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Native Backfill Materials for Mechanically Stabilized Earth Walls

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Supplementary Notes

Abstract

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FINAL CONTRACT REPORT

NATIVE BACKFILL MATERIALS FOR MECHANICALLY STABILIZED EARTH WALLS

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ABSTRACT

Mechanically stabilized earth walls are an attractive alternative to conventional reinforced concrete retaining walls. The economy of these walls for non-critical applications might be improved by using alternative backfills consisting of on-site soils or quarried materials for the reinforced zone. The results of this study indicate that use of alternative backfill soils is feasible for non-critical wall applications. Soils for use with metallic reinforcement should be well graded with a maximum size of 3 inches and have less than 20 percent fines with a plasticity index less than 6. Backfill for polymer-reinforced walls should be well graded with a maximum size of ¾ inch and less than 30 percent fines with a plasticity index less than 9. Use of wall systems such as geosynthetic reinforced modular block walls should be considered when using alternative backfills. The primary benefit of using alternative soils is in material procurement. Additional costs might be incurred for quality control testing and material placement; however there is presently insufficient data to provide quantitative estimates of these costs.

INTRODUCTION

Mechanically stabilized earth (MSE) walls are geotechnical composite systems composed of alternating layers of soil and tensile reinforcing elements. These walls are self-supporting gravity structures due to the interaction between the soil and reinforcement. MSE walls have become an attractive alternative to conventional reinforced concrete retaining walls during the past 30 years. Rapid acceptance of MSE wall systems can be attributed to their relative low cost, aesthetics, good performance and reliability, relatively simple construction, and ability to use in a wide variety of site conditions. The Virginia Department of Transportation (VDOT) has successfully used MSE structures as elements of approach embankments and bridge abutments and for general earth retention.

Soil reinforcement systems consist of three components: reinforcement, facing elements, and backfill material as shown on Figure 1. Reinforcing elements are broadly classified as inextensible (metal) or extensible (polymer). This classification is based on the deformation necessary to mobilize the full strength of the reinforcement relative to the deformation necessary to mobilize the full strength of the soil. Inextensible reinforcement includes steel strips and grids. Extensible reinforcement includes polymer geogrids and geotextiles. Reinforcing elements are produced by a variety of manufacturers and are readily available.

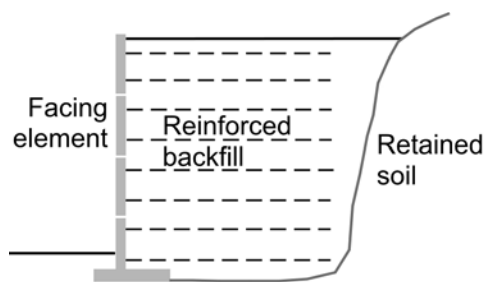

Figure 1. Schematic illustration of an MSE wall

Facing elements retain the backfill soil and generally consist of precast concrete panels or modular precast masonry units. Segmental panels have square, rectangular, cruciform or other polygon shape. Wall systems are available with precast panels that extend the full height of the wall. MSE walls constructed with precast masonry units are referred to as modular block walls (MBWs) by the Federal Highway Administration (FHWA) (2001) but are also commonly referred to as segmental retaining walls (SRWs). This report adopts the term MBW to be consistent with FHWA terminology. Panels and blocks are manufactured with a wide variety of aesthetically appealing textures and colors.

Well-graded, free-draining granular soils are specified as backfill within the reinforced zone of the wall for federal and most state highway projects. Granular materials are generally desirable for the following reasons (FHWA, 2001): good durability; high permeability, enabling better drainage and less pore pressure build-up; easy to moisture condition and compact; good frictional strength for good soil-reinforcement interaction; and, relatively high pullout resistance.

Consequently, the economic benefits of MSE walls are largely limited by the availability and cost of imported granular fill. Cost savings could potentially be realized by using on-site

native soils or lower cost quarried materials. These alternative materials would generally have a greater percentage passing the No. 200 sieve, and some degree of plasticity. Use of these materials would reduce or eliminate the costs of disposing native soil and/or importing select soil.

Concerns with using soils with higher percentage of fines as backfills in the reinforced zones of MSE walls (Zornberg and Mitchell, 1994) include:

- Low permeability could allow build-up of pore water pressures and reduction in soil strength.
- Drained soil friction angles are generally lower than those of freely draining granular soils.
- Presence of clay particles (less than 2 microns) increases the rate of corrosion of metallic reinforcements.
- More difficult to moisture condition and compact.
- Potential for increased construction inspection effort.
- Relatively greater construction and post-construction movements.

PURPOSE AND SCOPE

The purpose of this project was to develop guidelines for determining when the current MSE wall specifications can be relaxed to include alternative backfill materials for non-critical MSE walls constructed for VDOT projects. A new generic MSE wall special provision is being drafted in the Structure and Bridge Division and it is anticipated that the results of this study will be incorporated into that document.

VDOT has specified that use of alternative materials in MSE walls be limited to non-critical applications until a sufficient database of performance information is accumulated. This approach was used as the basic framework for this study.

Unless otherwise noted, non-critical walls include those walls that are 15 feet or less in height (H) and do not have a structure or traveled way on the retained soil within a horizontal distance of 2H behind the face of the wall. Non-critical walls also include structures used to retain cut slopes adjacent to a highway. MSE walls associated with bridge abutments, approach embankments, or high walls adjacent to the traveled way are considered critical walls, and the use of alternative backfills is not permitted at present.

This report summarizes the results of project phases I and II. The scope of this project includes the following:

1. Develop a pullout testing capability within Virginia.
2. Prepare decision guidelines and screening tools for use of native Virginia soils.
3. Provide recommendations for the draft special provisions for using native fill materials.
4. Develop selection guidance for inclusion materials other than steel.

5. Collaborate with and transfer results from an on-going National Cooperative Highway Research Program (NCHRP) study to Virginia practice.

METHODS AND MATERIALS

Methods

Phase I: Development of a Pullout Testing Capability

Mr. Robert Swan, President and CEO of Soil and Geosynthetic Interaction, Inc. (SGI) of Norcross, Georgia, donated a 2 feet wide by 5 feet long standard pullout device to the Commonwealth. A load cell and LVDT were also provided as part of the donation at no cost. The device required a hydraulic pump, valving, pneumatic cylinders used to apply the normal load and repair of several parts. The device is currently being housed at the Prices Fork research laboratory in Blacksburg, Virginia. Details regarding the pullout box are available from the authors.

Phase II: Assessment of Alternative Backfills

Task 1 - Review of literature and current practice

Published references were reviewed to determine the use and performance of alternative backfill soils in the construction of MSE structures. The published information was used to summarize current practice for design, specification, and construction of MSE wall backfill, and to identify laboratory and field values of engineering properties of soil within the reinforced zone of similar walls.

In addition, selected VDOT approved and probationary wall system manufacturers were interviewed to learn of their experience with alternative backfills and design issues particular to their systems. The approved and probationary lists as of June 2003 are summarized in Tables 1 and 2. Note that not all systems listed in the tables are MSE wall systems.

Table 1. Wall Systems on Approved List

Wall System	Company	Location
Hilfiker Walls	T & B Structural Systems	Hurst, TX
Reinforced Earth	The Reinforced Earth Company	Vienna, VA
Isogrid	The Neel Company	Springfield, VA
VSL Retained Earth	Foster Geotechnical Co.	Woodbridge, VA
T-Wall Retaining Systems	The Neel Company	Springfield, VA
Doublwal Corporation	Doublwal Corporation	Plainville, CT

Table 2. Wall Systems on Probationary List

Wall System	Company	Location
MSE Plus	SSL	Scotts Valley, CA
KEYSYSTEM I	Betco Block & Products, Inc.	Manassas, VA
Strengthened Earth Walls	Gifford-Hill & Company	Dallas, TX
ARES (Panel System)	Tensar Earth Technologies, Inc.	Atlanta, GA
MESA (MBW System)	Tensar Earth Technologies, Inc.	Atlanta, GA
Tricon Retained Soil Wall System	Tricon Precast, Ltd.	Houston, TX

Task 2 - Laboratory index and strength tests on alternative native Virginia soils

A laboratory testing program was conducted to provide strength data on three selected native Virginia soils. Additionally, select aggregate (VDOT No. 21A stone) was tested to serve as a control or reference soil. As the possible range of soils is large, this study focuses on soils with fines contents that are approximately 15 to 25 percent.

Soils were fully characterized in terms of index properties such as gradation, fines content, Atterberg limits of the fine fraction, and moisture-density relationships (AASHTO T-99). Strength envelopes were determined for each soil at three normal stresses using the consolidated-drained, direct shear test. It is believed that the direct shear test provides a relatively easy way to determine backfill friction angles that could potentially be used as a screening tool by the Materials Laboratory for specific projects.

Soils were compacted into a 2.5-inch diameter shear ring on the wet side of optimum to within 95 percent maximum dry density. Prior to the consolidation phase, specimens were allowed to soak overnight subject to a normal stress of approximately 5 pounds per square inch (psi). Samples were then allowed to fully consolidate under normal stresses within the range of 10 to 20 psi. These pressures are representative of field conditions for MSE walls of up to approximately 20 feet in height. Samples were sheared at a rate of 0.002 inches per minute. This slow rate was chosen to prevent excess pore pressure from generated during shear.

Isotropically consolidated drained (ICD) triaxial test were conducted on one native soil to validate the results of the direct shear test. Samples were prepared approximately two points wet of optimum moisture content, and were compacted to within 95 percent of AASHTO T-99 maximum dry unit weight using a Harvard Miniature compaction device. Samples measured 1.4 inches in diameter and 2.8-inches in height. Consolidation stresses were within the range of 10 to 20 psi. Samples were then sheared at a strain rate based on an approximate time to failure of 16 times t_{90}, where t_{90} is determined from the consolidation curve using Taylor's method. The strain rate corresponds to approximately 0.5 percent of specimen height per hour.

