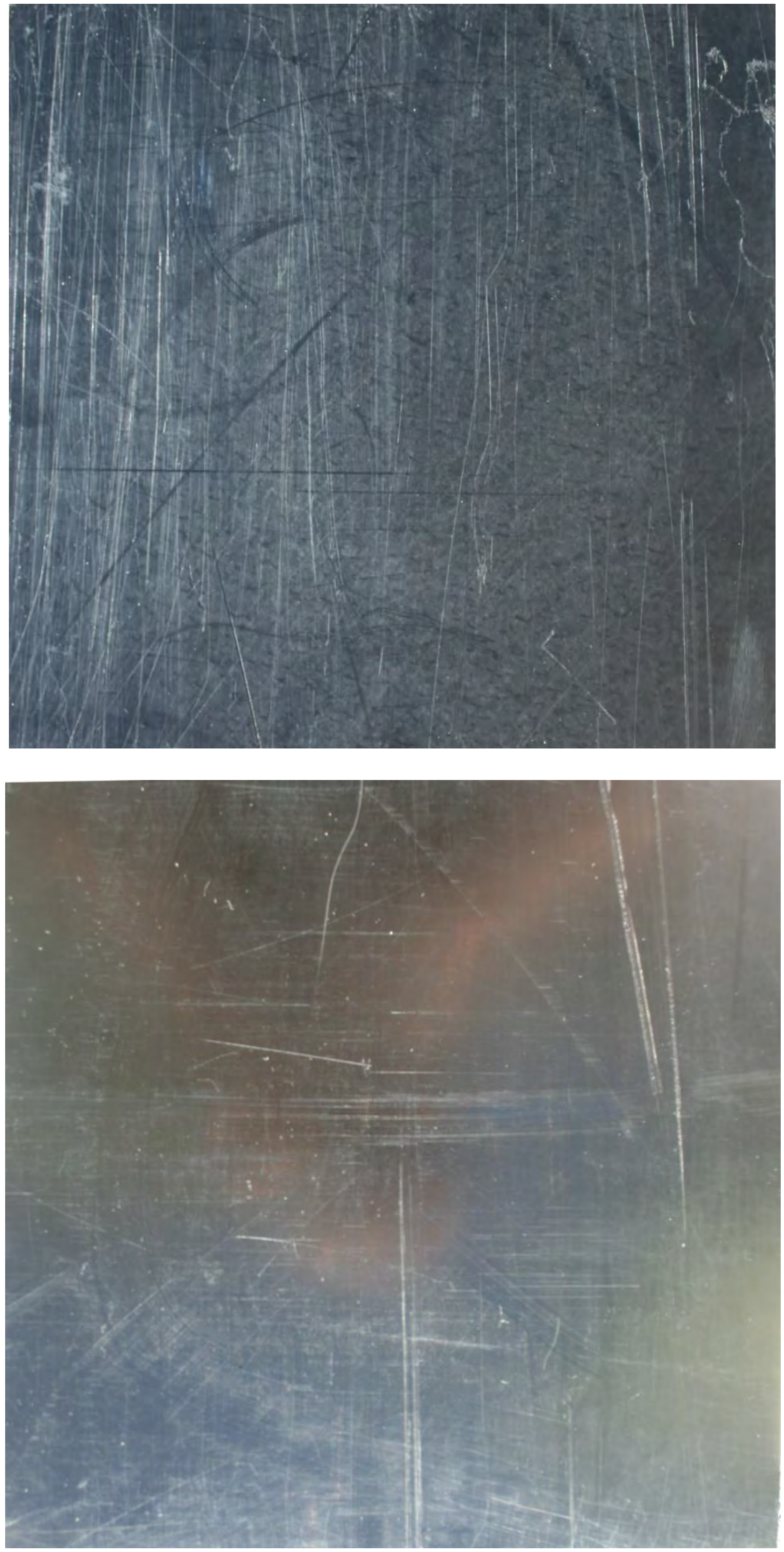


Figure 1. Photograph of PE4710 Test Coupons before 2-inch shear displacement at 20 lb/in^2 normal stress. Pea Gravel (Top), Crushed Rock (Bottom).

Figure 2. Photographs of Crushed Rock. Upper and Lower photographs are images of the same sample.

Figure 3. Image of Pea Gravel.

Figure 4. Image of Density Sand.

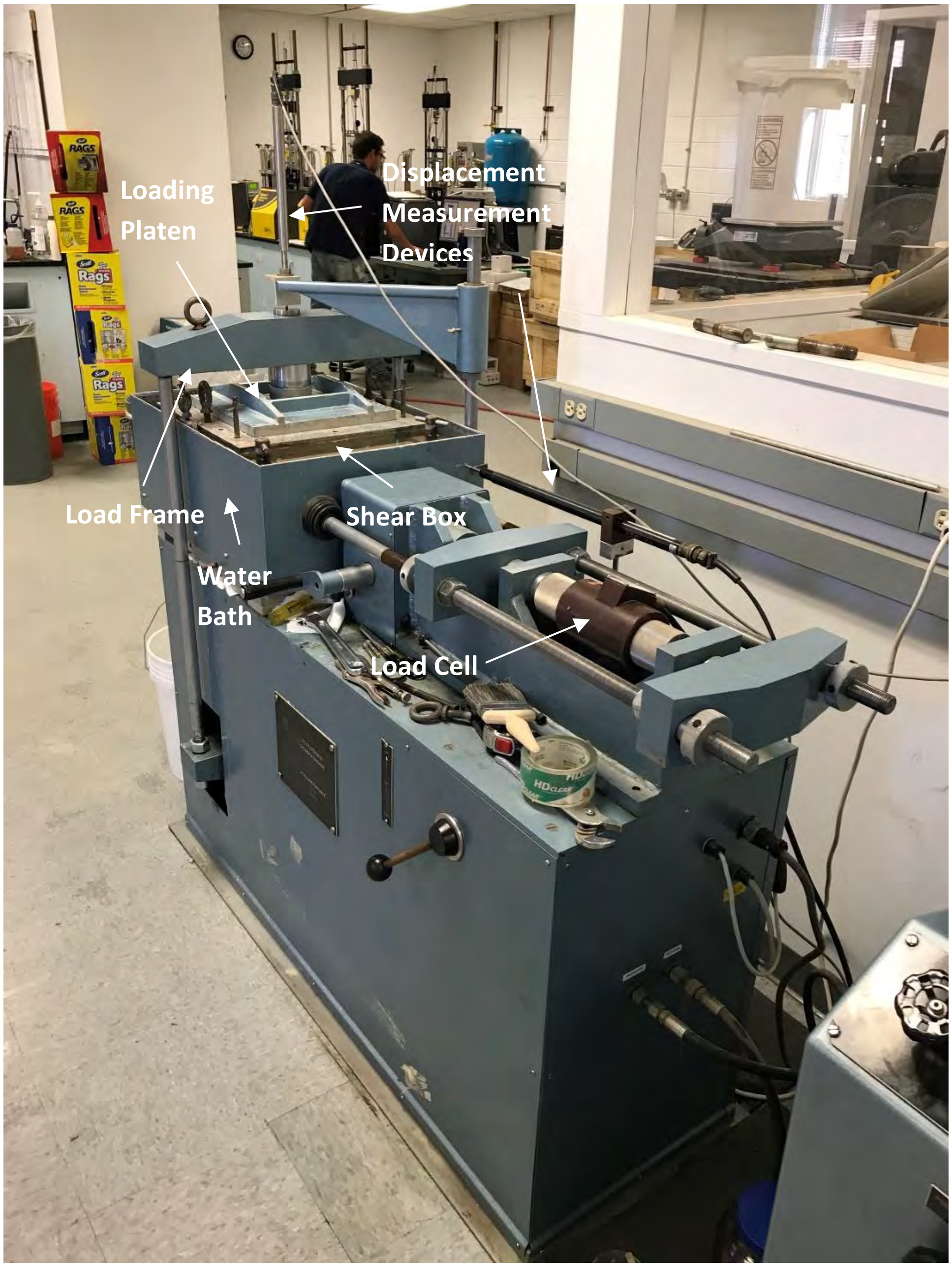


Figure 5. Image of the direct shear test machine.

Figure 6. Image of PE4710 HDPE test coupon positioned in lower shear box.

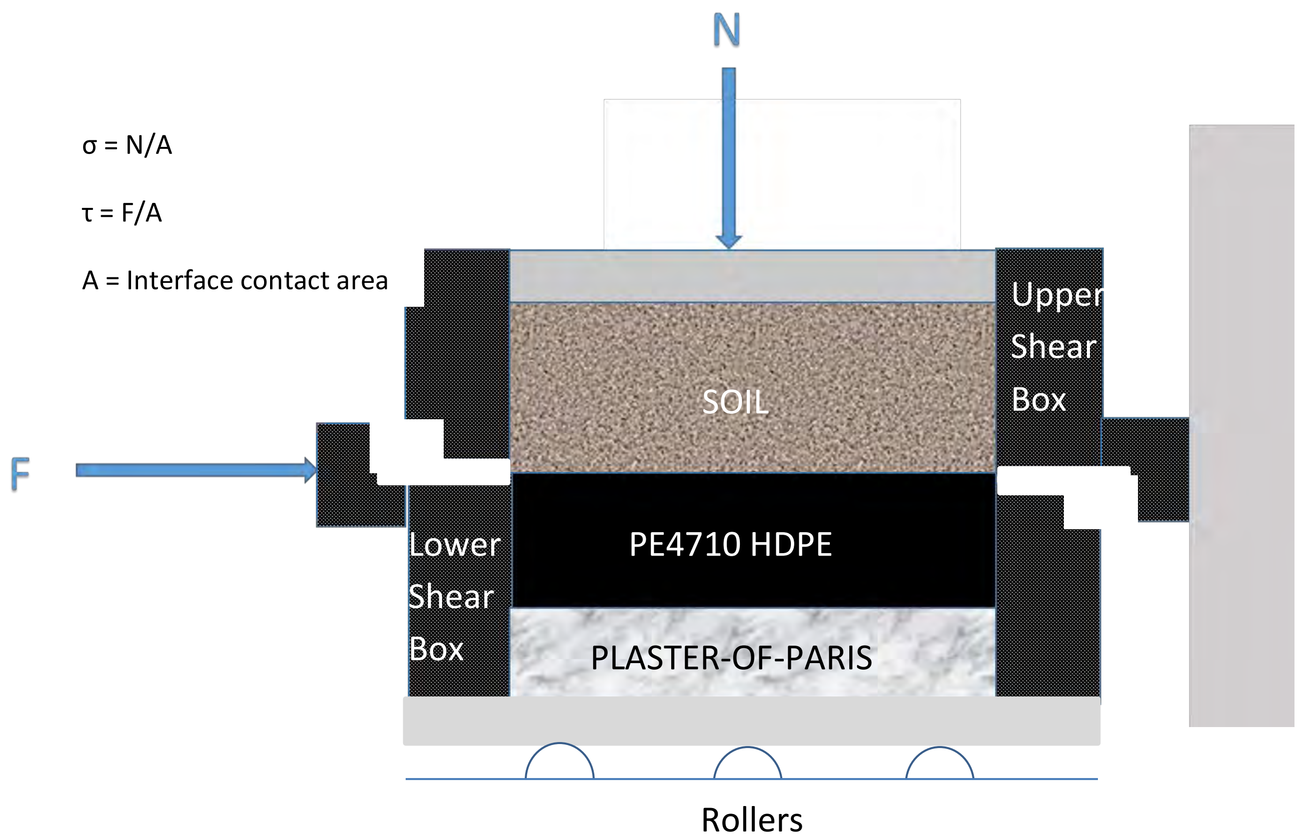


Figure 7. General schematic of the direct shear test setup.

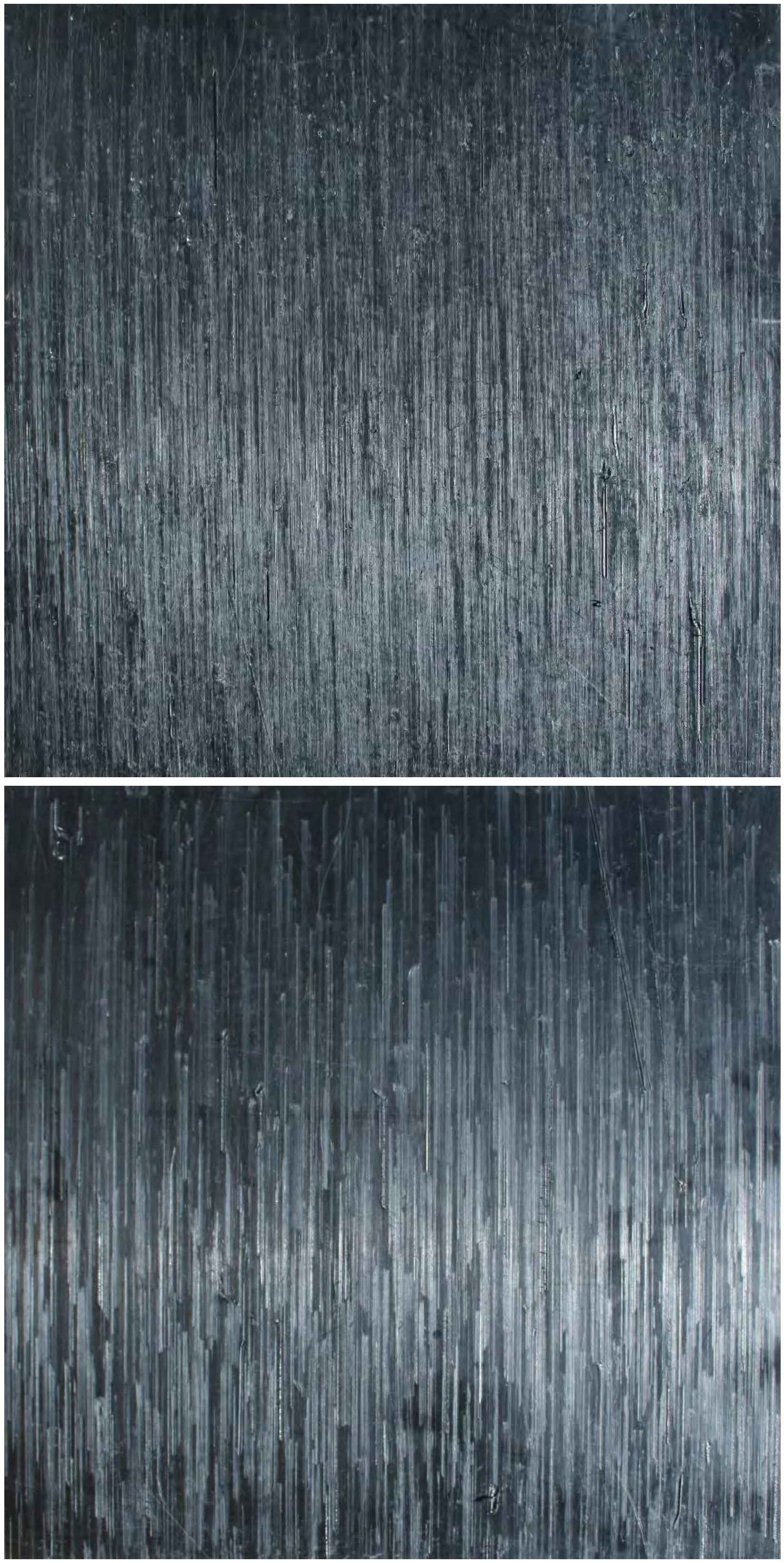


Figure 8. Photograph of PE4710 Test Coupons following 2-inch shear displacement at 20 lb/in^2 normal stress. Pea Gravel (Top), Crushed Rock (Bottom). Contrast with Figure 1.

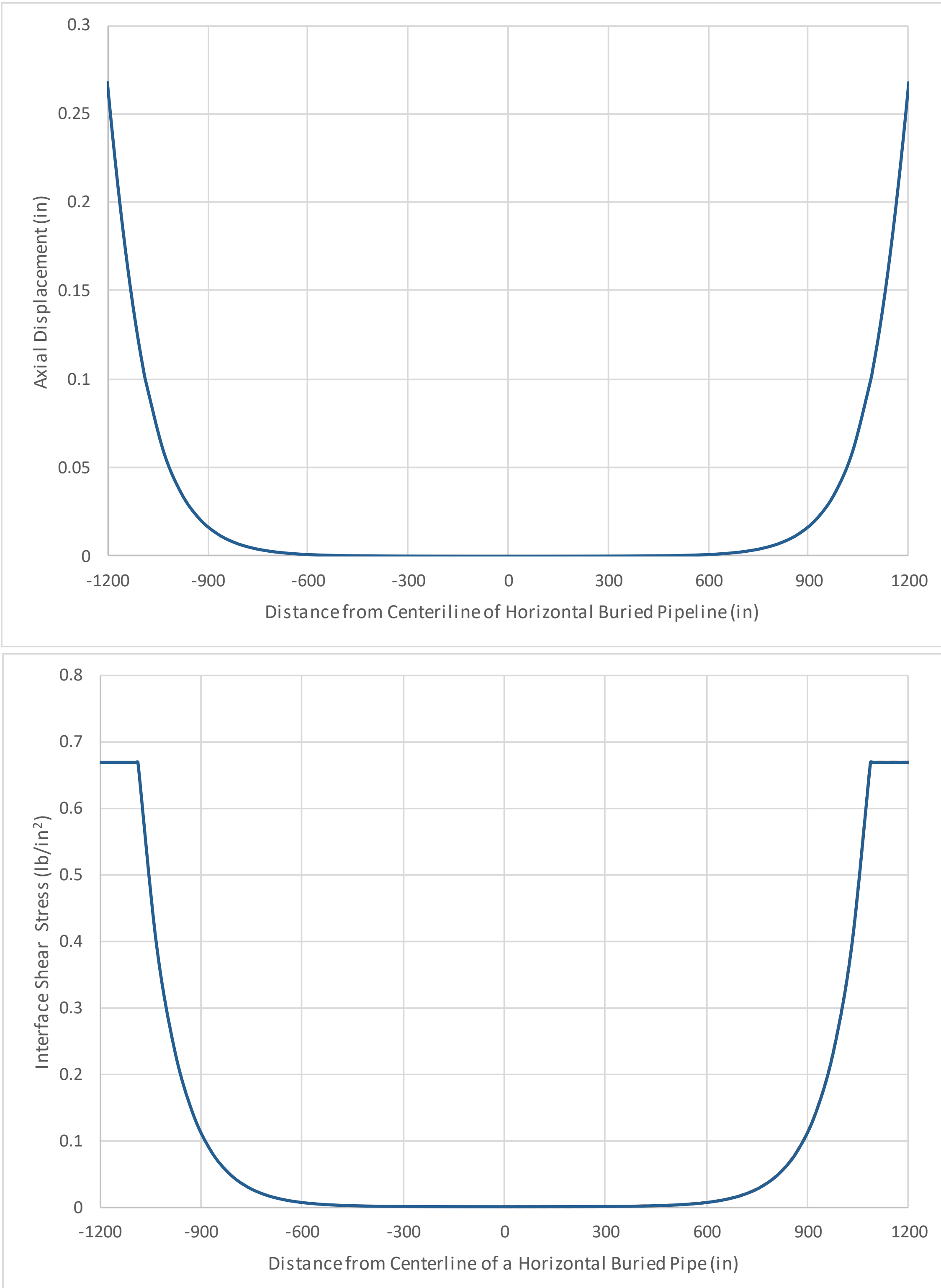
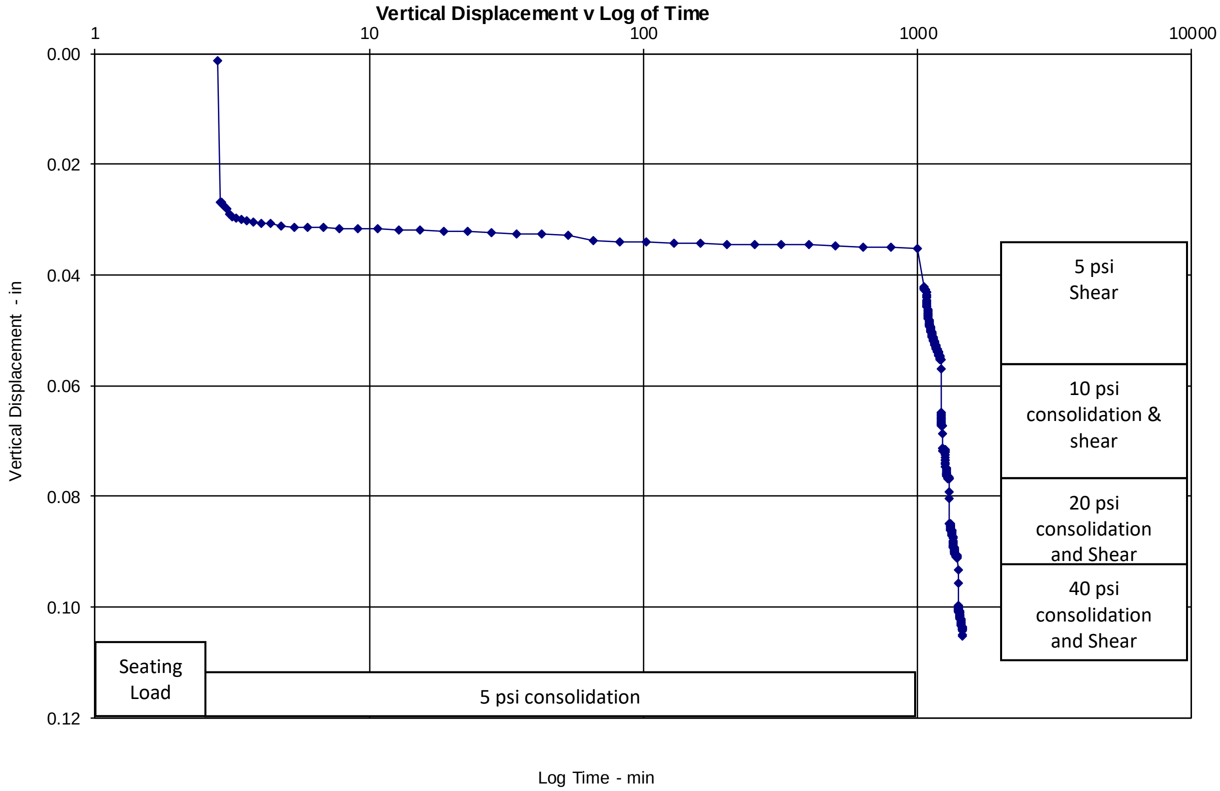
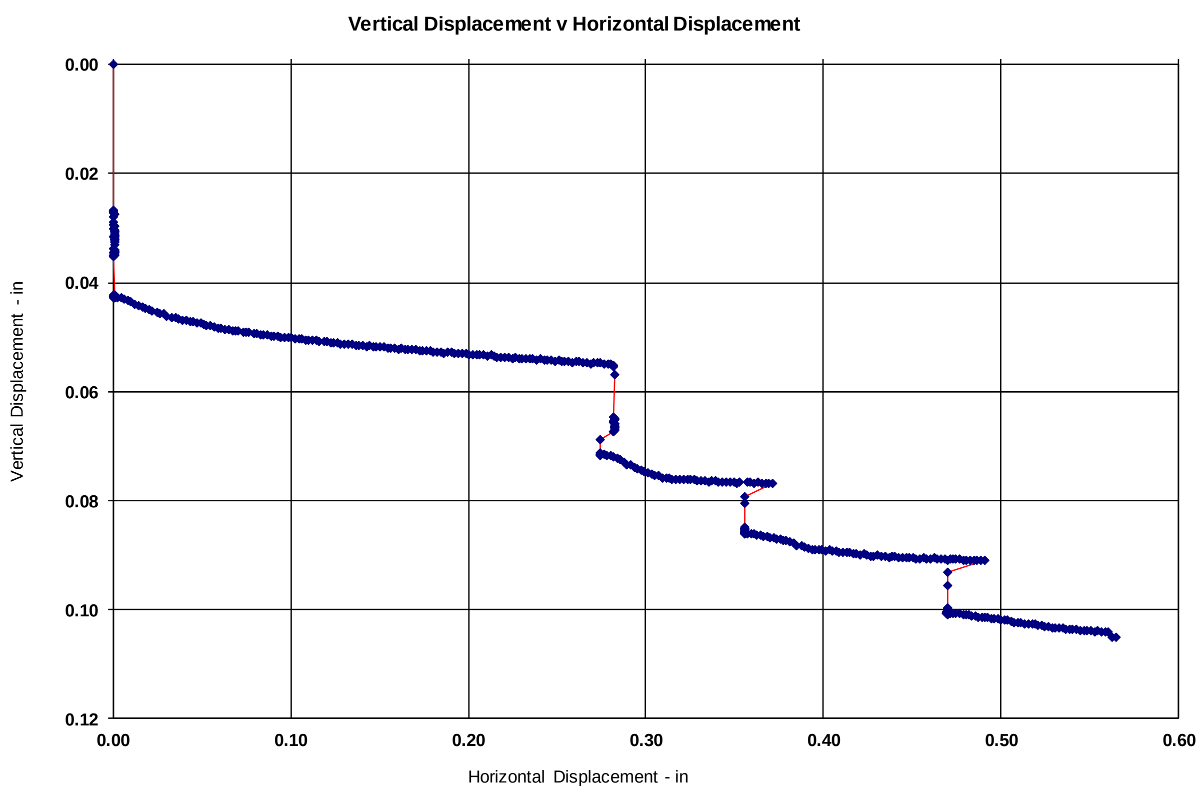


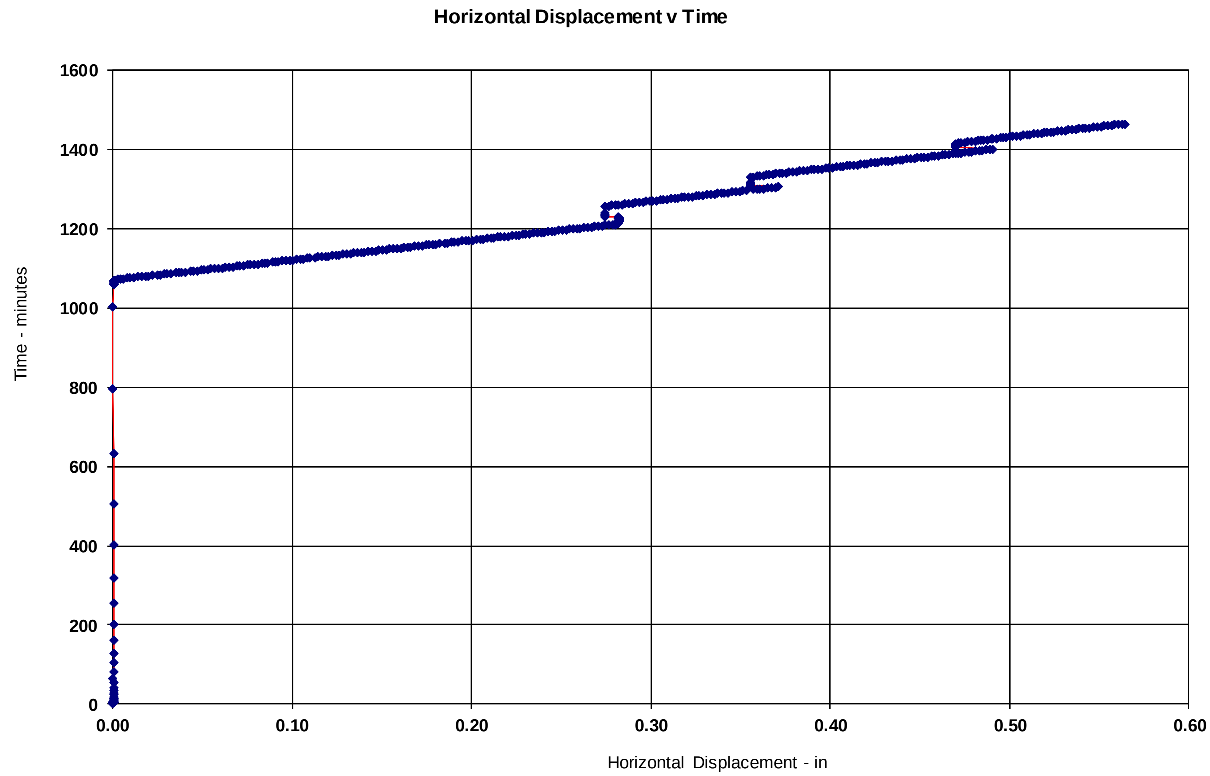
Figure 9. A conceptual representation of the change in axial displacement and interface shear stress with increased distance from the centerline of a long horizontally buried pipeline due to a change in temperature or pressure (Note: shear stress distribution theoretically differs with changes in pipe dimensions, pipe material, embedment, cover, and magnitude of temperature and/or pressure change.)

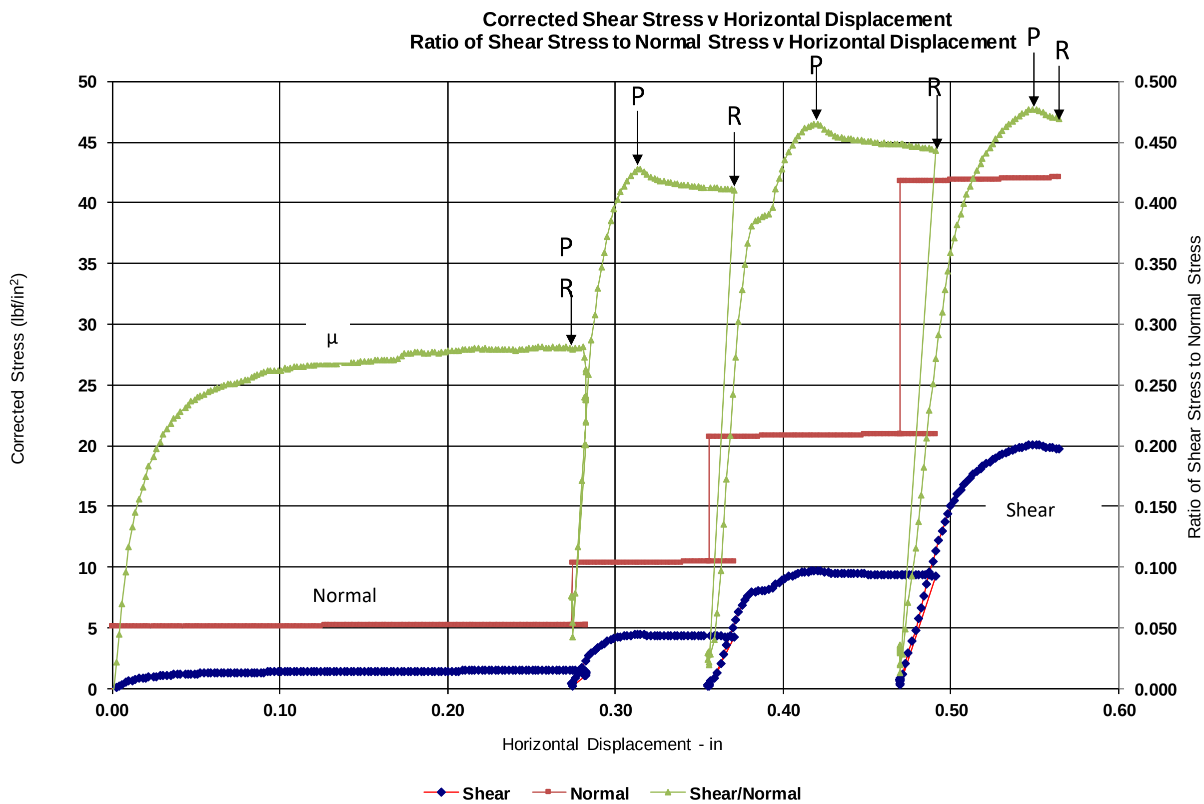
Test 1- Normal Stress = 5, 10, 20, 40 lb/in^2. Density Sand, POORLY GRADED SAND (SP), 100 percent hard, medium-size, quartz sand. Particles break with moderate hammer blow. Interface Coefficients of Friction: μ_{p5} = 0.28, μ_{p10} = 0.43, μ_{p20} = 0.46, μ_{p40} = 0.48; μ_{r5} = 0.28, μ_{r10} = 0.41, μ_{r20} = 0.44, μ_{r40} = 0.47.



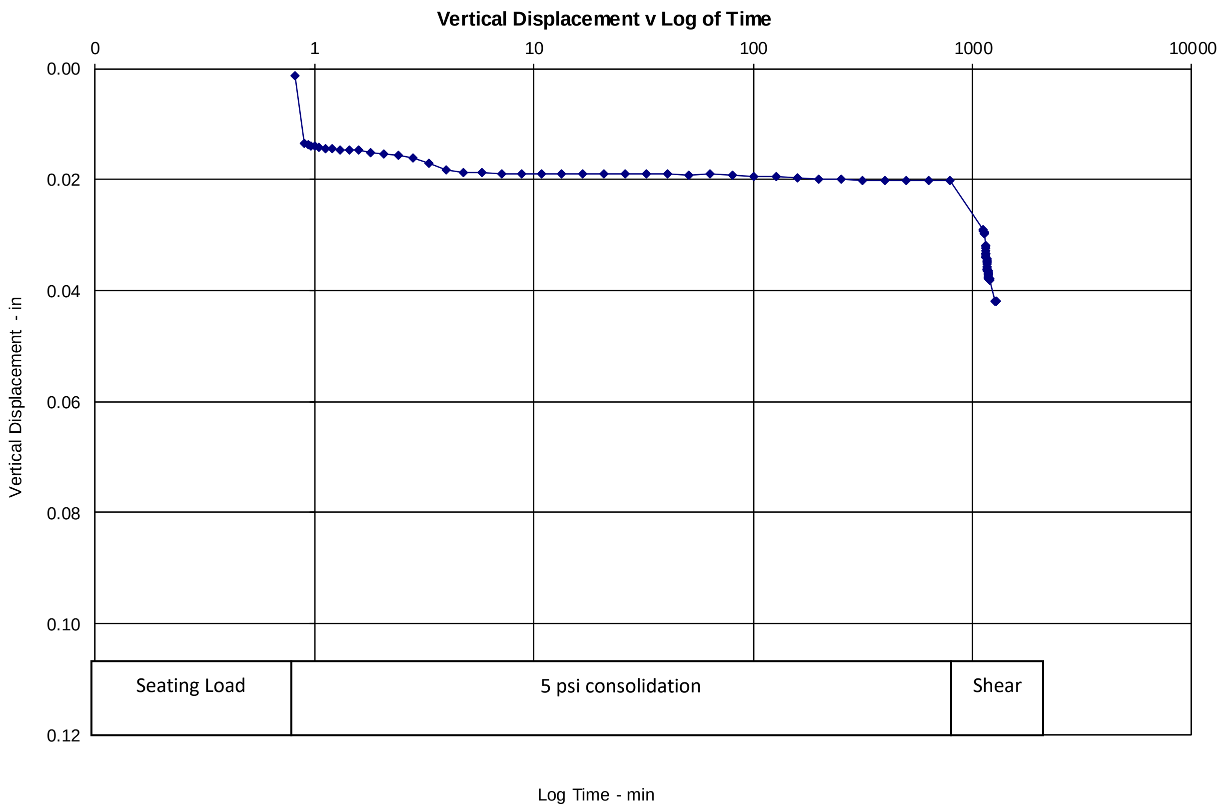


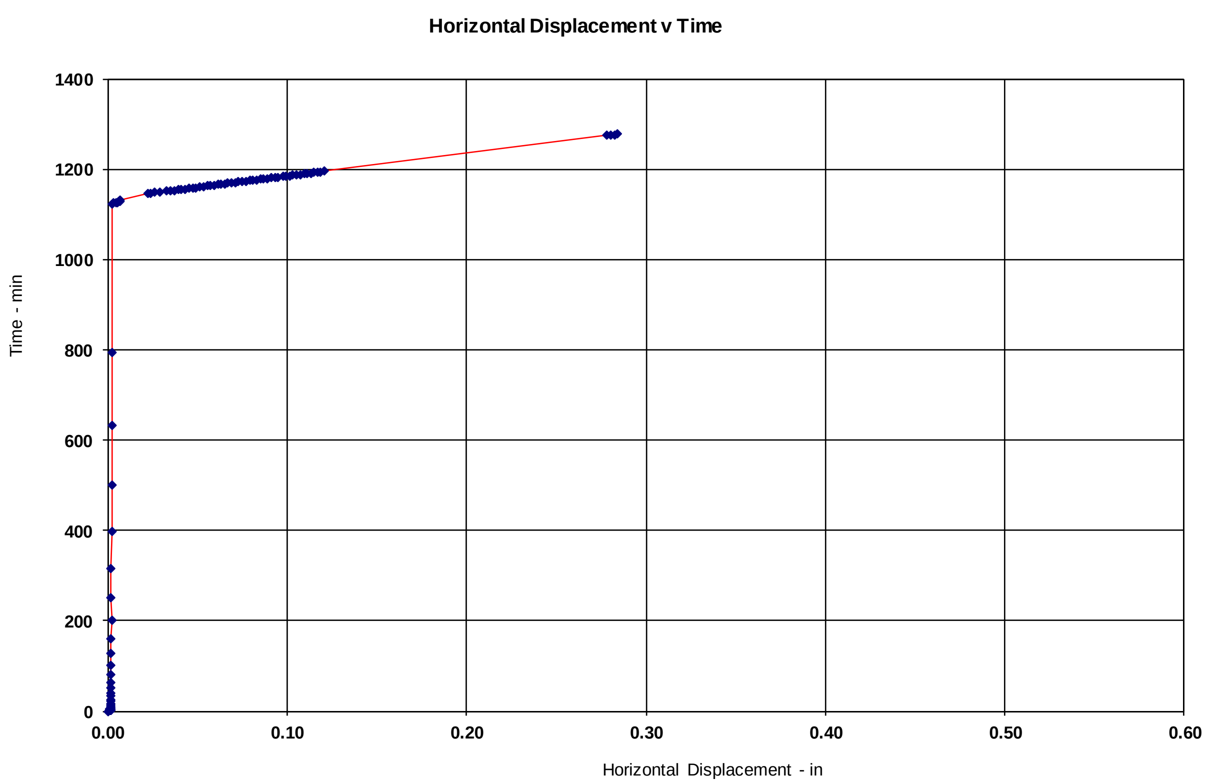
Test 1- Normal Stress = 5, 10, 20, 40 lb/in^2. Density Sand, POORLY GRADED SAND (SP), 100 percent hard, medium-size, quartz sand. Particles break with moderate hammer blow. Interface Coefficients of Friction: μ_{p5} = 0.28, μ_{p10} = 0.43, μ_{p20} = 0.46, μ_{p40} = 0.48; μ_{r5} = 0.28, μ_{r10} = 0.41, μ_{r20} = 0.44, μ_{r40} = 0.47.



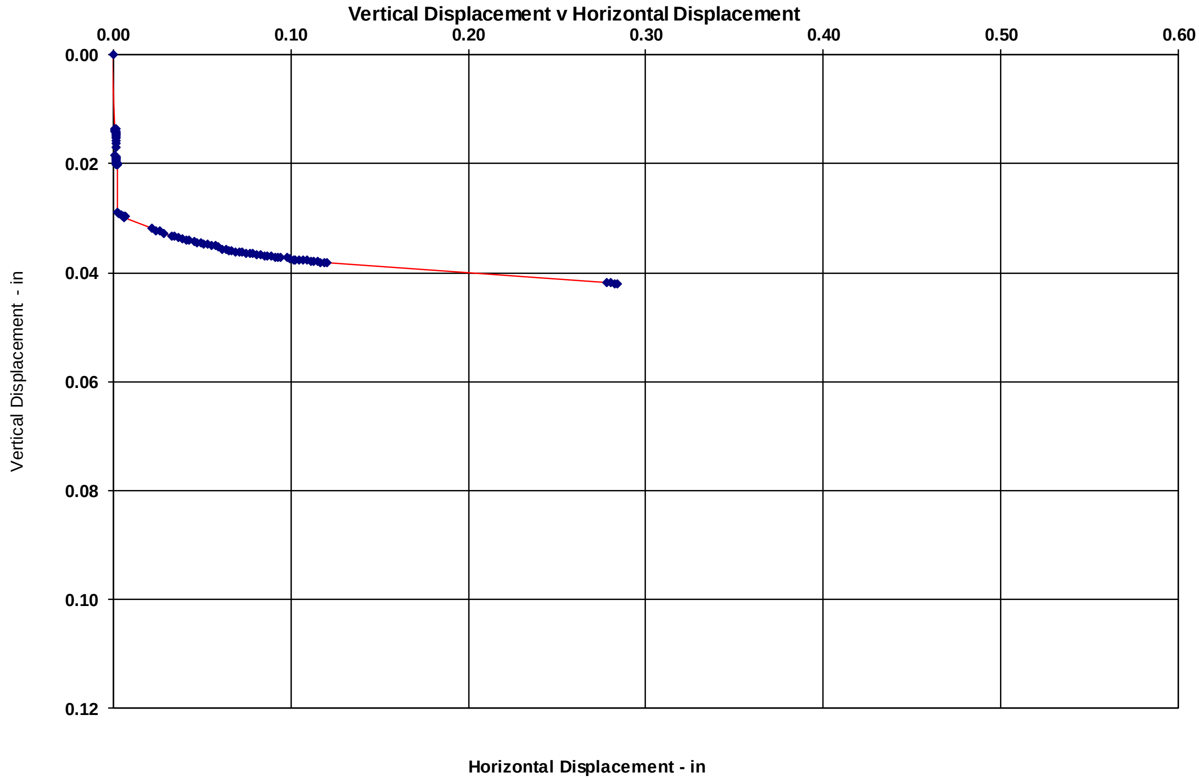


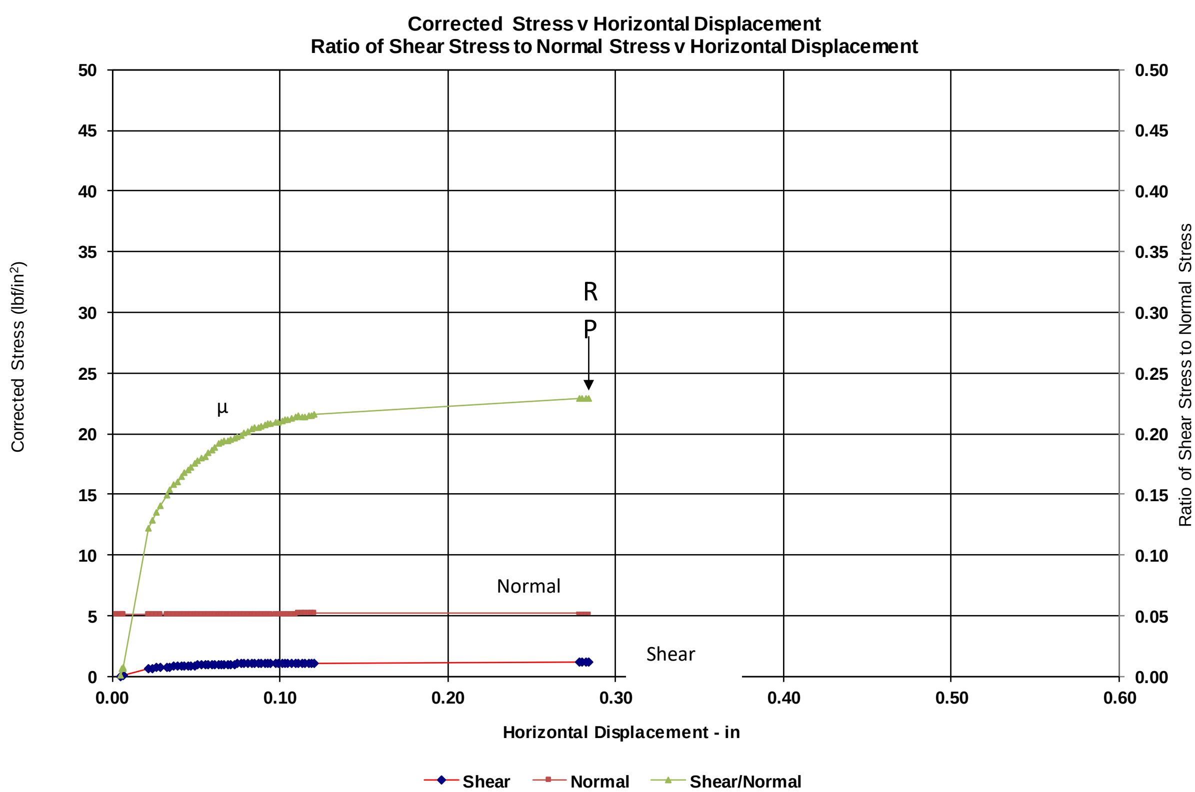
Test 2 Normal Stress = 5 lb/in^2. Density Sand, POORLY GRADED SAND (SP), 100 percent hard, medium-size, quartz sand. Particles break with moderate hammer blow. Interface Coefficients of Friction: μ_p = 0.23, μ_r = 0.23.