Task 3 - NCHRP test wall construction

The NCHRP has sponsored the research project “Selecting Backfill materials for MSE Retaining Walls” (Project 24-22). This demonstration project is aimed at developing selection guidelines, soil parameters, testing methods, and specifications for using on-site materials. The project’s thrust is to construct three instrumented wall sections about 20 feet in height and 60 feet in length. The walls will be monitored through two winter seasons and subjected to a variety of imposed backfill saturation conditions. This project will collaborate with and leverage the results of that project as applied to Virginia practice.

Future Phases

Additional phases of the project will potentially be completed at a later date. These include Phase III, Pullout Testing on Selected Soil and Reinforcing Inclusions, and Phase IV, NCHRP Test Wall Monitoring Program.

Materials

Soil Materials

Three native soils from Virginia plus a reference material were selected for laboratory index and strength testing. Nearly all VDOT District offices were contacted for location of potential borrow sources that could supply such material. Ideally the materials would be located on a project site, within VDOT right-of-way. Suitable materials at current projects were identified. However, access to those sites was not available in the time frame of this study. Therefore, considerable effort was expended to locate and contact approximately 20 quarries and sand pit operators. A brief summary of the soils obtained from that effort is provided in Table 3.

Table 3. Summary of Soil Materials Acquired in Study

Source	Location	Material Description
Sandy Bottom Materials	Suffolk, VA	Orange, Fine Silty SAND (SM)
Isle of Wight Materials, Co., Inc.	Smithfield, VA	Gray, Fine Silty SAND (SM)
Isle of Wight Materials, Co., Inc.	Smithfield, VA	Red-brown, Fine Silty SAND (SM)
Sisson & Ryan Quarries	Shawsville, VA	VDOT 21A
Rt. 661	Christiansburg, VA	Orange-brown, Sandy CLAY (CL with Decomposed Shale). Did not meet project requirements.

RESULTS

Summary of Literature Review

Case Histories: Performance of Reinforced Backfills

In order to develop guidelines for VDOT engineers, it is important to determine how MSE walls backfilled with alternative soils have performed with respect to those walls with select backfill. Well-documented case histories of MSE walls backfilled with both select and alternative materials were reviewed. Most case histories involving alternative soils incorporated geosynthetic reinforcement. Information for metallic reinforcements with alternative backfills was obtained from discussions with manufacturers. Alternative backfills are broadly defined in this section to include silty and clayey sands with fines contents greater than 15 percent, low plasticity clays and low plasticity silts.

Select Backfills

Allen et al. (2003) present a detailed review and summary of case histories involving geosynthetic-reinforced walls. The backfill material for each wall consisted of select material with low fines content. These results are important because they provide data that can be used to assess the performance of geosynthetic reinforced walls backfilled with alternative soil.

Table 4 provides a compilation of 15 case histories. Data are from instrumented walls constructed for various infrastructure projects as well as large-scale test walls constructed in the laboratory.

Alternative Backfills

Koerner and Soong (2001) summarized 26 case histories of problems with geosynthetic-reinforced walls. The majority of the walls incorporated geogrids; geotextiles were used in only a few cases. Twelve cases resulted in serviceability problems involving excessive deformation. Of these cases, five were contractor/construction related and seven were design related. The majority of the design related cases involved bulging at either the top or bottom. Five of the design related cases have alternative backfills in the reinforced zone. Movements are attributed to poor backfill drainage, unanticipated surcharges or strength loss of the backfill due to saturation.

Fourteen cases resulted in failures where a portion of the wall actually collapsed. They concluded that problems were attributable to either: 1) Fine-grained soil backfill in the reinforced zone (15 cases); or, 2) Inadequate construction inspection procedures (10 cases). Practically all failures occurred during rainy weather with saturation of the backfill.

Mitchell and Zornberg (1995) evaluated a number of published case histories of reduced and full-scale reinforced soil structures constructed with low plasticity clay and silt backfill. Post-construction movements were generally observed when pore pressures were generated in the fill, especially in those constructed with metallic reinforcements. The authors also evaluated

the suitability of different types of reinforcing inclusions when used in conjunction with higher fines content backfills. They make the following observations:

Table 4. Summary of Wall Performance: Select Backfill with Geosynthetic Inclusions (Modified from Allen et al., 2003)

Project	Height / Surcharge	Soil Description	Soil d_{max} (mm) d_{50} (mm) <75μm (%)	Design Friction Angle (deg.)	Measured Friction Angle (deg.)	Inclusion Type	Lateral Deformation (inches)
Snailback Wall, 1974	9.5 ft/ 2.9 ft	Fine to medium sand.	4.76 1.2 2	34	38	Geotextile	Little after shotcrete face applied
Olympic National Forest Wall, 1975	18.3 ft	Crushed rock	75 35 -	40	NA	Geotextile	1 to 2
CDOT Glenwood Canyon, 1982	15.7 ft	Pit run, well-graded, clean sandy gravel	100 19 -	35	42	Geotextile	2 to 3 (during construction)
Devils Punch Bowl, Wrapped-Face, 1982	28.9 ft	crushed basalt	50 4 to 5 10	40	NA	Geotextile	Little
Lithonia Georgia, 1985	20 ft	Well graded sandy gravel	NA	40	43	Tensar Geogrid	4 to 6
Algonquin, 1988	20 ft/ 6.9 ft	well graded clean gravelly sand	50 4 0	40	40	Tensar Geogrid	1.4 (during construction)
Algonquin Geotextile, 1988	19.3 ft		50 4 0	40	40	Geotextile	6 (during construction)
RMC Wrapped-Face, 1986	9.3 ft / 1.9 ft	clean uniform sand, some gravel	8 1.2 0	40	46 to 53	Tensar Geogrid	< 0.8 (during construction), < 0.4 (after construction)
RMC Timber Panel, 1987	9.8 ft/ 875 psf	clean uniform-size washed sand with some gravel	8 1.2 0	40	46 to 53	Tensar Geogrid	10.7 (after construction) 1.6 (after surcharging)

Table 4 – Continued

Project	Height / Surcharge	Soil Description	Soil d_{max} (mm) d_{50} (mm) <75μm (%)	Design Friction Angle (deg.)	Measured Friction Angle (deg.)	Inclusion Type	Lateral Deformation (inches)
RMC Full-Height Aluminum Panel, 1989	9.8 ft/ 1450 psf		8 1.2 0	40	46 to 53	Tensar Geogrid	4 (facing movement at failure)
RMC Aluminum Panel, 1989	9.8 ft/ 1253 psf		8 1.2 0	40	46 to 53	Tensar SS1 Geogrid	0.5 (after construction)
WSDOT Rainier Ave, 1989	41.3 ft/ 17.4 ft	well graded gravelly sand	60 2 -	36	45	Geotextile	5.5 (during construction) 1.2 (over 1 yr.)
Ontario Propped Panel, 1989	23.2 ft	silty sand and gravel	27 4.75 10	36	NA	Geogrid	1.7
New Brunswick Propped, 1990	20 ft	pit run coarse sand and gravel	76 - 12	40	NA	Geotextile	1.0 (after 6 mo.)
PET Strip St. Remy Test, 1993	20.1 ft	uniform fine to medium sand	1.5 0.15 to 0.2 4	37	39	Geogrid	<2

- Metallic reinforcements are not suited for backfills with high fines content. They do not provide lateral drainage, their interface friction relies on the dilatant behavior of granular materials, and they are subject to a higher rate of corrosion.
- Polymeric grid reinforcements and woven geotextiles provide adequate strength for permanent structures, but have limited in-plane drainage ability. Thus, low moisture content must be maintained in the fill using appropriate drainage.
- Non-woven geotextiles have high in-plane drainage ability, but strength and stiffness is generally too low for permanent structures.

Mitchell and Zornberg (1994) found that geosynthetic composites that combine hydraulic properties of non-wovens with mechanical properties of wovens and geogrids might provide a suitable alternative for reinforcing poorly draining backfills. Strong experimental evidence shows that permeable reinforcements can effectively reinforce poorly draining backfills. It has also been shown that permeable reinforcements can aid in the compaction of alternative soils by (1) providing better distribution of the compaction effort, and (2) draining excess pore pressures

generated during compaction. The most improvement in compaction has been reported for low plasticity clays and silts (Christopher et al., 1998).

Elias and Swanson (1983) describe a case history of a reinforced earth wall constructed in Virginia in 1978-1979. The wall had a maximum height of about 23 feet, reinforcing elements consisted of ribbed galvanized steel strips, and specifications restricted backfill to less than 15 percent fines. No drainage system was installed. Before construction was complete, several days of above normal precipitation resulted in excessive movements of the wall (10-12 inches out of plumb at tallest section). Subsurface investigation revealed that in areas of severe wall distress, the backfill contained 30-50 percent fines, with more than 15 percent finer than 15 micron size (medium silt), and a plasticity index (PI) outside of project specifications. For mitigation, areas with more than 25 percent fines were identified, excavated, and replaced with backfill limited to less than 25 percent fines. No movement was originally noted in areas with less than 25 percent fines. The results of this study indicate the following:

- Fines content and moisture content are important factors in the construction of reinforced earth with residual soils.
- When backfill soils contain more than 10-20 percent finer than 15 microns (medium silt), significant reduction in pullout capacity and decrease internal stability can occur.
- Reduction in frictional strength is more pronounced in saturated soils.
- Fines content of residual soil can vary greatly over small distances.
- Highly micaceous sands of the Piedmont physiographic province are extremely sensitive to moisture variations, as significant strength reduction was observed with increased compaction water content.

According to Christopher et al. (1998), good wall performance strongly depends on prevention of excess pore water pressures generated within the reinforced fill. These three conditions are of primary concern.