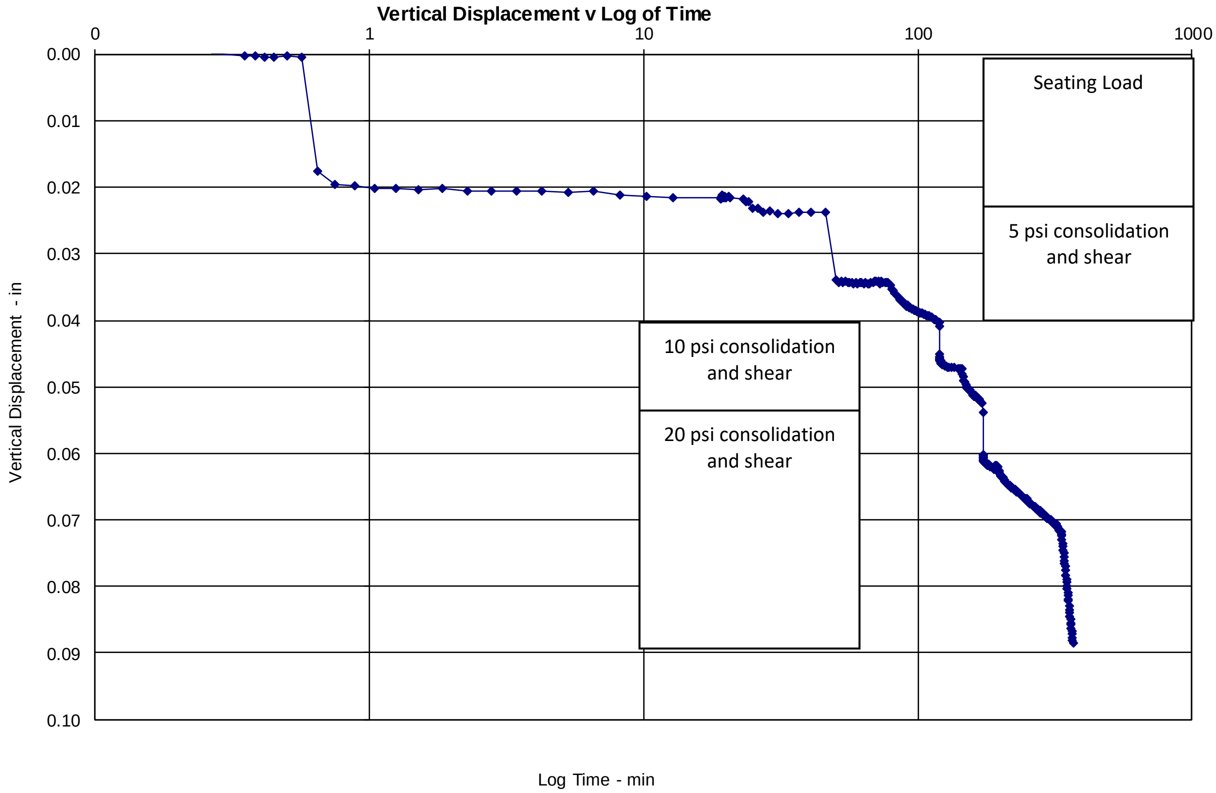


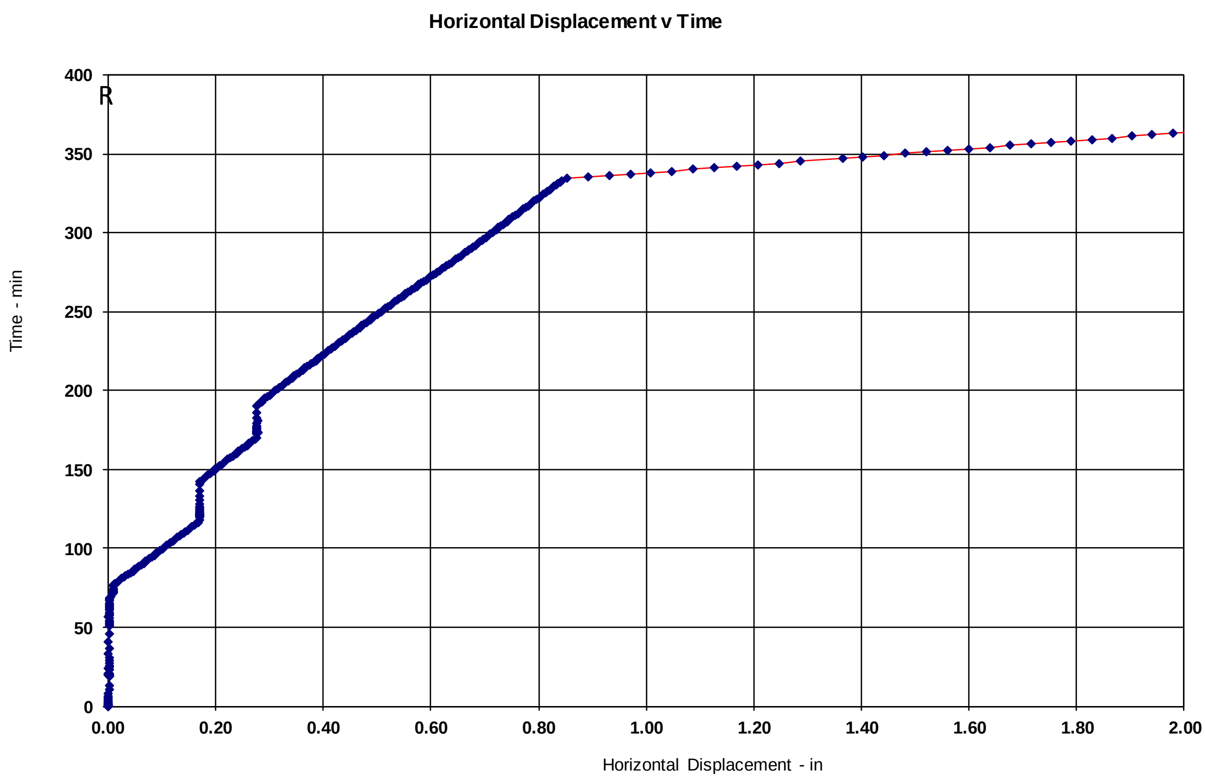
Test 2 Normal Stress = 5 lb/in^2. Density Sand, POORLY GRADED SAND (SP), 100 percent hard, medium-size, quartz sand. Particles break with moderate hammer blow. Interface Coefficients of Friction: μ_p = 0.23, μ_r = 0.23.



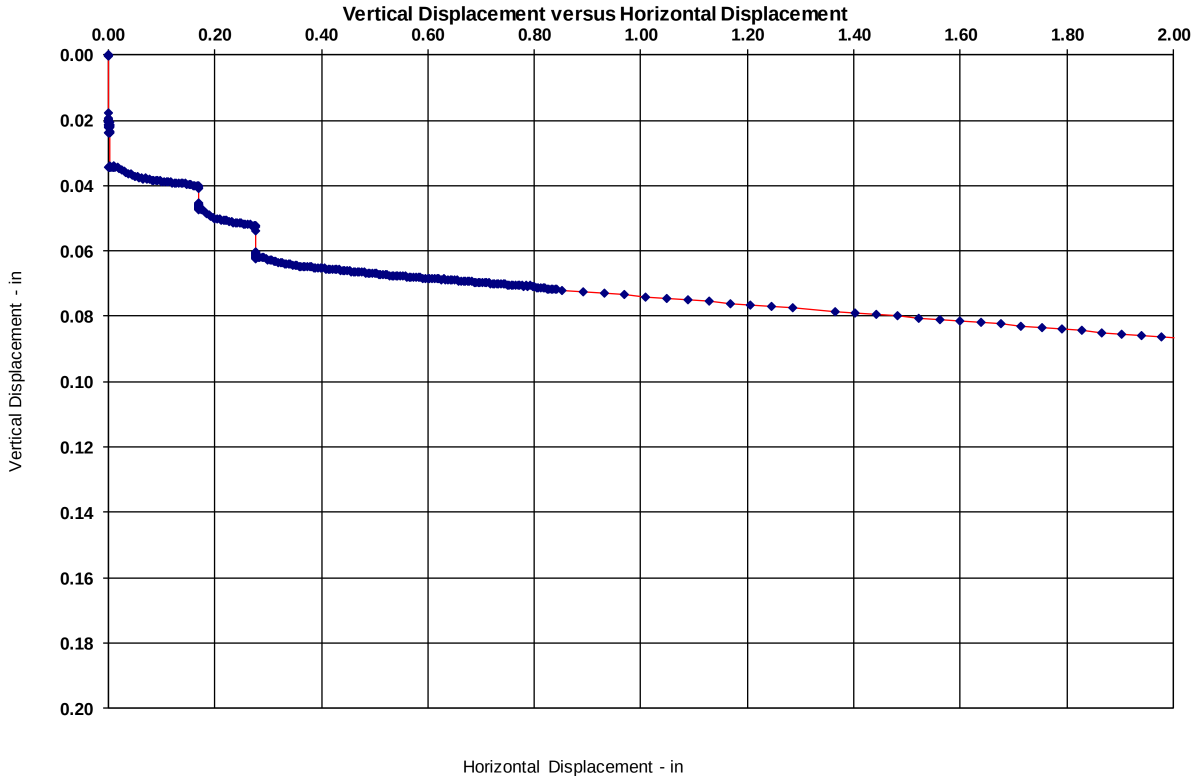


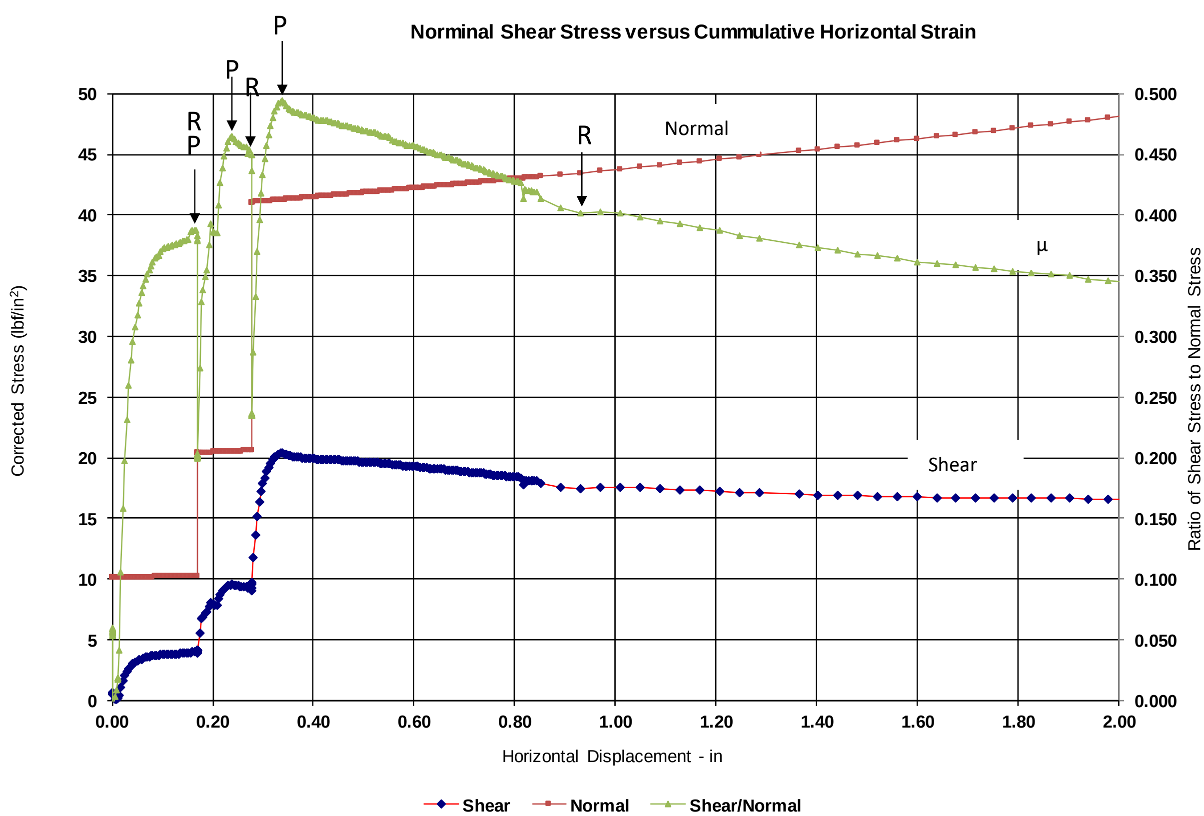
Test 3 - Normal Stress = 5, 10, 20 lb/in^2. Density Sand, POORLY GRADED SAND (SP), 100 percent hard, medium-size, quartz sand. Particles break with moderate hammer blow. Interface Coefficients of Friction: μ_{p5} = 0.39, μ_{p10} = 0.46, μ_{p20} = 0.49; μ_{r5} = 0.39, μ_{r10} = 0.45, μ_{r20} = 0.42.



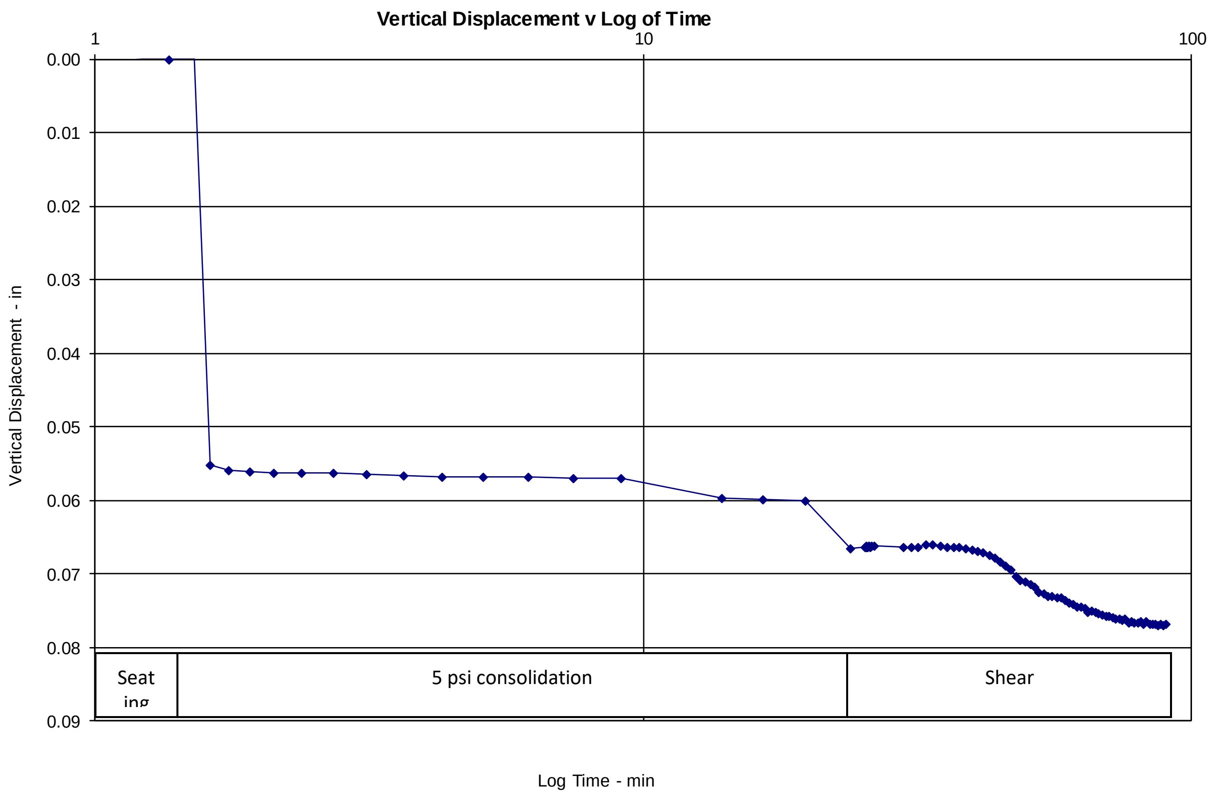


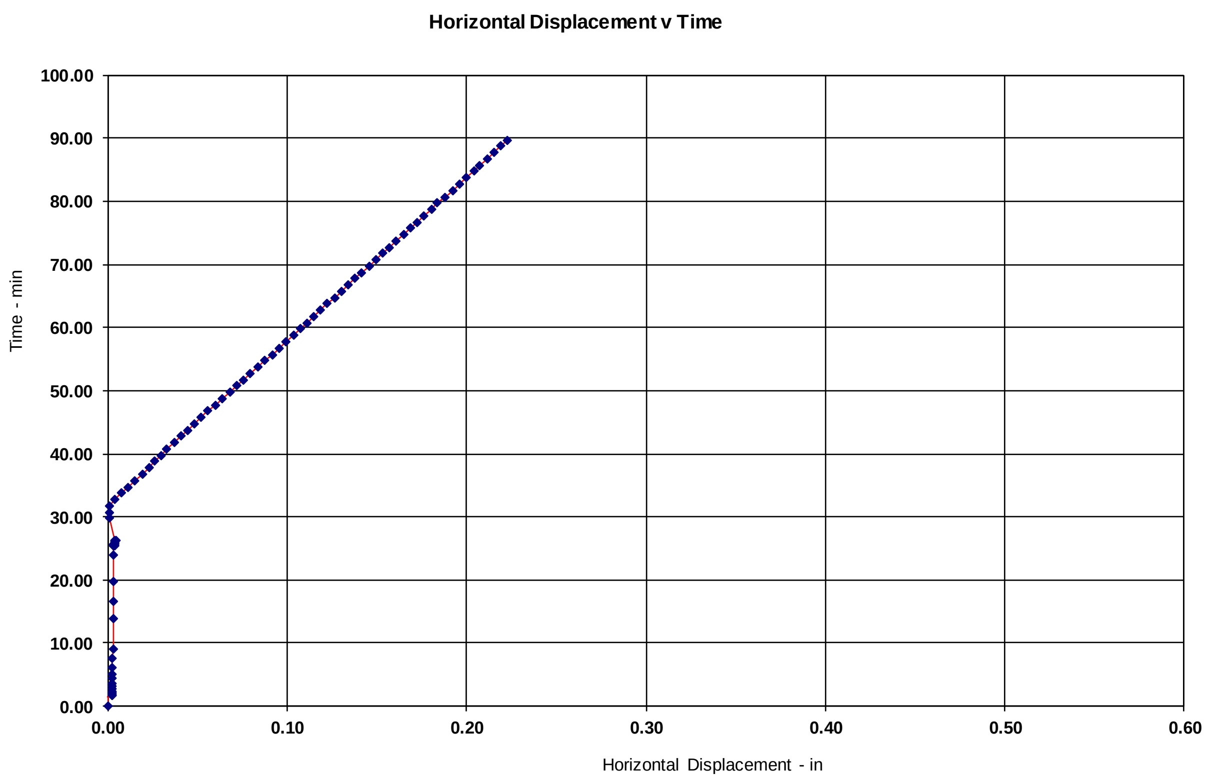
Test 3 - Normal Stress = 5, 10, 20 lb/in^2. Density Sand, POORLY GRADED SAND (SP), 100 percent hard, medium-size, quartz sand. Particles break with moderate hammer blow. Interface Coefficients of Friction: μ_{p5} = 0.39, μ_{p10} = 0.46, μ_{p20} = 0.49; μ_{r5} = 0.39, μ_{r10} = 0.45, μ_{r20} = 0.42.



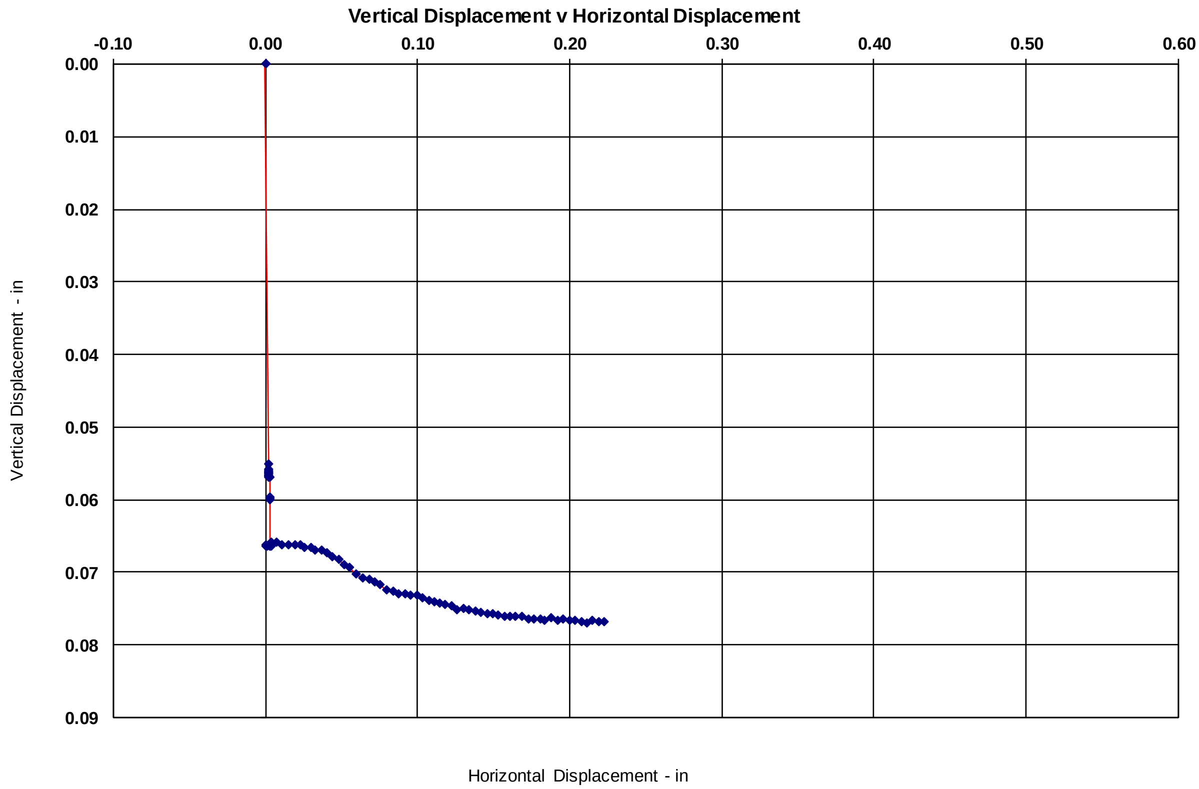


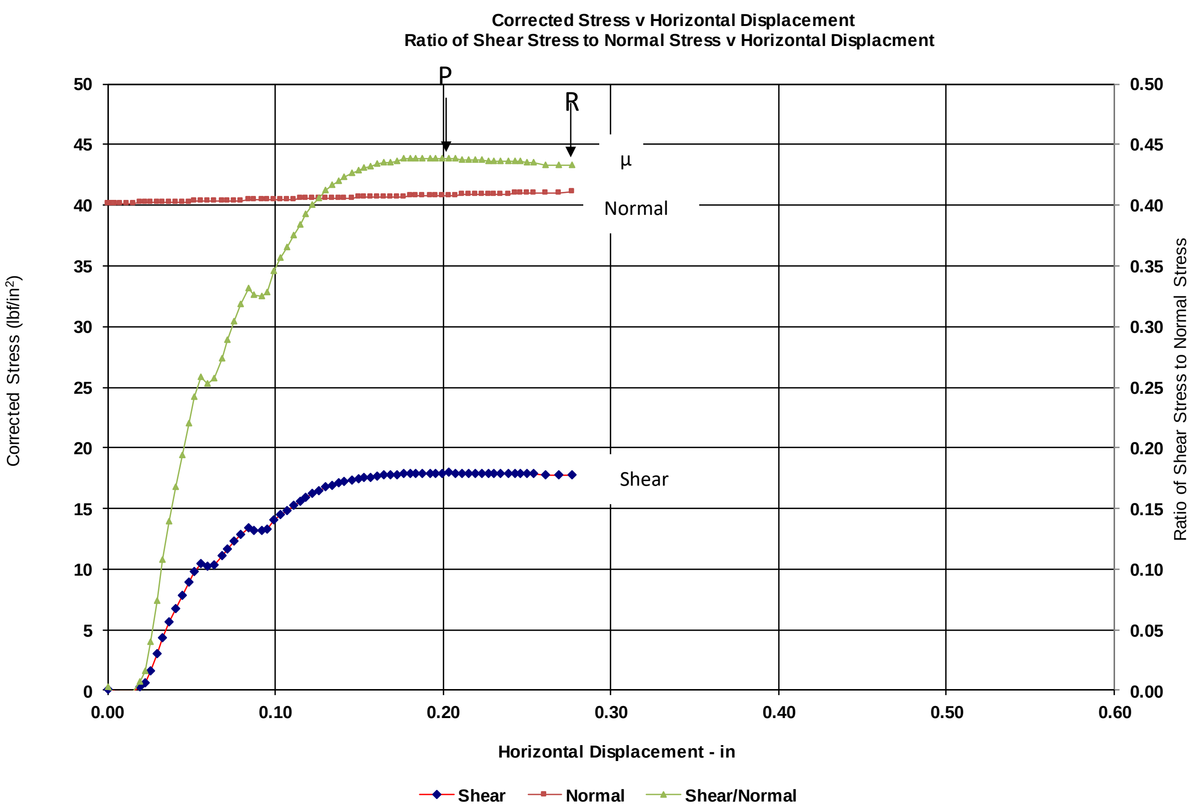
Test 4 - Normal Stress = 40 lb/in^2. Density Sand, POORLY GRADED SAND (SP), 100 percent hard, medium-size, quartz sand. Particles break with moderate hammer blow. Interface Coefficients of Friction: $\mu_p = 0.44$, $\mu_r = 0.43$.



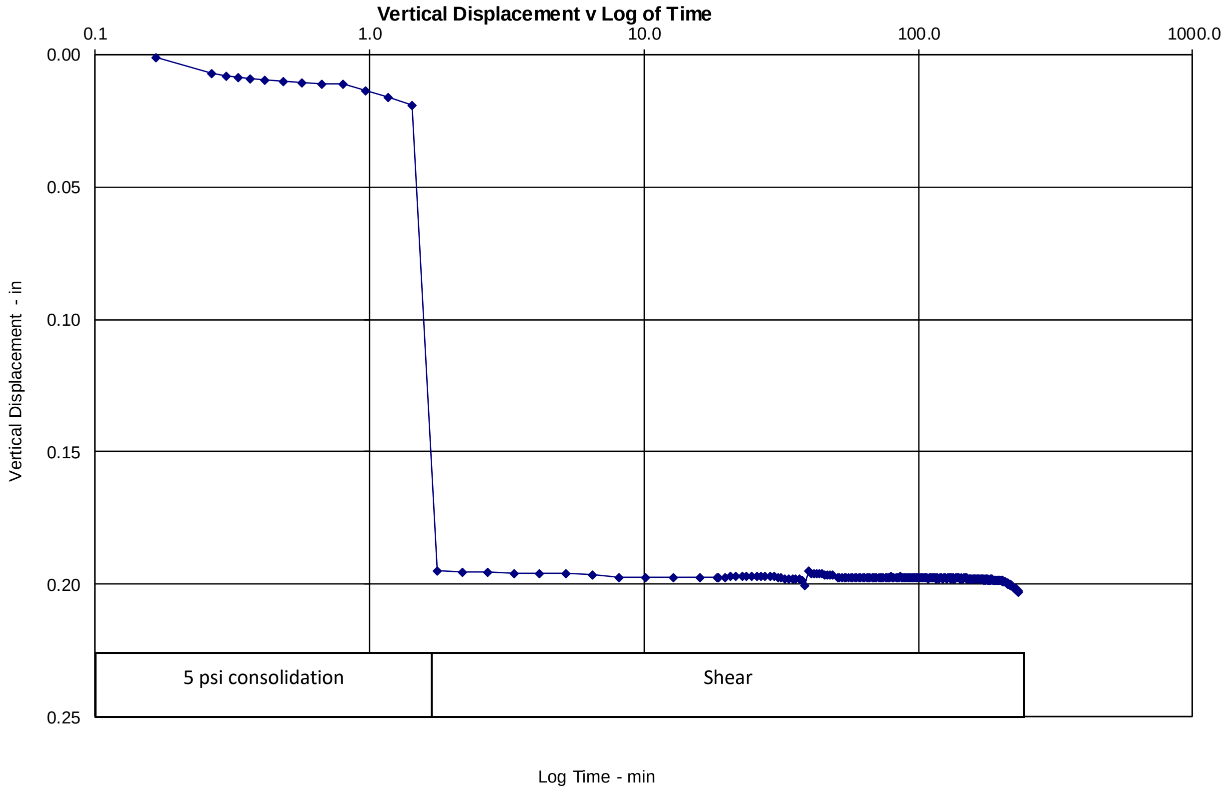


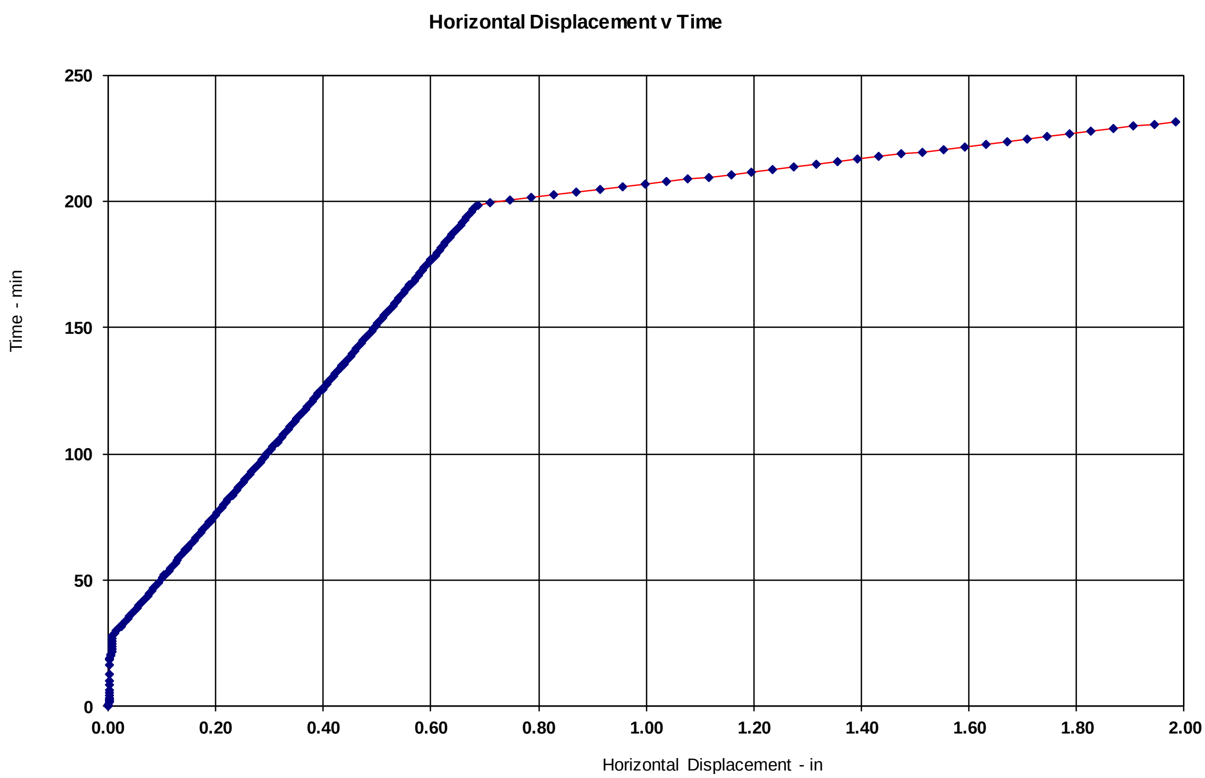
Test 4 - Normal Stress = 40 lb/in^2. Density Sand, POORLY GRADED SAND (SP), 100 percent hard, medium-size, quartz sand. Particles break with moderate hammer blow. Interface Coefficients of Friction: μ_p = 0.44, μ_r = 0.43.



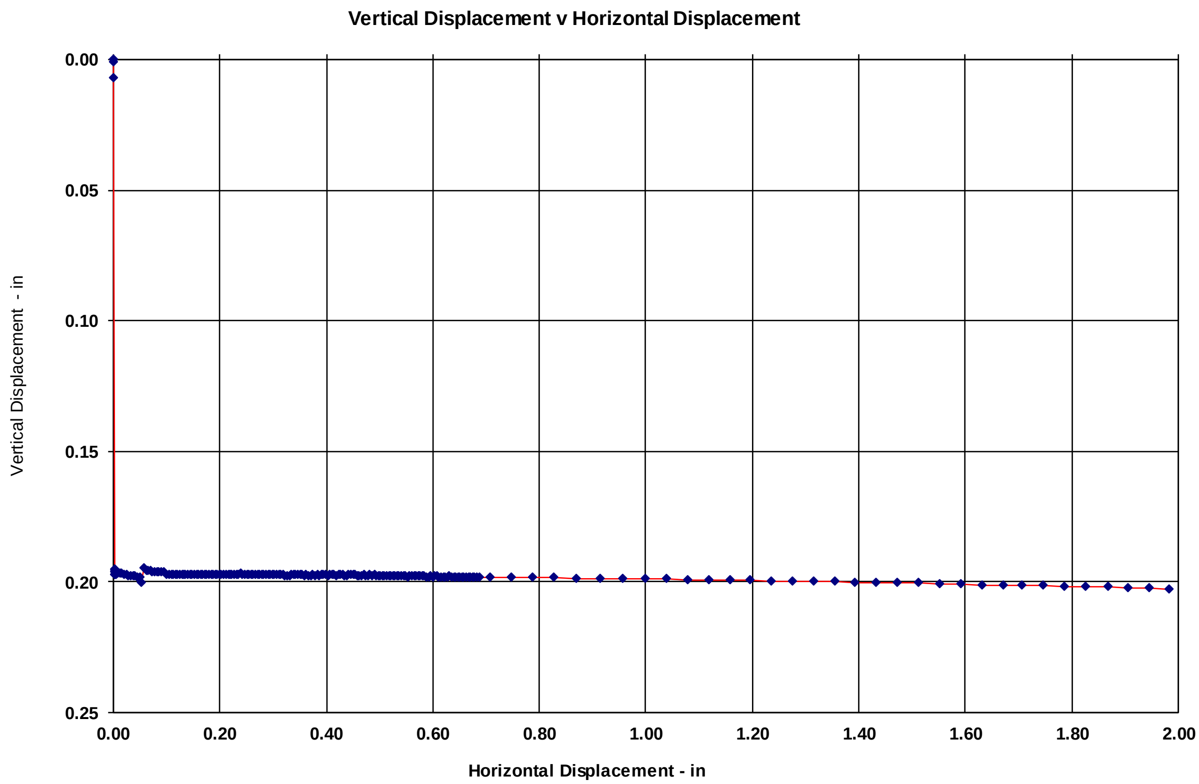


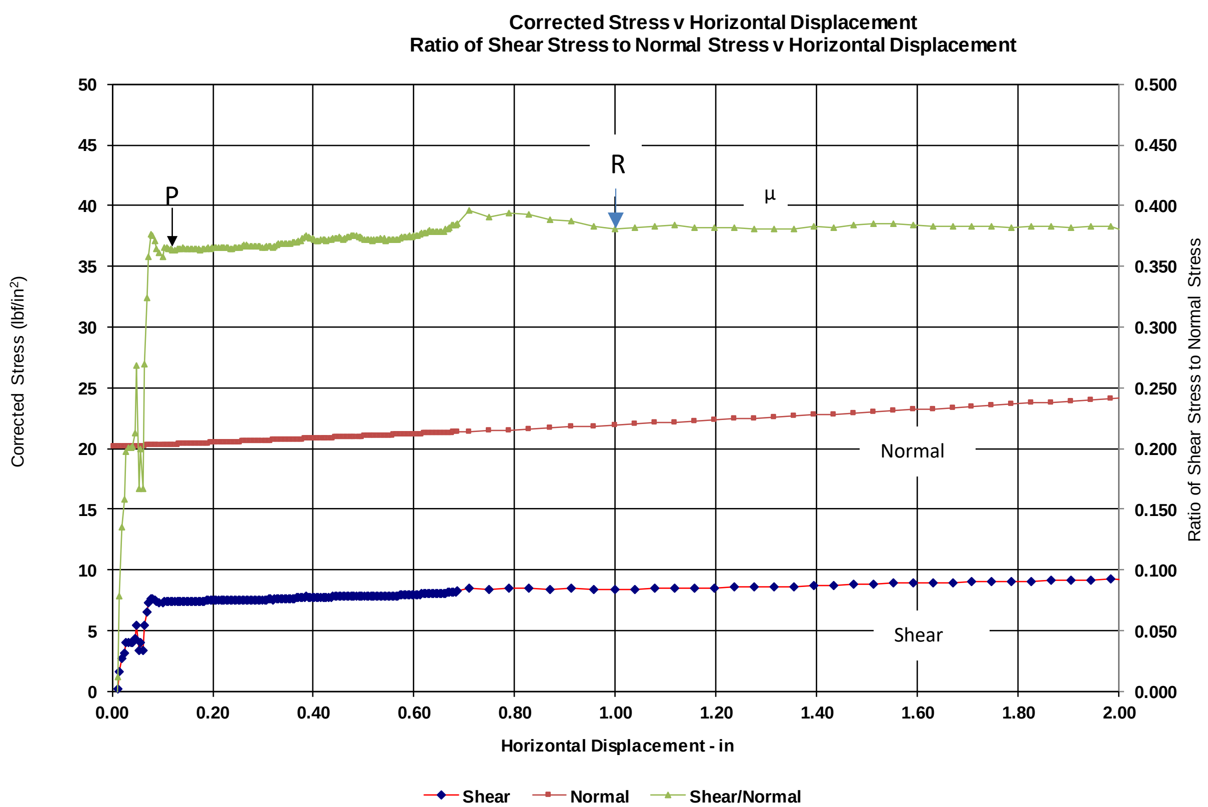
Test 5 - Normal Stress = 20 lb/in^2. Pea Gravel, POORLY GRADED GRAVEL (SP), 100 percent hard, subrounded to subangular gravel. Maximum particle size ½-inch. Particles break with moderate hammer blow. Interface Coefficients of Friction: μ_p = 0.38, μ_r = 0.36.



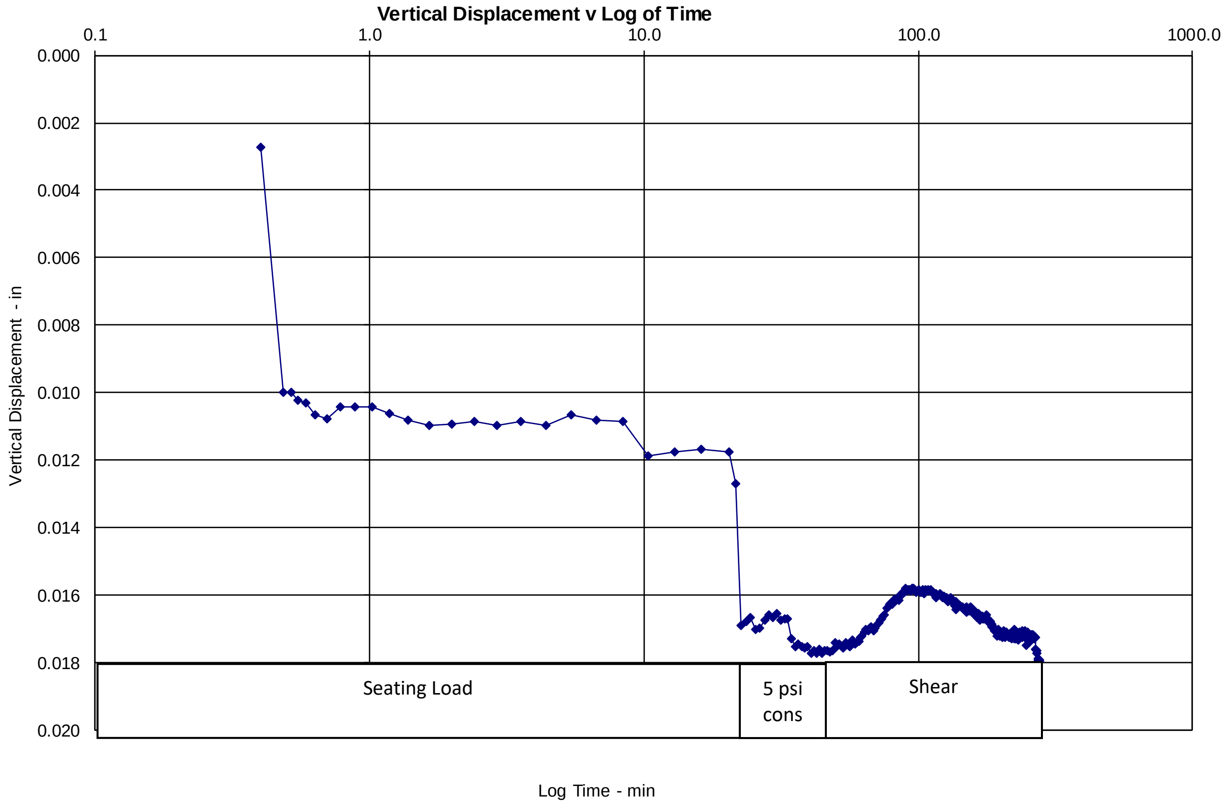


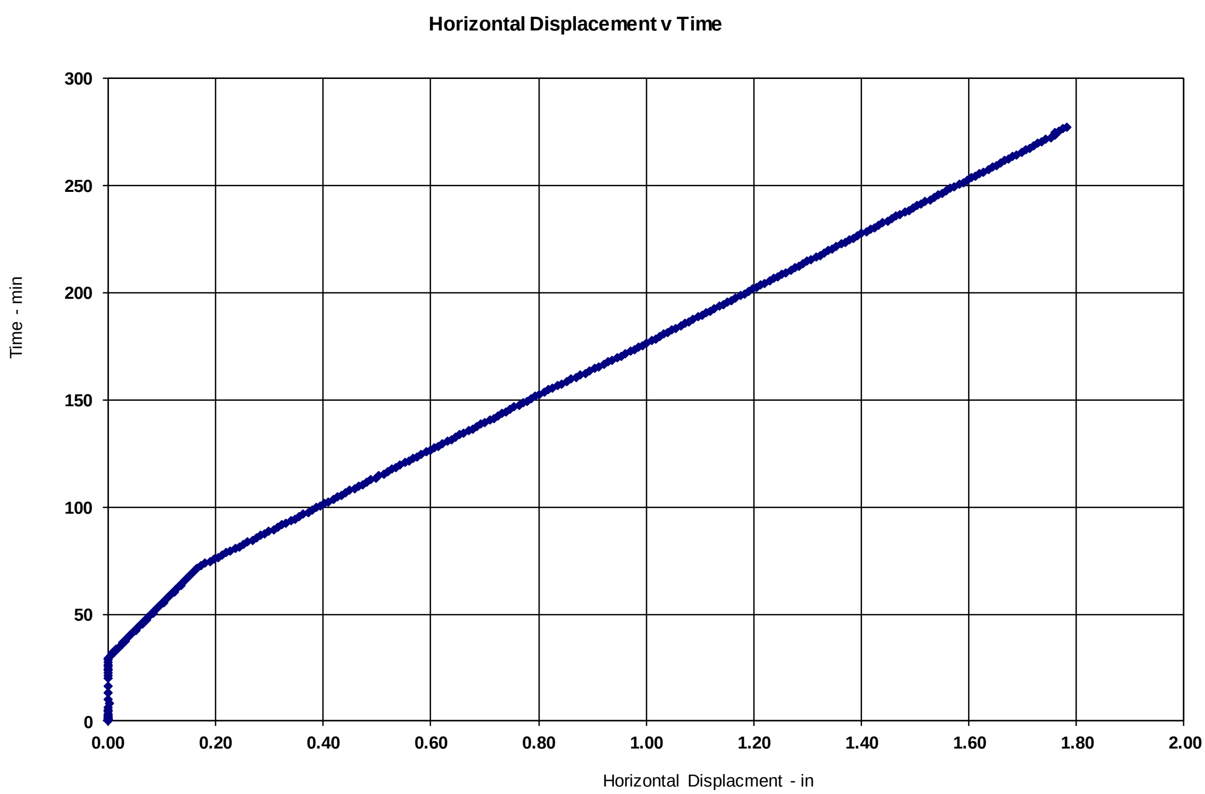
Test 5 - Normal Stress = 20 lb/in^2. Pea Gravel, POORLY GRADED GRAVEL (SP), 100 percent hard, subrounded to subangular gravel. Maximum particle size ½-inch. Particles break with moderate hammer blow. Interface Coefficients of Friction: μ_p = 0.38, μ_r = 0.36.



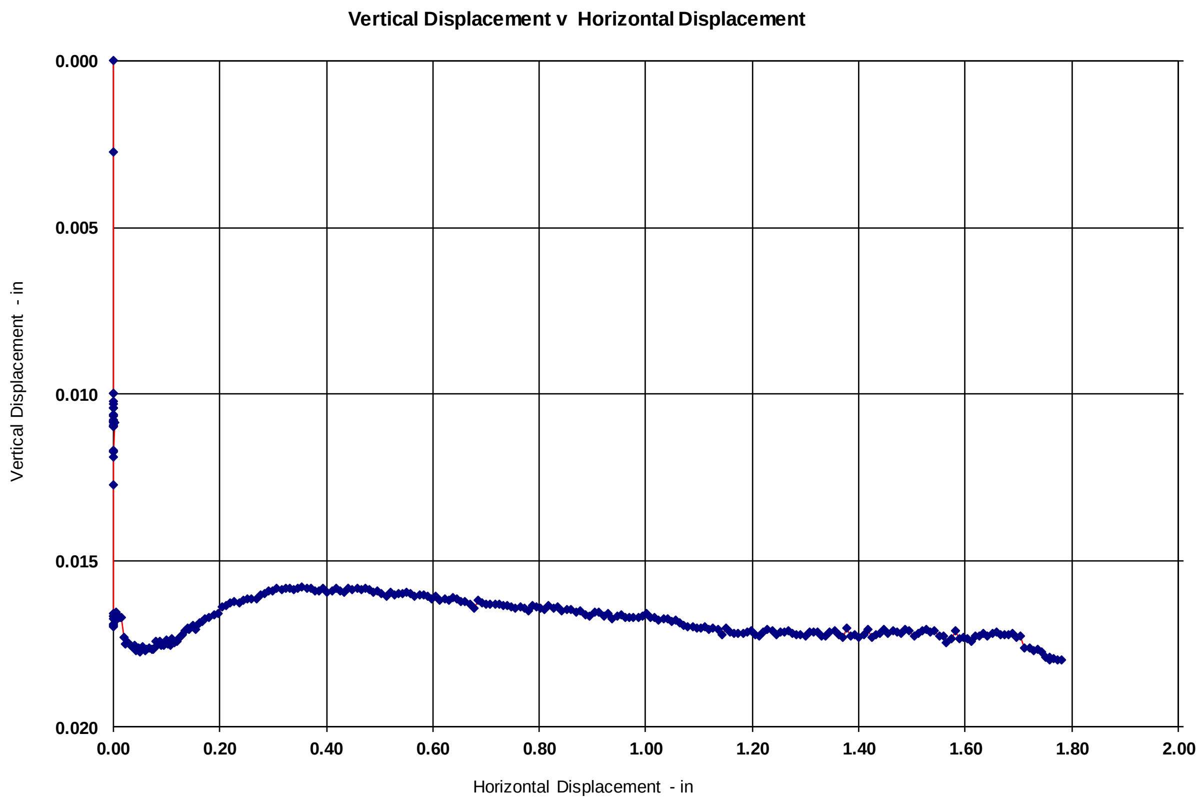


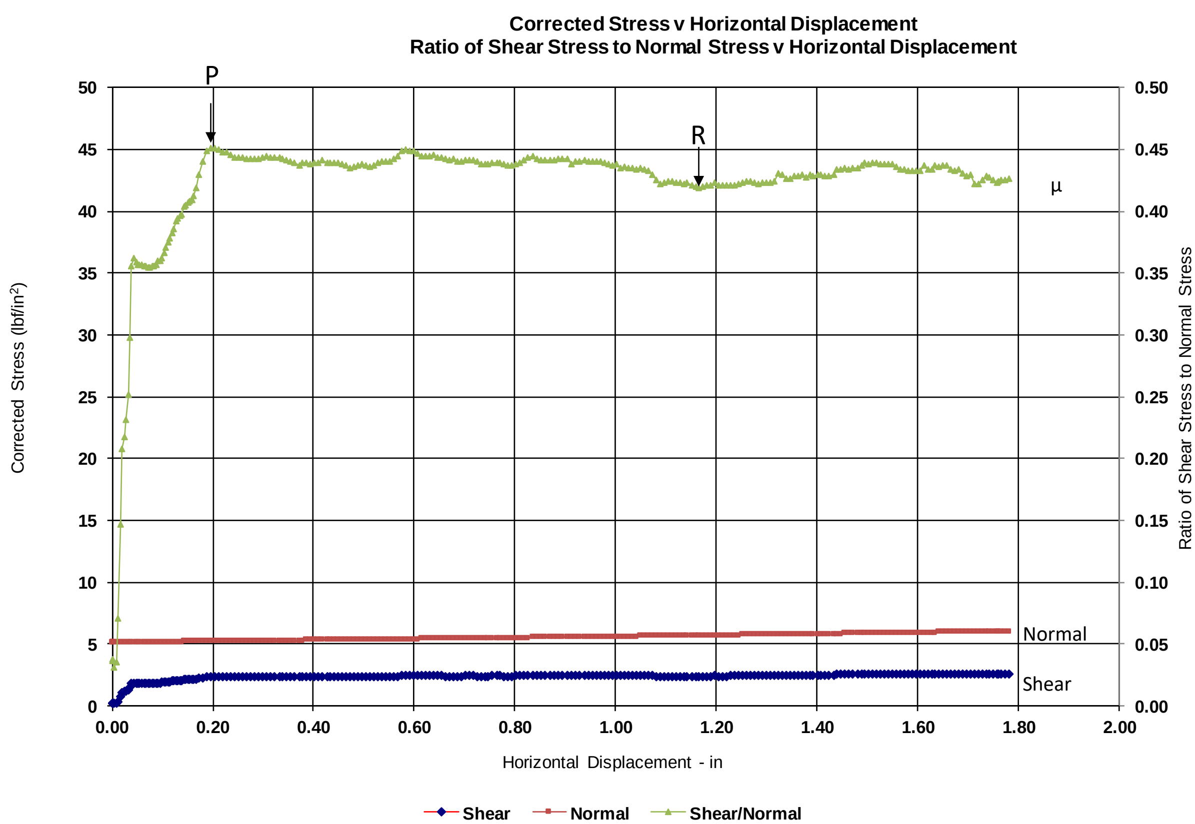
Test 6 - Normal Stress = 5 lb/in^2. Pea Gravel, POORLY GRADED GRAVEL (SP), 100 percent hard, subrounded to subangular gravel. Maximum particle size ½-inch. Particles break with moderate hammer blow. Interface Coefficients of Friction: μ_p = 0.45, μ_r = 0.42.



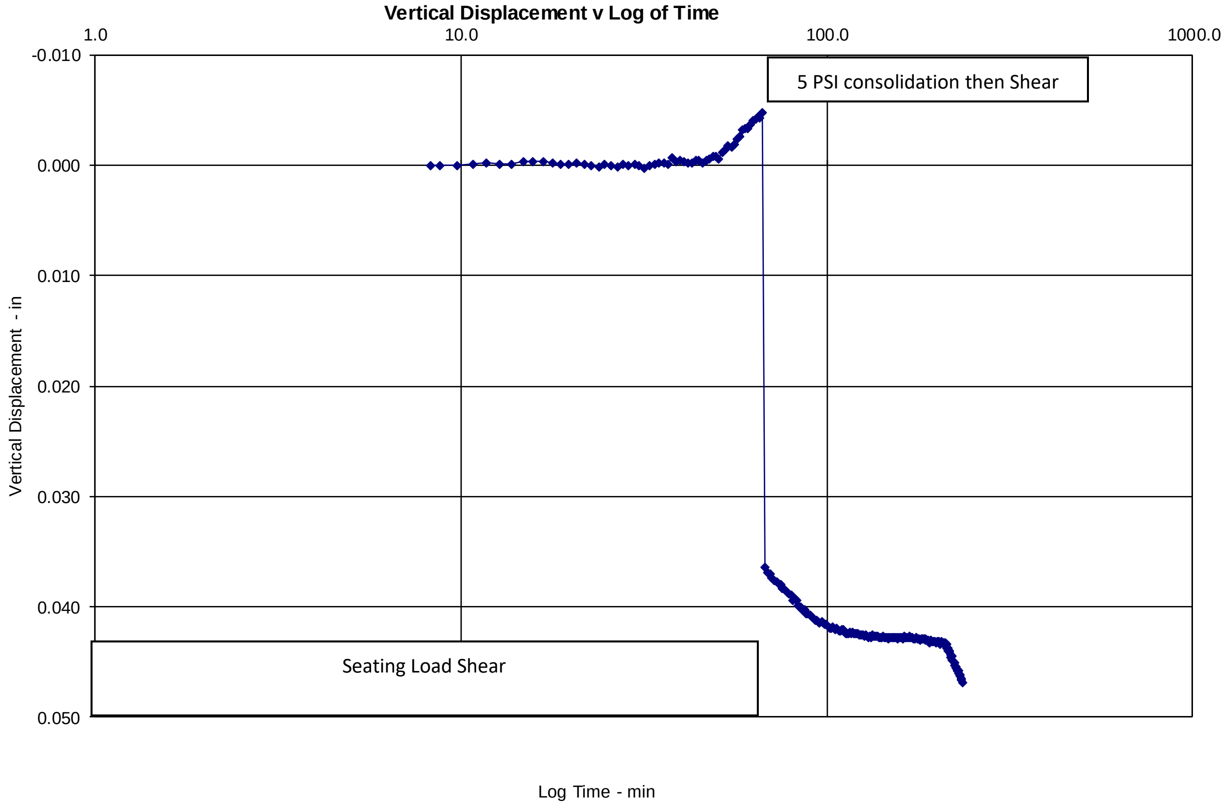


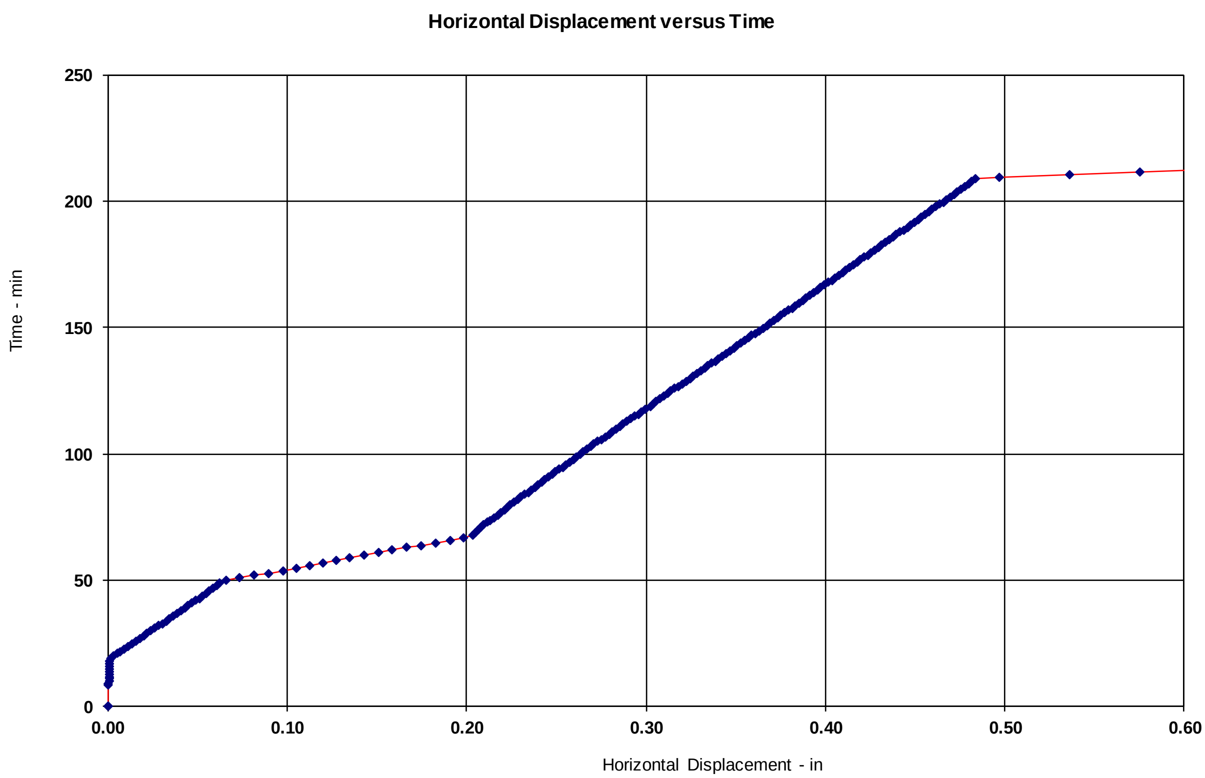
Test 6 - Normal Stress = 5 lb/in². Pea Gravel, POORLY GRADED GRAVEL (SP), 100 percent hard, subrounded to subangular gravel. Maximum particle size ½-inch. Particles break with moderate hammer blow. Interface Coefficients of Friction: $\mu_p = 0.45$, $\mu_r = 0.42$.



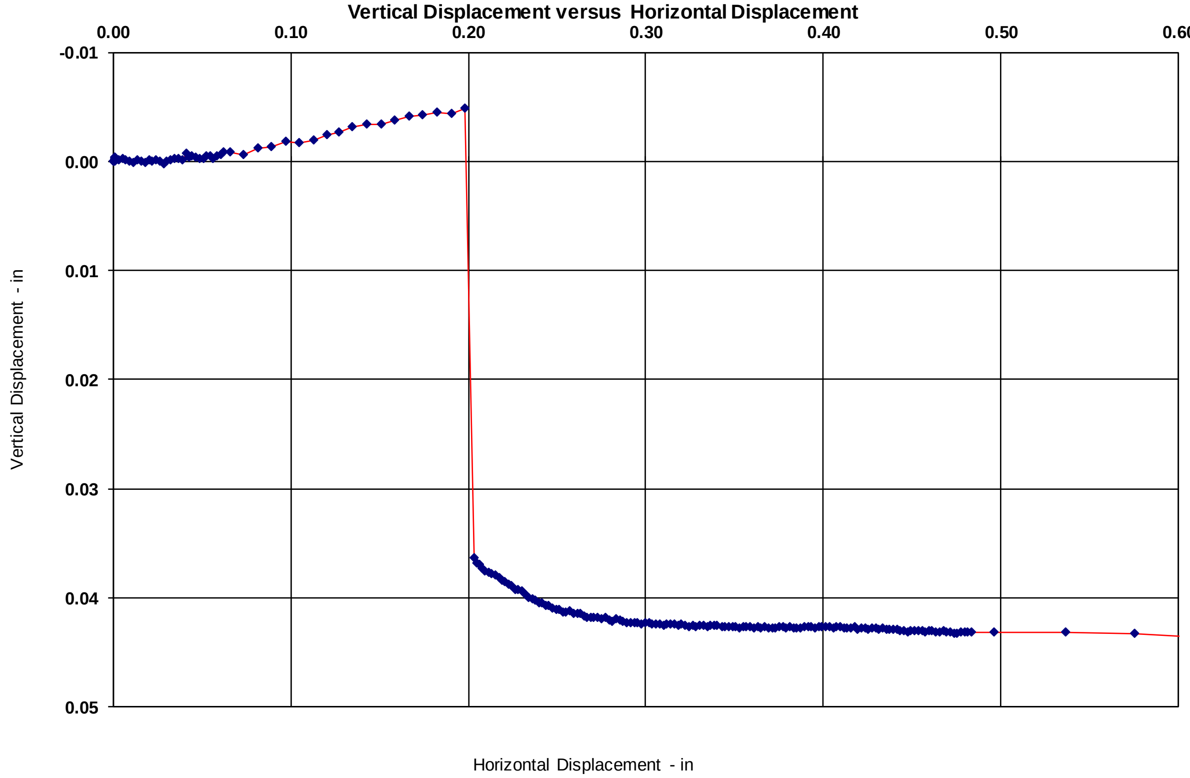


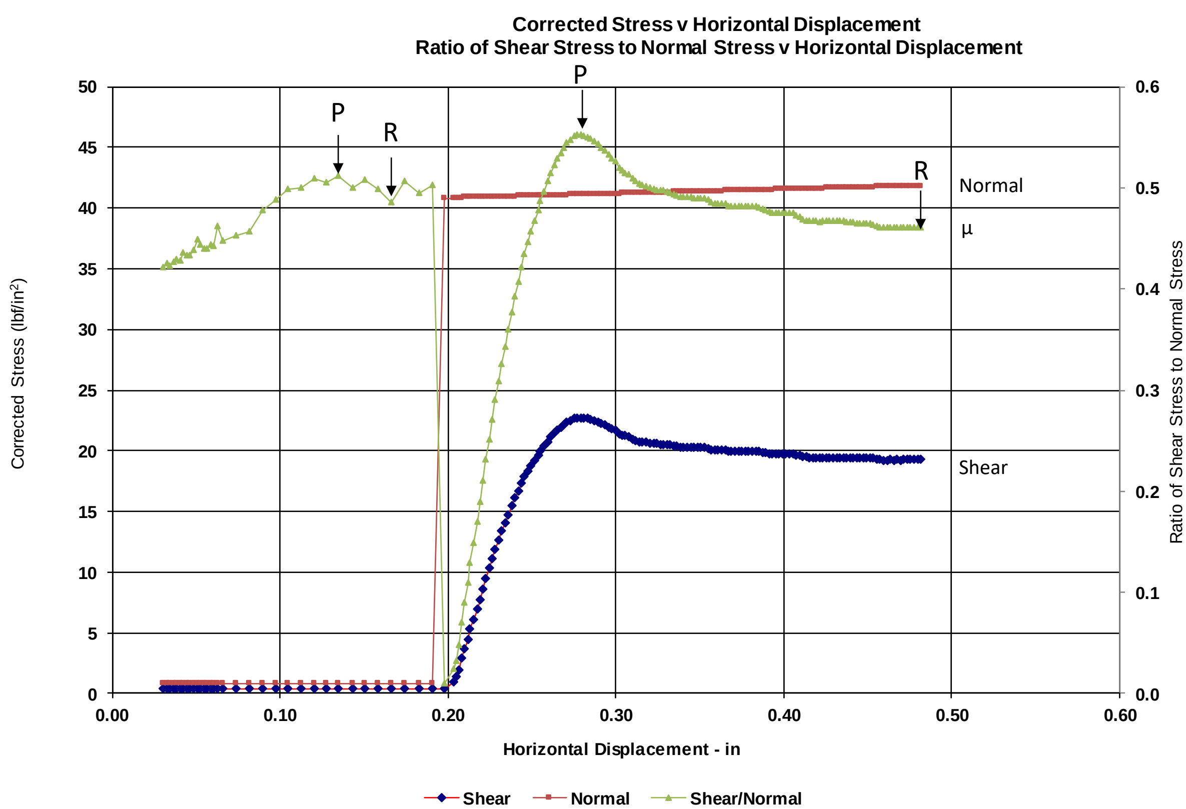
Test 7 - Normal Stress = 1 lb/in^2,40 lb/in^2. Pea Gravel, POORLY GRADED GRAVEL (SP), 100 percent hard, subrounded to subangular gravel. Maximum particle size ½-inch. Particles break with moderate hammer blow. Interface Coefficients of Friction: $\mu_{p1} = 0.51$, $\mu_{p40} = 0.55$; $\mu_{r1} = 0.49$, $\mu_{r40} = 0.46$.



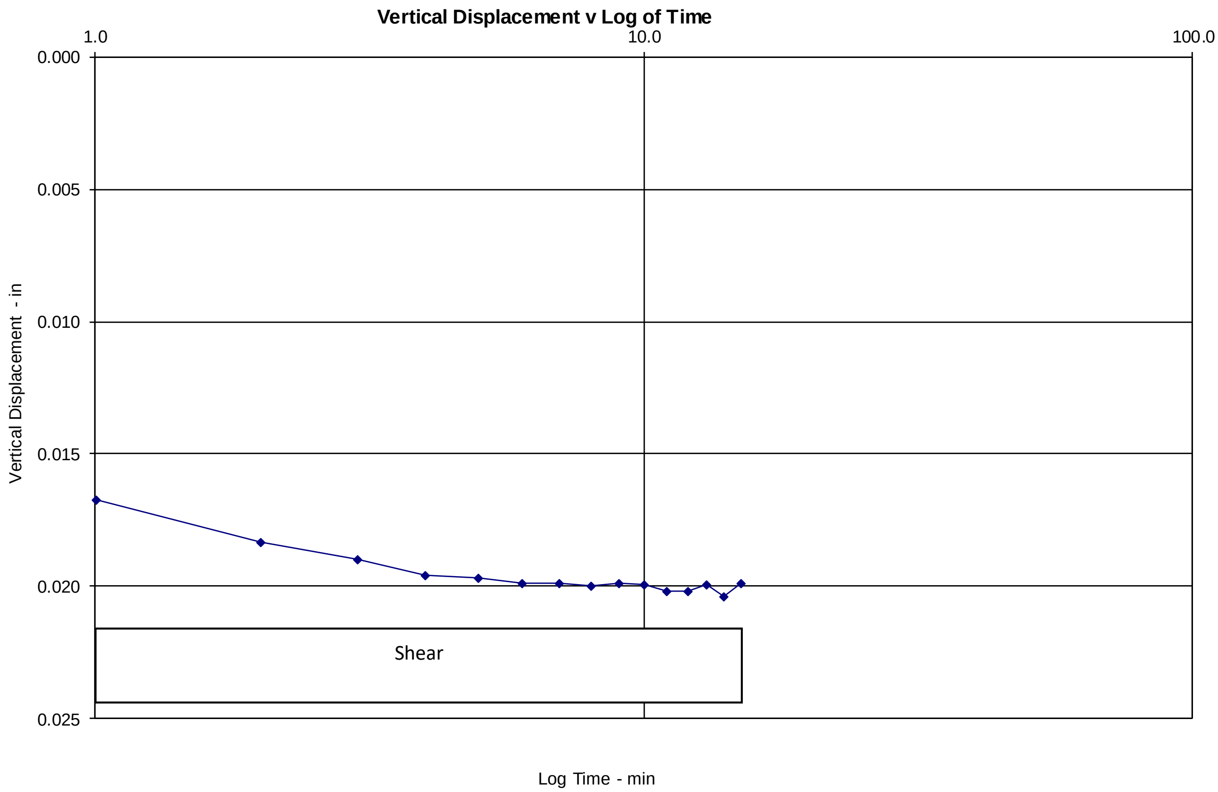


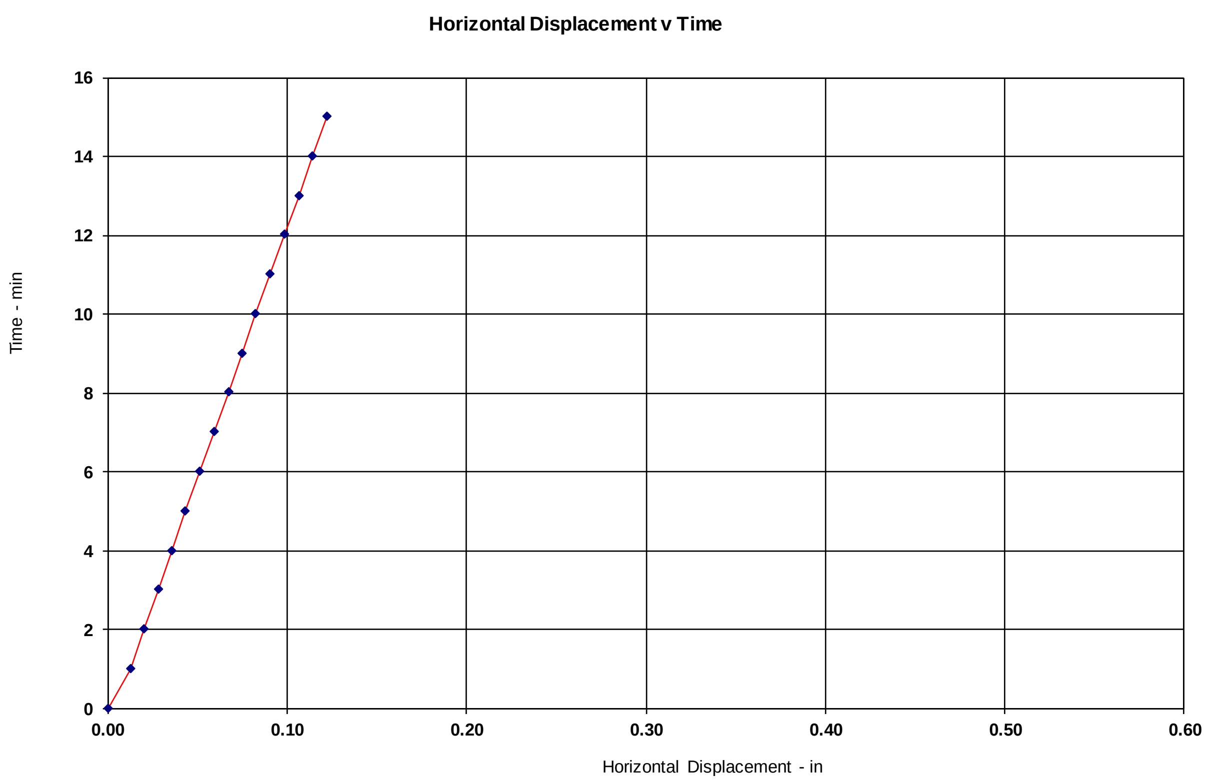
Test 7 - Normal Stress = 1 lb/in^2,40 lb/in^2. Pea Gravel, POORLY GRADED GRAVEL (SP), 100 percent hard, subrounded to subangular gravel. Maximum particle size ½-inch. Particles break with moderate hammer blow. Interface Coefficients of Friction: μ_{p1} = 0.51, μ_{p40} = 0.55; μ_{r1} = 0.49, μ_{r40} = 0.46.



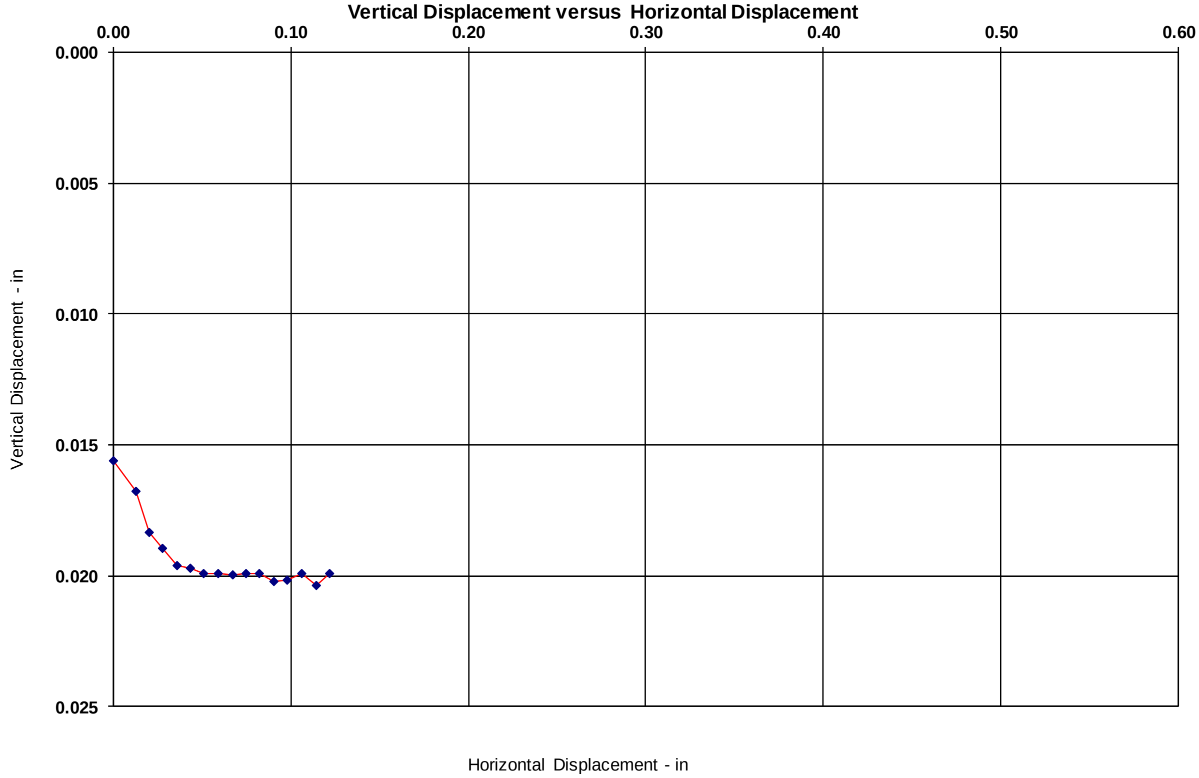


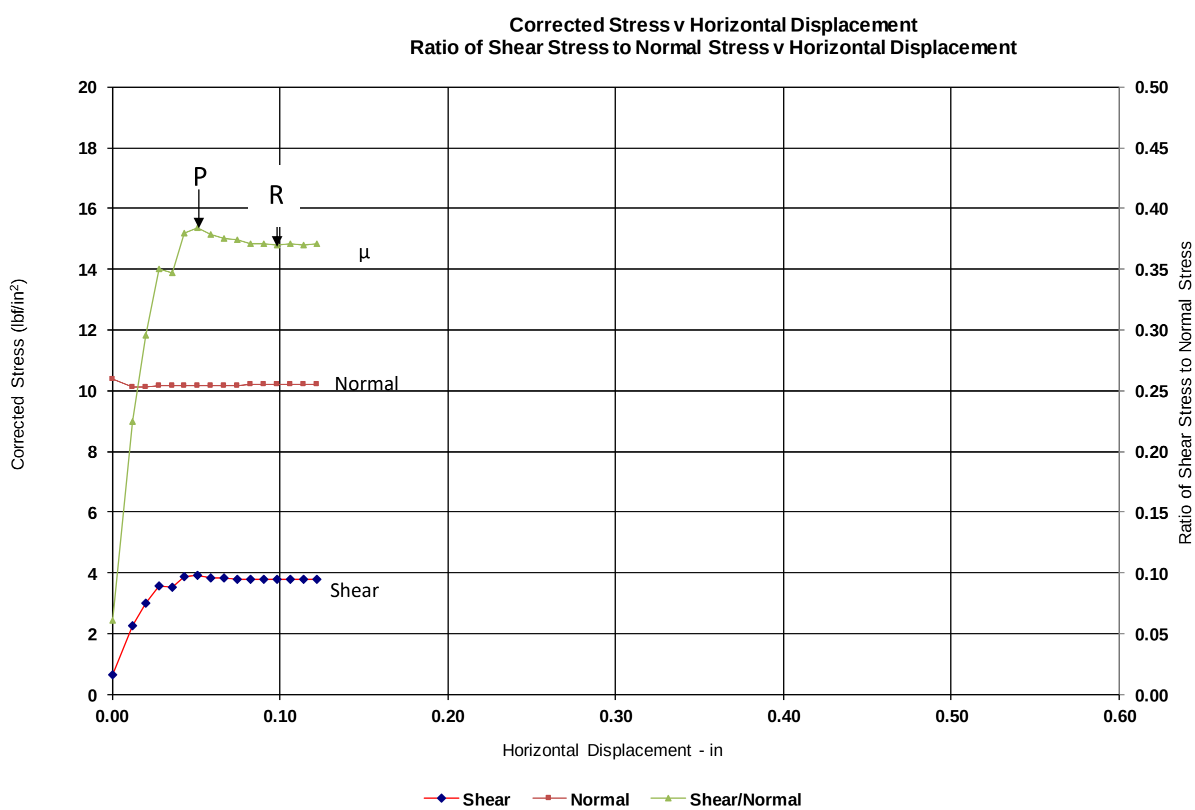
Test 9 - Normal Stress = 10 lb/in^2. Pea Gravel, POORLY GRADED GRAVEL (SP), 100 percent hard, subrounded to subangular gravel. Maximum particle size ½-inch. Particles break with moderate hammer blow. Interface Coefficients of Friction: $\mu_p = 0.38$, $\mu_r = 0.36$.



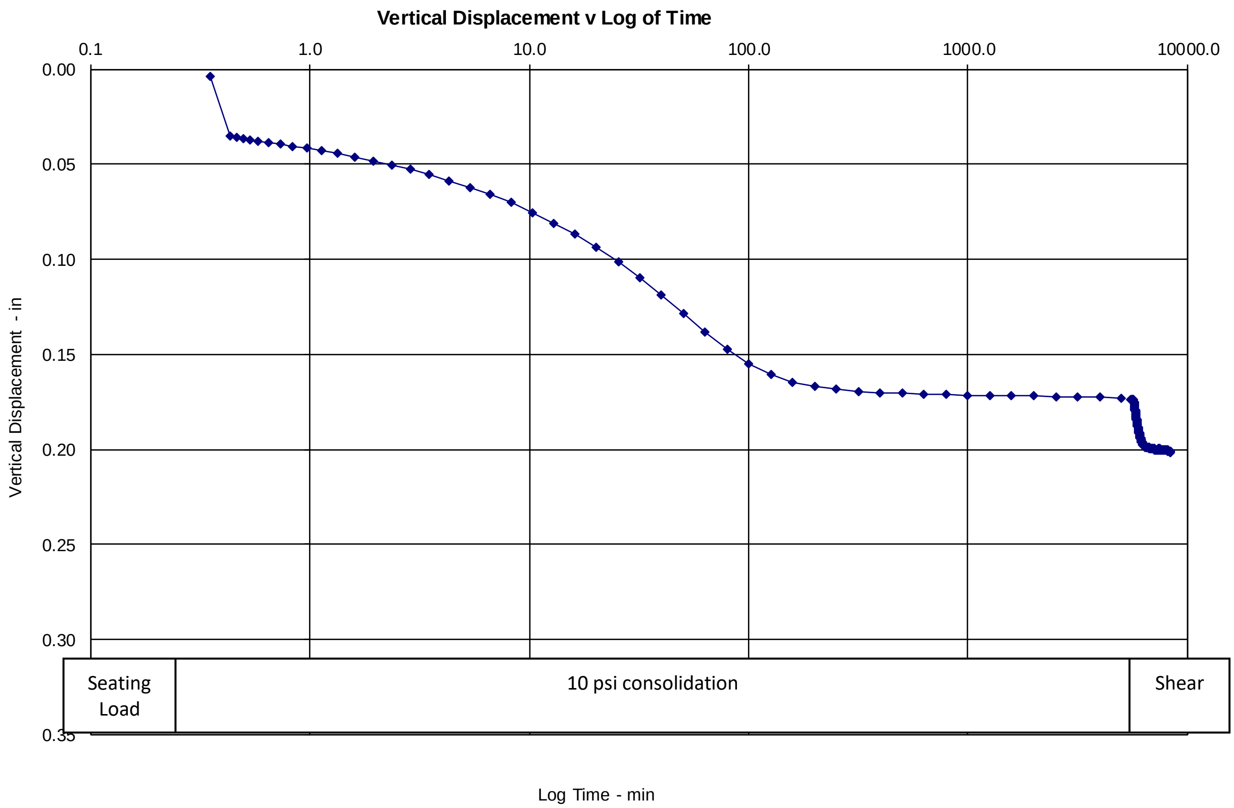


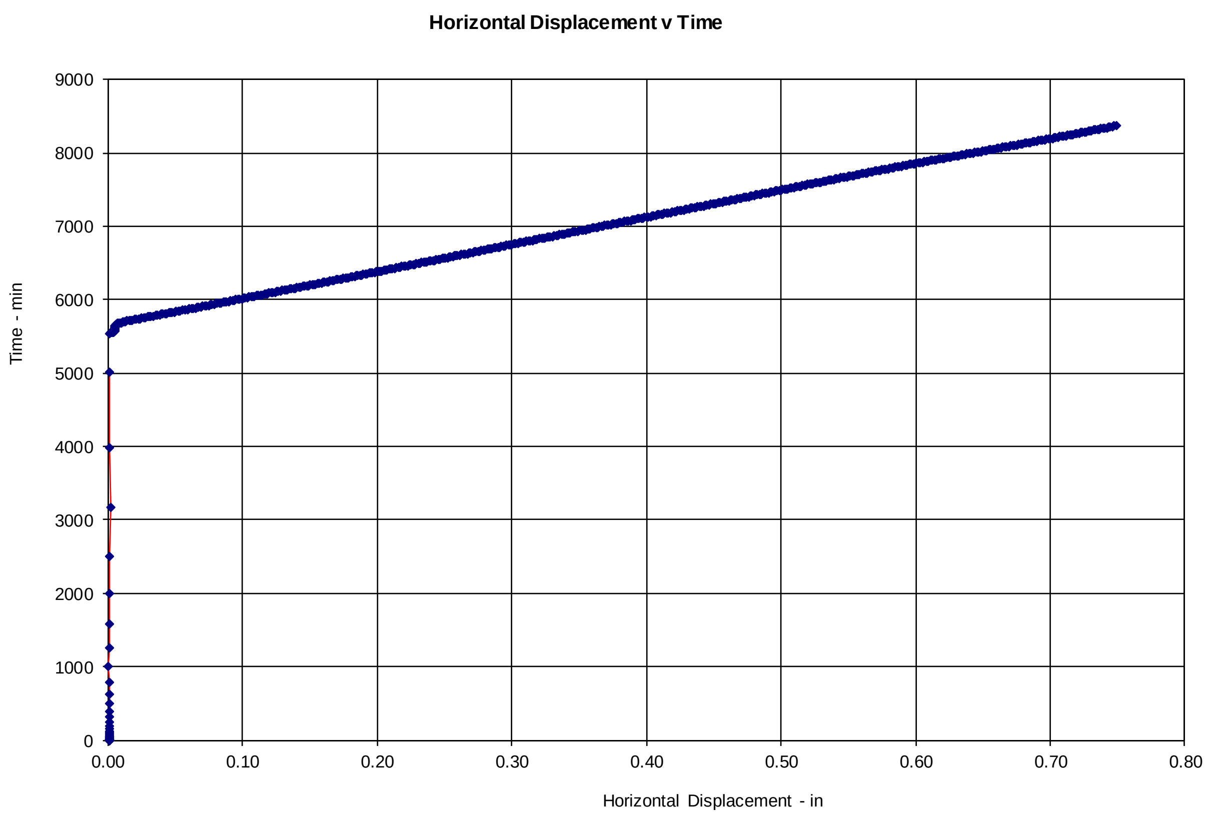
Test 9 - Normal Stress = 10 lb/in^2. Pea Gravel, POORLY GRADED GRAVEL (SP), 100 percent hard, subrounded to subangular gravel. Maximum particle size ½-inch. Particles break with moderate hammer blow. Interface Coefficients of Friction: $\mu_p = 0.38$, $\mu_r = 0.36$.



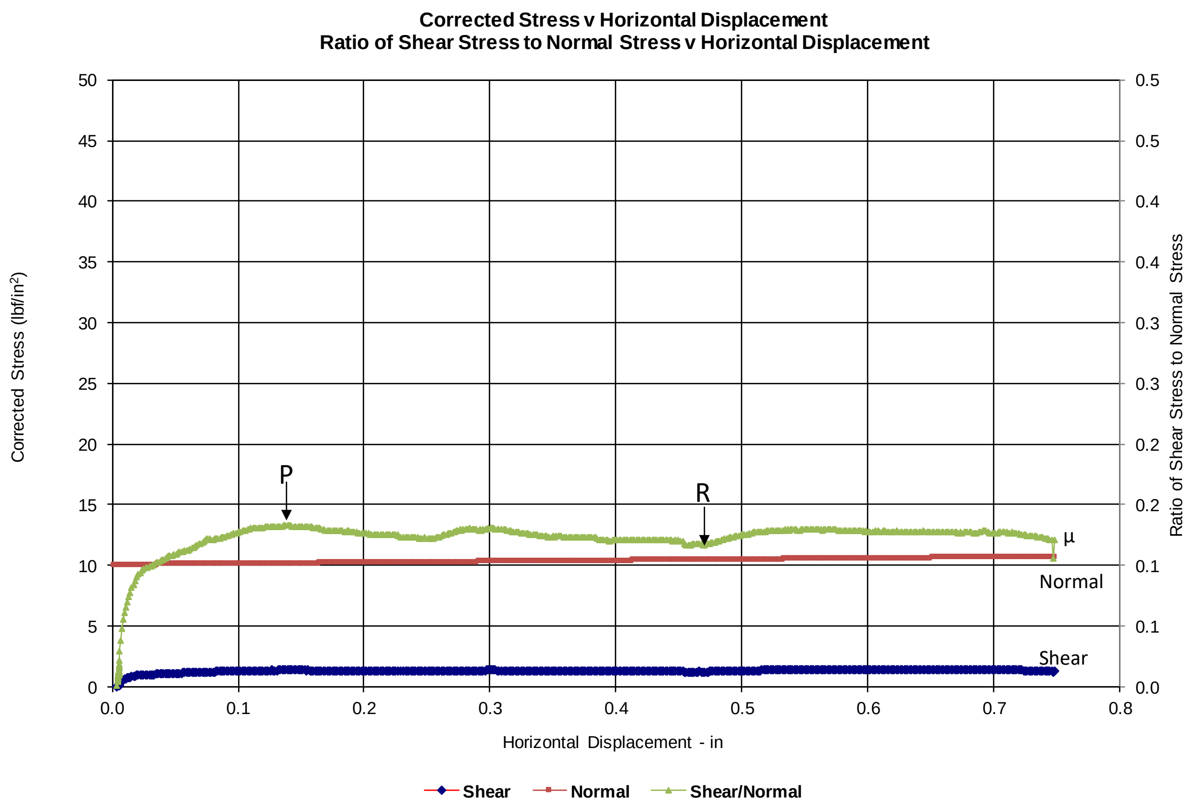


Test 10 – Normal Stress = 10 lb/in^2. SILTY CLAY (CL-ML), Fines with a trace of fine sand.
Coefficients of Friction: $\mu_p = 0.13$, $\mu_r = 0.12$.