1. Excess pore water pressure can develop during compaction, subsequent loading, and surcharging, particularly when the soil is placed wet of optimum.
2. Soils placed comparatively dry of optimum (no excess pore pressure during construction) may be subject to post construction water infiltration. Wetting results in a loss of strength.
3. The seepage configuration that may be established in fills constructed on existing embankment slopes and cut slopes when water from adjacent ground flows into the reinforced fill.

Moisture content control and implementation of an appropriate drainage system resulted in good performance when four MSE walls with metallic reinforcements were constructed for the widening of Interstate 80 near Baxter, California. Two of the walls, with a maximum height of 16 feet, were instrumented to monitor the performance of a sandy silt backfill with up to 50 percent fines. A subsurface drainage system was constructed, and construction was stopped several times to allow saturated material to dry out. Monitoring through heavy rainfall the

following year showed no significant vertical or lateral wall movements (Hannon and Forsyth, 1984).

Keller (1995) describes U.S. Forest Service practice with MSE walls. Hundreds of walls utilizing various reinforcing and facing materials have been successfully constructed with native backfill soils on Forest Service and rural roads in the past 20 years. Reinforcements have included geotextiles, geogrids, and welded wire mesh. Facing materials have included timbers, gabions, tires, geocells, and segmental concrete blocks. Backfills typically include soils with up to 50 percent fines, a PI of less than 20, and a peak friction angle between 25 and 30 degrees. Seven case histories involving Forest Service walls utilizing native alternative backfills are summarized in Table 5. Information regarding the cost of Forest Service walls is presented in the Costs and Benefits Assessment section.

Table 5. Summary of Wall Performance: Alternative Backfill with Geosynthetic Inclusions (Keller, 1995)

Location	Inclusion/ facing	Height (ft)	Backfill	Friction Angle (deg.)	Movement	Comments
Shaver Lake, CA	Welded wire/1:6 batter	15-22	SM to SC < 42% fines PI = 15	Not Reported	3% of Height	All walls and drainage systems performed well, some typical face bulging
Oroville, CA	Non-woven Geotextile/ 1:6 batter	9	SM <26% fines NP	35	7% face settlement	Performed well with some bulging and face irregularity
Canyon Dam, CA	Geotextile/ Tire face	10	SC 30-38% fines PI = 8-9	26	10% of wall height at midwall on face	Wall appears stable and partially vegetated, most settlement occurred in first 2 years
Oakridge, OR	Woven geotextile	28	Wood chips	34	5% of total height	Performed well, chips decompose with moisture, geotextile is disintegrating
Eagle Lake, CA	Welded wire/ concrete panel,	12.5	GW with minimal fines, NP	30+	minor	Performed well, minor face deformation, some offset and cracking of panels
Oroville, CA	Welded wire	10.5 ft	SM to ML < 55% fines PI = 3	33-34	minor	Supports 50 ft 1:1 reinforced slope, performed well, good vegetative growth, several local face slumps
Yreka, CA	Geogrid/ timber	15 ft	SM < 27% fines NP	30+	None visible	Excellent performance

Forest Service experience has shown that the use of alternative materials requires laboratory testing of individual samples of potential materials for strength properties and strength-density relationships; positive drainage; some additional care in construction; and, allowance for some settlement or deformation.

The performance of a geosynthetic reinforced soil test wall backfilled with silty clay (91 percent passing the No. 200 sieve, a PI of 15) was evaluated by the Louisiana Transportation Research Center (LTRC). The wall was constructed on a soft clay foundation. It was monitored for four months after completion for deformation, foundation settlement, reinforcement strain, vertical and horizontal soil stresses, and pore pressures. The wall was purposely designed with low factors of safety to produce measurable deformations in the test wall sections and higher reinforcement loads. Coupled with the high foundation soil settlement, the instrumentation program revealed higher deformations than in conventionally designed walls. Deformations, however, occurred mostly during construction. The results of this study showed promising performance of reinforced alternative backfill material provided that proper design and control of soil compaction and moisture is ensured. Long-term performance of the wall has not been evaluated (LTRC, 2004).

Design and Performance Issues with Alternative Backfills

Long-Term Deformation

Major factors controlling the creep behavior of the backfill material are the plasticity and quantity of the fine fraction, and moisture content. Because of creep deformation, conventional walls backfilled with fine-grained soil are designed with at-rest pressures rather than active pressures. Unfortunately, little long-term movement data from instrumented MSE walls with alternative backfills are available in the literature.

Table 6 presents data from two published case histories for full-scale Reinforced Earth (steel inclusions) walls using CL and ML soils where lateral deformation was recorded over time.

Table 6. Long-Term Movements of walls with Alternative Backfill

Source	Wall Height (ft)	Backfill	Time after construction (days)	Lateral Displacement	Vertical Displacement
Hashimoto, 1979	26	Clay, 80% fines, PI = 40, $w_c > 50\%$	40	1.6 in	28 in.
Battelino, 1983	11.5	Clayey Silt, $w_c = 20\%$	152	1.4 in	-

Water content is believed to have remained relatively constant during and after construction in the two case histories. Other studies indicate that the increase of water content during construction (through rain, for example) can increase long-term movements when higher fines content soils are used.

Current specifications do not require specific tests to evaluate creep potential of backfill material. However, current backfill selection criteria limit the nature and content of fines, thereby ensuring suitable creep characteristics. Mitchell (1997) indicates that creep strain rate of soils with a PI less that 10 and between 20 and 30 percent clay fraction is very low. Furthermore, it is known that creep deformation of soils compacted dry of optimum are small if saturation is prevented.

Mitchell and Zornberg (1994) present reviews of studies that indicate the actual long-term performance of geosynthetic walls backfilled with fine-grained soil is much better than predicted. This is because geosynthetic creep magnitudes and rates are known to be lower when confined in soil.

Pullout Resistance

The assumed active zone of a reinforced soil structure lies just behind the facing panels and in front of the failure plane (Figure 2). As the soil in the active zone moves away from the backfill, forces in the reinforcement develop and are transferred to the resisting zone.

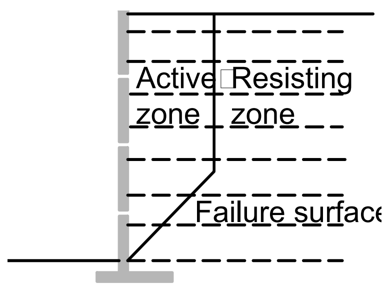

Figure 2. Driving and resisting zones within an MSE wall. Failure plane shown is for inextensible reinforcements.

Pullout tests are used to determine the available resistance of the reinforcements to forces tending to pull them out of the reinforced zone. The resistance is a function of frictional resistance over the surface of the inclusion and passive resistance (bearing) on transverse elements. The basic mechanisms responsible for the pullout resistance for alternative soils are the same as for coarse-grained soils. These include surface friction between soil particles and reinforcing, and mobilization of soil shear strength.

For sheet and smooth strip materials where there is complete separation between upper and lower soil layers, pullout resistance is controlled by the friction on the two inclusion surfaces. Where soil particles can lie within the plane of the reinforcing, such as grids and wire mesh, bearing resistance is developed on transverse elements (oriented perpendicular to the pullout direction) in addition to frictional resistance on the surface of the elements. Bearing resistance for these materials can contribute as much as 85-90 percent of the total pullout resistance (Bergado et al., 1993). Ribbed strip inclusions generate the majority of their pullout resistance by promoting soil dilation and a minor amount from surface friction.

In general, factors that tend to reduce the shear strength of the backfill soil will reduce pullout capacity. Well-graded, coarse-grained soils will provide the highest resistance whereas resistance decreases with increases in fines content and compaction water content (Elias and Swanson 1983; Bergado et al., 1993). These same studies found that pullout resistance of fine-grained soils was drastically reduced when compaction water contents were greater than 2 percent above optimum. LTRC (2004) and Bergado et al. (1992) found that lab pullout tests generally provide conservative approximations of field pullout resistance.

Shear Strength Properties

As discussed above, the effective stress shear strength of backfill soil is a critical design parameter that controls much of the behavior of MSE walls. Many of the correlations for MSE wall design in FHWA 2001 and the AASHTO specifications are based on select backfill soils with friction angles ranging from about 34 degrees to over 40 degrees. Experimental data between soil index properties and soil strength parameters were collected from the literature review and from the laboratory testing conducted for this study to provide some approximate estimation of strength. As additional data are collected, a potentially useful screening tool for soils that may be candidates for use as alternative backfill can be developed.

The influence of percentage passing the No. 200 sieve and a PI on the peak effective stress friction angle are shown in Figures 3 and 4. These data are briefly discussed below.

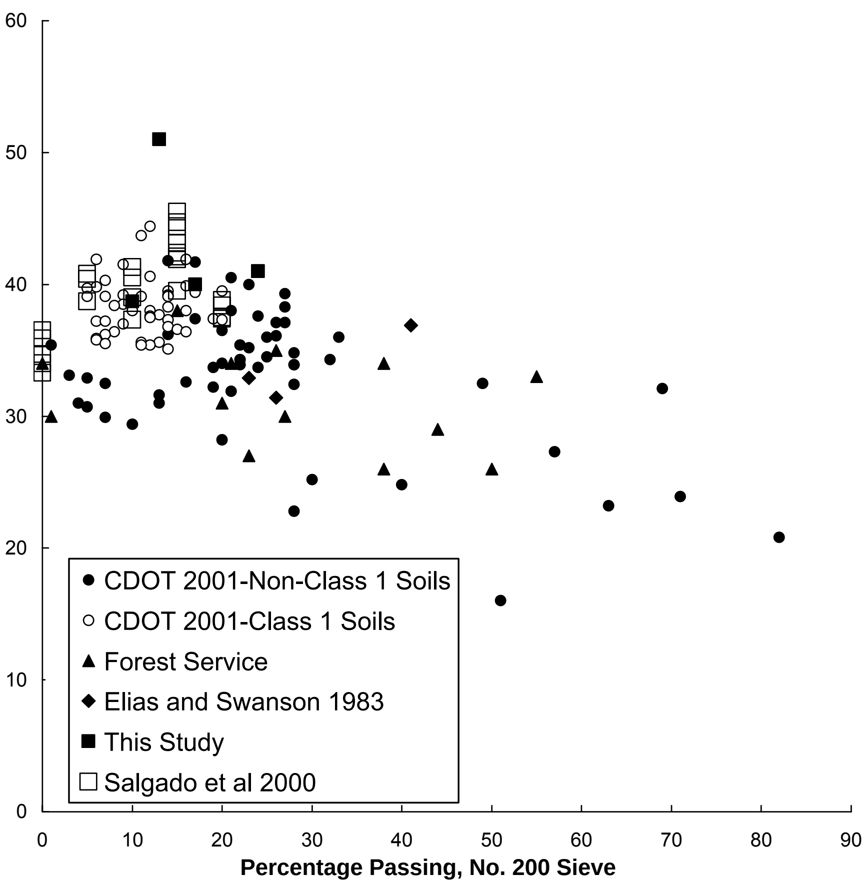

Figure 3. Peak Backfill Effective Stress Soil Friction Angle as Related to Percentage of Fines

In 2001, the Colorado Department of Transportation (CDOT) compiled index and strength parameters for 100 different soils, including liquid limit, PI, moisture-density relationships, and friction angles obtained from the direct shear test. Forty-seven of the soils were considered Class 1 Structural Backfill. These soils have the following index properties: 100 percent passing the 2-inch, 30-100 percent passing No. 4, 10-60 percent passing No. 50, and 5-20 percent passing the No. 200, PI < 6, LL < 35. The remaining 53 soils were classified as

non-Class 1. Samples were composed of the fraction passing the No. 4 sieve and were compacted 2 percent wet of optimum moisture to 95 percent of AASHTO T99 unit weight or 90 percent of AASHTO T180 unit weight.

CDOT concluded that the current practice of assigning a friction angle of 34 degrees was conservative based on the fact that all 47 Class 1 soils had a peak friction angle of greater than 35 degrees. Nineteen of the non-Class 1 soils also had a peak friction angle of greater than 35 degrees, many of which had fines contents between 20 and 30 percent. The remaining non-Class 1 soils with a friction angle of less than 35 degrees typically had a larger percentage of fines. However, no useful correlation between the internal friction angle and any single soil parameter was found (CDOT, 2001).

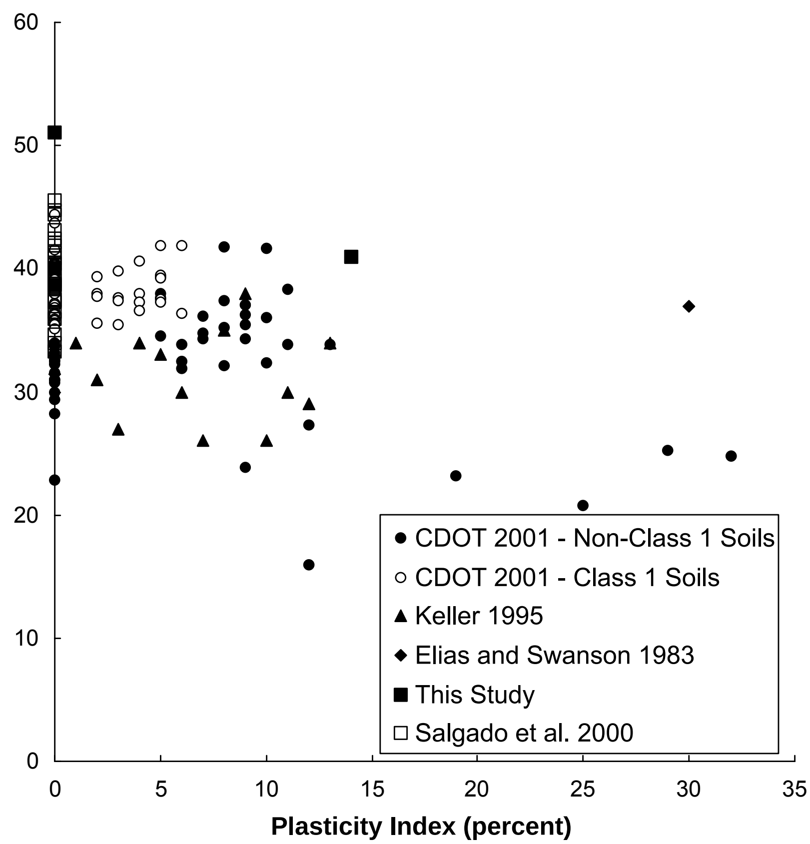

Figure 4. Peak Backfill Effective Stress Soil Friction Angle as Related to Plasticity Index

Salgado et al. (2000) studied the effects of non-plastic fines on the small-strain stiffness and shear strength of sands. Mixtures of clean, subrounded Ottawa sand and non-plastic ground silica fines were prepared with fines contents ranging from 0 to 20 percent. The Ottawa sand is poorly graded with 100 percent passing the No. 30 sieve and approximately 50 percent passing the No. 40 sieve. Isotropically consolidated drained triaxial compression tests were conducted to determine the strength parameters. At any given initial relative density, it was shown that even small additions of silt to the clean sand resulted in an increasing peak friction angle.

Keller (1995) presented effective stress friction angles from the results of 13 consolidated undrained triaxial tests on alternative soils that were successfully used in Forest Service Structures. Test specimens were compacted to 95 percent of their maximum dry density based standard Proctor. Fines contents ranged from 0 to 55 percent passing the No. 200 sieve, and the PI ranged from 0 to 15.

Elias and Swanson (1983) documented the performance and problems of a project that used fine-grained residual soils for reinforced earth backfill. They presented effective stress friction angles from three triaxial shear tests conducted on project backfill. Fines contents for the test specimens ranged from 23 to 41 percent passing the No. 200 sieve, and the PI ranged from 0 to 30.

Results of Survey of Wall System Manufacturers

A telephone survey of MSE wall system manufacturers on the VDOT approved list and one manufacturer on the probationary list was conducted to obtain their unique perspective on use of alternative backfill materials, and their requirements for backfill. Those on the approved list included Reinforced Earth Company, Foster Geotechnical Co., T&B Structural Systems, and The Neel Company. One company on the probationary list, Tensar Earth Technology, was surveyed.

The survey targeted senior design or supervisory engineers with sufficient experience to comment on past practices, knowledge of relevant case histories and current practice. Specific common questions included:

1. What are the backfill requirements with their system?
2. On projects where alternative backfill was used, how did the wall perform?
3. What are your opinions regarding use of alternative materials?

Results of the survey are summarized in Table 7. The Reinforced Earth Company manufactures steel strip (reinforced earth systems) and steel mesh (Pyramid systems) inclusions. Discussions indicated that they had used backfill materials with greater than 25 percent fines in the past. However, performance of the wall was poor. They now limit all backfill materials to less than 20 percent fines with a PI less than 6.

Foster Geotechnical Company manufactures VSL Retained Earth wall systems that employ a steel wire mesh inclusion. They also limit backfill fines content to less than 20 percent primarily due to corrosion and deformation. They can design walls with materials having friction angles of less than 34 degrees. Alternative soils are problematic, in their opinion, because of backfill variation, compaction problems, and increased inspection oversight.

T&B Structural Systems manufactures Hilfiker Walls that use steel wire grid inclusions. An A-2-4 material with a PI of 6 to 10 is the lowest quality material they will use. However, they have worked with materials with a PI up to 20 but have experienced problems with deformations during construction.

The Neel Company manufactures Isogrid walls that use a steel grid inclusion. They indicate that their policy is to adhere to the AASHTO specification for backfill.

Table 7. Summary of Discussions with Wall System Manufacturers

Manufacturer	Inclusion Types	VDOT status	Fines Limit (%)	Comments
Reinforced Earth Co.	Steel strip, steel mesh	Approved	20	Poor experience with soils containing greater than 25% fines. Require less than 20% fines with PI < 6.
VSL Retained Earth	Steel wire mesh	Approved	20	Upper limit due to corrosion concerns and deformation. Can design for lower friction angles.
Tensar	Polymer grid	Probationary	35-50	Feel their experience base is good enough to use alternative backfill. Backfill drainage is critical.
T&B Structural Systems	Steel wire mesh	Approved	35	A-2-4 with PI 6 to 10 is the lowest quality backfill they will work with.
The Neel Company	Steel wire mesh	Approved	15	Materials must meet AASHTO specifications.

Tensar Earth Technology manufactures polymer grid reinforcement systems. The Mesa system is on the probationary list. They feel their experience base is sufficient to allow them to comfortably use lower quality backfills. Silty sand backfills with fines contents of up to 50 percent have been successfully used. Backfill drainage is with higher fines content is extremely important.

Several issues were common among the manufactures surveyed. The major underlying issue is that the contractor building the wall is critical to overall performance. They feel that deformations during and after construction can be greatly reduced if the contractor has previous successful experience with a particular system. It was felt that this was the single most important factor in wall performance.

Each stated that facing panel distortions increase during construction with lower quality materials due to the low pullout capacity at the current layer and the increased difficulty with compaction relative to a select fill. However, distortion during construction can be controlled by experienced contractors. Manufacturers of steel grid reinforcing materials stated that friction angles of the reinforced fill should generally be greater than 34 degrees and have a PI less than 10.