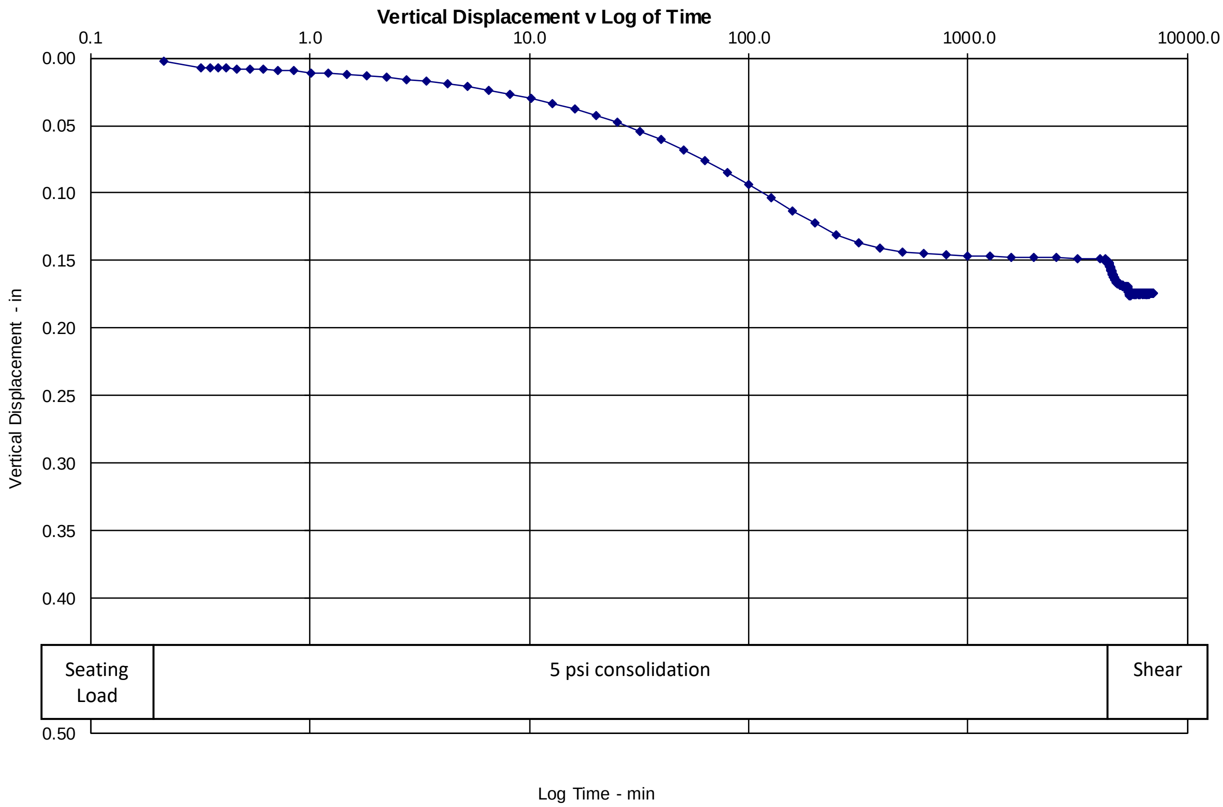


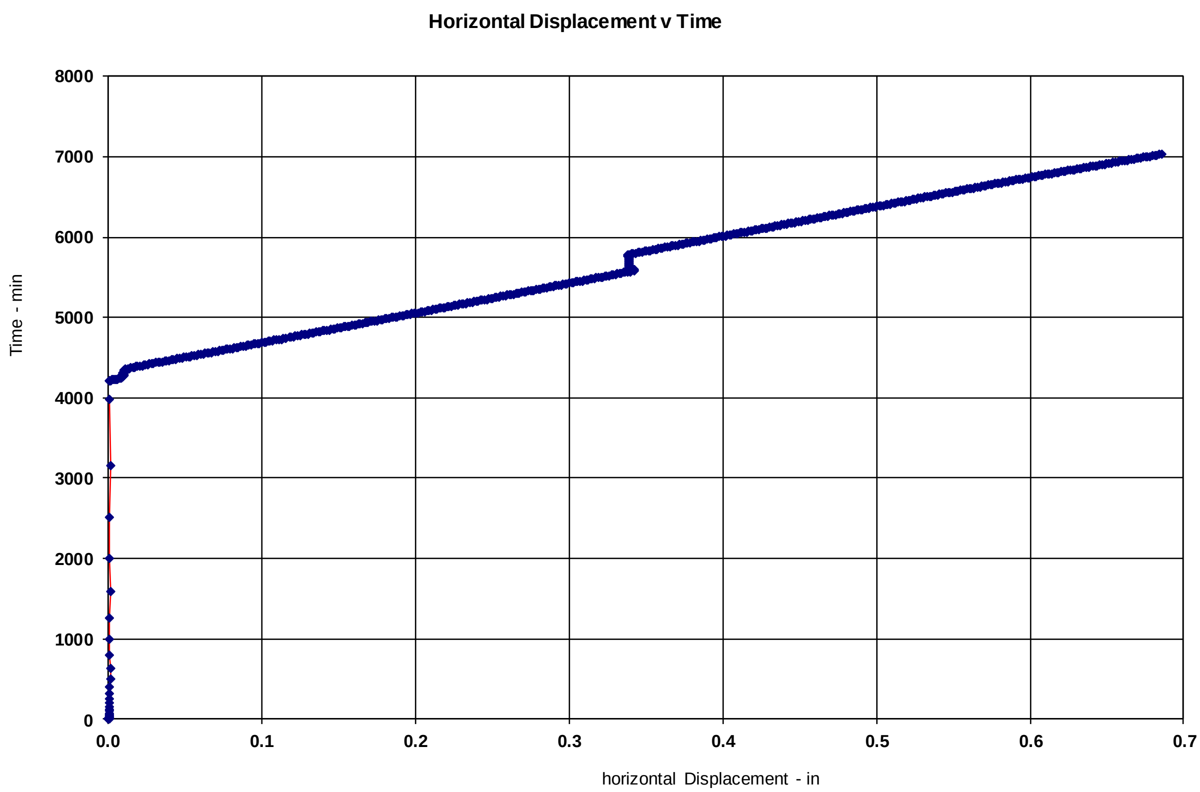


Test 10 – Normal Stress = 10 lb/in^2. SILTY CLAY (CL-ML), Fines with a trace of fine sand.
Coefficients of Friction: μ_p = 0.13, μ_r = 0.12.

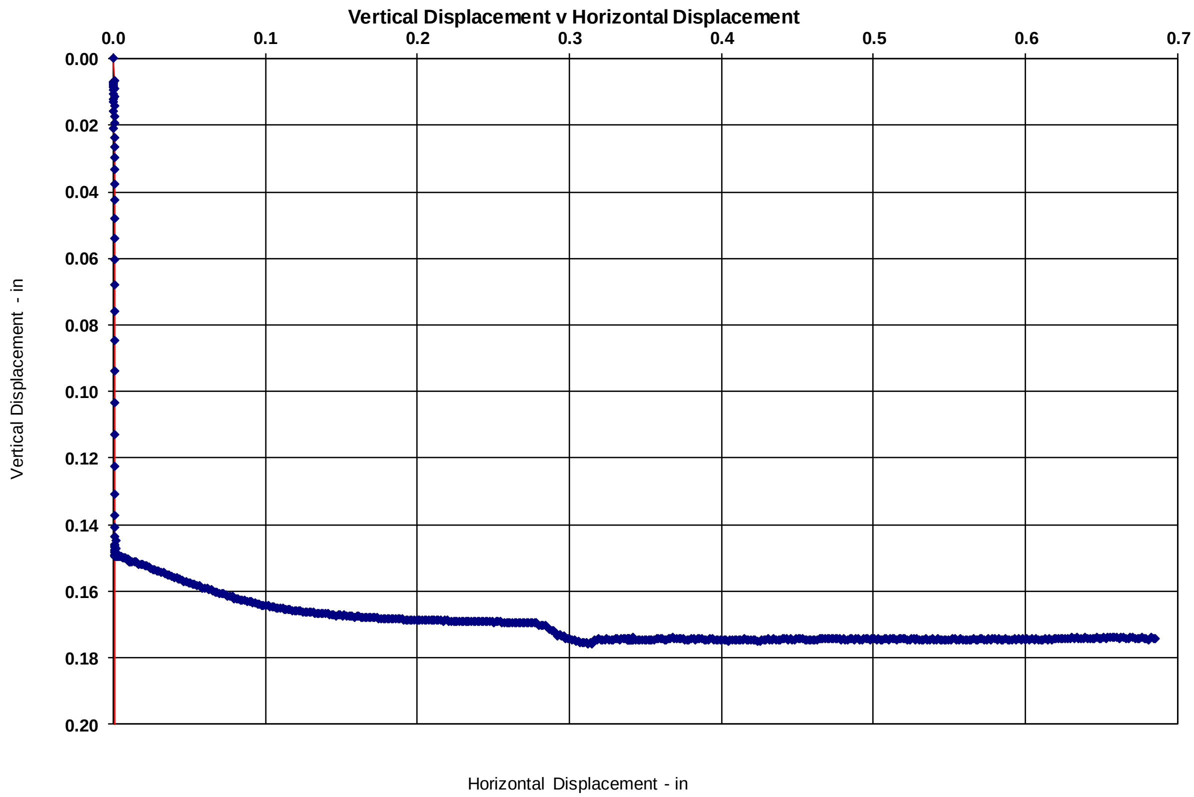


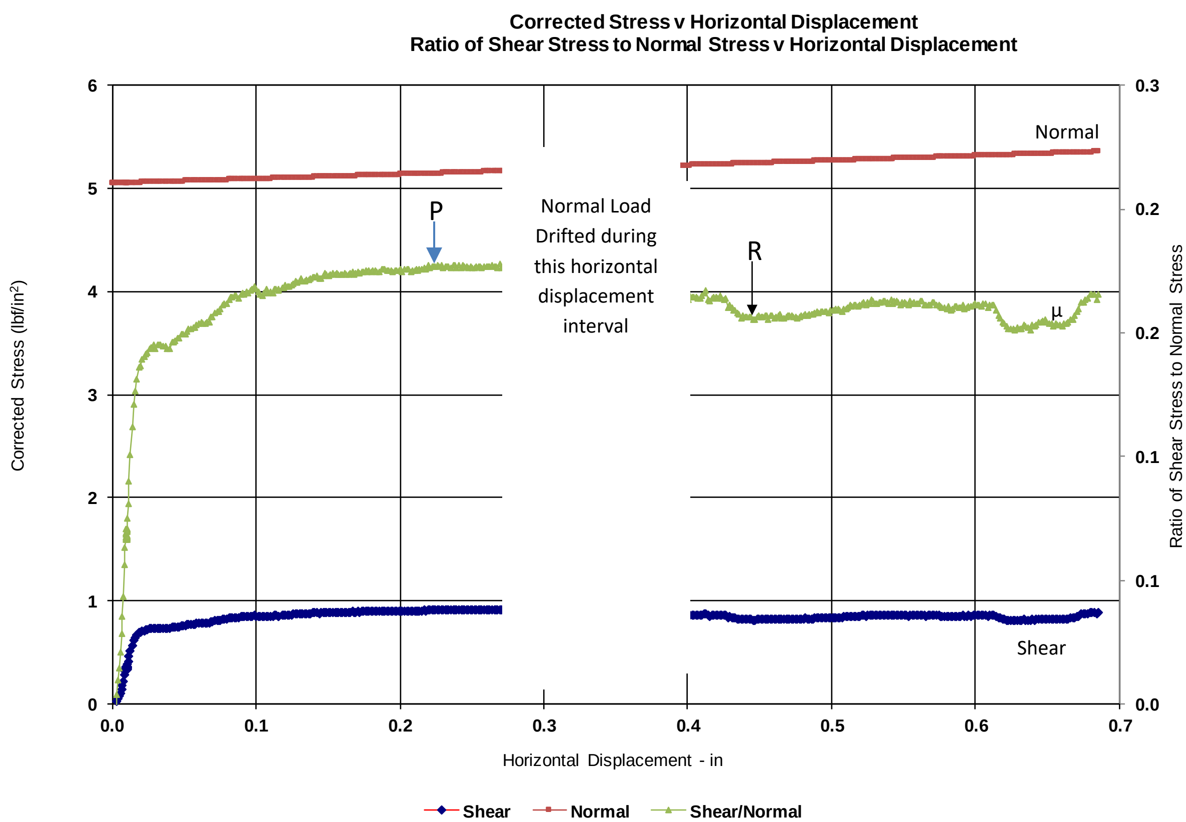
Test 11 – Normal Stress = 5 lb/in^2. SILTY CLAY (CL-ML), Fines with a trace of fine sand.
Coefficients of Friction: $\mu_p = 0.18$, $\mu_r = 0.16$.



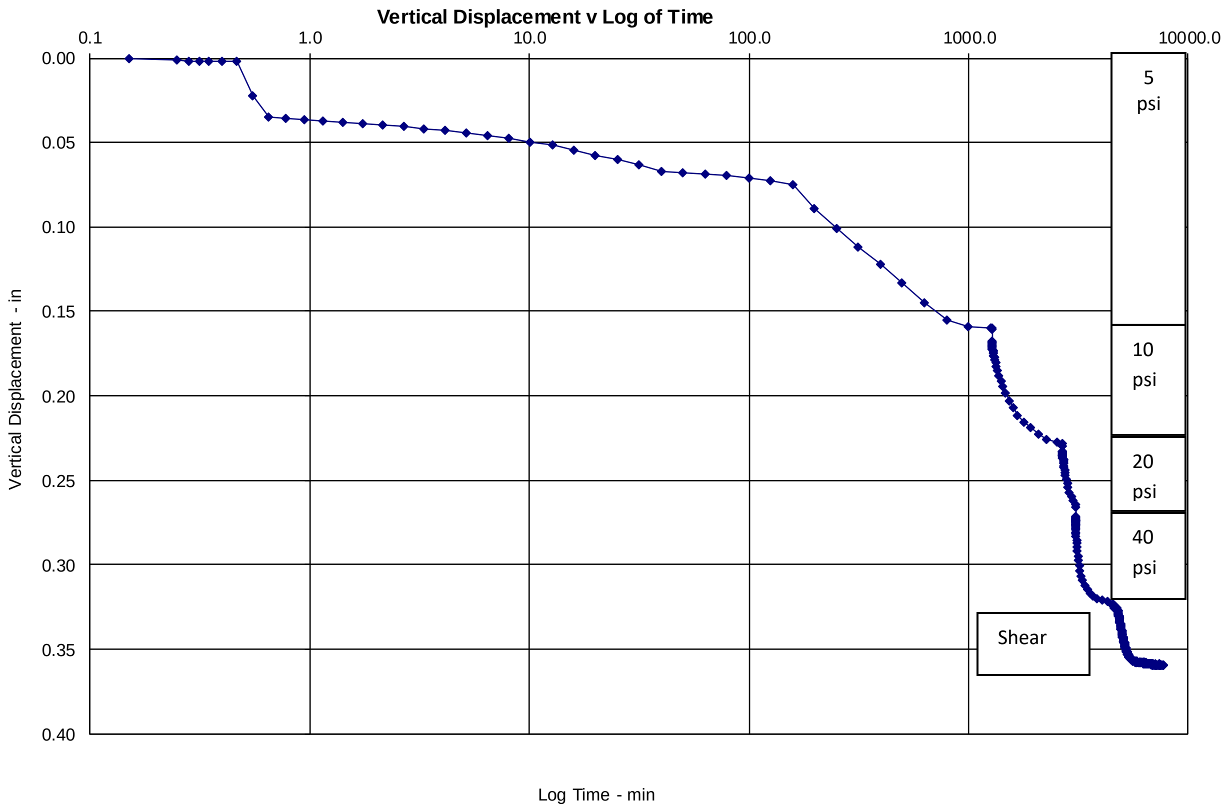


Test 11 – Normal Stress = 5 lb/in^2. SILTY CLAY (CL-ML), Fines with a trace of fine sand.
Coefficients of Friction: $\mu_p = 0.18$, $\mu_r = 0.16$.

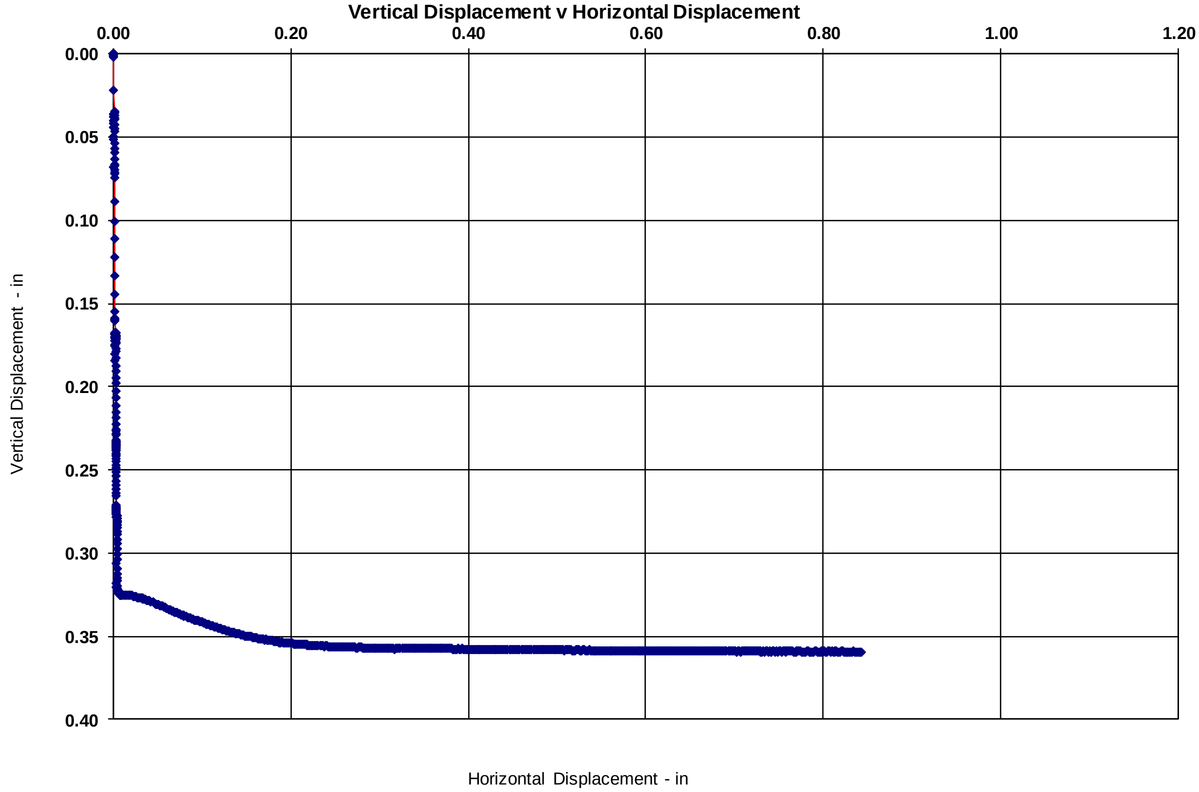


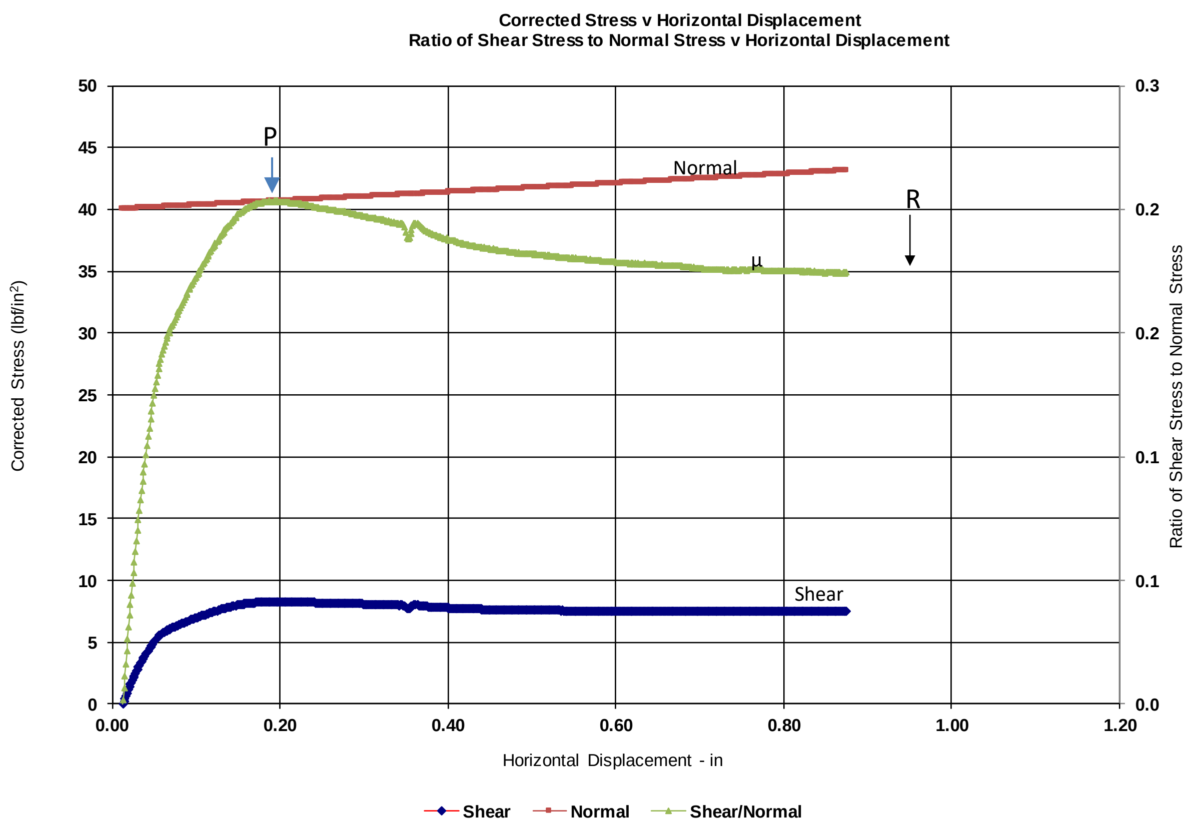


Test 12 – Normal Stress = 40 lb/in^2. SILTY CLAY (CL-ML), Fines with a trace of fine sand.
Coefficients of Friction: $\mu_p = 0.20$, $\mu_r = 0.17$.

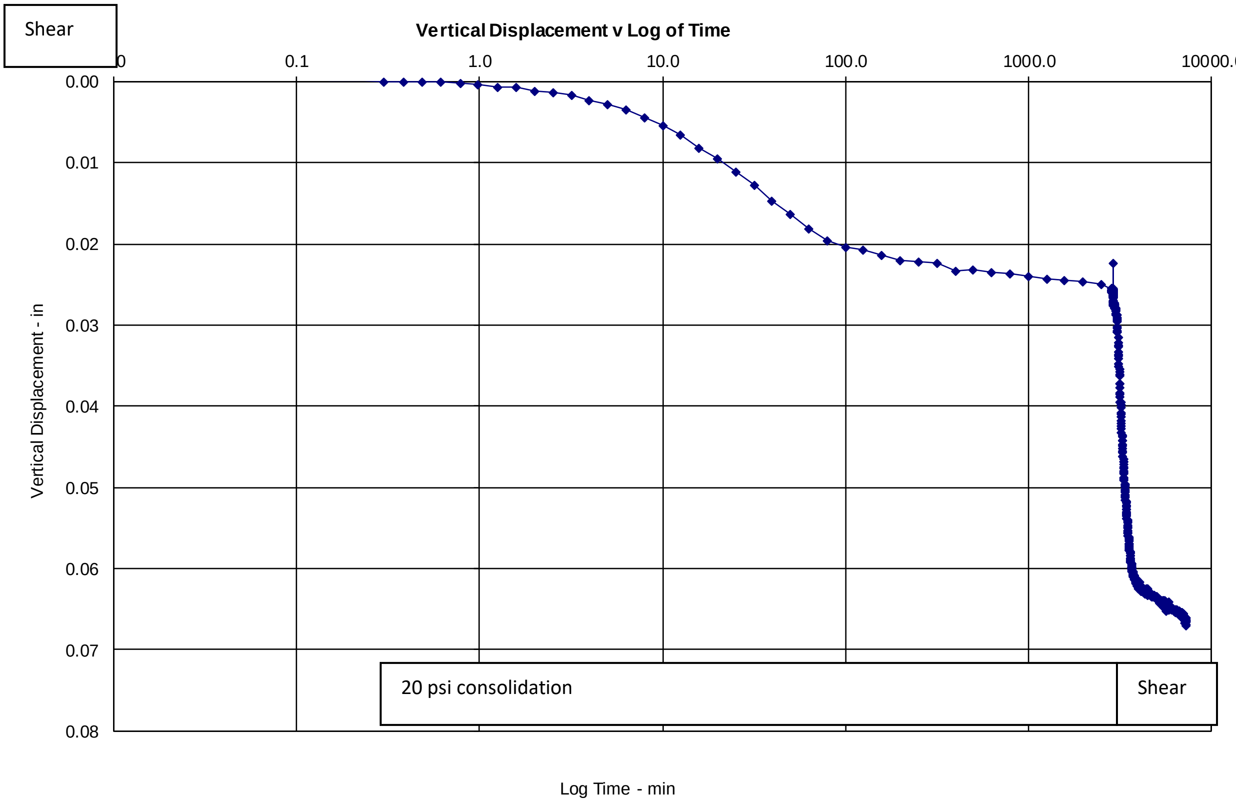


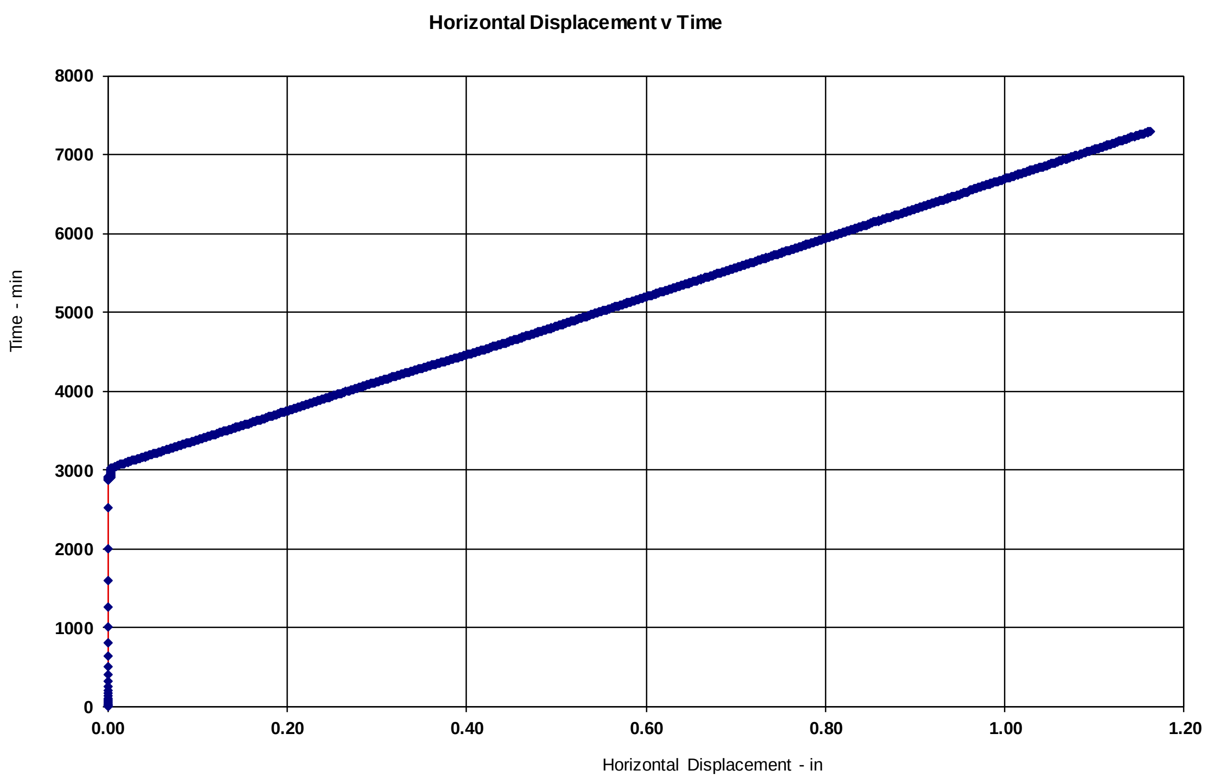
Test 12 – Normal Stress = 40 lb/in^2. SILTY CLAY (CL-ML), Fines with a trace of fine sand.
Coefficients of Friction: $\mu_p = 0.20$, $\mu_r = 0.17$.



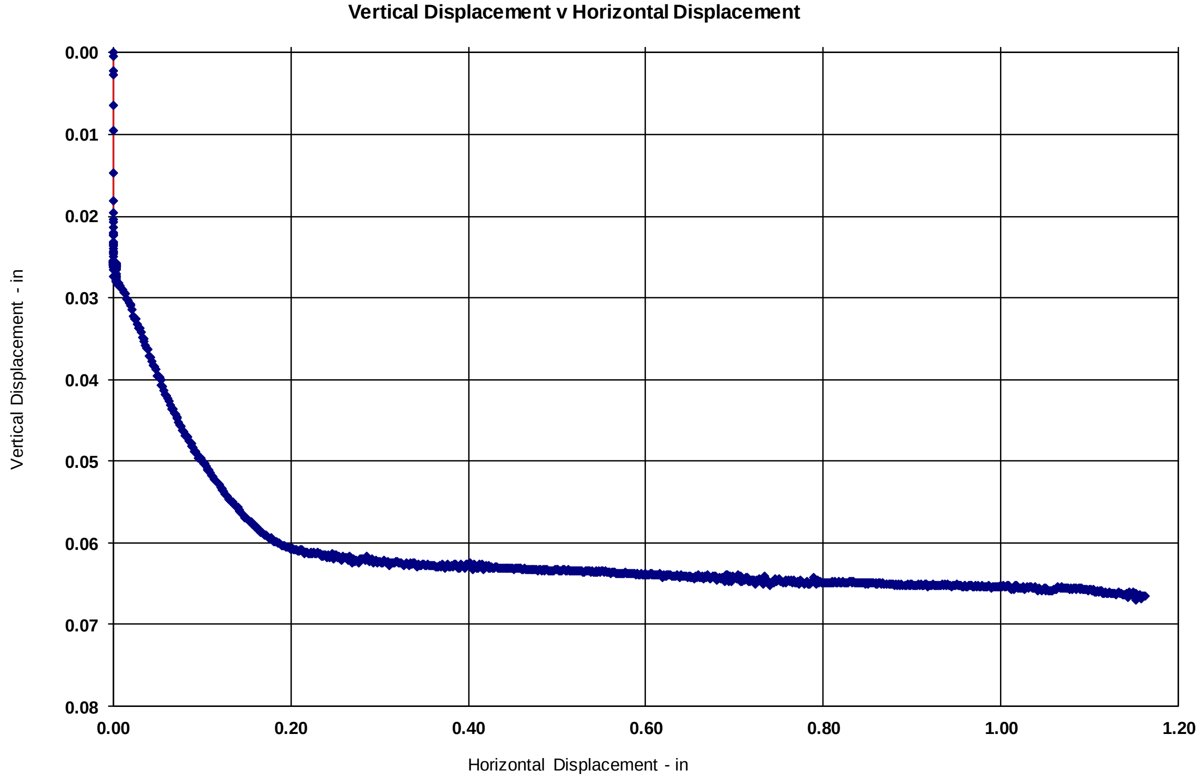


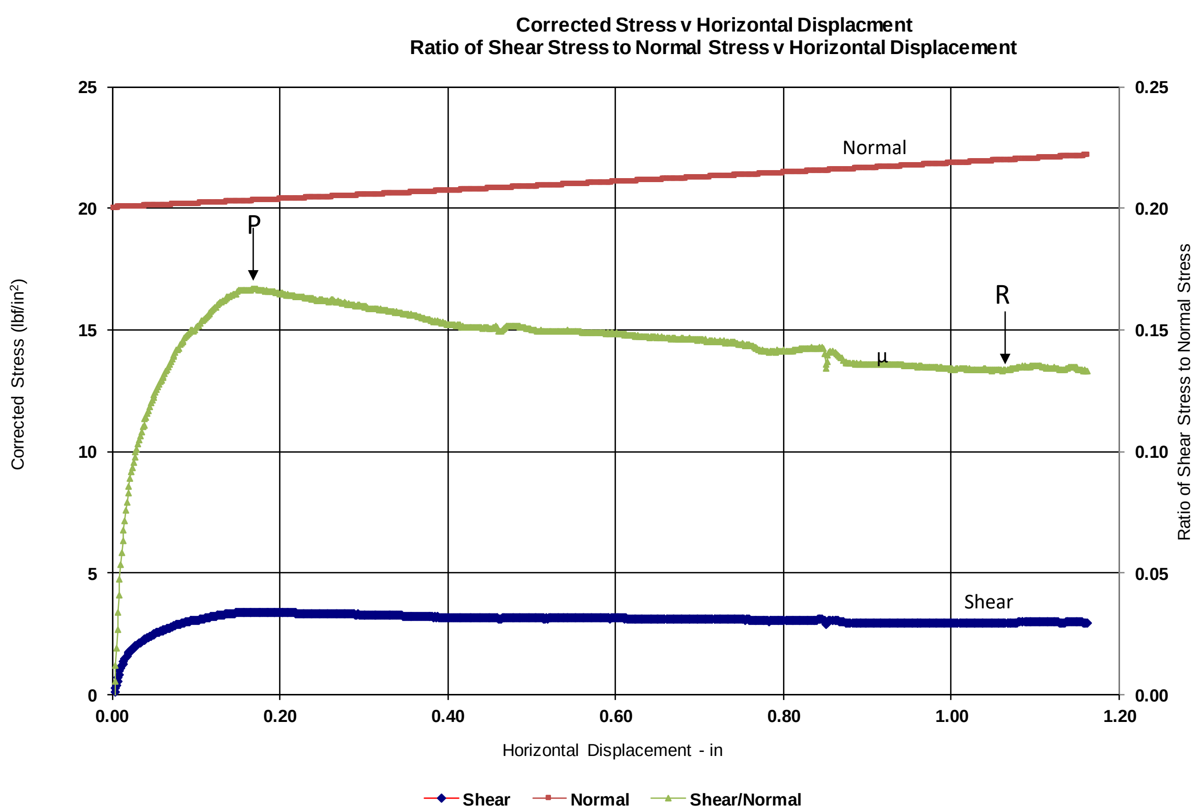
Test 13 – Normal Stress = 20 lb/in^2. SILTY CLAY (CL-ML), Fines with a trace of fine sand.
Coefficients of Friction: $\mu_p = 0.17$, $\mu_r = 0.13$.



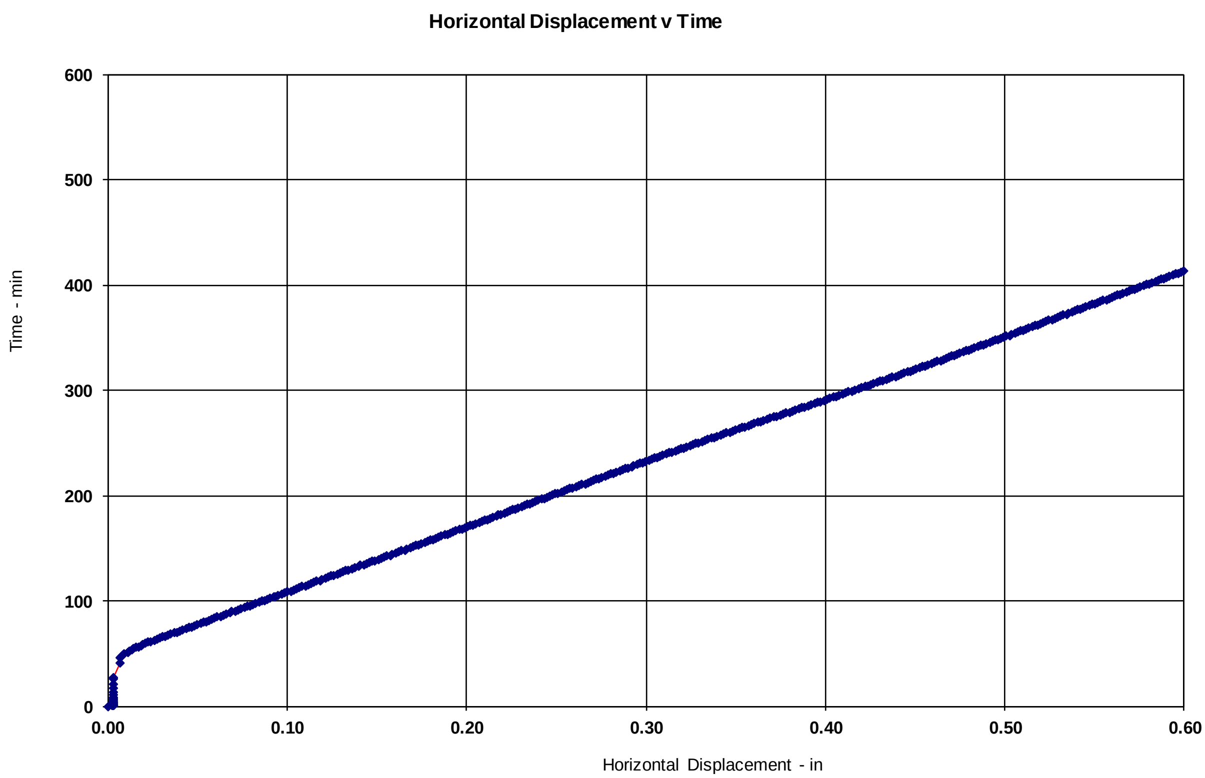


Test 13 – Normal Stress = 20 lb/in^2. SILTY CLAY (CL-ML), Fines with a trace of fine sand.
Coefficients of Friction: $\mu_p = 0.17$, $\mu_r = 0.13$.

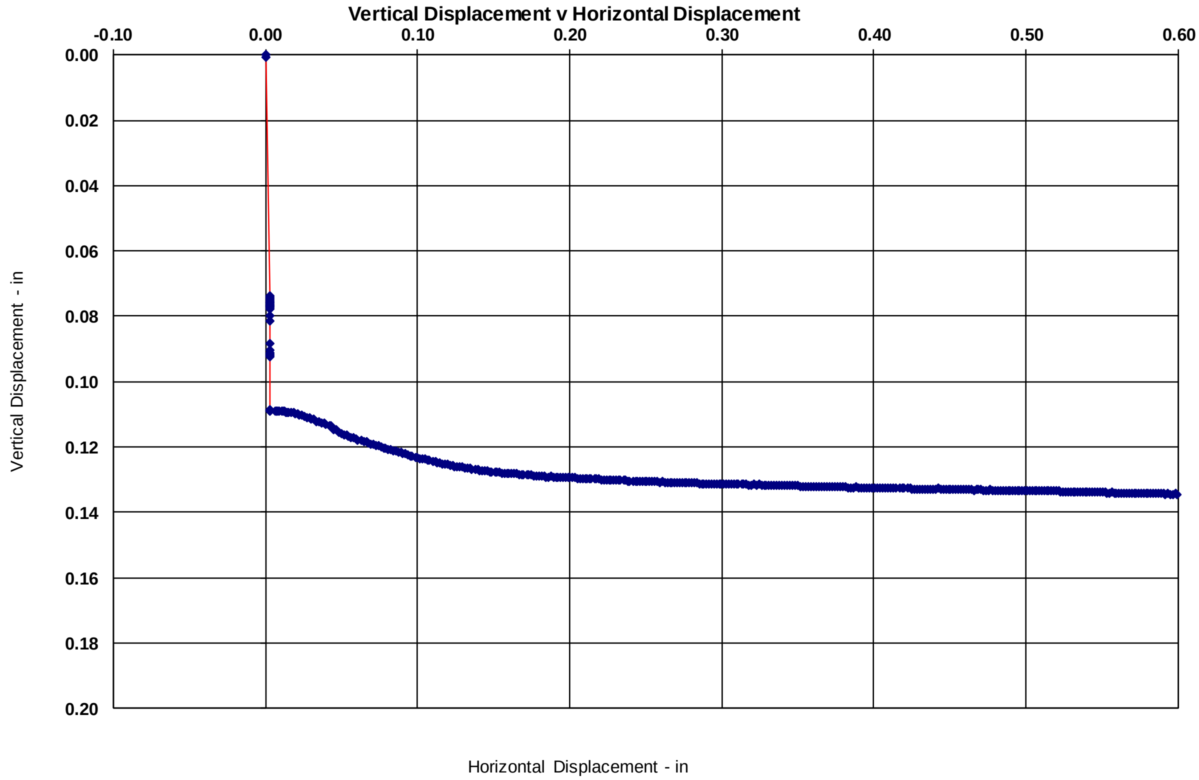


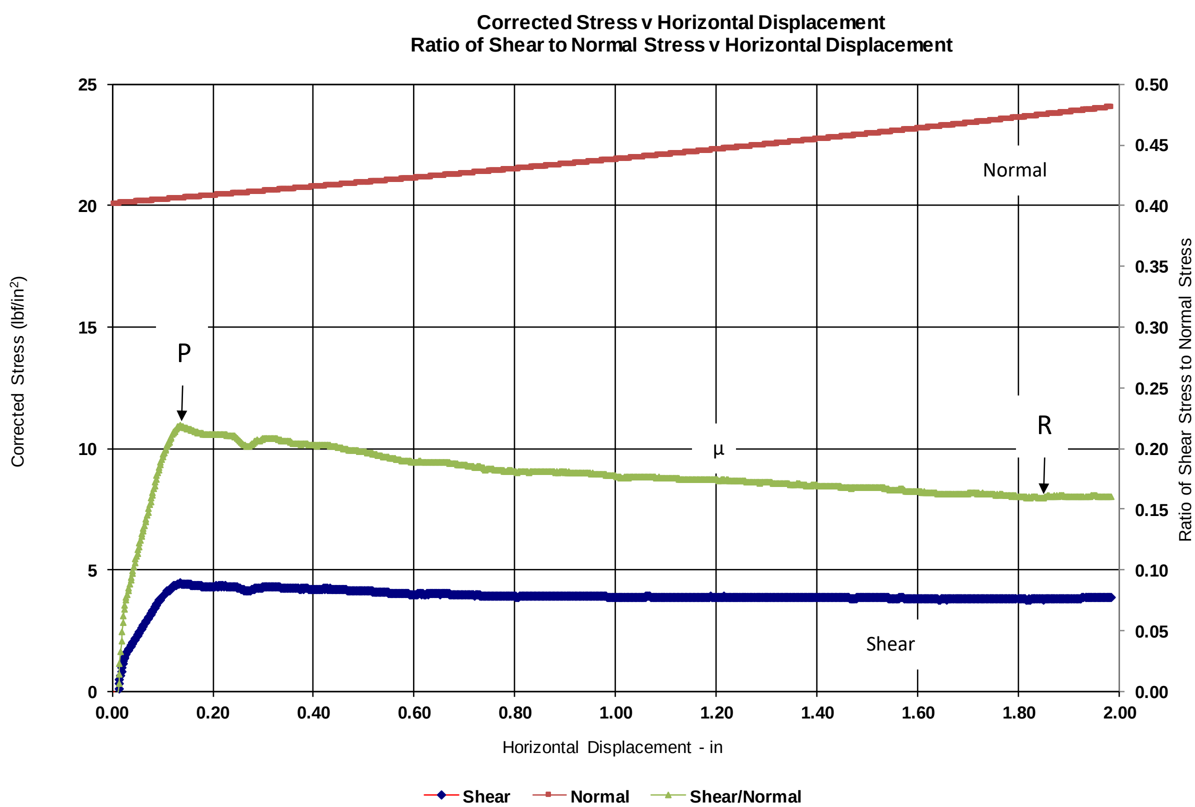


Test 14 - Normal Stress = 20 lb/in2. Crushed Rock, POORLY GRADED GRAVEL (GP), 96 percent gravel, 3 percent sand, 1 percent clayey fines, approximately 20 percent flat and 20 percent elongated particles. Particles break with moderate hammer blow. Interface Coefficients of Friction: $\mu_p = 0.22$, $\mu_r = 0.16$.

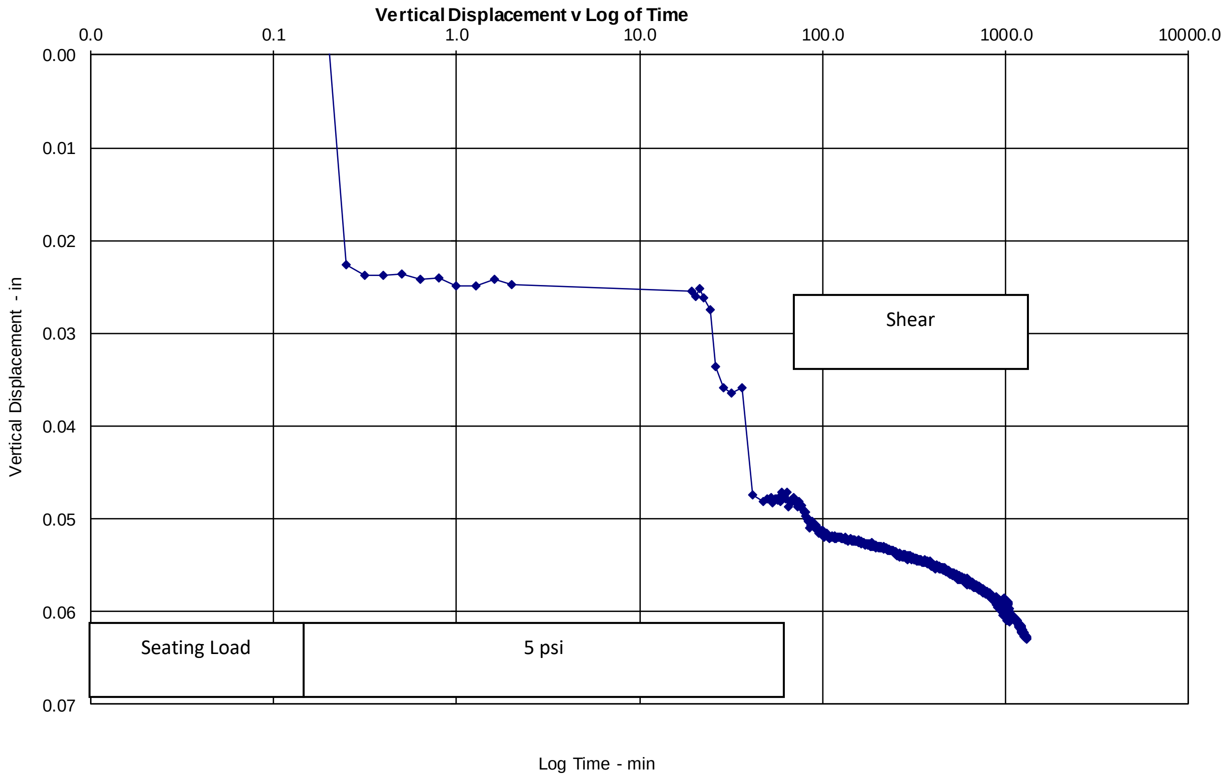


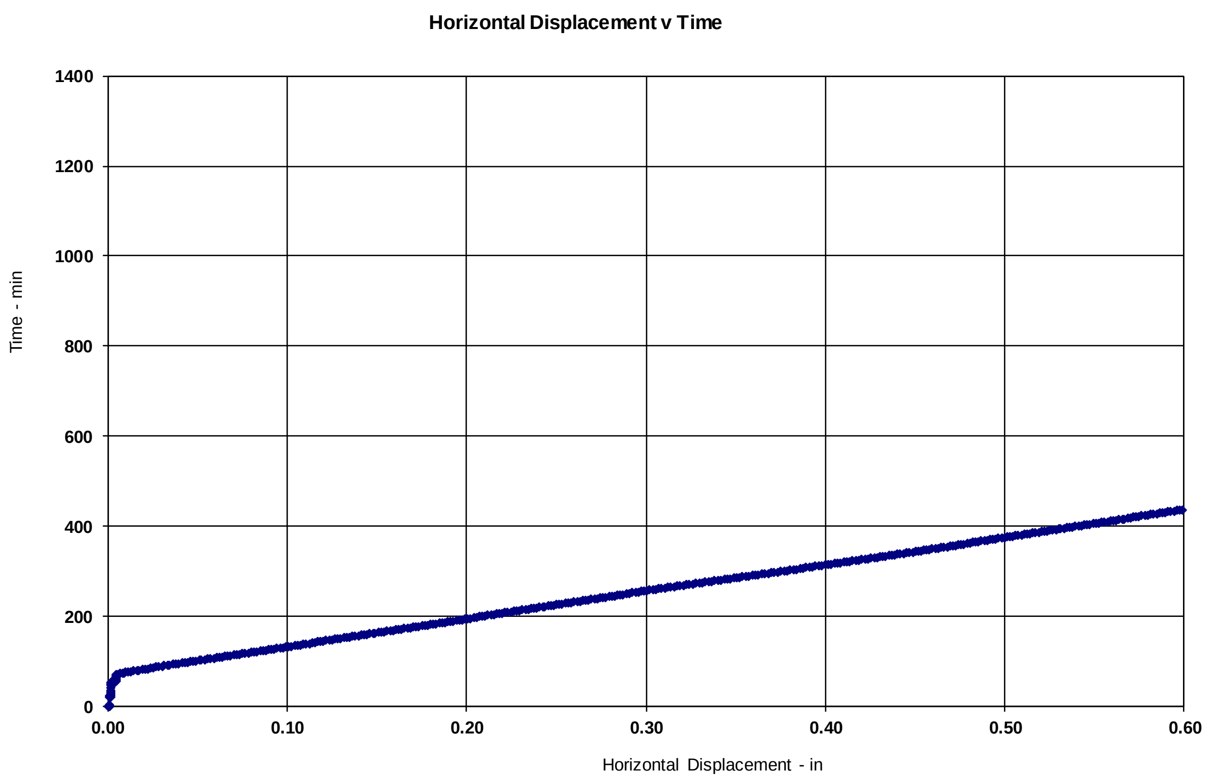
Test 14 - Normal Stress = 20 lb/in2. Crushed Rock, POORLY GRADED GRAVEL (GP), 96 percent gravel, 3 percent sand, 1 percent clayey fines, approximately 20 percent flat and 20 percent elongated particles. Particles break with moderate hammer blow. Interface Coefficients of Friction: $\mu_p = 0.22$, $\mu_r = 0.16$.



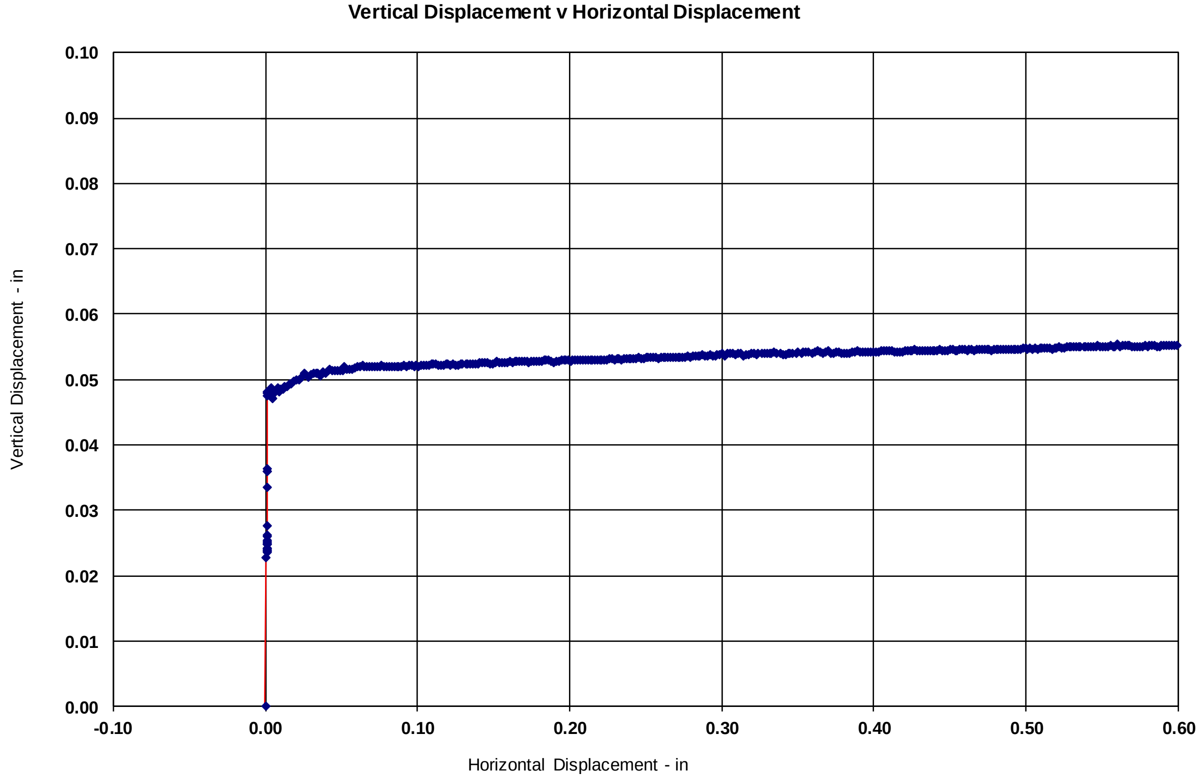


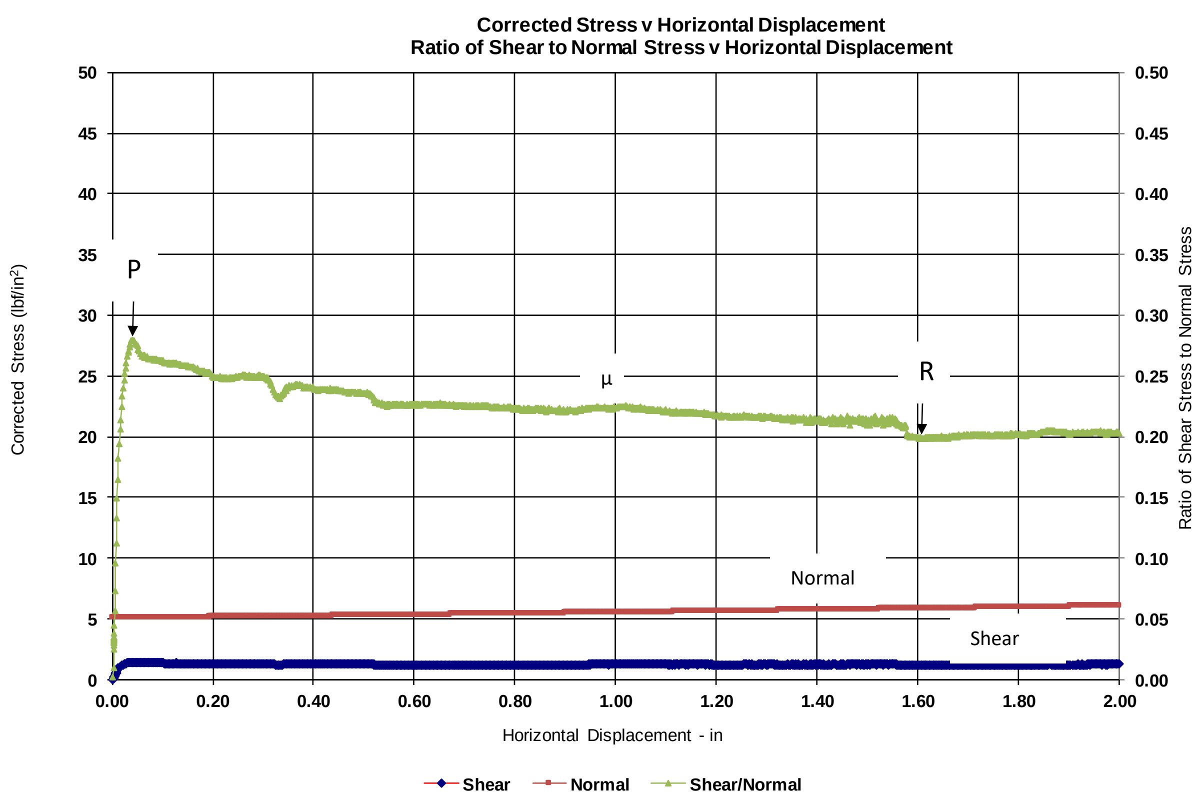
Test 15- Normal Stress = 5 lb/in2. Crushed Rock, POORLY GRADED GRAVEL (GP), 96 percent gravel, 3 percent sand, 1 percent clayey fines, approximately 20 percent flat and 20 percent elongated particles. Particles break with moderate hammer blow. Interface Coefficients of Friction: $\mu_p = 0.28$ $\mu_r = 0.20$.



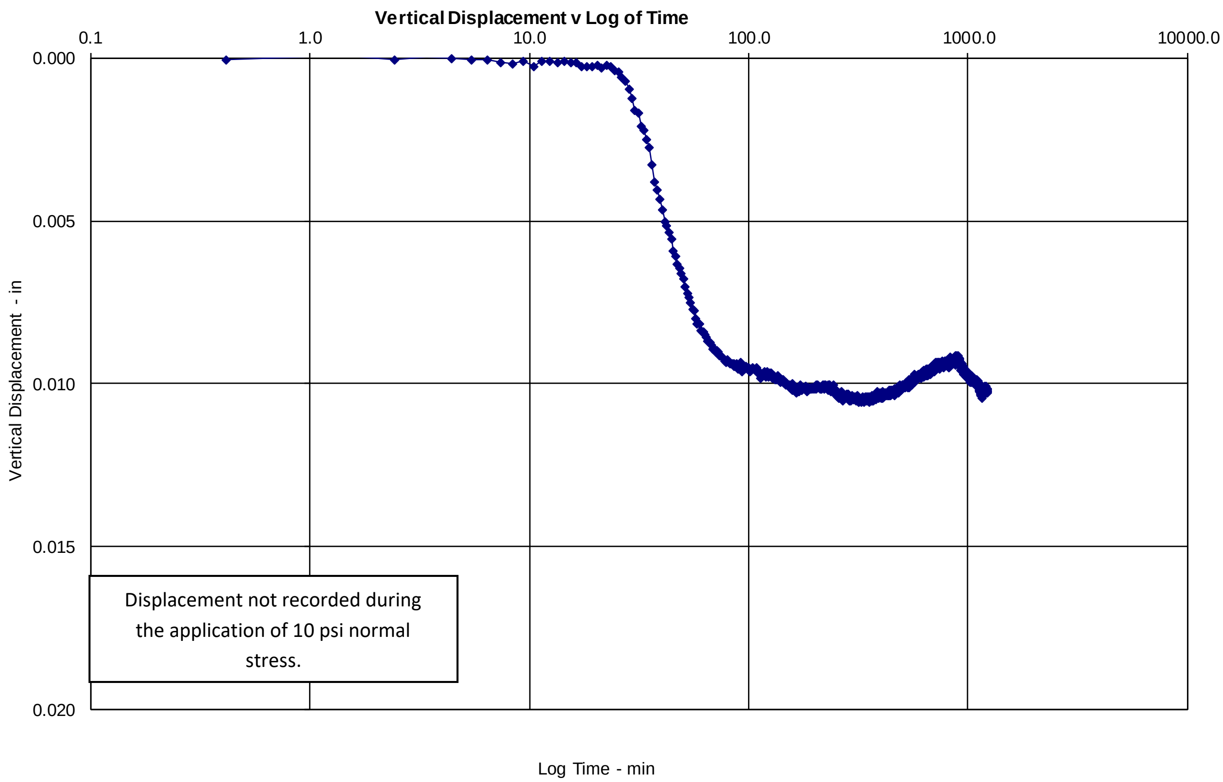


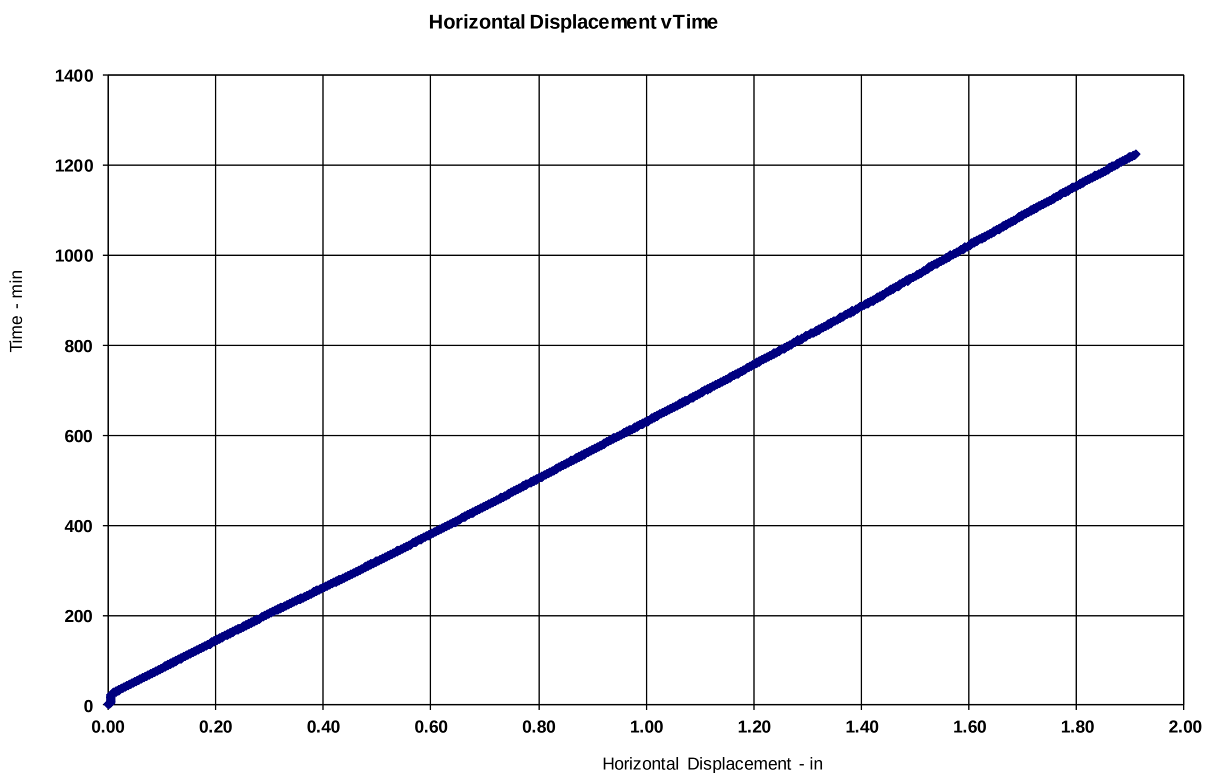
Test 15- Normal Stress = 5 lb/in2. Crushed Rock, POORLY GRADED GRAVEL (GP), 96 percent gravel, 3 percent sand, 1 percent clayey fines, approximately 20 percent flat and 20 percent elongated particles. Particles break with moderate hammer blow. Interface Coefficients of Friction: $\mu_p = 0.28$ $\mu_r = 0.20$.



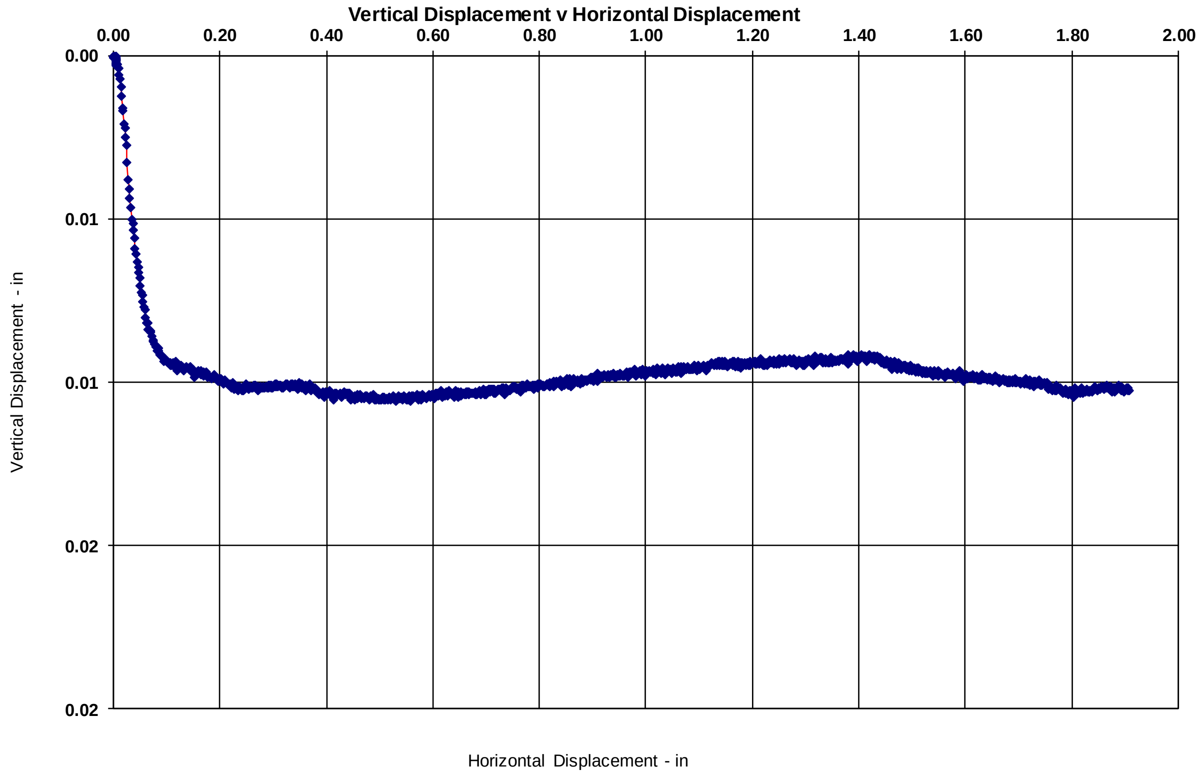


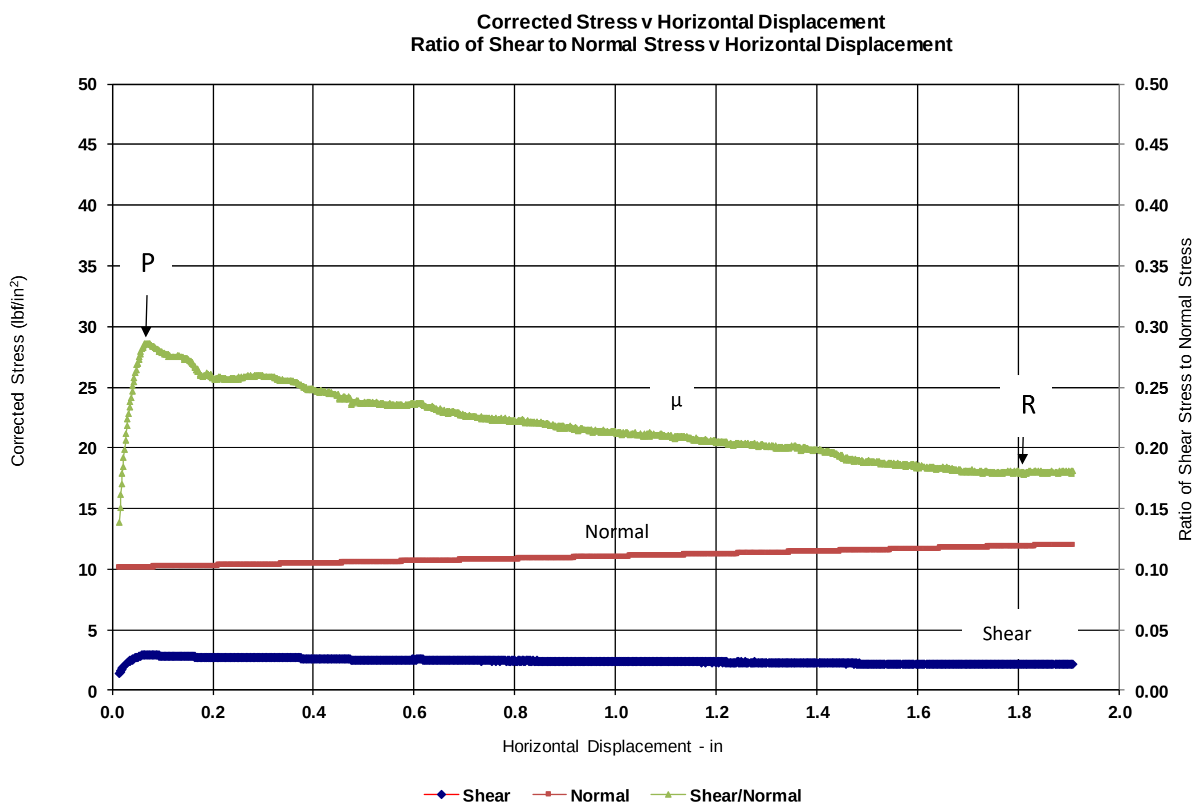
Test 16 - Normal Stress = 10 lb/in2. Crushed Rock, POORLY GRADED GRAVEL (GP), 96 percent gravel, 3 percent sand, 1 percent clayey fines, approximately 20 percent flat and 20 percent elongated particles. Particles break with moderate hammer blow. Interface Coefficients of Friction: $\mu_p = 0.29$ $\mu_r = 0.18$.



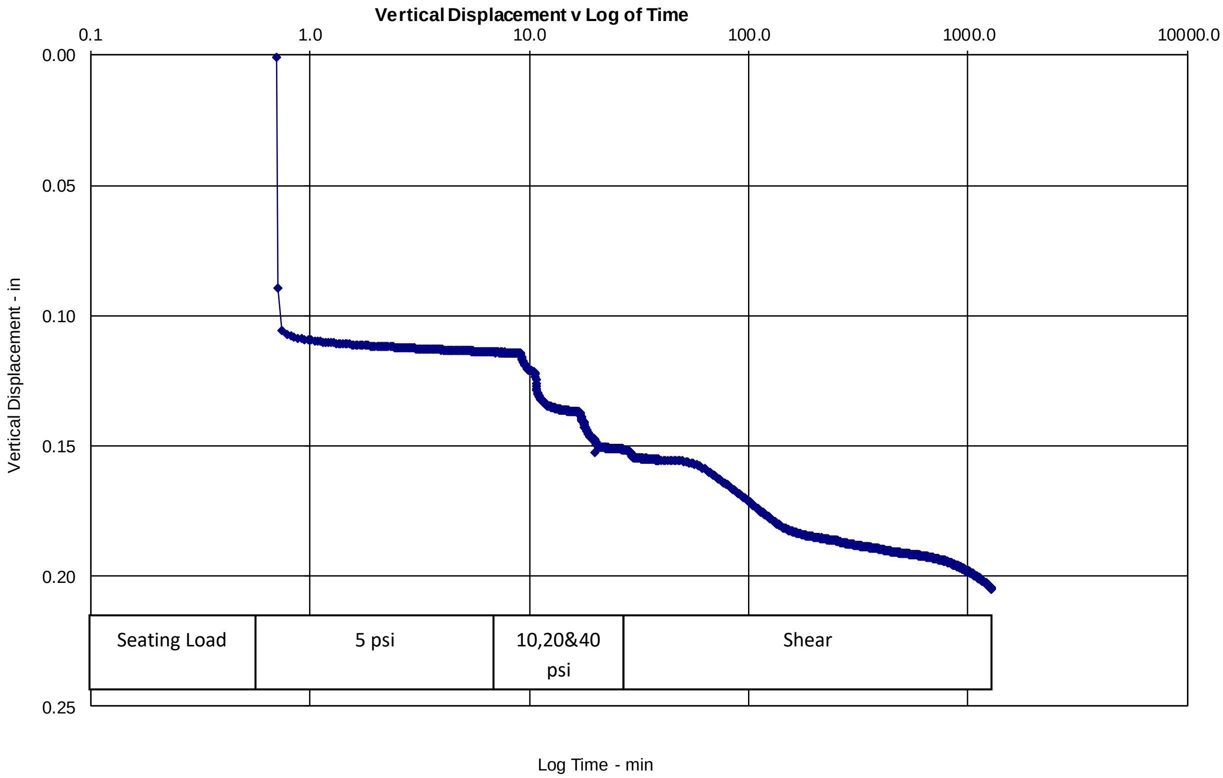


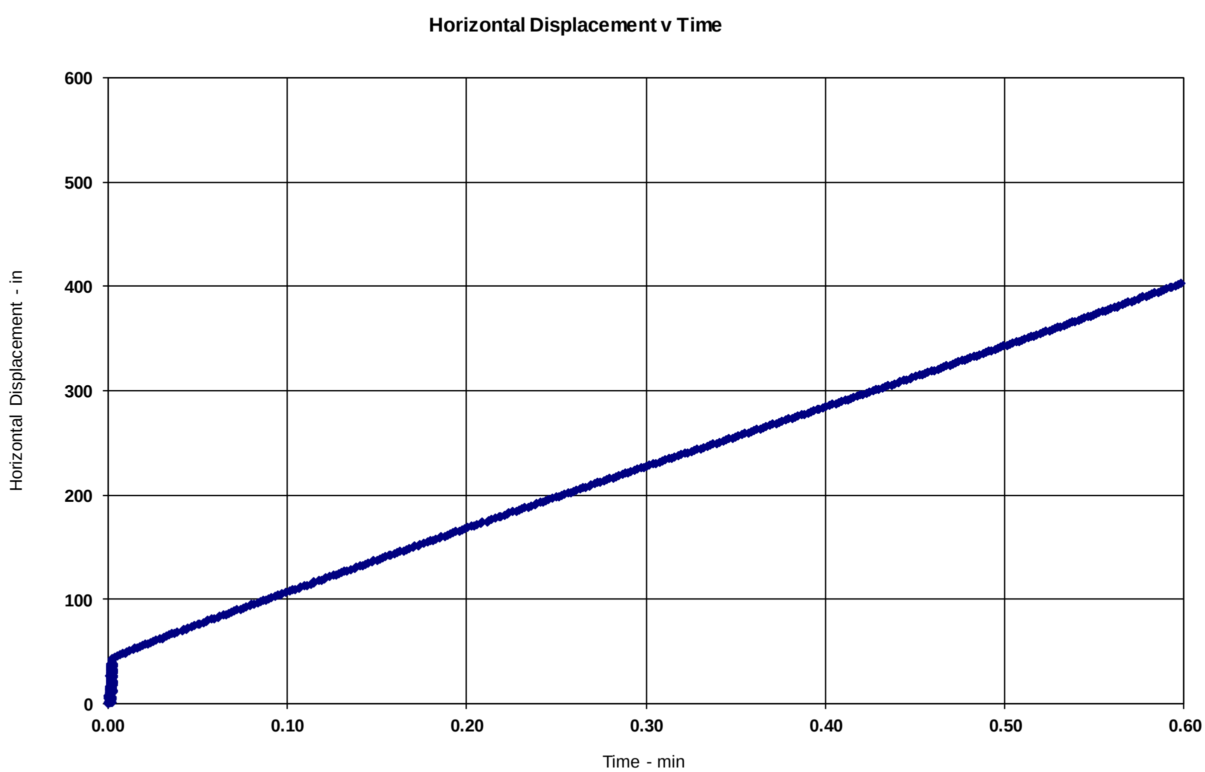
Test 16 - Normal Stress = 10 lb/in2. Crushed Rock, POORLY GRADED GRAVEL (GP), 96 percent gravel, 3 percent sand, 1 percent clayey fines, approximately 20 percent flat and 20 percent elongated particles. Particles break with moderate hammer blow. Interface Coefficients of Friction: μ_p = 0.29 μ_r = 0.18.



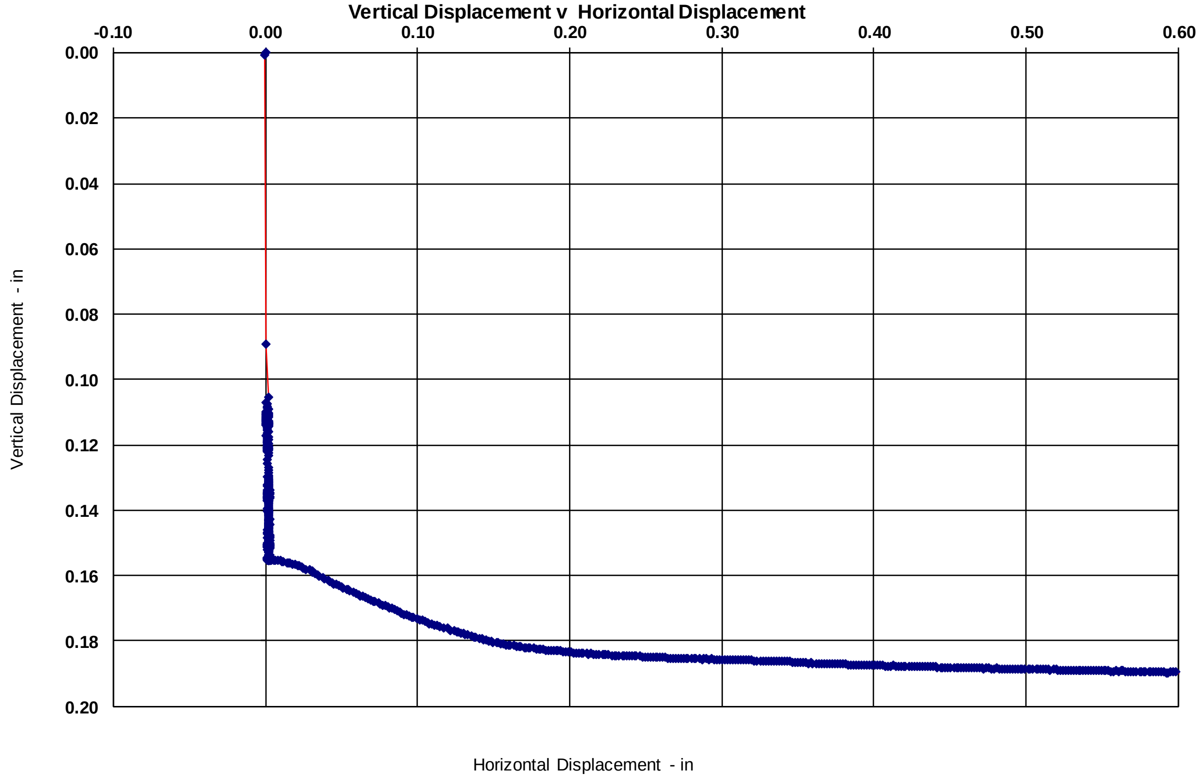


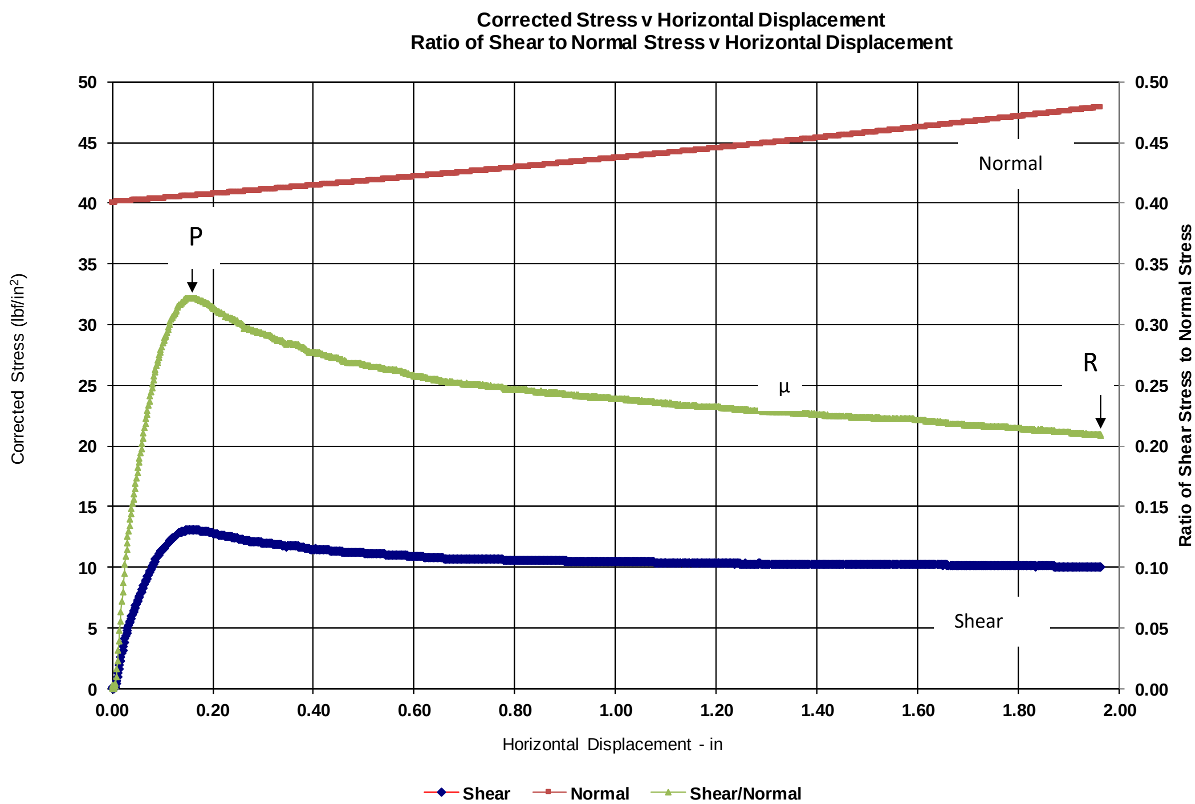
Test 17 - Normal Stress = 40 lb/in2. Crushed Rock, POORLY GRADED GRAVEL (GP), 96 percent gravel, 3 percent sand, 1 percent clayey fines, approximately 20 percent flat and 20 percent elongated particles. Particles break with moderate hammer blow. Interface Coefficients of Friction: μ_p = 0.32 μ_r = 0.21.



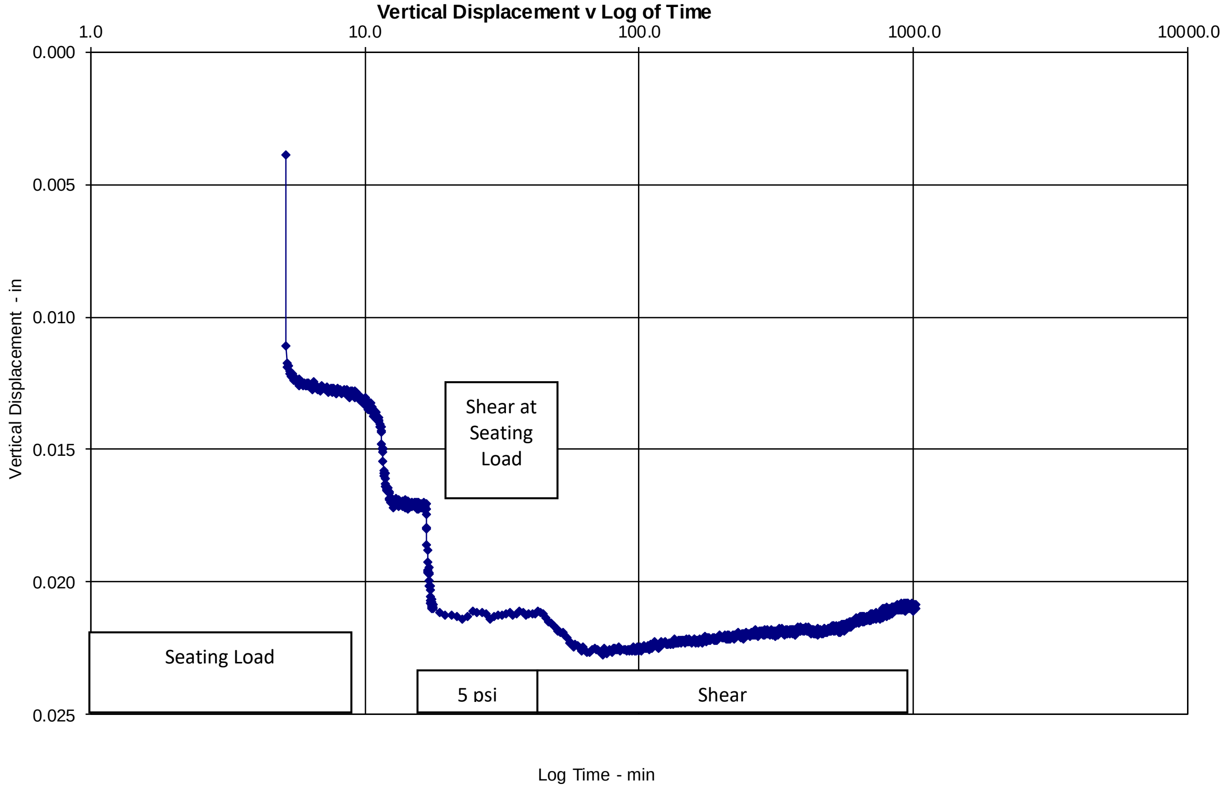


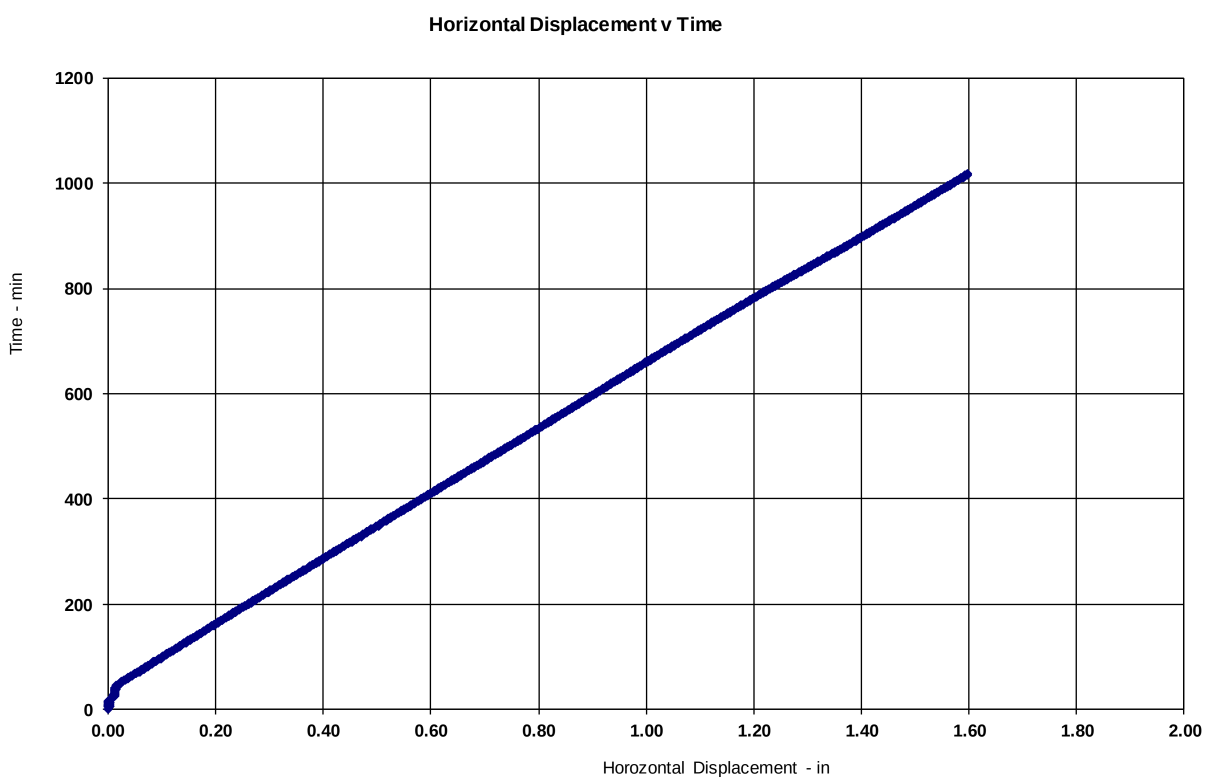
Test 17 - Normal Stress = 40 lb/in2. Crushed Rock, POORLY GRADED GRAVEL (GP), 96 percent gravel, 3 percent sand, 1 percent clayey fines, approximately 20 percent flat and 20 percent elongated particles. Particles break with moderate hammer blow. Interface Coefficients of Friction: $\mu_p = 0.32$ $\mu_r = 0.21$.