Results of Interview with California Department of Transportation (CALTRANS)

Kevin Riley, a geotechnical engineer responsible for MSE wall construction in California, was contacted to discuss current California Department of Transportation (CALTRANS) practice. For the last 20 years, CALTRANS has allowed the reinforced backfill

zone to contain up to 25 percent fines with a PI less than 10. Approved reinforcing elements consist of steel strips and mesh. Compaction requirements are 90 percent of AASHTO T-99.

They have experienced numerous problems with excessive panel movement. This problem is attributed to the low relative compaction requirement and the low level of workmanship displayed by contractors. CALTRANS feels that the overall experience level of contractors is declining as more companies are formed by less experienced individuals. Marketplace competition, and the need to produce profits, is forcing good contractors to accept lower levels of workmanship. Because of these problems, CALTRANS is seeking to implement the AASHTO backfill specifications of no more than 15 percent passing the No. 200 sieve and a relative compaction of 95 percent.

NCHRP Test Wall Design & Construction

Throughout the study, the authors have had regular telephone conversations and information exchange with Mr. Richard Stoulgis, Project Manager for the NCHRP-sponsored project "Selecting Backfill Materials for MSE Retaining Walls." GeoTesting Express, Inc. of Boxborough, Massachusetts, is the contractor for this project. The original project schedule called for construction of a series of three test walls during late summer 2004. The sections were to be constructed with reinforced-zone backfills that do not meet AASHTO specifications.

Monitoring of the test walls is to be conducted over a period of approximately 2 years. An innovative water handling system will permit controlled saturation of the backfill materials. Deformation monitoring will permit systematic evaluation of backfill/reinforcement performance.

The decision was made by a NCHRP review panel to delay construction of the test walls until spring 2005. At the time this report was prepared the PI was waiting for the first interim project report to be made available. The following description of proposed wall test sections is based on information available in late October 2004.

Because of the interest in using alternative backfills by other organizations, the number of test walls to be constructed has increased. It is understood that up to eight test sections may be constructed. Table 8 provides details on six of the proposed sections.

NCHRP-sponsored test sections will incorporate geosynthetic reinforcing with a flexible, welded wire facing. It is proposed at this time to correlate the performance of the flexible system with that of a rigid facing system such as a concrete panel but details on how the correlation would be accomplished were not available. The NCMA is to sponsor two or more sections that incorporate polyester and polyethylene geogrid reinforcement and PMU facing. The polyester sections will have both fixed (pinned) and friction connections at the block face. An unidentified steel reinforcement manufacturer has proposed a test section using and A-2-6 backfill. Details of this section are not available. It should be noted that participation of the metallic reinforcing manufacturers in the NCHRP study came after the interviews described in a previous section.

Table 8. Proposed Test Wall Sections for NCHRP Project

Wall Sponsor	Wall Facing	Reinforcement	Backfill
NCHRP	Welded wire	Geosynthetic	A-1-a (15% fines, control section)
NCHRP	Welded wire	Geosynthetic	A-2-6 (35% fines, PI<6)
NCHRP	Welded wire	Geosynthetic	A-4 (50% max. fines, PI<10)
NCMA	Block	Geosynthetic (two types)	A-4 (50% fines, PI approx. 20)
Steel manufacturer	Unknown	Steel	A-2-6

Results of Laboratory Testing

Index Tests

Laboratory index tests including grain size analysis, Atterberg Limits, and standard Proctor moisture-density relationships were performed on the four soils obtained for the project. Tests were conducted in general accordance with applicable AASHTO standards. Table 9 summarizes the results of the index testing. Refer to the Appendix for more detailed results of laboratory index testing.

Table 9. Summary of Laboratory Index Tests

Soil	Gravel (%)	Sand (%)	Fines (%)	LL	PI	Maximum Dry Unit Weight (pcf)	Optimum Water Content (%)
Sandy Bottoms (SB Orange)	0	89	11	NP	NP	110.5	14.0
Isle of Wight – non plastic (IOW Gray)	0	83	17	NP	NP	114.0	12.0
Isle of Wight - plastic (IOW Red-Brown)	3	74	23	41	14	105.0	19.0
VDOT 21A	52	35	13	NP	NP	--	--

Strength Tests

Strength of four soils was characterized in the laboratory using the consolidated-drained direct shear test. Additionally, the Isle of Wight soil with PI=14 was tested using the isotropically consolidated drained triaxial test to verify that strengths measured in the direct shear test were fully drained. Tests were conducted in general accordance with applicable AASHTO standards. Table 10 summarizes the results of the strength testing. Refer to the Appendix for more detailed results of laboratory strength testing.

Table 10. Summary of Laboratory Strength Tests

Soil	As-Molded Dry Unit Weight (pcf)	As-Molded Water Content (%)	RC (%)	ϕ'_p (deg)	c'_p (psi)	ϕ'_r (deg)	c'_r (psi)	Water Content after Shear (%)
SB Orange	107.2	17.3	97.0	38.7	2.4	30.0	1.4	19.4
IOW Gray	108.7	17.1	95.4	40.0	1.7	30.6	1.5	18.0
IOW Red-Brown	100.4	23.7	95.6	41.0	1.4	40.3	0.8	25.0
VDOT 21A	138.5	10.4	--	50.9	2.3	--	--	--

DISCUSSION

Synthesis of Literature Review and Discussions with Others

Current State of Practice for Reinforced Backfill

The paragraphs below provide a brief description of the current state of MSE wall practice, based on the results of our literature review. Discussion is included regarding how current VDOT practice is related to the practice of others. This section focuses on (1) reinforcing systems, (2) design, (3) backfill gradation, (4) selection of backfill shear strength parameters, and (5) backfill placement and compaction.

Reinforcing Systems

Current VDOT practice is to provide a contractor with general provisions for wall system options from the approved and provisionary lists. For critical applications, the wall systems are selected from the approved list. The approved systems use steel mesh or strip inclusions. Probationary systems must initially be used on non-critical wall projects and receive approval from the Structure and Bridge Division Assistant Division Administrator for Geotechnical Design.

Design

VDOT currently assumes responsibility for external stability, which includes overturning, sliding, bearing capacity, settlement and deep stability failure below reinforced zone. Internal stability, which includes reinforcement spacing and tensile strength, soil pullout length, and facing connection strength, is the responsibility of the general contractor. Reinforcing inclusions for MSE walls constructed by VDOT generally consist of steel strips or mesh with a minimum length of 0.7H behind the wall face. This is consistent with the practice of other state transportation departments.

Internal design of MSE walls is typically based on the "Simplified Method" contained in the AASHTO specifications and in FHWA (2001). The NCMA design method (NCMA 1997) is used in the design of MBW walls. A detailed discussion of the design procedures commonly

used in North America is presented in FHWA (2001) and NCMA (1997) and is not contained herein.

Koerner and Soong (2001) compared the three design methods for evaluating external and internal stability in geosynthetic reinforced MBW's. They conclude that all three approaches are acceptable; although results on a typical wall example were found to be most conservative using modified Rankine, least conservative by NCMA (1997), and intermediate by FHWA (1997). It has been shown in two publications, Allen et al. (2004a) and Allen et al. (2004b), that the Simplified Method over predicts the reinforcement forces for geosynthetic inclusions. The new K-Stiffness approach introduced by those authors offers promise for more efficient wall design, however, it the method has not been adopted for use on federal highway projects.

Backfill Specification

VDOT currently specifies select backfill material for all walls in accordance with FHWA (1997) gradation recommendations, which requires less than 15 percent fines with a PI of less than 6. Backfill must conform to specifications for magnesium sulfate soundness loss and electrochemical requirements. A minimum direct shear friction angle of 34 degrees for the fraction finer than the No. 10 sieve is required.

Based on the literature review there appears to be no clear concurrence among geotechnical engineers regarding fines content for the reinforced zone, especially for polymer inclusions. Table 11 provides three widely accepted gradations in comparison with the VDOT requirements. Gradations recommended by Koerner and NCMA are for polymer reinforced MBWs, while the FHWA gradation is intended for steel reinforcements. FHWA recommends a maximum particle size of ¾-inch for polymer reinforcements. NCMA also recommends a maximum particle size of ¾-inch for polymer reinforcement unless tests are performed to assess the potential for construction damage and strength reduction to the reinforcements. Note that fine-grained soils are not included in Table 11.

Table 11. Summary of Backfill Gradation Requirements

Sieve No.	Size (mm)	Percent Passing			
		Koerner (1998)	VDOT	FHWA (2001)	NCMA (1997)
4-inch	100	---	100	100	100 – 75
¾-inch	19	---	---	100 (polymer inclusions)	100 (polymer inclusions)
No. 4	4.8	100	---	---	100 – 20
No. 10	2.0	90 – 100	---	---	---
No. 40	0.42	0 – 60	0 – 60	0 – 60	0 – 60
No. 100	0.15	0 – 5	---	---	---
No. 200	0.075	0	0 - 15	0 – 15	0 – 35
Comments		Geosynthetic reinforced MBW walls.	-PI < 6 -Steel strip and mesh reinforcement.	-PI < 6 -Free of organics	Geosynthetic reinforced MBW walls, PI < 20.

It is anticipated that, in general, sufficient quantities of native Virginia soils can be found for project purposes. Recovery of native soils must meet VDOT specifications for control of materials. The most likely sources of material will be located in sand and gravel deposits situated in stream and river terraces, and in Coastal Plain deposits. River or stream dredge material might also be suitable depending on the source material and recovery methods.

It is unlikely that residual soils of Virginia can be directly utilized because they tend to have high fines contents and Plasticity Indices. Saprolite and partially weathered bedrock zones underlying native residual soils are possible sources of borrow provided that the rock fragments are not friable and otherwise meet VDOT durability requirements. Lower quality quarried material is also a potential source of alternative backfill material.

Backfill Placement and Compaction

Current VDOT construction requirements for select backfill placement and compaction are as follows:

1. 8-inch maximum loose lift thickness.
2. Compact to 95 percent of maximum dry density as determined by AASHTO T-99.
3. Placement moisture content should be at optimum moisture content, or a maximum of two percentage points dry of optimum moisture content.
4. Compaction adjacent to the backside of the wall in a strip 3 feet wide is to be achieved using mechanical hand tampers.

These methods are consistent with those of other departments of transportation and general engineering practice for MSE walls using select backfill materials.

Selection of Strength Parameters

Shear strength of the backfill soil for MSE wall design is defined in terms of the effective stress (drained) friction angle. Drained conditions are representative of both the short and long-term conditions of select backfill as pore pressures are not likely to be present, and any failure that takes place will likely be fully drained. AASHTO requires a maximum effective stress peak friction angle of 34 degrees be used for determination of horizontal earth pressures within the reinforced soil zone and for pullout resistance. It is recognized that this friction angle is likely conservative for select fill, and if used in design will result in a wall system that typically has small construction and post-construction deformations.

From the test results of Figure 3, it can be seen that peak drained friction angle for soils compacted at optimum or 2 percent wet of optimum decreases with increasing fines content. However, there is large scatter in the data so that, at any given fines content, the friction angle can vary by up to 50 percent. There is a tendency for friction angle to increase in the range of 0 to about 20 percent fines. This is caused by a reduction in the initial void ratio at a given relative compaction due to filling of void space by the fines. For a compacted soil, this would increase the magnitude of dilation during the initial stages of shear, which will increase the peak friction angle. Test results from this study are in agreement with the data from others.

Figure 4 shows the variation in peak friction angle with a PI for the soils of Figure 3. Again, there is a large amount scatter in the data. However there appears to be a decrease in strength for Plasticity Indices greater than 10. Below a PI of 10, there is no clear trend in the data.

Analysis of the limited data in Figure 3 suggests that the peak friction angle of these compacted soils with fines contents less than 30 percent and a PI less than 10 is approximately 36 degrees. Friction angles of these soils with fines content of 15 percent or less and a PI less than 6 is 37 degrees. Therefore, considering the average drained peak friction angle from this data set alone, there is little difference between the lower bound friction angle for select fill and the alternative soils. It must be emphasized that this is a small database with strength values determined in direct shear and triaxial compression. Site-specific data should always be used for design. However, it suggests that the drained strength of alternative soils with up to 30 percent fines compares well with that of select material. It is expected that backfills with a lower percentage of fines will have greater stiffness than soils with higher fines content. At any given working stress the potential deformation will be smaller for the stiffer soils.

In most design methods for reinforced soil structures, reinforcement strength and length requirements are based on the shear strength of the soil through which potential failure surfaces are likely to occur. Generally, free-draining compacted backfill soils exhibit strain-softening behavior. Thus, failure of the soil and the shear strength parameters can be quantified in terms of peak or residual strength. The question of which to use in design is often debated. Which strength parameter to use in design is generally determined based on strain compatibility between the soil and the reinforcement. Reinforcements are classified as "extensible" if the strain at failure in the reinforcement exceeds the strain required in the soil to develop an active pressure. Geosynthetic reinforcements are considered extensible. Conversely reinforcements are "inextensible" if the strain at failure in the reinforcement is less than the strain required in the soil to develop an active pressure. Steel reinforcements are considered inextensible.

Peak soil strength is generally used with inextensible, metallic reinforcements because the peak strength in the reinforcements is mobilized more quickly than the soil's peak strength. With geosynthetic reinforcement applications, the soil strength is expected to reach its peak before the reinforcements achieve their ultimate strength. It follows that residual strengths may be appropriate for use in geosynthetic reinforced structures. Residual friction angles will be either lower or the same as the peak friction angles presented in Figures 3 and 4, depending on actual stress-strain behavior. Table 10 shows that residual friction angles from tests conducted on two of the Virginia soils are 8 to 10 degrees less than the peak values.

Discussion of Case Histories and Wall Manufacturer Interviews

General Comments

Published case histories show that coarse-grained alternative soils can be effectively used in the reinforced zones of walls, provided that moisture content and compaction during construction are controlled, and appropriate drainage systems are implemented to prevent generation of excess pore pressures after construction. Poor performance was observed where

excess pore pressures were generated in the fill, especially where impermeable metallic reinforcements were used.

Fine-grained soils appear to provide acceptable performance where special consideration was given in design to these materials and construction was carefully controlled. Because of the difficulties associated with fine-grained soils, they are not amenable to VDOT's design and construction methods. Fine-grained soils are not considered further in this report and the term "alternative backfill" as used henceforth refers to coarse-grained soils (less than 50 percent passing the No. 200 sieve). Perhaps as experience is gained, fine-grained soils may be included.

Experience with the reinforcement of alternative soils has indicated a number of issues to consider during the design and construction phases of non-critical walls:

1. Strength properties and moisture-density relationships of candidate backfill soils must be tested on an individual basis. It would be unconservative to use presumptive values of shear strength for alternative soils.
2. Walls should be designed to tolerate larger total and differential settlements. Therefore flexible face systems should be considered.
3. The corrosive effect of the fine-grained fraction on metallic reinforcements must be anticipated. Corrosion is controlled by the clay content (less than 2 microns) of the soil. The PI of backfill soils for use with steel reinforcement should be less than 6.
4. Some additional construction effort to compact and moisture condition soils should be anticipated.
5. Polymer reinforcement should be considered.
6. Provisions must be provided to prevent saturation of the backfill.
7. Because of previously discussed performance issues, traditional systems incorporating metallic reinforcements and segmental facing panels may not be suitable for use with alternative backfills with greater than 20 percent fines.
8. Wall system manufacturers on the approved list indicated varying levels of experience and confidence with using alternative fills in MSE wall applications. All agree that the contractor building the wall is the single most important factor in wall performance and ultimate satisfaction with the wall system. They feel that deformations during and after construction can be greatly reduced if the contractor has previous successful experience with a particular wall system.

Lateral Wall Deformation

Tables 4, 5, and 6 provide data that permit estimation of an index of relative deformation for reinforced walls backfilled with select materials versus alternative soils. Deformations in this discussion are expressed in terms of lateral movement divided by the wall height (δ/H) so that direct comparison of case history data can be made. It is unfortunate that results from construction of the NCHRP test walls are not available for this discussion.

For reference, the empirical relationship contained in FHWA (2001) indicates that for a wall height of 18 feet and reinforcement length of 0.7H, normalized lateral deformations could be on the order of 0.004 for steel reinforcements to 0.013 for polymer reinforcements. Long-term

plumbness for VDOT walls is currently considered to be approximately ½ inch over 10 feet (0.0042).

The data contained in Table 4 suggest that the average normalized lateral deformation in select fill is 0.007 for geogrid reinforcement and 0.011 for geotextile reinforcement. Data in Table 5 suggest the normalized deformation for geotextile walls with alternative backfill is 0.09. Three cases for walls with metallic reinforcement and alternative backfill suggest that the maximum normalized deformation is on the order of 0.03. Table 5 suggests that no or little movement was observed in welded wire and geogrid reinforced walls backfilled with alternative soils.

This analysis is based on the limited published information and includes a wide variation in backfill, inclusion materials and construction methods. Nevertheless, it does suggest that lateral movements are, in general, greater with alternative backfills and polymer inclusions. Therefore determining what magnitude of movement is considered acceptable for non-critical structures is important. In addition, with potentially greater lateral deformations, a facing system that is relatively flexible would have advantages over conventional segmental panels.

One method to reduce movements would be to increase the minimum reinforcement length to 1.0H. For select fill, increasing the reinforcement length reduces the normalized deformations to a range of 0.003 to 0.010 based on the empirical FHWA (2001) relationship. Alternatively, the number of reinforcing layers could be increased.

It is important that a database of measured lateral deformations from MSE walls backfilled with alternative materials be developed. Data from controlled test walls supplemented with numerical modeling are needed to obtain more refined estimates of deformations.

For design life of 75 to 100 years, long-term deformation under constant load, or creep, of either the backfill material or the reinforcement is a concern when the backfill contains fines or is reinforced with geosynthetics. Creep in a reinforced backfill is a coupled process whereby deformation of one component (either the soil or reinforcement) influences the deformation of the other component. Based on the available information, long-term deformations are less than those predicted from current design methods. Backfill soils that contain a high percentage of coarse-grained particles are less subject to creep deformations due to the degree of particle interlocking. As these particles are pushed apart by increasing fines content, this interlocking action is suppressed making the material more prone to creep deformation.

Current AASHTO backfill requirements of less than 15 percent fines preclude high creep rates of the backfill soil. Well-graded silty sands with fines contents less than 30 percent should have low creep rates if saturation is prevented. Measurements from instrumented test walls, such as the NCHRP test sections, and a series of creep pullout tests are critical for verifying this observation.

Pullout Resistance

Pullout resistance is of critical importance within the upper 10 feet of a MSE wall. Once overburden pressures become high, breakage, not pullout, controls design. It also controls much of the deformation observed in walls.

Based on the literature review there is little information available on the pullout behavior of silty/gravelly sand materials. Wall system manufacturers maintain proprietary databases of pullout test results for different soil gradations paired with their particular reinforcing inclusion. These data are not generally available outside these companies. As such, engineers must rely on results of research or project specific tests.

However, based on discussions with manufacturers it is believed that the materials considered for reinforced backfill in this report will have adequate pullout resistance. A comprehensive database on pullout resistance using candidate reinforcements and alternative backfills would provide valuable information and a basis for selecting alternative backfills.