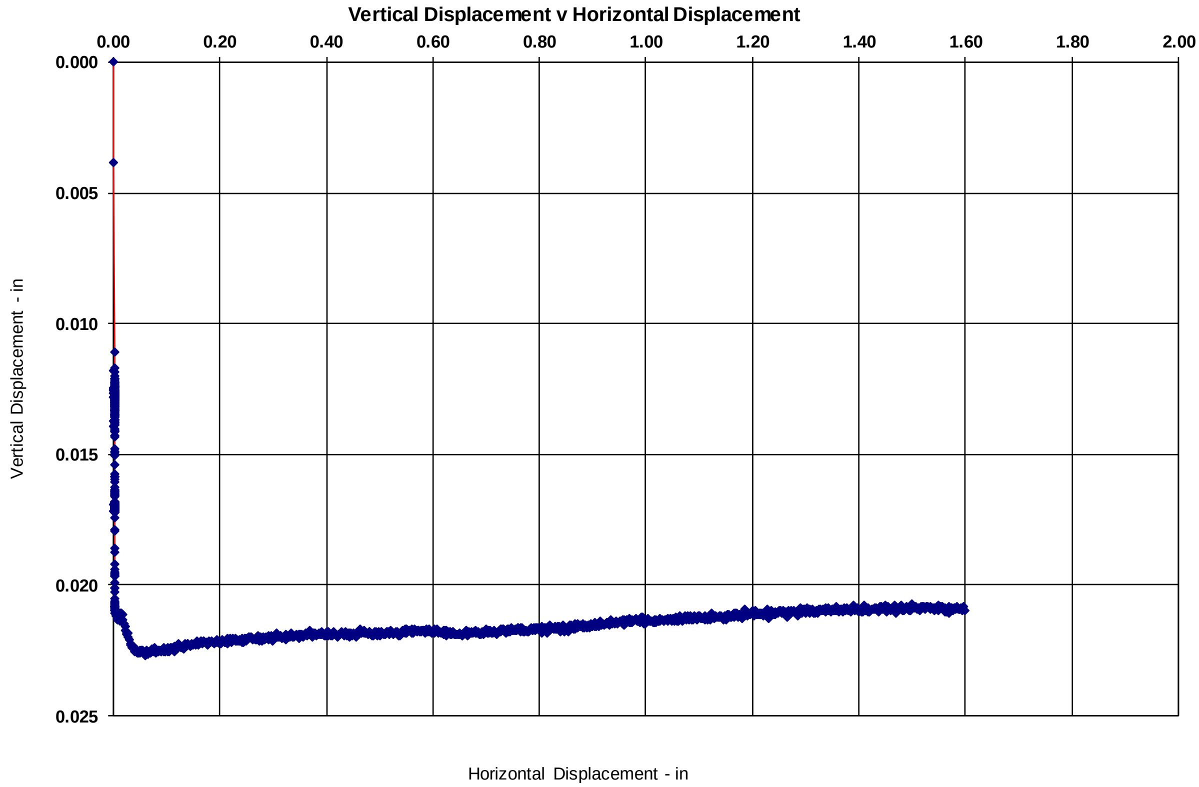


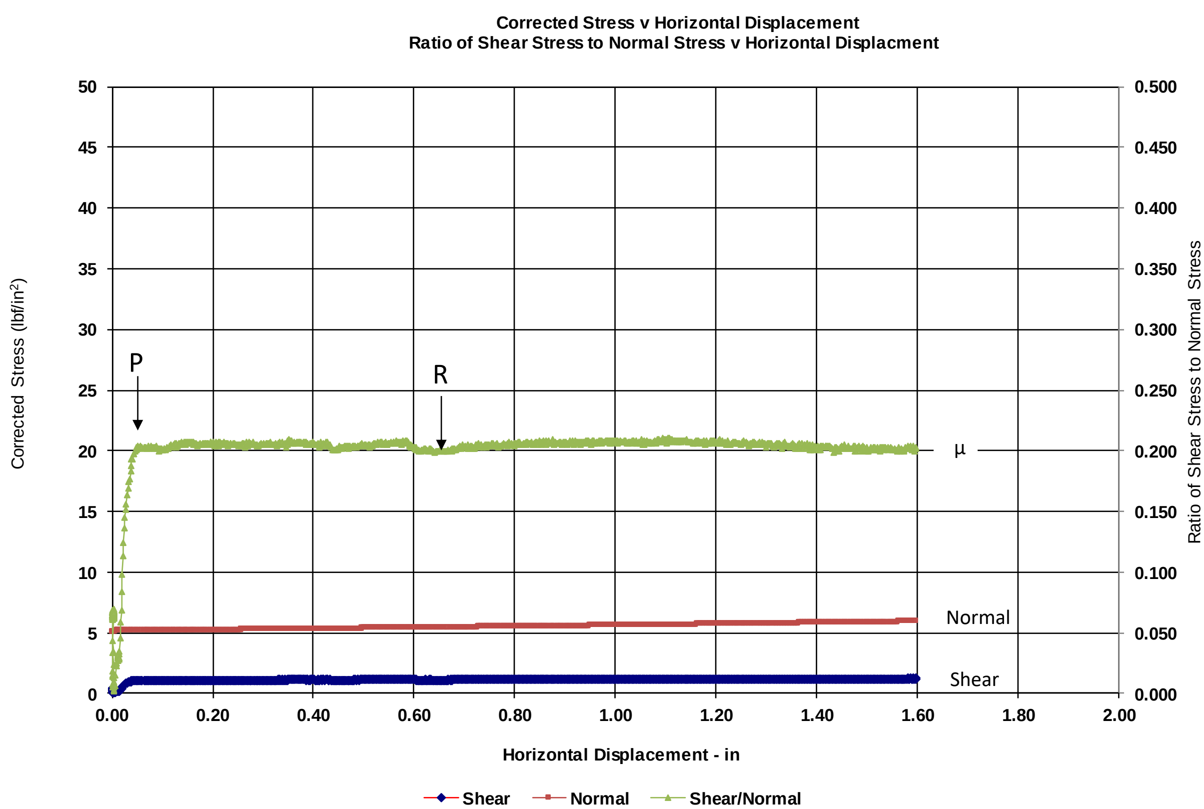
Test 18 - Normal Stress = 5 lb/in^2. POORLY GRADED SAND (SP), 100 percent hard, medium-size, quartz sand. Particles break with moderate hammer blow. Interface Coefficients of Friction:
$\mu_p = 0.20$, $\mu_r = 0.20$.



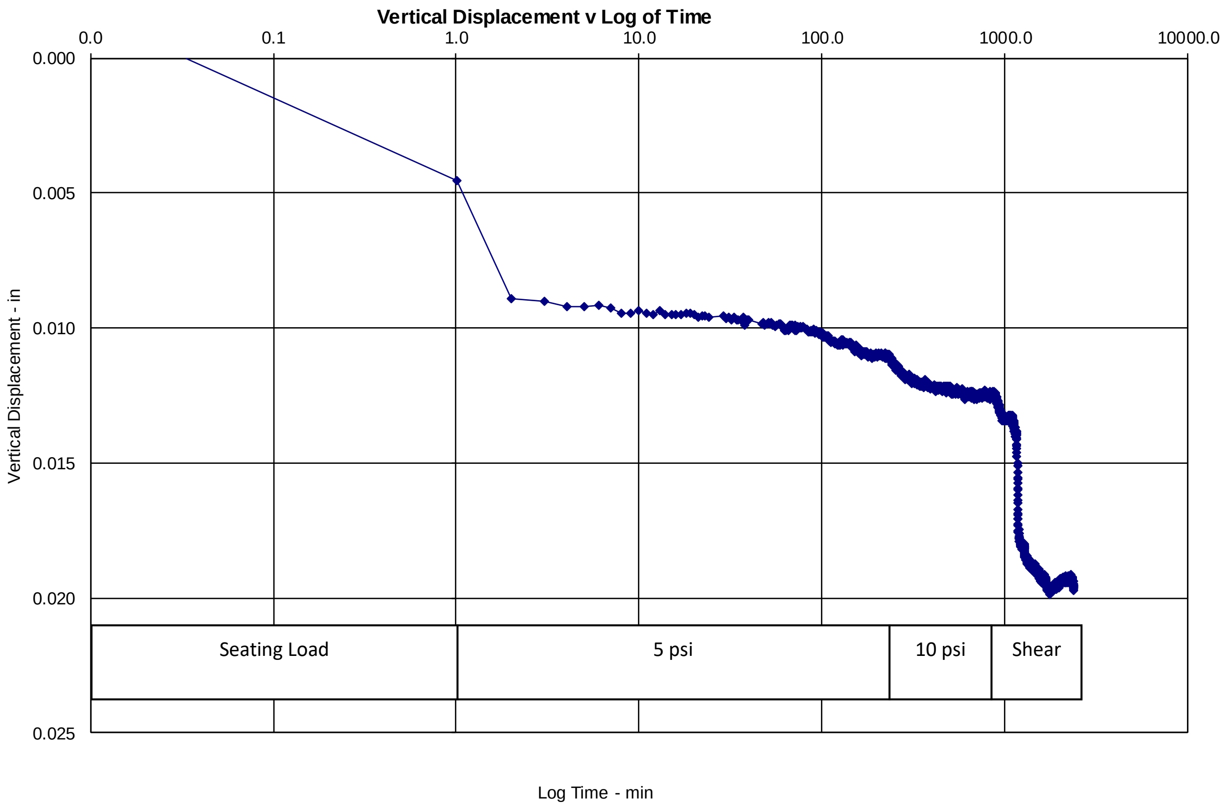


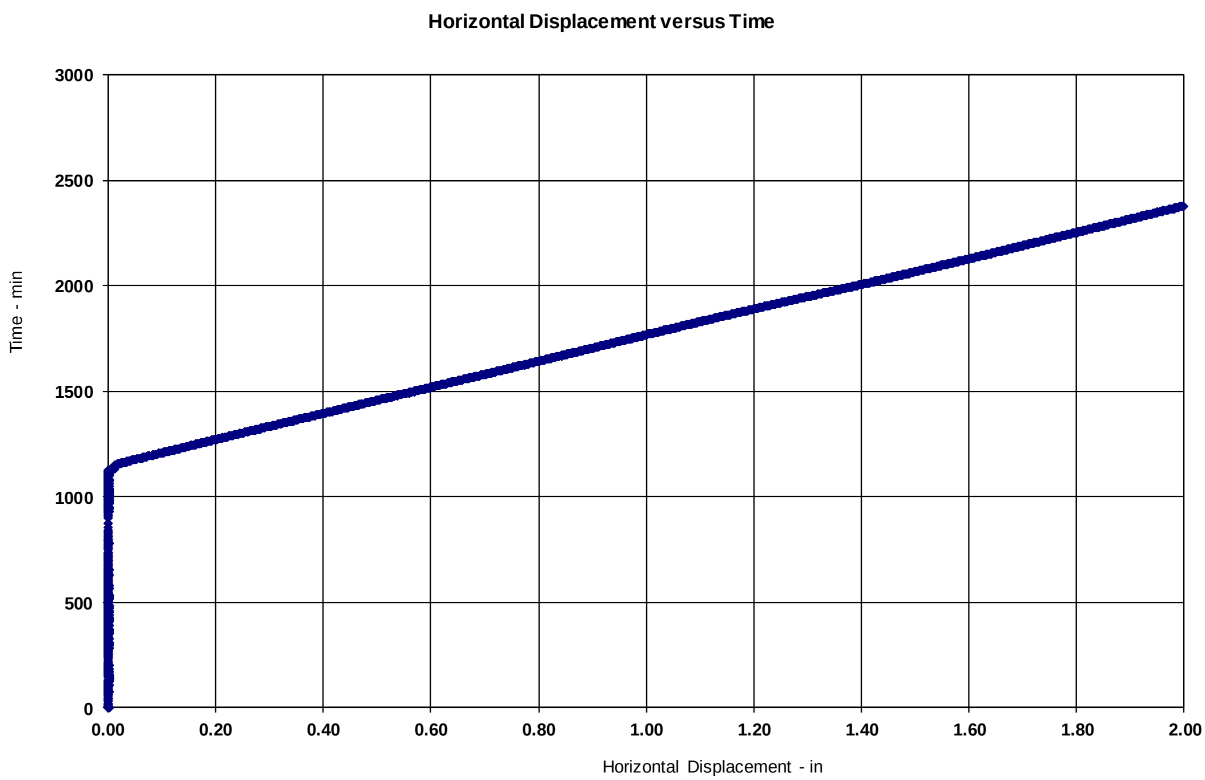
Test 18 - Normal Stress = 5 lb/in^2. POORLY GRADED SAND (SP), 100 percent hard, medium-size, quartz sand. Particles break with moderate hammer blow. Interface Coefficients of Friction:
μ_p = 0.20, μ_r = 0.20.



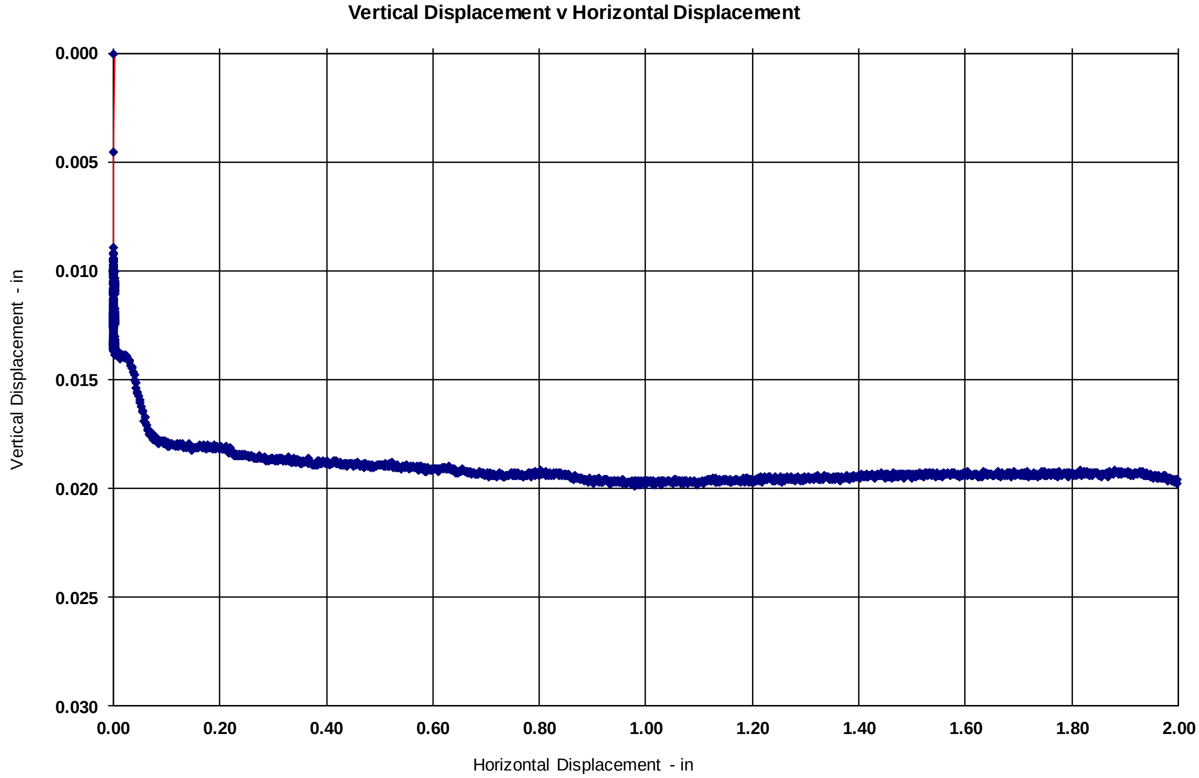


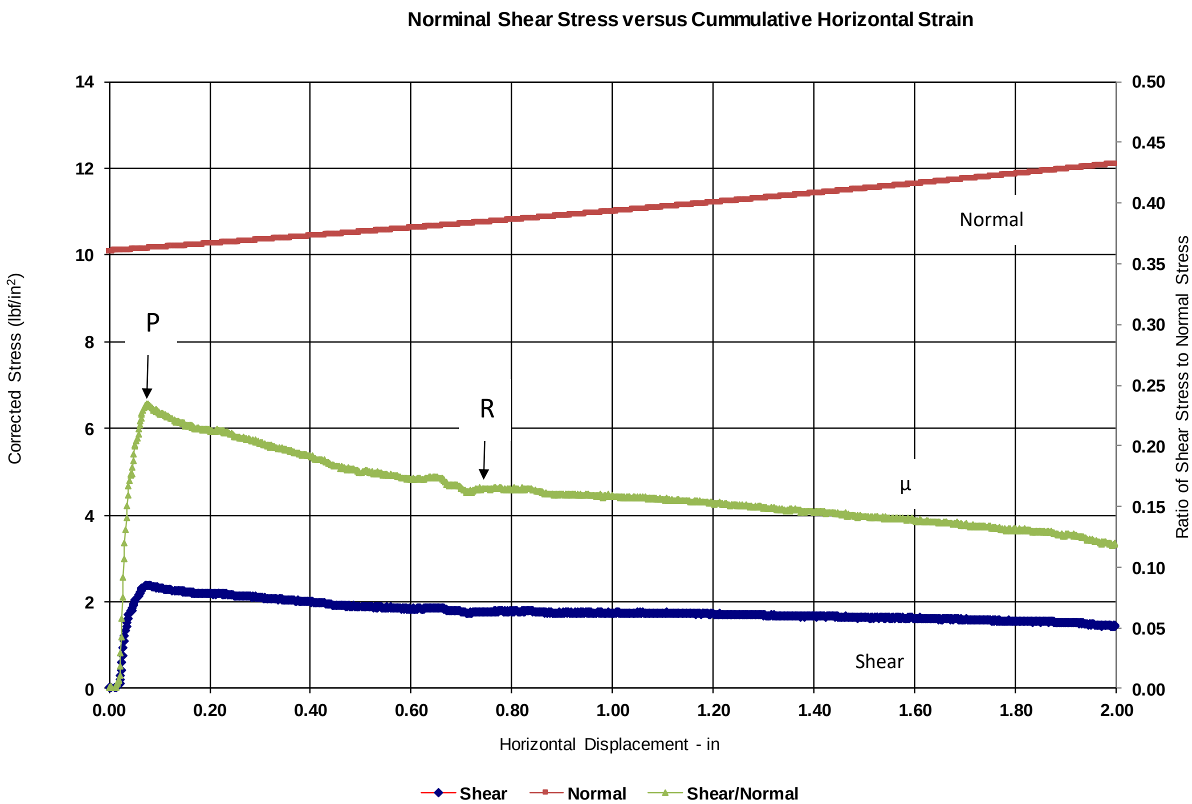
Test 19 - Normal Stress = 10 lb/in2. Crushed Rock, POORLY GRADED GRAVEL (GP), 96 percent gravel, 3 percent sand, 1 percent clayey fines, approximately 20 percent flat and 20 percent elongated particles. Particles break with moderate hammer blow. Interface Coefficients of Friction: μ_p = 0.23, μ_r = 0.16.



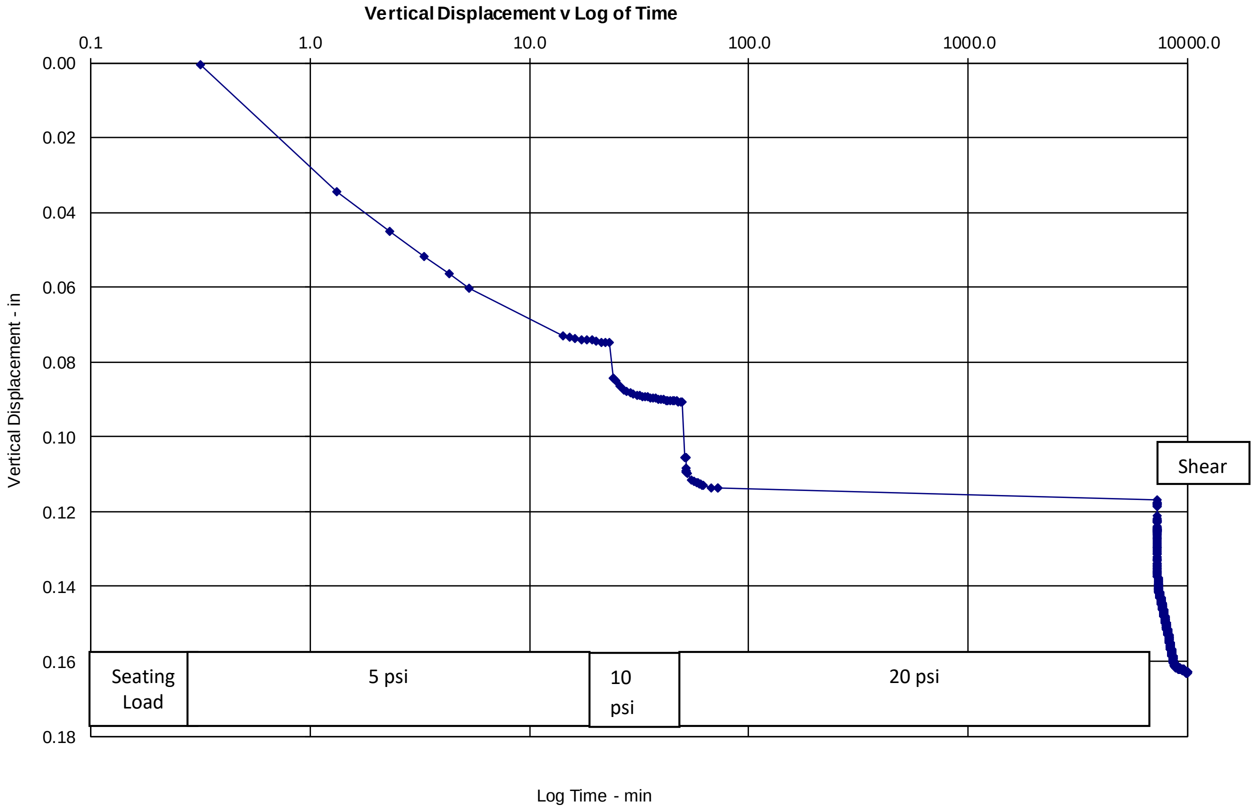


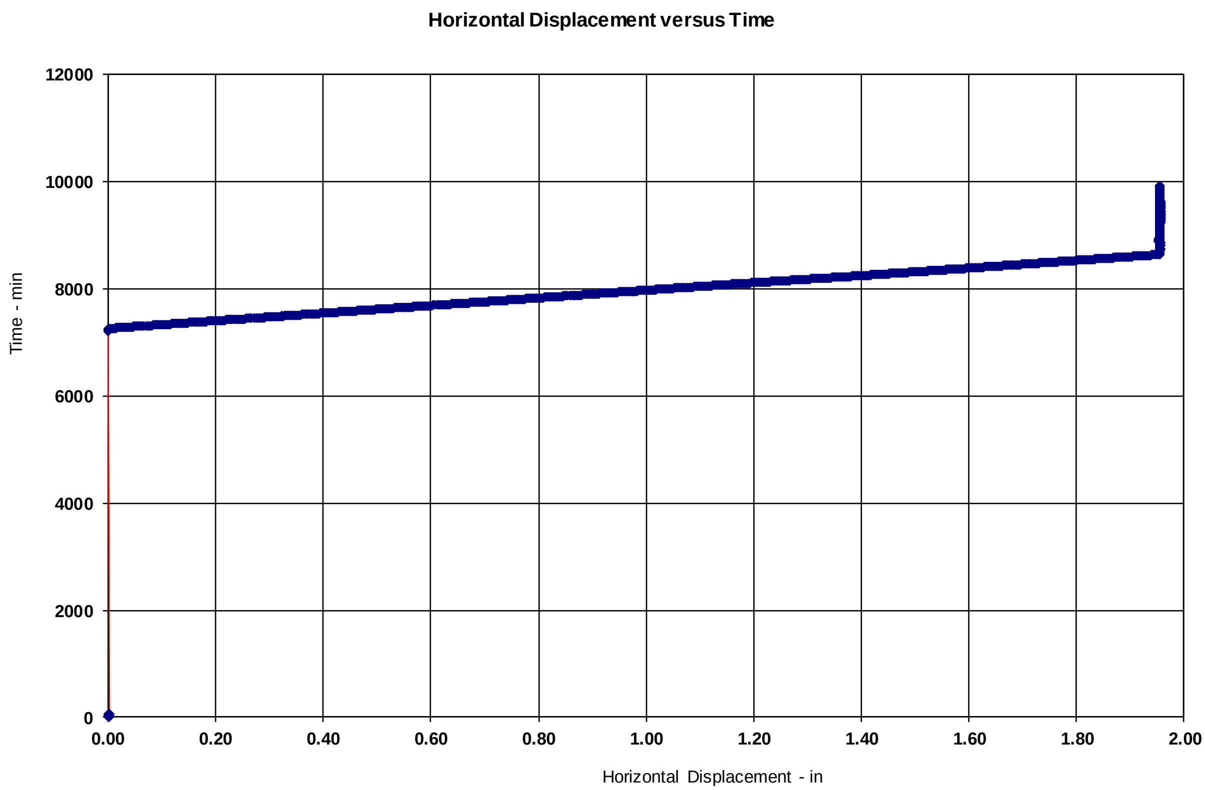
Test 19 - Normal Stress = 10 lb/in2. Crushed Rock, POORLY GRADED GRAVEL (GP), 96 percent gravel, 3 percent sand, 1 percent clayey fines, approximately 20 percent flat and 20 percent elongated particles. Particles break with moderate hammer blow. Interface Coefficients of Friction: μ_p = 0.23, μ_r = 0.16.



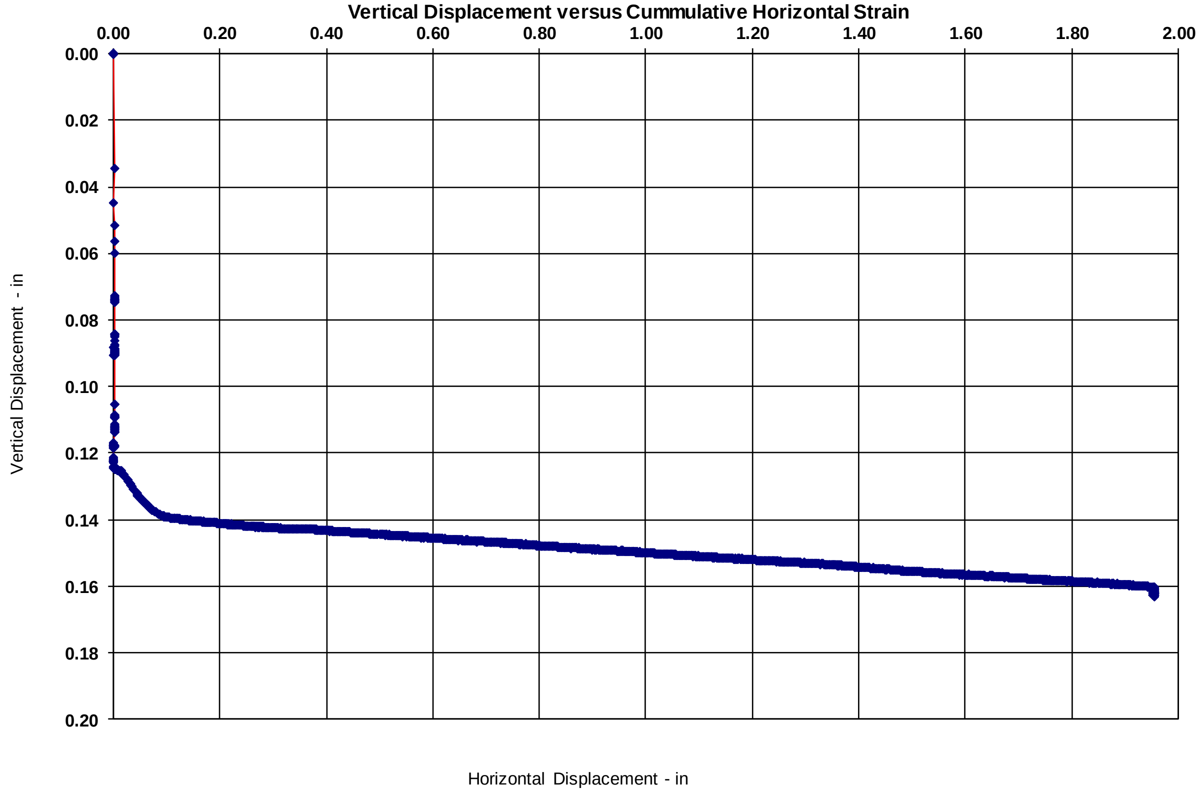


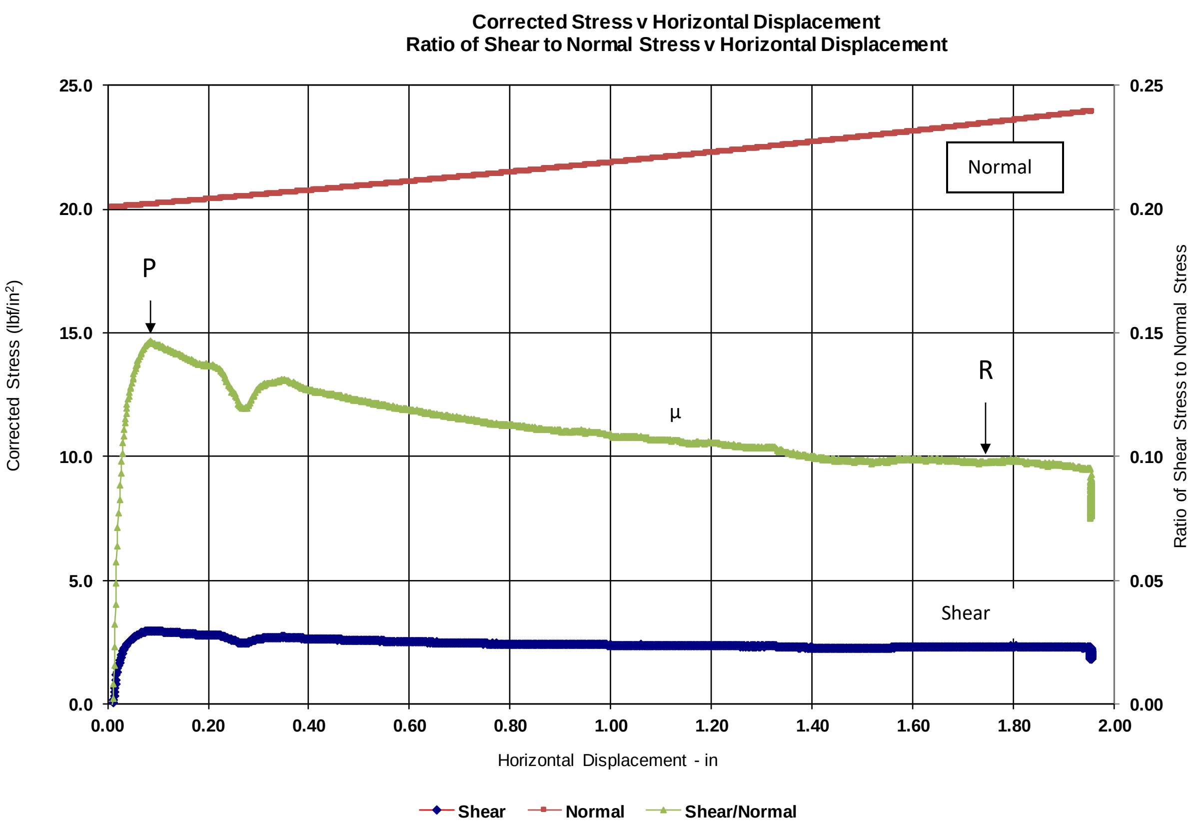
Test 20 - Normal Stress = 20 lb/in^2. SILTY SAND (SM), 73 percent hard sand, 27 percent nonplastic fines. Maximum size medium subangular to angular sand. Particles break with moderate hammer blow. Interface Coefficients of Friction: μ_p = 0.15, μ_r = 0.10.



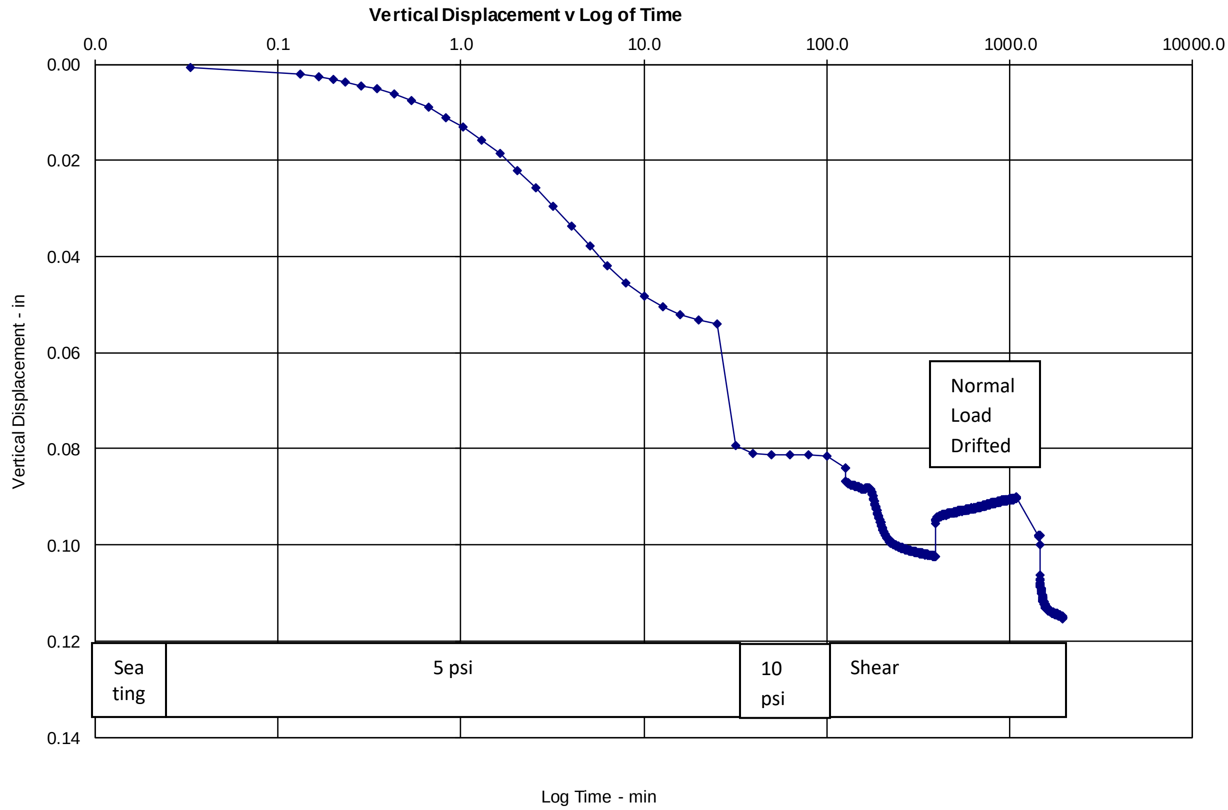


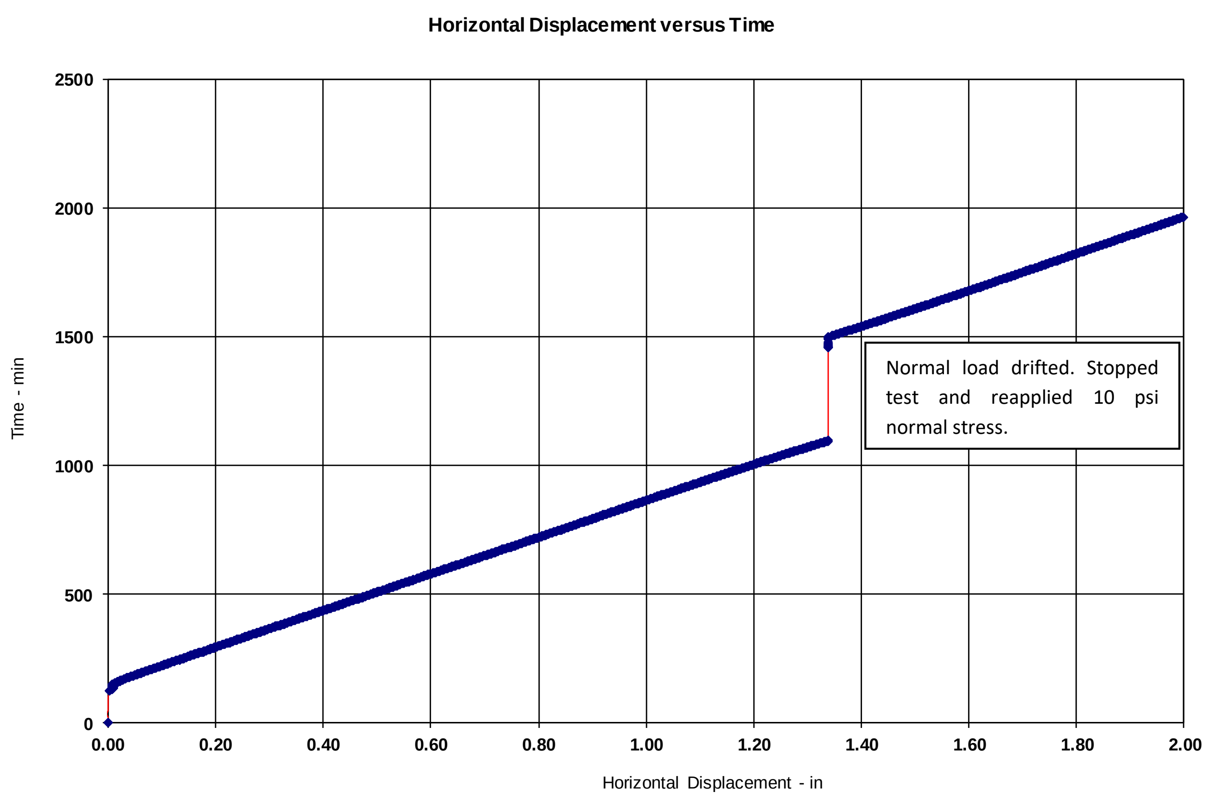
Test 20 - Normal Stress = 20 lb/in^2. SILTY SAND (SM), 73 percent hard sand, 27 percent nonplastic fines. Maximum size medium subangular to angular sand. Particles break with moderate hammer blow. Interface Coefficients of Friction: μ_p = 0.15, μ_r = 0.10.