Synthesis of Laboratory Testing and Shear Strength Parameters

Soil Testing

Because the friction angle of a soil is not a function of any one soil property, alternative native soils that are being considered for reinforced backfill should be tested on an individual basis for shear strength properties. Currently, the VDOT draft special provision for MSE Walls specifies that the angle of internal friction of a soil should be determined by the direct shear test in accordance with AASHTO T236. The direct shear test is interpreted as a drained test. Pore pressures are allowed to completely dissipate during the consolidation phase before shear and the deformation rate is sufficiently slow to allow dissipation during shear. Coarse-grained, select fill materials have drainage rates that are sufficiently rapid to permit practically instantaneous pore pressure dissipation. Several potential problems with applying the current method of testing to alternative backfills were noted in this study.

For determining the approximate strain rate, AASHTO T236 prescribes a time to failure of $50 \cdot t_{50}$, where t_{50} is the time required for the specimen to achieve 50 percent consolidation under the applied normal stress. ASTM D3080, Note 16 offers more guidance with respect to strain rate determination for a consolidated drained direct shear test. It states that for dense sand with more than 5 percent fines, a time to failure of 60 minutes may be used if t_{50} cannot be determined by a well-defined time-settlement curve. This guidance may result in a strain rate that is too fast for complete dissipation of excess pore pressures during shear when testing candidate backfill soils with even a relatively small percentage of fines. For densely compacted soils that have a tendency to dilate during shear, this can result in the generation of negative pore pressures, which increase the effective normal stress to an unknown quantity. This can result in misinterpreted data and an unconservative (high) estimation of friction angle.

In addition to careful consideration of the strain rate, silty sands with higher percentage of fines should be inundated with water and allowed to fully consolidate under the applied normal stress prior to shearing. For densely compacted soils that tend to dilate during shear,

capillary suction forces can be generated in moist samples that have no access to free water. This will also result in an unknown increase in effective normal stress on the sample, which can lead to a high estimation of friction angle.

These two problems were observed during direct shear testing with the red-brown silty sand (24 percent fines, PI of 14). A brief summary of the testing conditions and results is summarized in Table 12.

Table 12. Summary of Test Conditions for Red-Brown Silty Sand

Shear Box Description	Strain Rate (mm/min)	Inundated	Friction Angle (deg)
12-inch square	1.0	No	44
4-inch square	0.25	No	46
2.5-inch diameter	0.005	Yes	41

Although the effect of shear apparatus size is not evaluated here, it can be seen that strain rate and sample inundation have an effect on the resulting friction angle of the soil. A drained friction angle of approximately 41 degrees was confirmed for this soil from a consolidated-drained triaxial test.

It is critical that personnel conducting laboratory tests on alternative backfill materials recognize the potential difficulties with pore pressures and plan accordingly. This statement applies to consultants as well as VDOT internal practice. Shear strength values should be reviewed by a geotechnical engineer before use in design. If necessary, confirmation tests using a drained triaxial test may be necessary.

Alternative Wall Systems and Reinforcements

VDOT typically specifies MSE walls with steel reinforcing inclusions and precast concrete facing panels. It has been shown, however, that metallic reinforcements may not be suitable for use with backfills containing higher percentages of fines due to the increased potential for corrosion. Also noted is the fact that segmental concrete facing panels may be too rigid for walls with alternative backfills where larger deformations will be generally expected. Therefore, geosynthetic-reinforced Modular Block Walls (MBW's) may be an attractive optional system for non-critical applications.

MBWs are a form of MSE wall in which the front of the wall consists of dry-stacked (without mortar), concrete masonry units. These are also commonly referred to as Segmental Retaining Walls (SRW's). Masonry units are precast, machine produced concrete blocks without internal reinforcement. MBW's are generally reinforced with geosynthetics that extend through the interface between the MBW units and into the backfill to create a composite gravity mass structure. Use of this type of geosynthetic-reinforced wall is undergoing enormous growth and acceptance by public agencies. The abutment walls for the Founders/Meadows Bridge designed by the Colorado DOT are a good example. The spread footings for the bridge superstructure

bear directly on the reinforced zone of the select backfill. These abutment walls have performed well (Abu-Hejleh et al., 2001).

Modular block retaining walls offer several advantages over traditional precast concrete facing panels. Alternative soils can more readily be used within the reinforced zone of the backfill. While high-quality granular soils are still preferred, even fine-grained soils have been successfully used for reinforced backfill. The geosynthetic reinforcements are composed of polymers that resist corrosion. MBW units have a pre-defined batter angle and are not bonded by mortar. Therefore they can move and adjust relative to one another, thus enabling the wall to tolerate movement and settlement. Maintaining proper drainage, moisture content, and density is of primary importance with these systems. The Tensar MESA MBW system is on the probationary list.

Polymer reinforcements are also used for traditional precast segmental panels and full height precast facing systems. Presently, the Tensar ARES system is on the probationary list. This system employs geogrids and either full height or segmental precast concrete facing panels. Tensar recommends that the backfill for the full height system have less than 25 percent fines with a PI less than 20. Backfill for the segmental panel system should have less than 15 percent fines with a PI less than 6. Therefore the material requirements recognize the potential for larger deformations for the segmental system over the full-height panel system.

CONCLUSIONS

The following conclusions pertaining to non-critical MSE walls can be drawn from this study:

- Fine-grained soils (more than 50 percent passing the No. 200 sieve) are not appropriate alternative backfill materials for VDOT applications.

- Alternative native or quarried backfill materials consisting of well-graded coarse-grained soils containing silty or clayey fines can be successfully used in the reinforced backfill zone of MSE walls.

- The direct shear test is suitable for assessing the shear strength parameters of silty sands provided that samples are inundated, allowed to consolidated completely under the applied normal stress prior to shear, and are sheared at strain rates slow enough to ensure fully drained conditions.

- A properly constructed and functional internal drainage system is essential in walls backfilled with alternative native soils.

- Geosynthetic reinforcements and modular block facing are an attractive alternative to metallic reinforcements and precast concrete panel facing when considering backfills with a greater percentage of fines.

- Good contractor workmanship and quality control by the owner's representative is essential for projects involving alternative backfills.

RECOMMENDATIONS

Backfill for Reinforced Zone

Based on the findings of this limited study, the fines content and a PI for backfills within the reinforced zone of non-critical MSE Walls are presented in Table 13. It is recommended that the values in Table 13 be used as a starting point for modifications to current backfill specification for non-critical walls. The current VDOT gradation requirements for portions coarser than the No. 200 sieve should be maintained except that the maximum particle size for metallic reinforcements could be reduced to be less than 3 inches. The maximum particle size for use with geosynthetic reinforcing materials should be limited to ¾ inch to minimize installation damage. All materials should be well graded.

Table 13. Backfill Fines Contents for Non-Critical Walls

Reinforcement Type	Metallic	Geosynthetic
Percent passing No.200	< 20	< 30
Plasticity Index (PI)	< 6	< 9

All current additional VDOT material property specifications for the reinforced zone, such as chemical requirements and soundness, should apply to these materials. Depending on the performance of actual wall systems the fines content limits, gradation requirements and Plasticity Indices could be increased or decreased as needed.

In addition to native materials described earlier, the above gradation limits would include several standard VDOT dense graded aggregates. VDOT 21A, 21B, and 22 appear to be potential candidate materials. Because of installation damage potential, 21A should not be used with geosynthetic materials. Quarry waste material could potentially be used but should be evaluated on a case-by-case basis until sufficient experience is developed with these materials.

Based on the available information, it is likely that the peak drained friction angle of the material passing the No. 10 sieve compacted to 95 percent of AASHTO T-99 density at optimum moisture will be at least 34 degrees. However, tests should be conducted on candidate materials before being approved for use to verify the actual peak and residual drained strengths. Residual strength should be used for design of walls incorporating geosynthetic reinforcement.

Wall Systems and Reinforcements

It is recommended that VDOT consider the use of geosynthetic-reinforced, MBWS for non-critical wall applications where alternative soils are being considered as potential backfill. Information on MBWs is included in the Discussion section. MBWs are able to accommodate

wall movements that might otherwise be unacceptable with segmental panels. Only one system is currently on the probationary list (Tensar Mesa) but it is understood that additional systems have undergone HITEC approval.

Soil Testing

It is recommended that shear strength of alternative soils being considered for reinforced backfill be determined on an individual project basis. The following additional guidance when testing a soil with more than 15 percent passing the No. 200 sieve are recommended:

- Compact soils in the direct shear apparatus on the wet side of optimum to 95 percent of maximum dry density as determined by the Standard Proctor test (AASHTO T-99).

- Inundate samples and allow full consolidation under the applied normal stress. Monitor the vertical deformation of the sample over time under the applied normal stress.

- Assumed or typical strain rates for granular soils should not be used. If a well-defined time deformation curve is not obtained, use as slow of a strain rate as is practicable. This study employed a strain rate of 0.005 mm per minute.

- Check measured friction angle obtained against commonly accepted values. If results are questionable, triaxial compression tests such as the isotropically consolidated undrained (CU) or the consolidated drained (CD) test can be used to check the results.

Improper test methods will most likely result in a higher friction angle that will not be representative of the long-term conditions (i.e., unconservative).

Drainage

It is imperative that all walls backfilled with alternative backfill soils have an internal drainage system installed during construction. Proper drainage systems should include continuous freely draining aggregate for a minimum distance of 12 inches behind the wall face, beneath the reinforced zone of the backfill, and behind the reinforced zone of the backfill. Positive drainage at the surface should be away from the reinforced zone and should be maintained at all times during and after construction. An impermeable boundary, such as a high-density polyethylene geomembrane, should be installed above the first layer of reinforcement to prevent saturation of the backfill after construction. The drainage system should be monitored to ensure functionality after precipitation events. MBW masonry unit fill, where required, should also be a free-draining material.