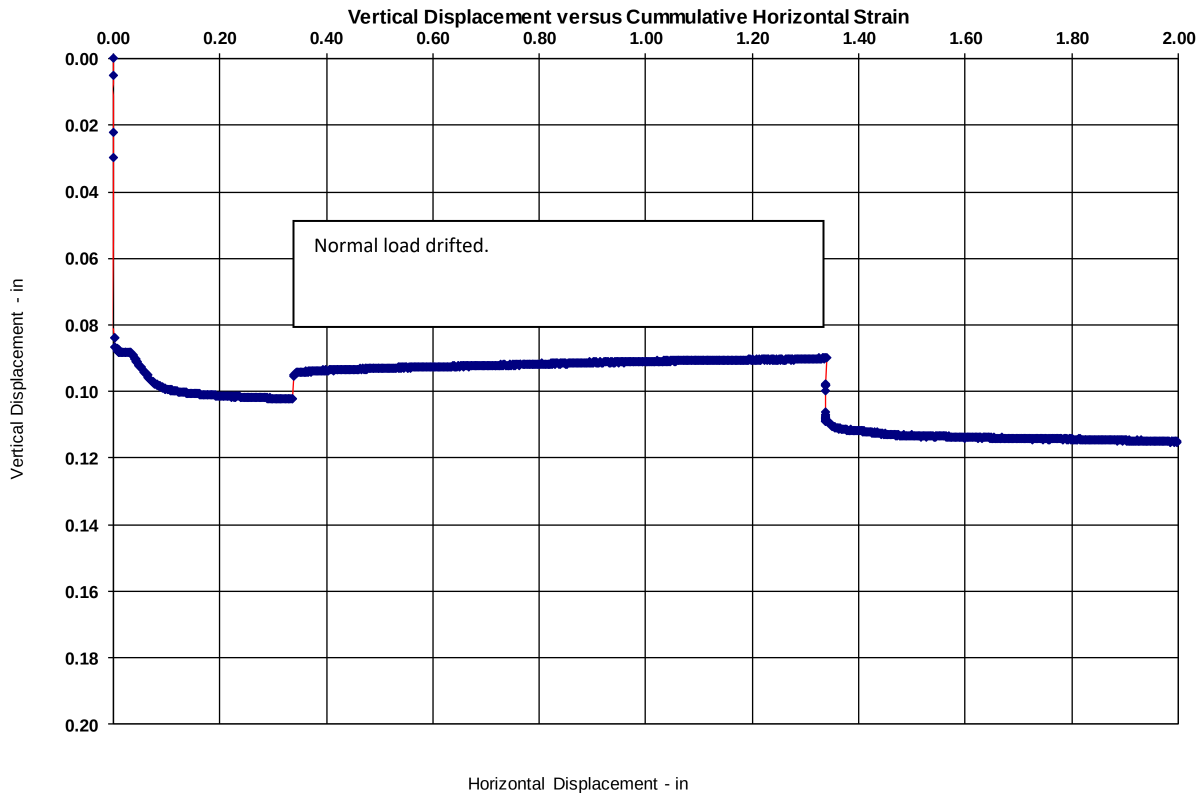


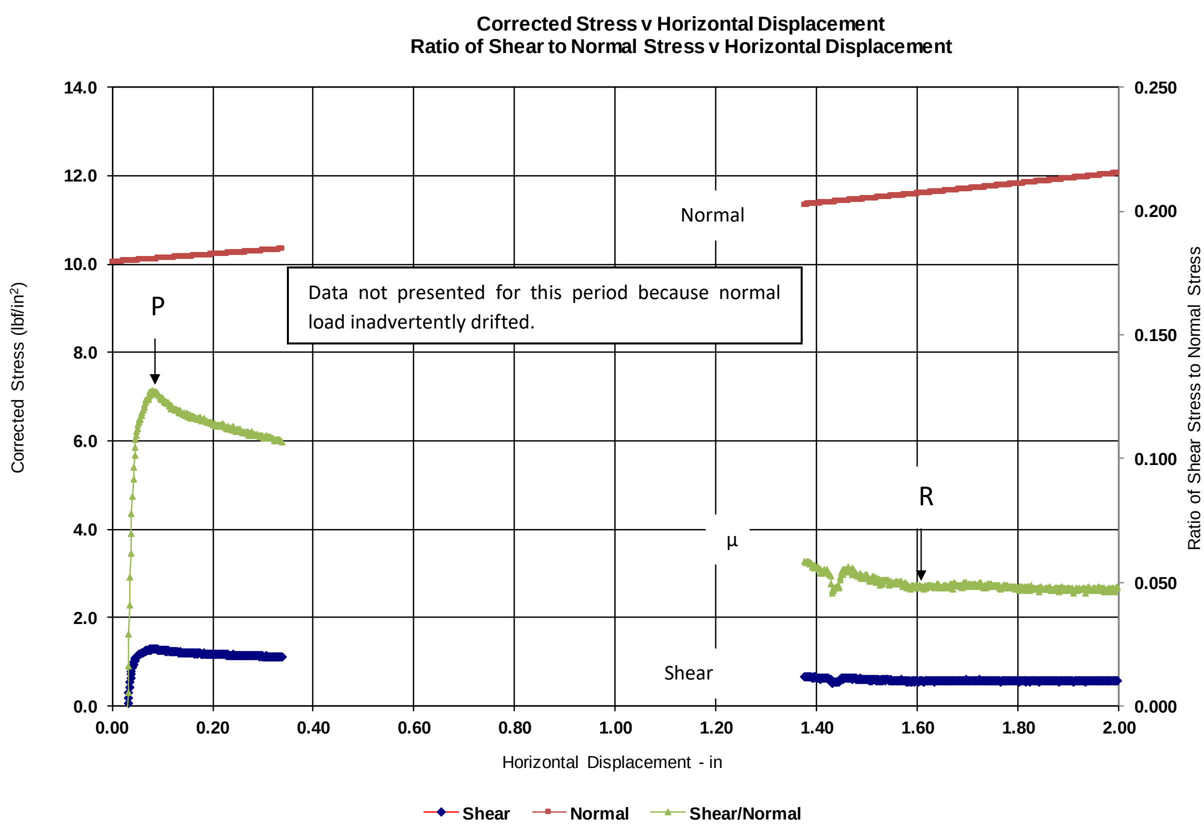
Test 21- Normal Stress = 10 lb/in^2. SILTY SAND (SM), 73 percent hard sand, 27 percent nonplastic fines. Maximum size medium subangular to angular sand. Particles break with moderate hammer blow. Interface Coefficients of Friction: μ_p = 0.13, μ_r = 0.05.



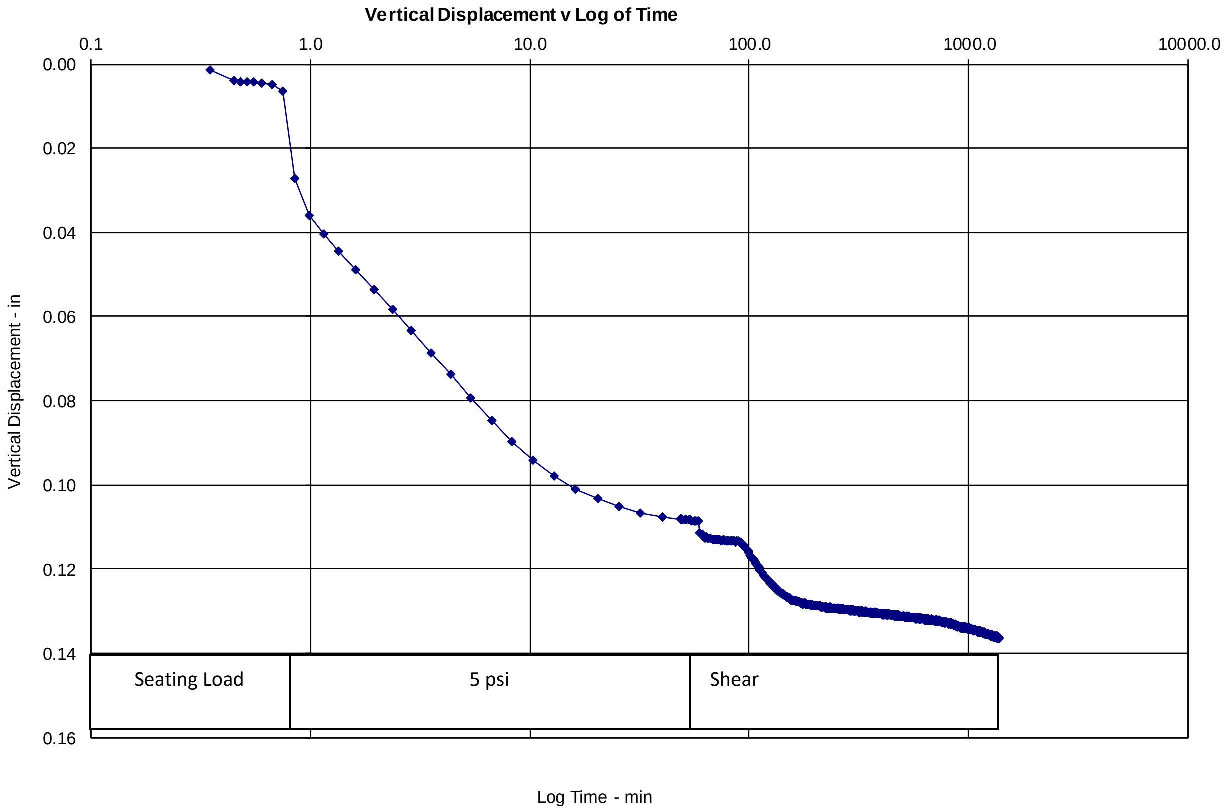


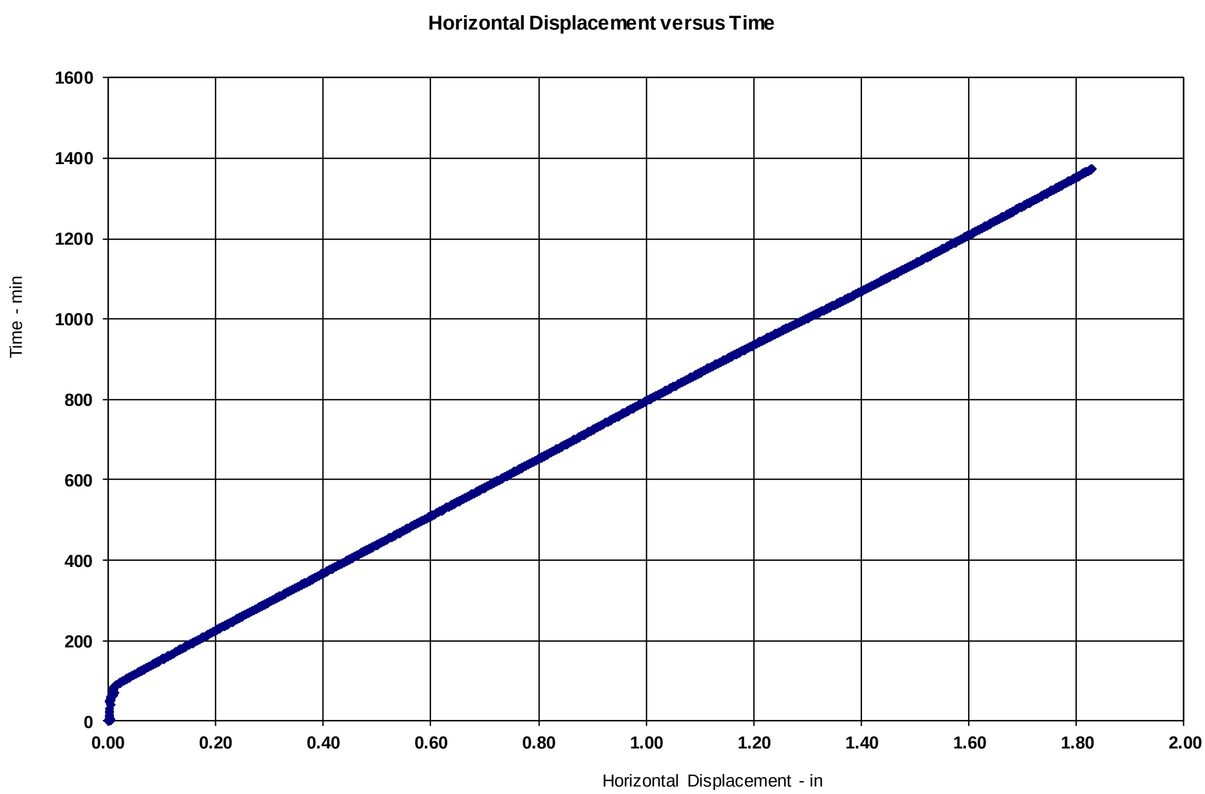
Test 21- Normal Stress = 10 lb/in^2. SILTY SAND (SM), 73 percent hard sand, 27 percent nonplastic fines. Maximum size medium subangular to angular sand. Particles break with moderate hammer blow. Interface Coefficients of Friction: μ_p = 0.13, μ_r = 0.05.



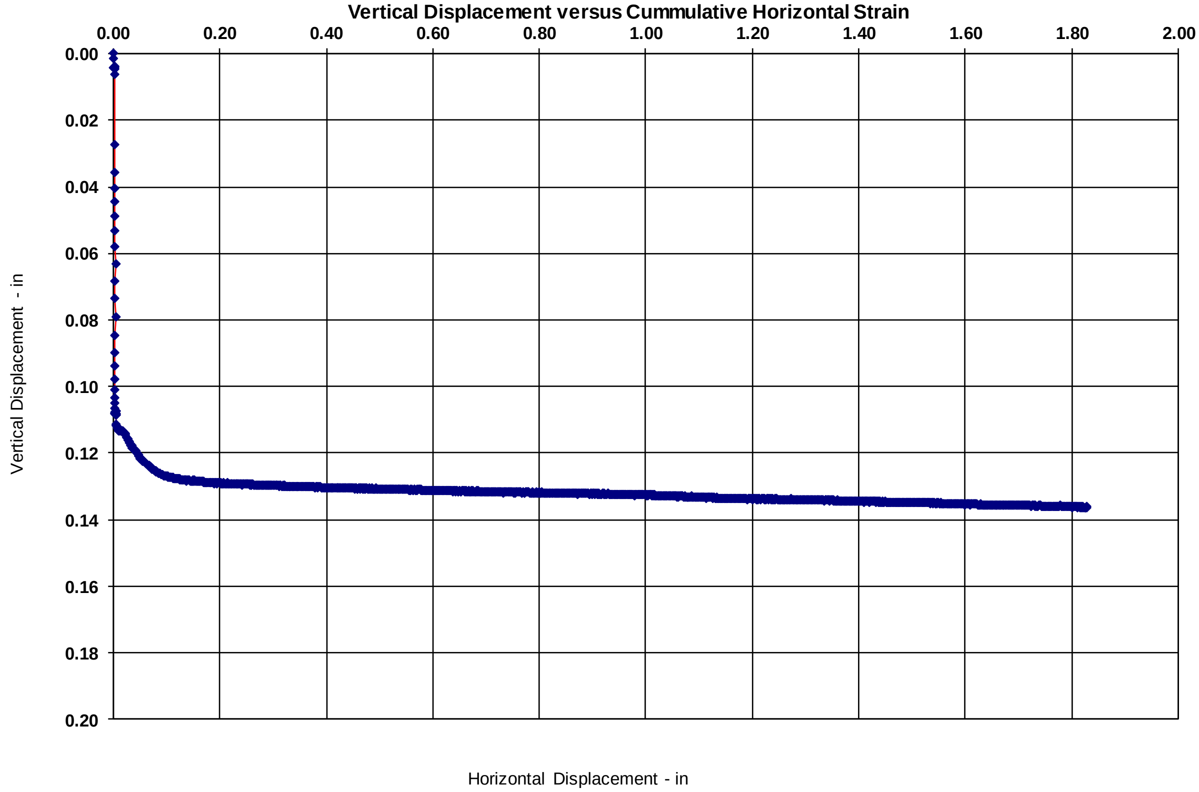


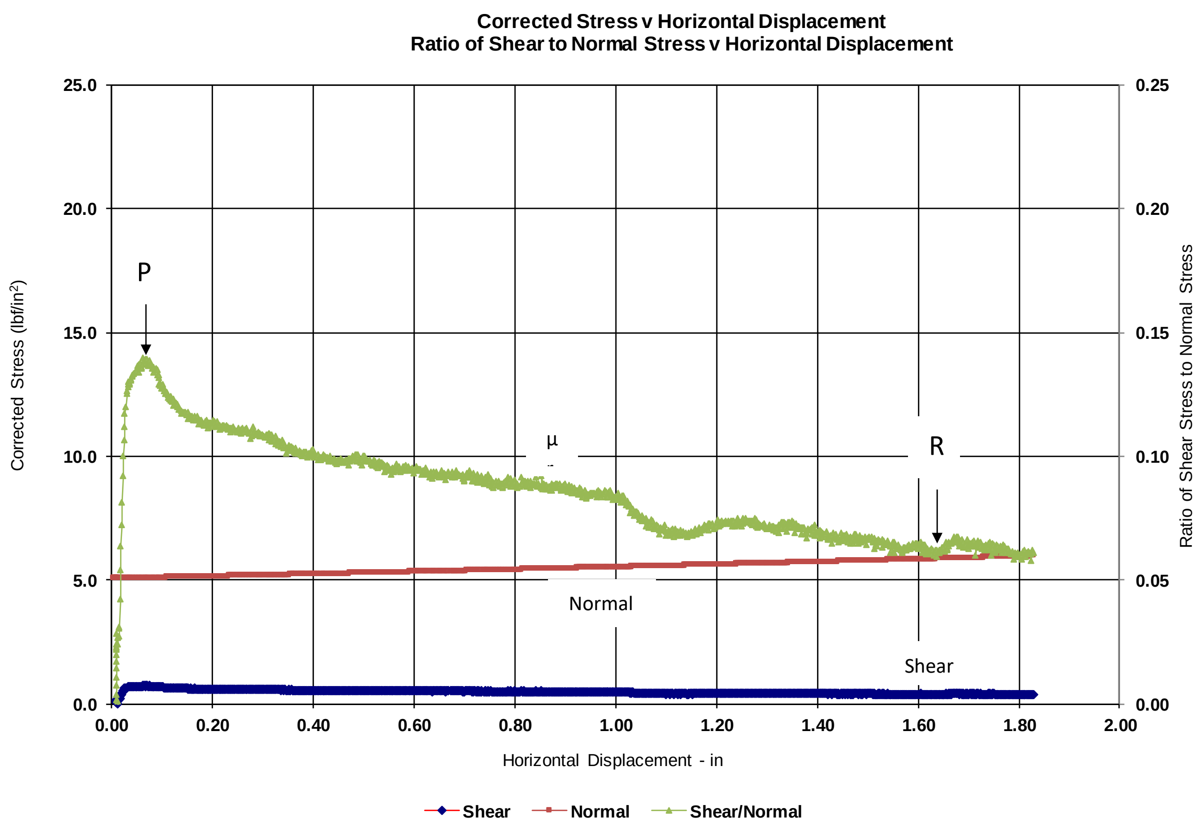
Test 22- Normal Stress = 5 lb/in^2. SILTY SAND (SM), 73 percent hard sand, 27 percent nonplastic fines. Maximum size medium subangular to angular sand. Particles break with moderate hammer blow. Interface Coefficients of Friction: μ_p = 0.14, μ_r = 0.06.



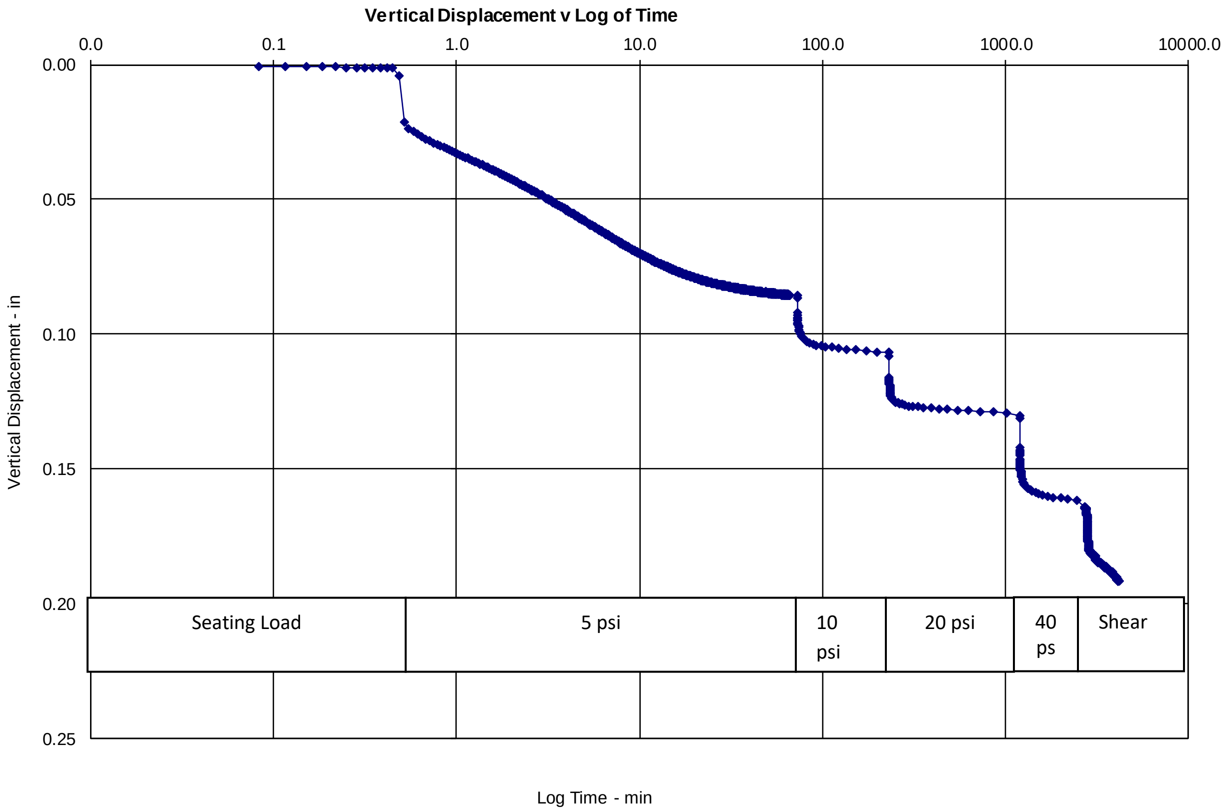


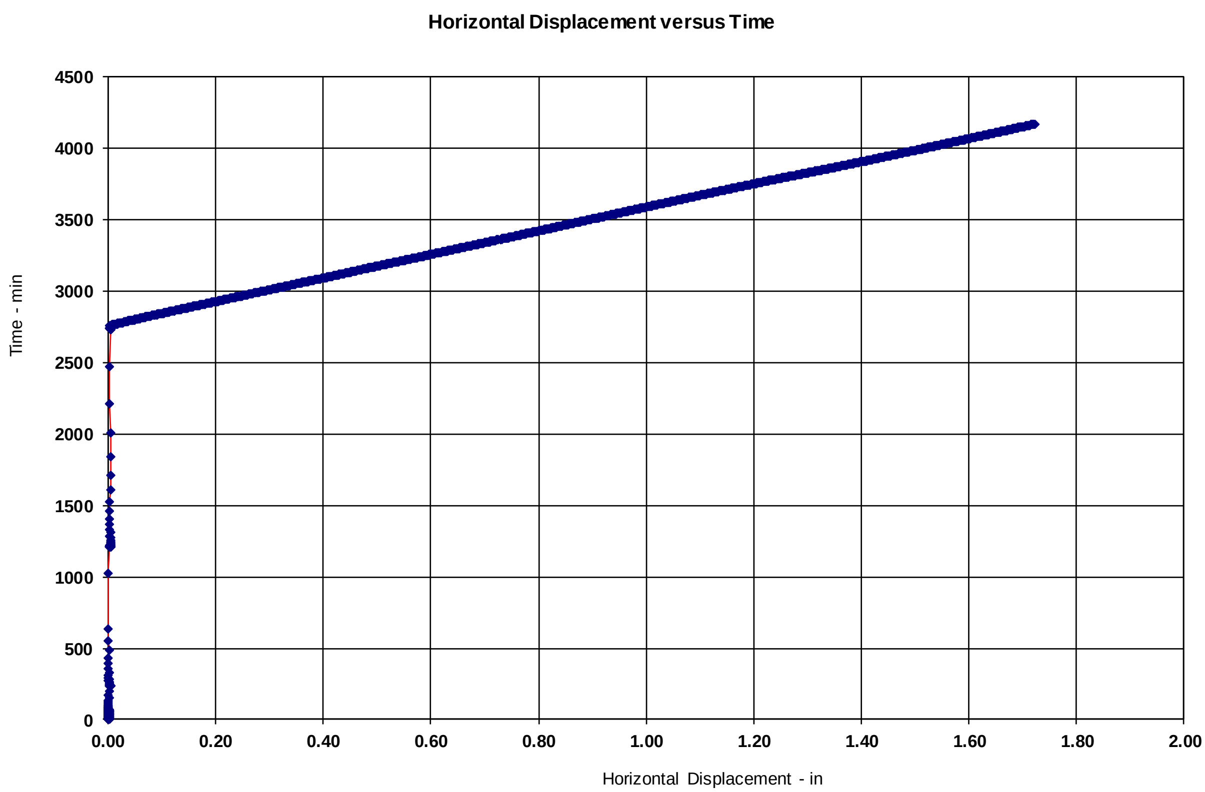
Test 22- Normal Stress = 5 lb/in^2. SILTY SAND (SM), 73 percent hard sand, 27 percent nonplastic fines. Maximum size medium subangular to angular sand. Particles break with moderate hammer blow. Interface Coefficients of Friction: μ_p = 0.14, μ_r = 0.06.



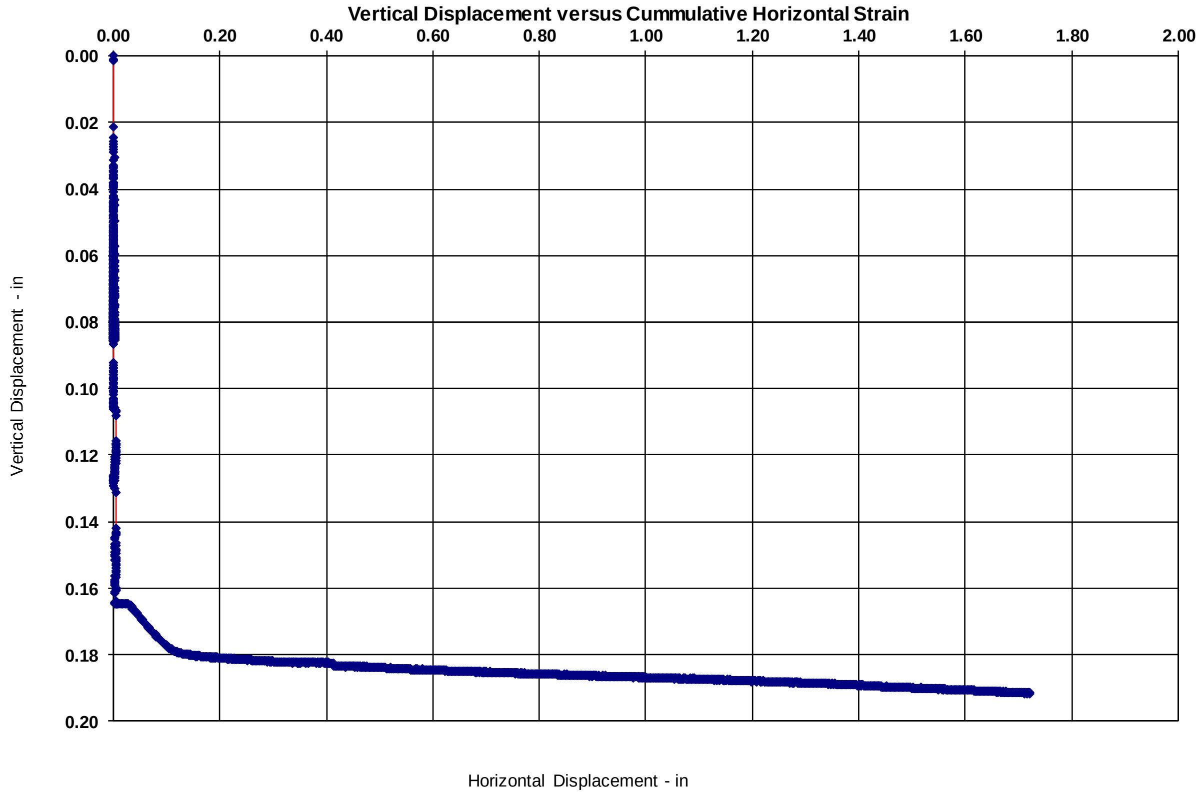


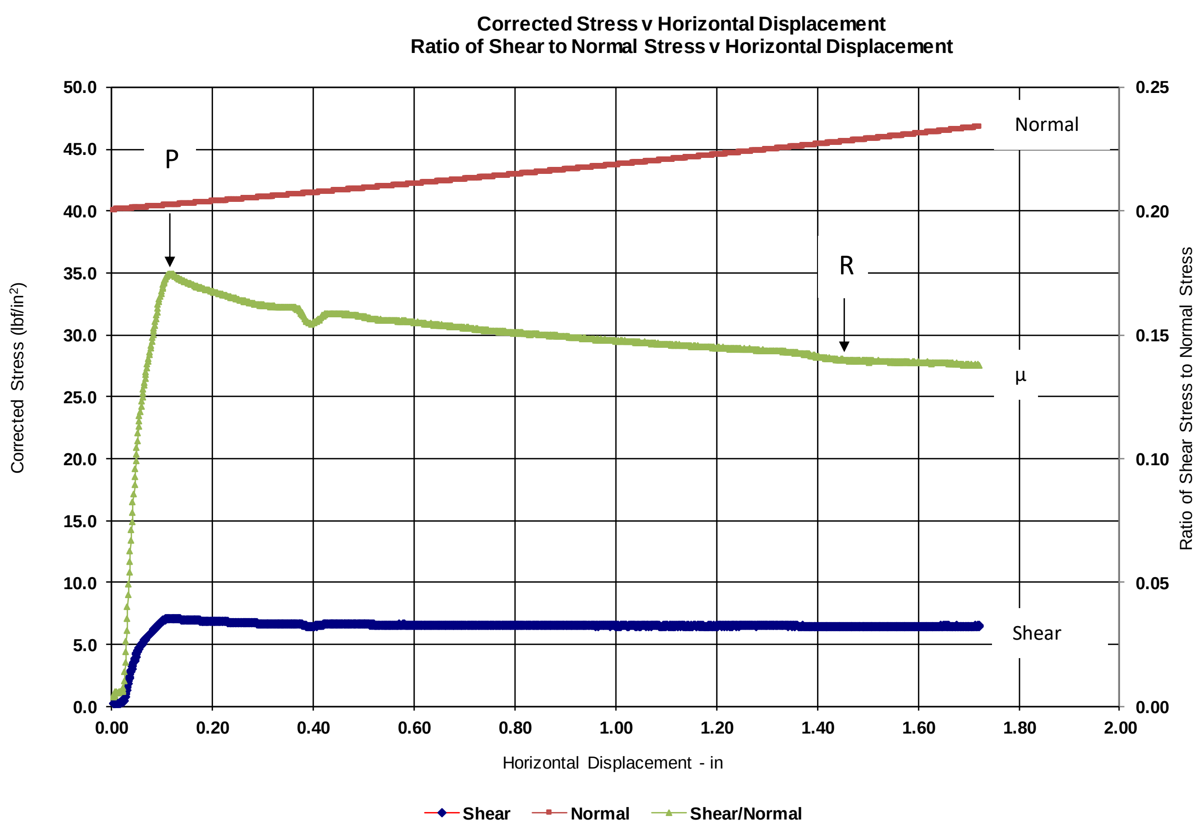
Test 23 - Normal Stress = 40 lb/in^2. SILTY SAND (SM), 73 percent hard sand, 27 percent nonplastic fines. Maximum size medium subangular to angular sand. Particles break with moderate hammer blow. Interface Coefficients of Friction: μ_p = 0.22, μ_r = 0.16.



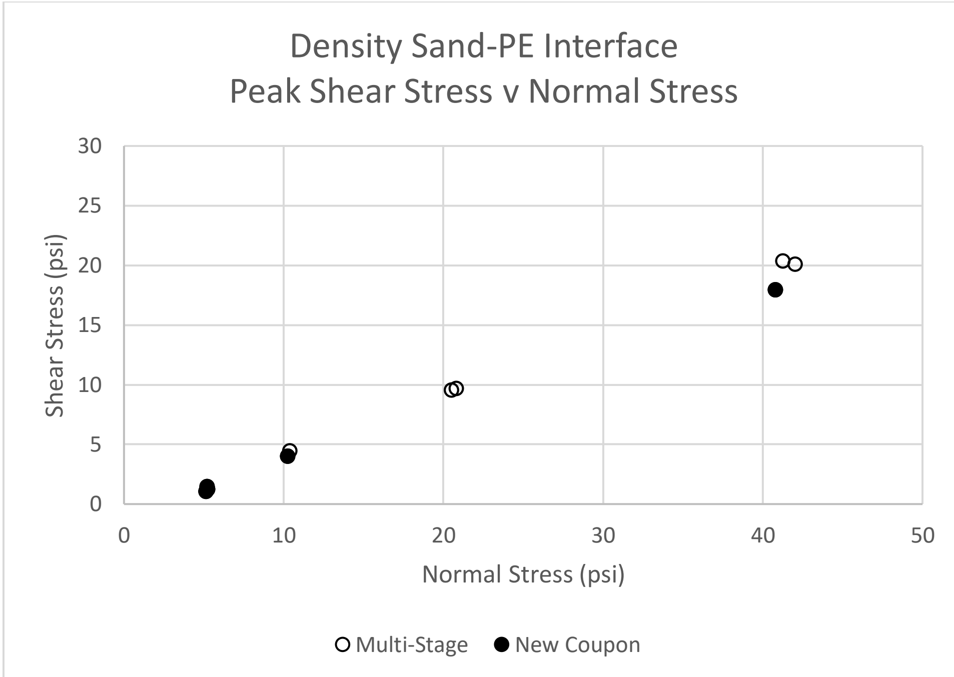


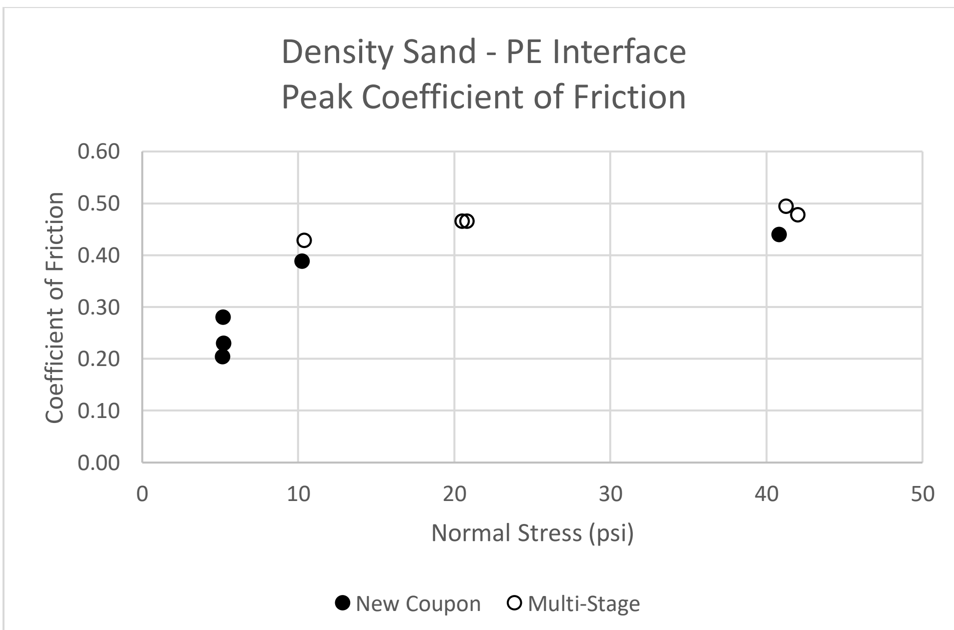
Test 23 - Normal Stress = 40 lb/in^2. SILTY SAND (SM), 73 percent hard sand, 27 percent nonplastic fines. Maximum size medium subangular to angular sand. Particles break with moderate hammer blow. Interface Coefficients of Friction: μ_p = 0.22, μ_r = 0.16.



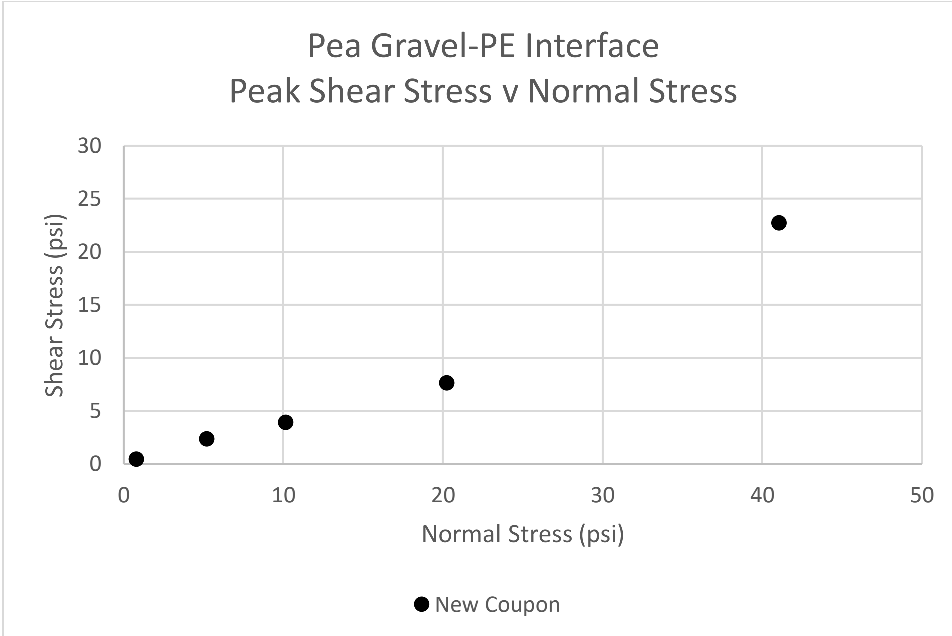


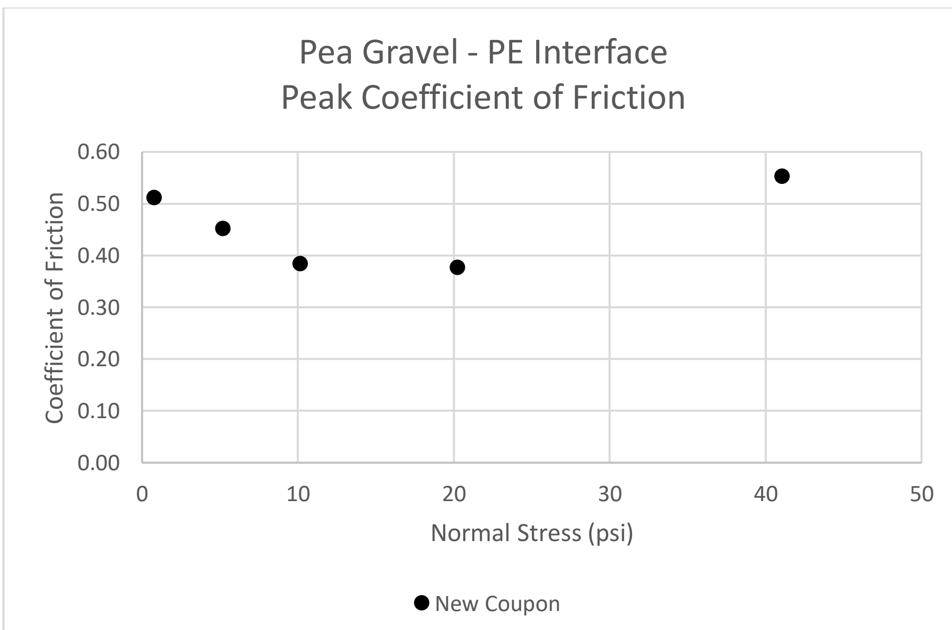
Appendix B – Shear Stress v Normal Stress and Coefficient of Friction v Normal Stress Plots



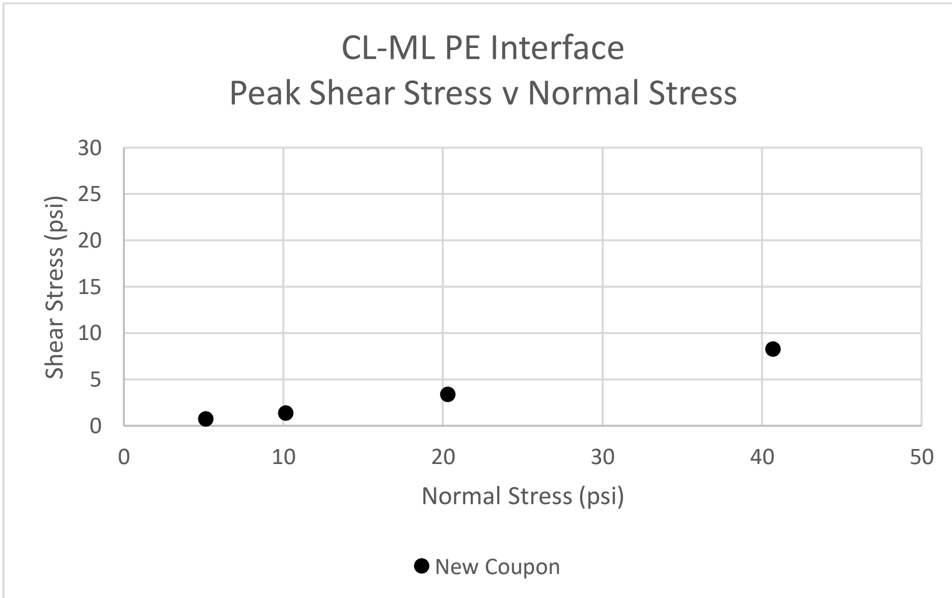


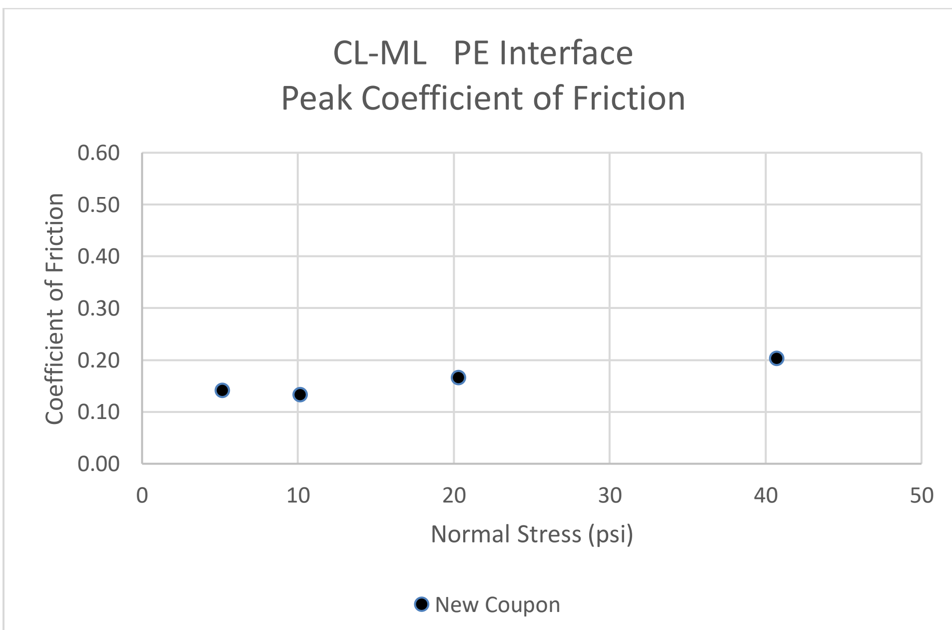
Appendix B – Shear Stress v Normal Stress and Coefficient of Friction v Normal Stress Plots





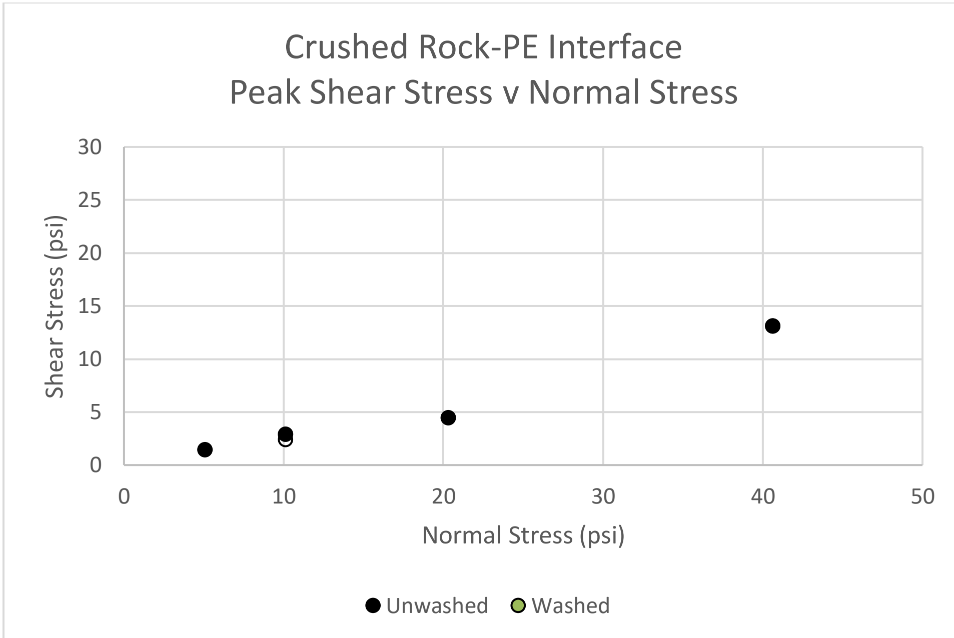
Appendix B – Shear Stress v Normal Stress and Coefficient of Friction v Normal Stress Plots

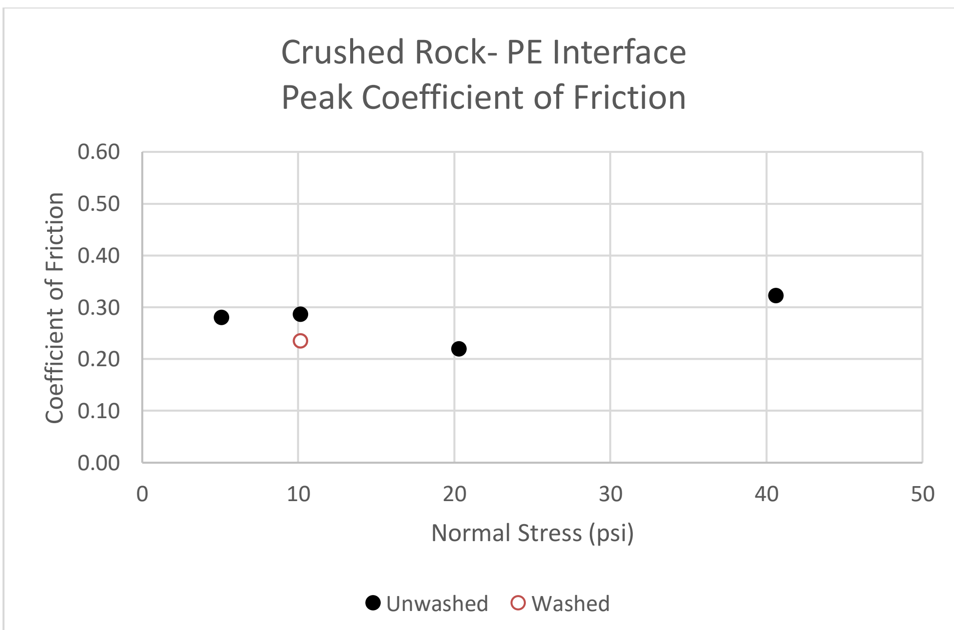




Results of tests on Bonny Loess.

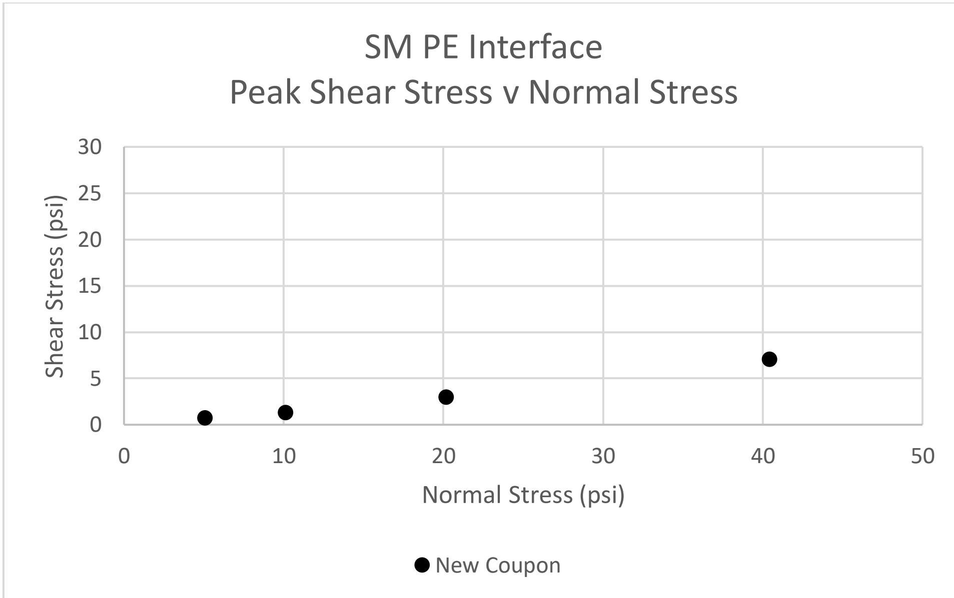
Appendix B – Shear Stress v Normal Stress and Coefficient of Friction v Normal Stress Plots

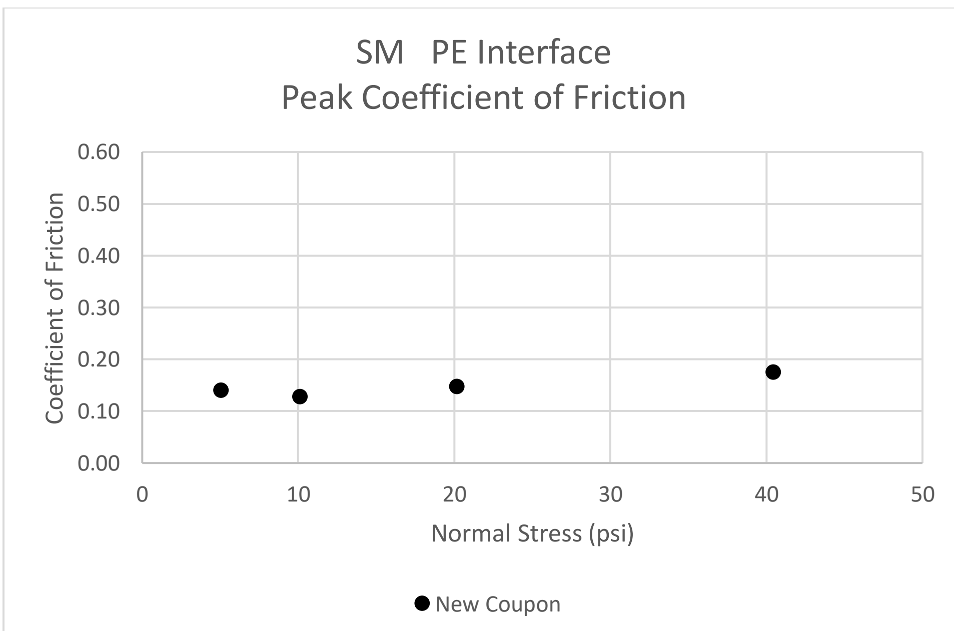




Results of tests on Crushed Rock.

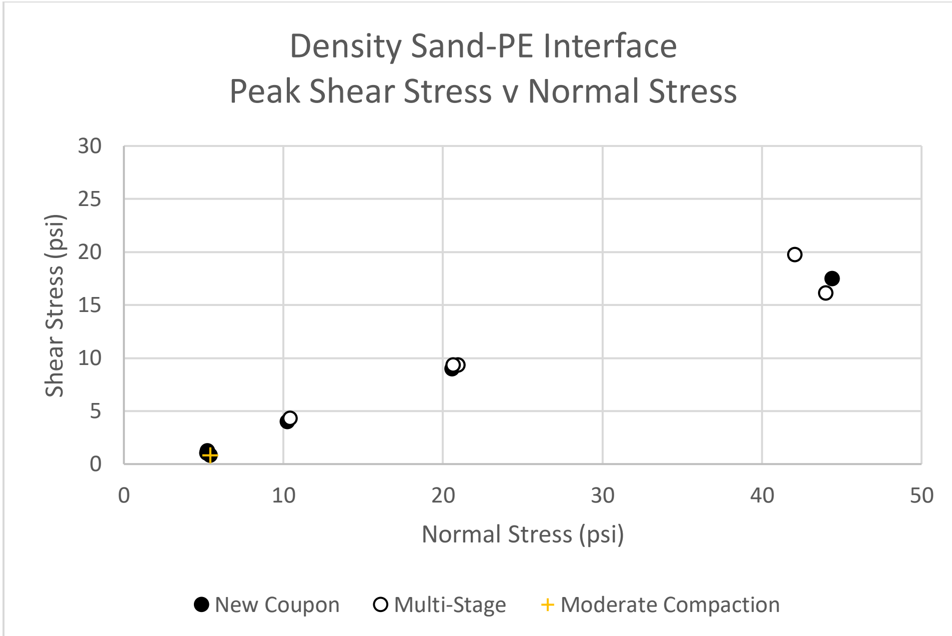
Appendix B – Shear Stress v Normal Stress and Coefficient of Friction v Normal Stress Plots

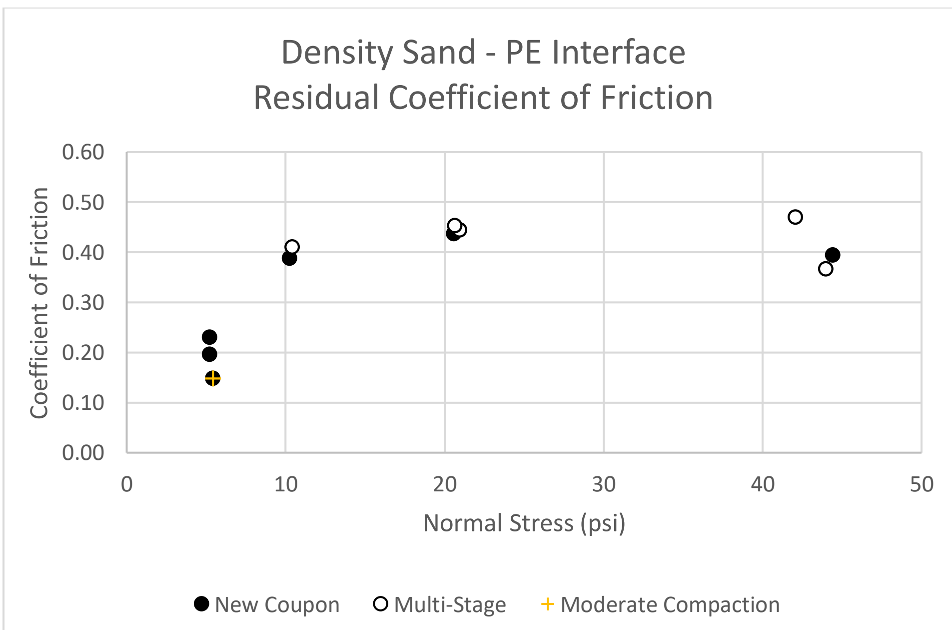


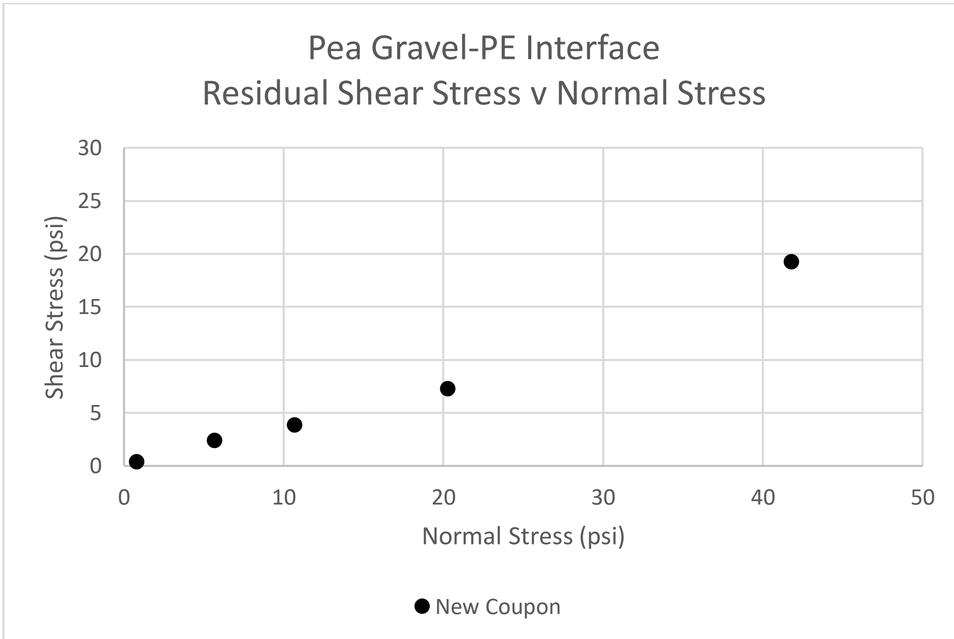


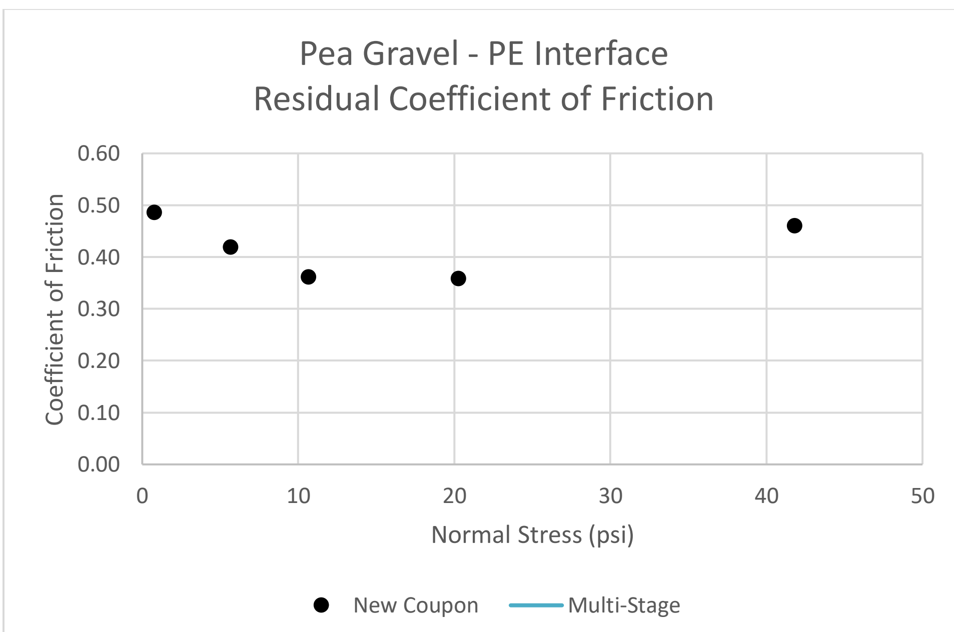
Results of Tests on SILTY SAND (SM)

Appendix B – Shear Stress v Normal Stress and Coefficient of Friction v Normal Stress Plots

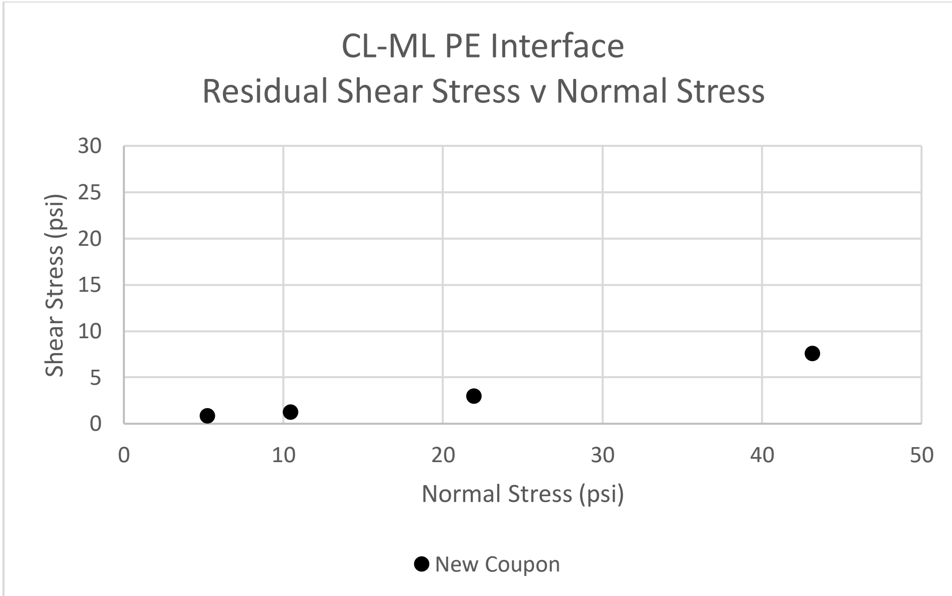


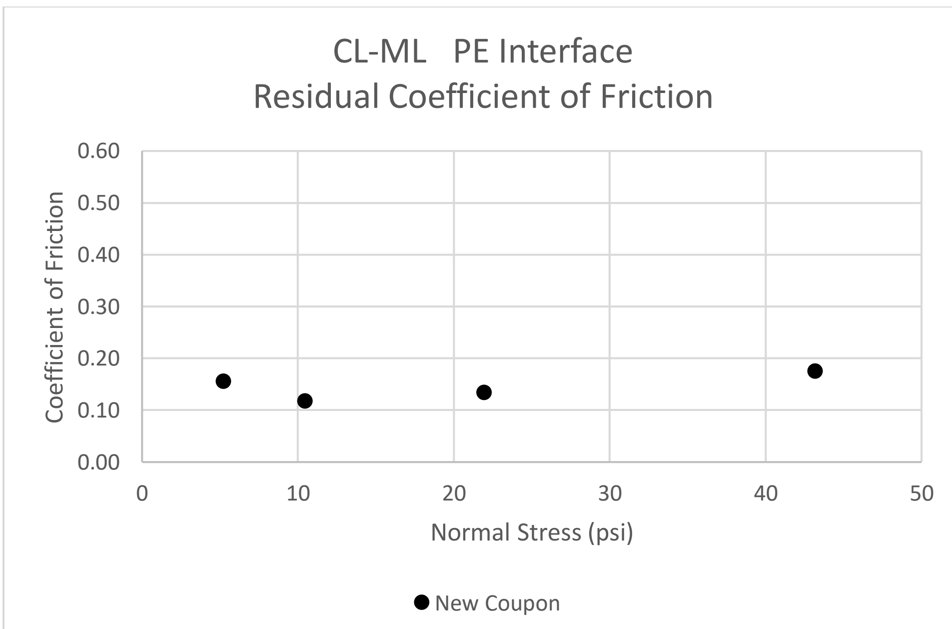



Pea Gravel-PE Interface
Residual Shear Stress v Normal Stress
Shear Stress (psi)
30
25
20
15
10
5
0
0
10
20
30
40
50
Normal Stress (psi)
New Coupon

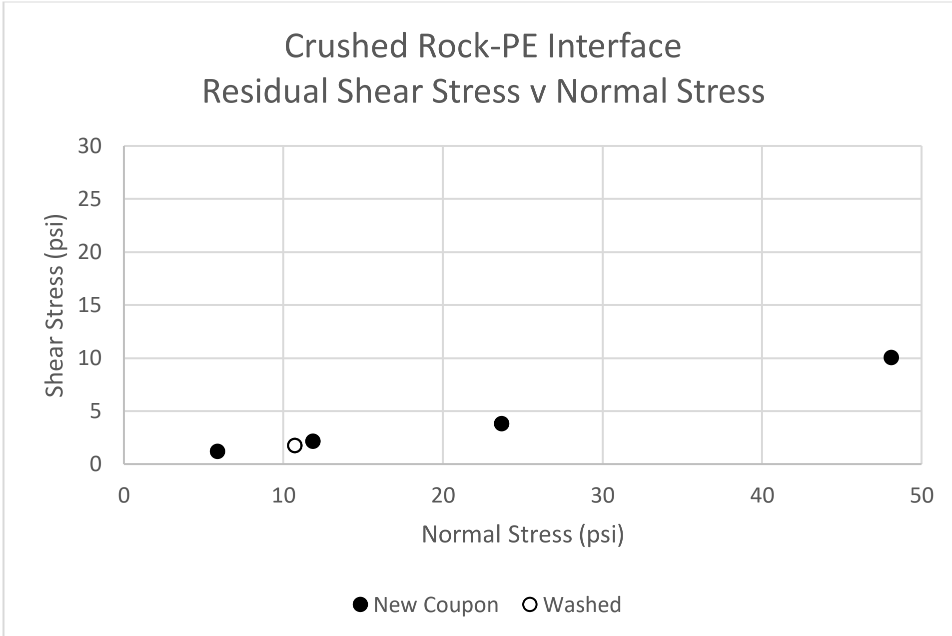

Pea Gravel - PE Interface
Residual Coefficient of Friction
Coefficient of Friction
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
10
20
30
40
50
Normal Stress (psi)
New Coupon
Multi-Stage

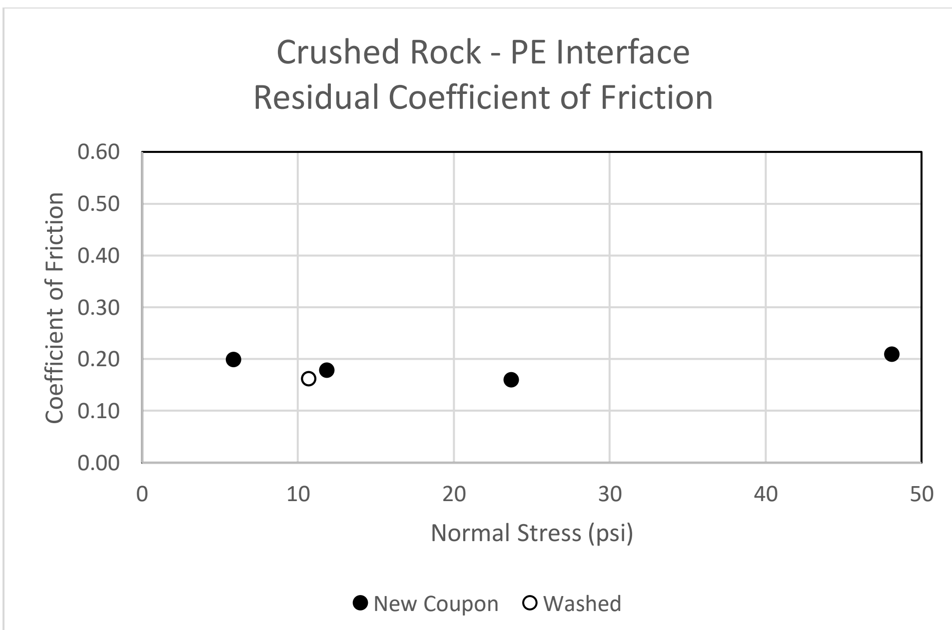
Appendix B – Shear Stress v Normal Stress and Coefficient of Friction v Normal Stress Plots



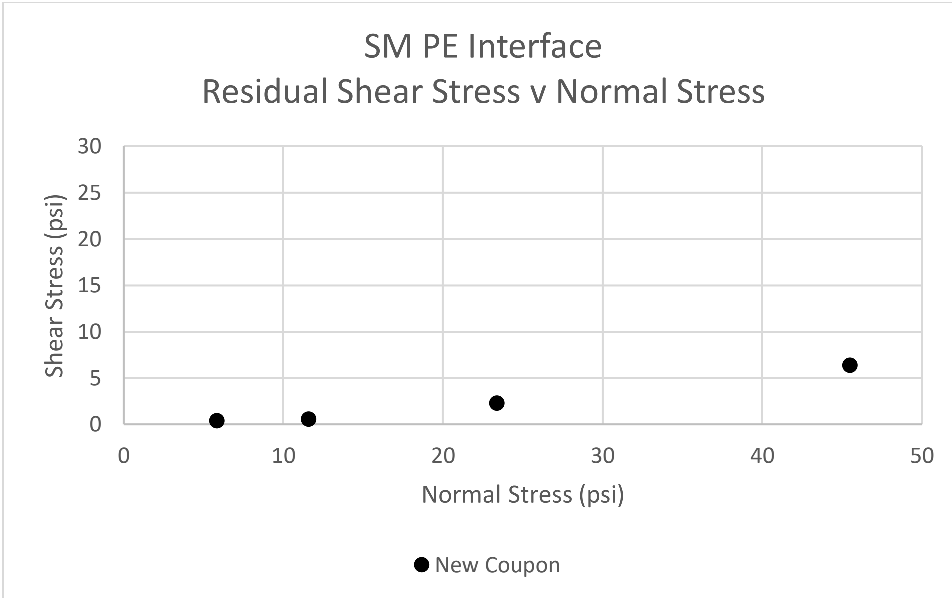


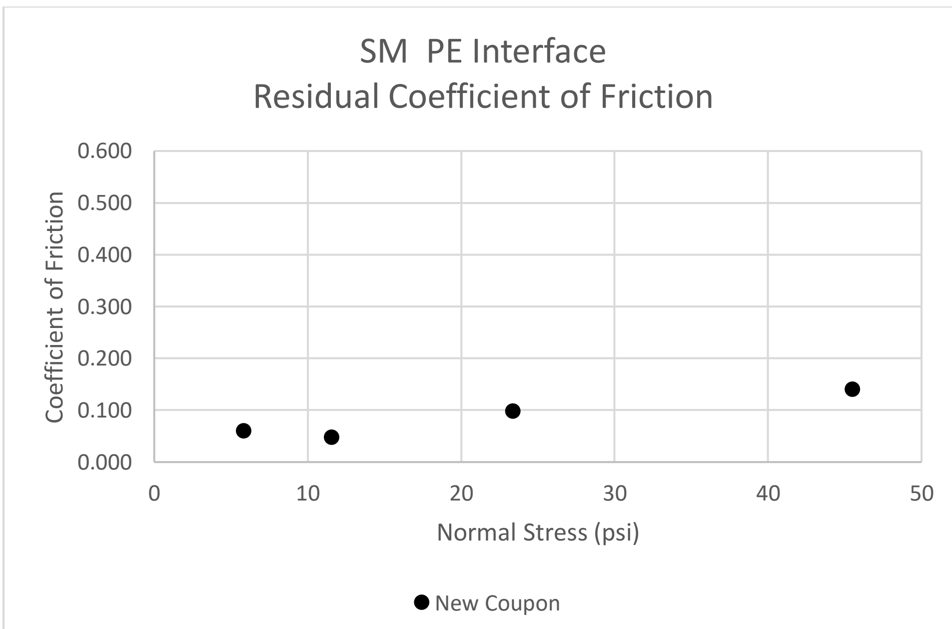
Appendix B – Shear Stress v Normal Stress and Coefficient of Friction v Normal Stress Plots





Appendix B – Shear Stress v Normal Stress and Coefficient of Friction v Normal Stress Plots





Appendix C. Data Plots Overlain on Drexel University Data Plots (Koerner 2005)

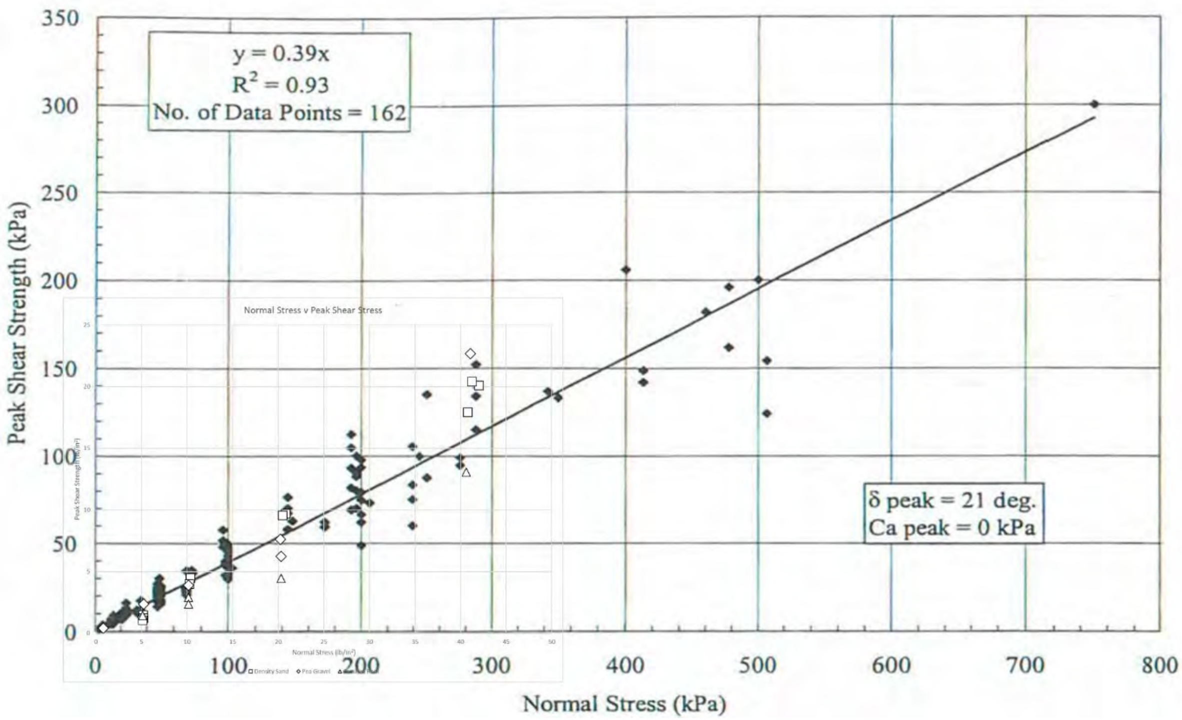


Figure C1. Peak data for Density Sand, Pea Gravel and Crushed Rock overlain on Drexel database plot of peak values for coarse soil. Study data is represented by open symbols: Density Sand-square; Pea Gravel-diamond; Crushed Rock-Triangle.

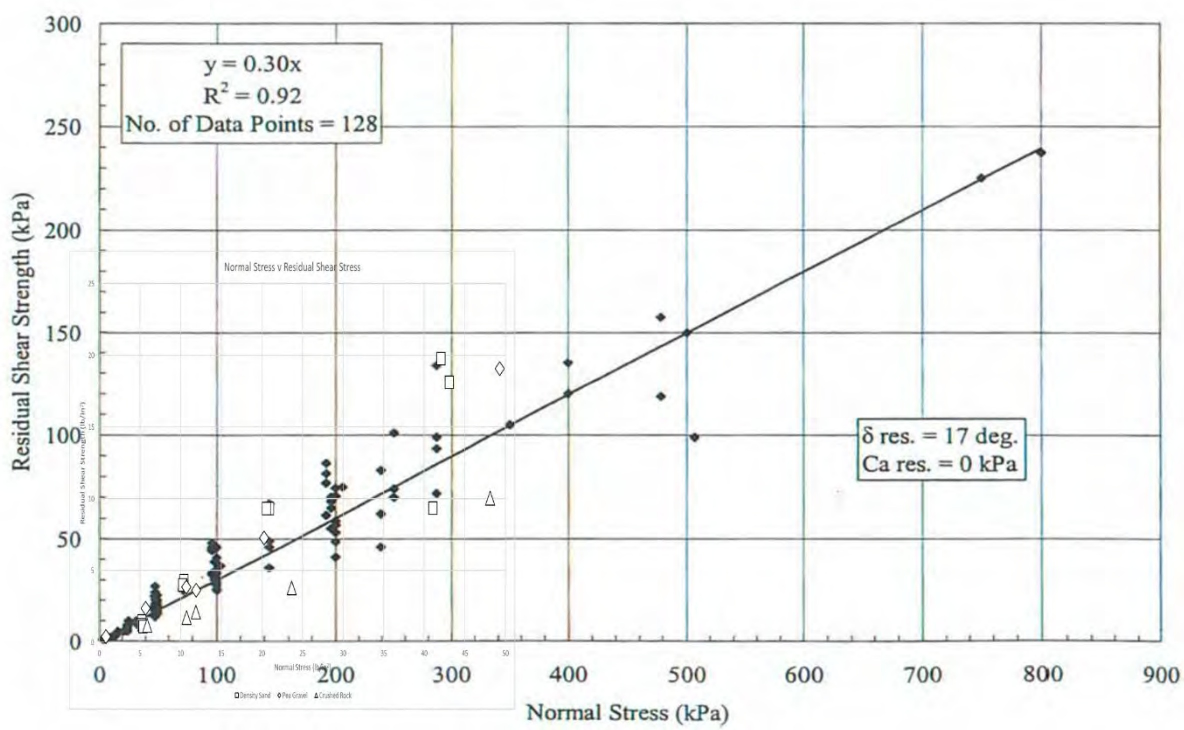


Figure C2. Residual data for Density Sand, Pea Gravel and Crushed Rock overlain on Drexel database plot of residual values for coarse soil. Study data is represented by open symbols: Density Sand-square; Pea Gravel-diamond; Crushed Rock-Triangle.

Appendix C. Data Plots Overlain on Drexel University Data Plots (Koerner 2005)

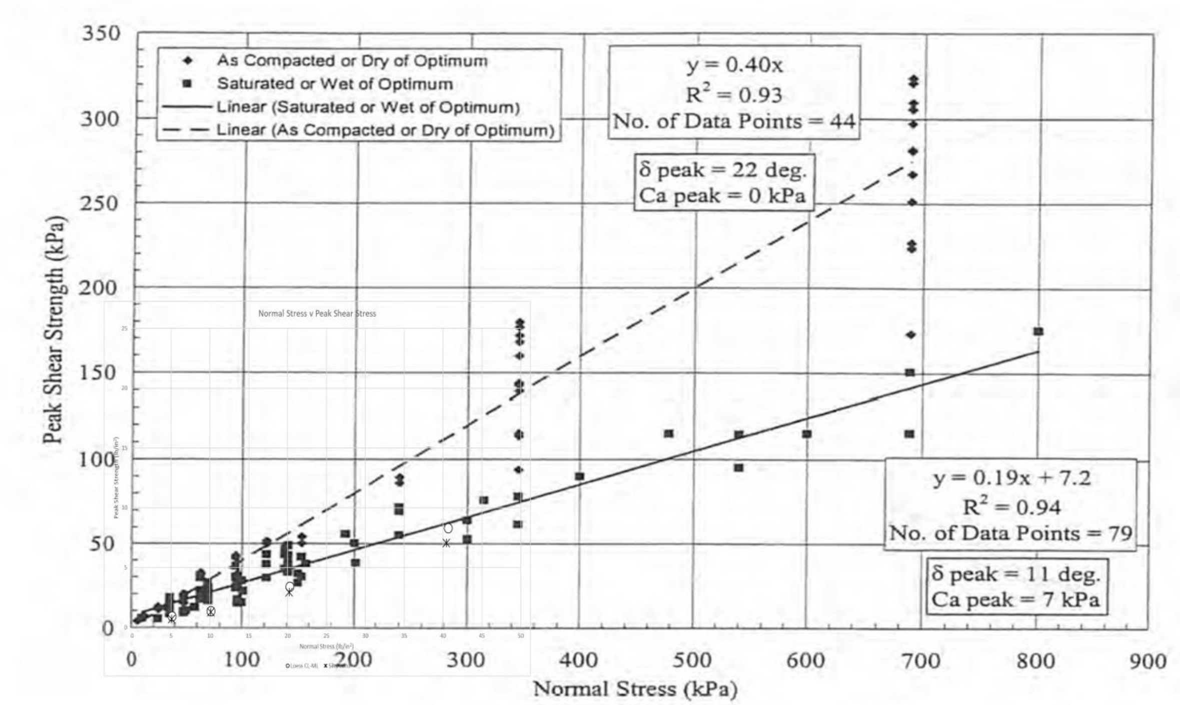


Figure C3. Peak data for Silty Sand and Silty Clay overlain on Drexel database plot of peak values for fine soil. Silty Sand data is represented by x; Silty Clay data is represented by open circles.

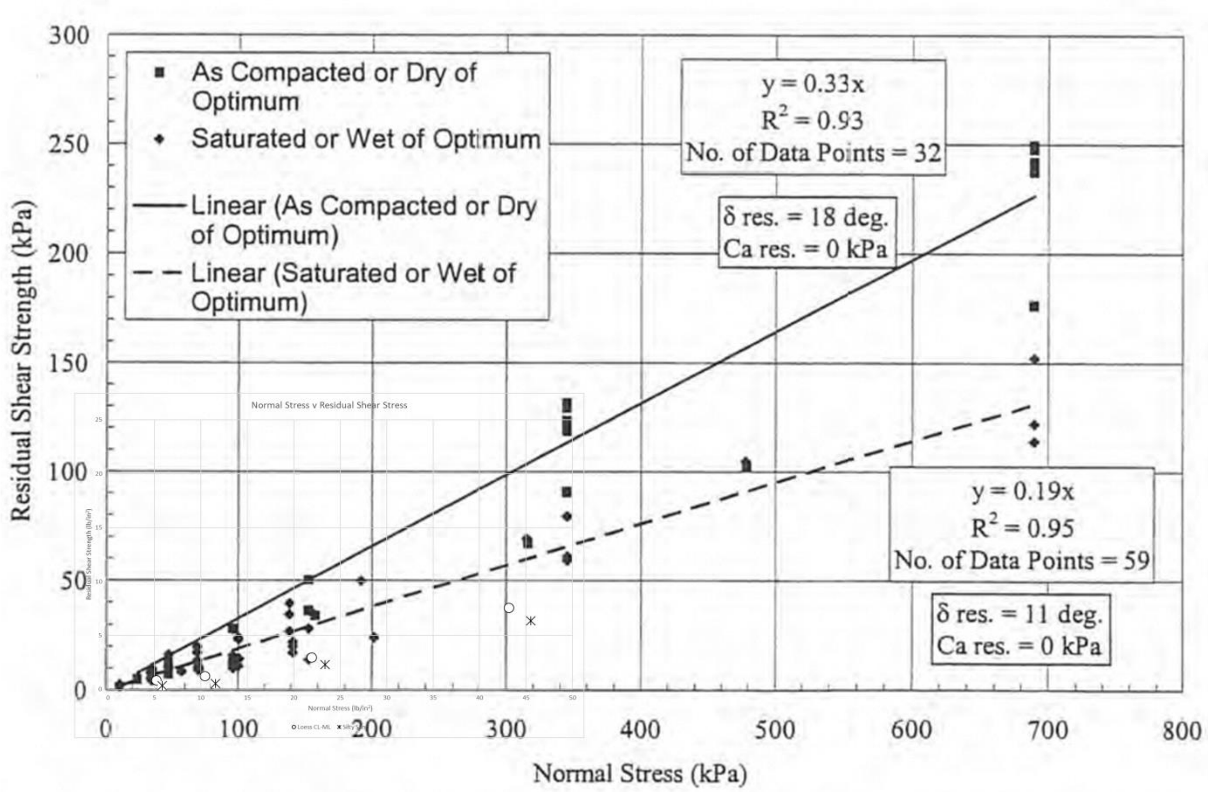


Figure C4. Residual data for Silty Sand and Silty Clay overlain on Drexel database plot of residual values

Appendix C. Data Plots Overlain on Drexel University Data Plots (Koerner 2005)

for fine soil. Silty Sand data is represented by x; Silty Clay data is represented by open circles.

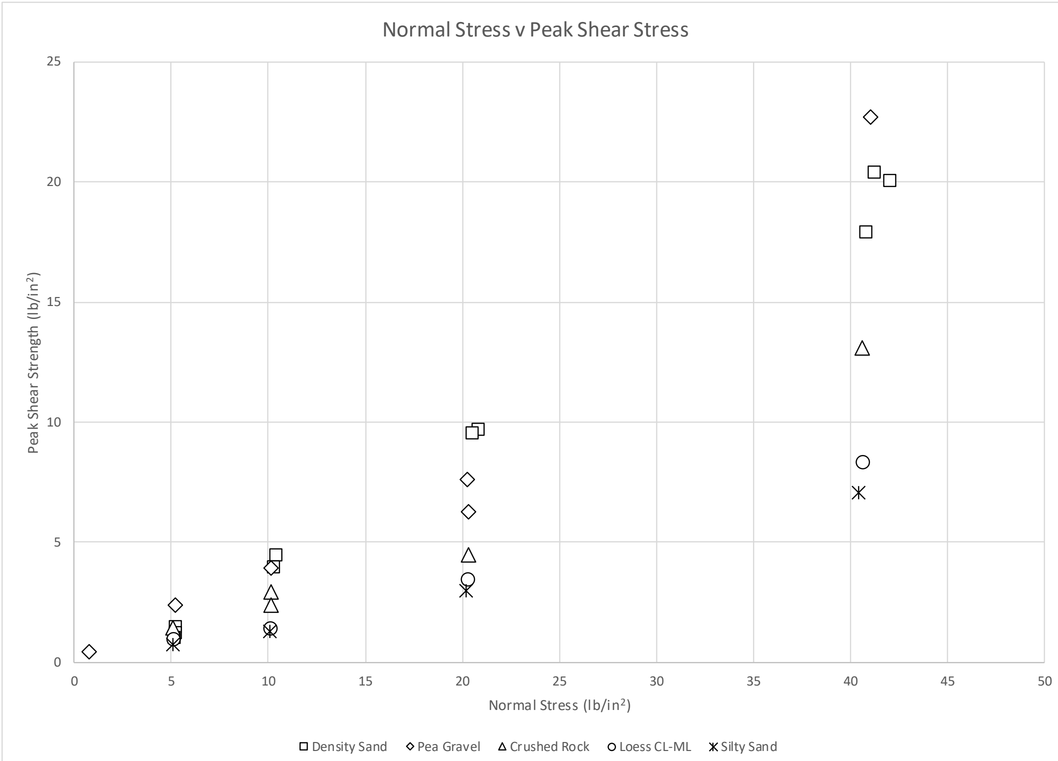


Figure C5. Peak Shear Stress v Normal Stress for all data from this study.

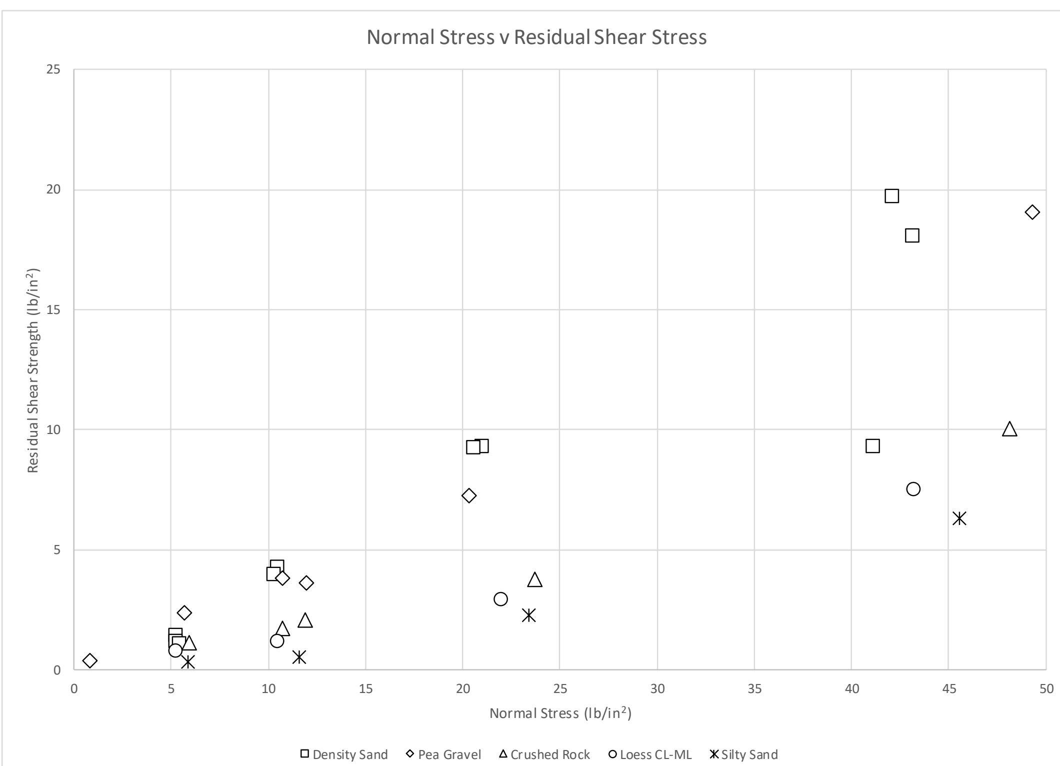


Figure C6. Residual Shear Stress v Normal Stress for all data from this study.