Geocomposite drainage materials are a widely used method of water control. They could possibly be used for the drainage system for MSE walls backfilled with alternative soils. However, use of geocomposites directly behind the facing is complicated due to the presence of the connections between the reinforcement and facing. This will require numerous penetrations

or use of strips of material placed between reinforcement layers. It is possible that a geocomposite that has a geotextile covering both sides could be utilized behind the reinforced zone to intercept groundwater from the native retained soil. The composite drain could extend under the reinforced zone to a collection pipe located just behind the wall. Design for geocomposite drains must account for the decrease in flow rate due to compression of the core from soil pressures.

Construction

It has been shown that contractor workmanship and quality control during construction are perhaps the most important factors contributing to the success of reinforced earth wall project. Earthwork operations should be monitored at all times by an experienced field representative of the geotechnical engineer. Moisture content and density of the controlled fill must be monitored closely to ensure proper compaction. The fill source (onsite or otherwise) should be evaluated often for variability and compliance with the new recommended backfill specification.

There does not appear to be any need at this time for modifications to VDOT's current compaction requirements for use with alternative materials.

Information Needs

Information needs have been identified and are summarized here:

- An instrumentation program should be implemented to monitor selected full-scale MSE walls constructed with both select fill and alternative soil as backfill within the reinforced zone. The walls should be monitored for settlement, pore pressure, lateral deformation and reinforcement strains. It is also possible that pullout resistance of reinforcements could be measured in the field on these full-scale structures. With new wireless communication systems, monitoring can be significantly less manpower intensive than in the past.

- In conjunction with a monitoring program, a series of test walls could be constructed with alternative soils, reinforcing materials and facing elements. These walls could be monitored over for two to three years to assess their performance. Additionally, the collaboration with the NCHRP test wall program should be continued. Results of the long-term monitoring and inundation testing planned for that project also should be transferred to Virginia practice.

- While the engineering properties of clean sands and fine-grained soils have been studied extensively, the behavior of sand and silt/clay mixtures has received little attention. A basic research study to assess properties and behavior of mixtures consisting primarily of sand with varying percentages of low plasticity silts and clays (say, 5 to 45 percent) could be used to validate existing theoretical developments for soil mixtures. A consistent

quantitative framework would aid in the determination of material properties and provide more accurate screening tools for assessing native backfill materials.

- A detailed analysis should be completed to assess the potential savings and assess the additional risks associated with the use of alternative backfills.

COSTS AND BENEFITS ASSESSMENT

This section provides a brief, preliminary discussion of the cost benefits and risks of using alternative backfill material for MSE walls. It is difficult to provide realistic values of potential costs at this time for all aspects of constructing these wall systems, as many factors are site and contractor specific. The table below presents a listing of areas where there are potential cost savings and benefits and where there are perceived risks. It should be noted that this list is not static. As projects are performed with alternative soils, additional economies can be realized and risks minimized.

Table 14. Listing of potential costs, benefits, and risk

Benefits	Risks
1. Select fill not required.	1. Construction testing and quality control potentially increased.
2. Disposal of native soil reduced.	2. Material testing costs increased.
3. Use of on-site soil or lower cost quarried material.	3. Internal drainage system is required.
4. Potentially greater cost savings in material costs where projects are located in rural areas.	4. Contractor costs potentially increased.
5. Potentially lower cost, easier to construct wall systems can be used.	5. Weather delays potentially more severe.

The greatest potential for cost savings is in material supply, disposal and transportation. For instance, Keller (1995) reported that, for Forest Service walls, the average cost of local backfill material was $8 per cubic yard including procurement, placement, compaction and haulage. In comparison, the average cost of imported select fill was $18 per cubic yard including procurement, placement, compaction and haulage. For a wall with 1,500 square feet of wall face, and 750 cubic yards of backfill Keller (1995) estimated that the average cost is $4 per square foot of wall face for local material versus $9 per square foot of wall face for imported material. These costs do not include the wall or facing system.

The cost differential between native and imported fill would not be expected to decrease over time. On the contrary, as fuel costs have increased, transportation of imported material could result in even greater differentials. If lower cost quarried materials were used instead of on-site materials, the transportation costs would be added to the cost of the alternative soil, reducing the cost differential.

For Virginia, realistic approximate costs for material, placement and compaction could be obtained from recent VDOT embankment fill construction projects. It is expected that these costs would be nearly the same as those for alternative soils used in MSE walls. The influence of placement issues would implicitly be included.

Another area in which cost savings might be realized is in the use of lower cost wall systems. Geosynthetic-reinforced MBW's might prove to be lower cost than traditional panel walls.

There is the potential of incurring additional quality control (QC) costs for construction control on the part of VDOT. It is recommended in this report to have close inspection of the backfilling operation. This requires a VDOT representative to be present at the site. The contractor will also have additional testing costs to prove that the soils meet the backfill requirements for soundness, chemical stability, and strength.

A detailed assessment should be conducted to determine the additional cost for construction (inspection and testing) quality control, contractor personnel, equipment time required to compact the alternative backfill, and the cost of an internal drainage system.

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APPENDIX
LABORATORY TEST RESULTS

TABLE A-1
Summary of Laboratory Test Results

Sample Description and Source			**Index Test Results**				
Sample No.	**USCS Description**	**Borrow Source**	**%-Finer No. 200**	**LL**	**PI**	**$\gamma_{d\ max}$ (pcf)**	**$w_{c\ opt}$ (%)**
1	Gray, gravel and fine to coarse sand, little silt. (GW - VDOT 21A Stone)	Sisson & Ryan Quarries, Shawsville, VA	13	NP	NP	--	--
2	Orange, fine to medium SAND, little silt. (SP)	Sandy Bottoms Materials, Suffolk, VA	10	NP	NP	110.5	14.0
3	Gray, fine to medium SAND, little silt. (SP)	Isle of Wight Materials Co., Smithfield, VA	17	NP	NP	114.0	12.0
4	Red-brown, fine to medium SAND, some silt. (SP)	Isle of Wight Materials Co., Smithfield, VA	24	41	14	105.0	19.0

Sample Description And Source			**Consolidated Drained Direct Shear Test Results**					
Sample No.	**USCS Description**	**Borrow Source**	**Molded w_c (%)**	**Molded γ_d (pcf)**	**% RC**	**ϕ_p (deg)**	**ϕ_r (deg)**	**Soaked w_c (%)**
1	(GW - VDOT 21A Stone)	Sisson & Ryan Quarries, Shawsville, VA	10.4	138.5	--	50.9	--	--
2	SP	Sandy Bottoms Materials, Suffolk, VA	17.3	107.2	97.0	38.7	30.0	19.4
3	SP	Isle of Wight Materials Co., Smithfield, VA	17.1	108.7	95.4	40.0	30.6	18.0
4	Red-brown, fine to medium SAND, some silt. (SP)	Isle of Wight Materials Co., Smithfield, VA	23.7	100.4	95.6	41.0	40.3	25.0

FIGURE A-1

Grain Size Analysis

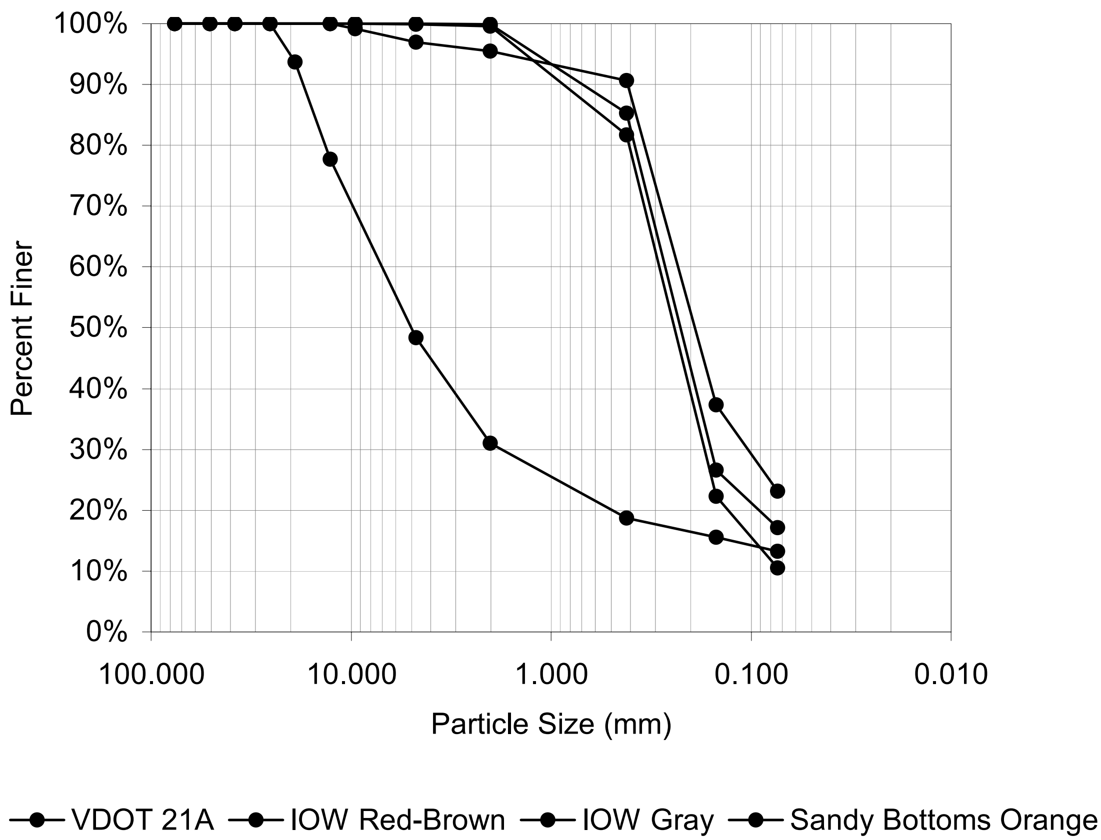


FIGURE A-2
Moisture-Density Relationship

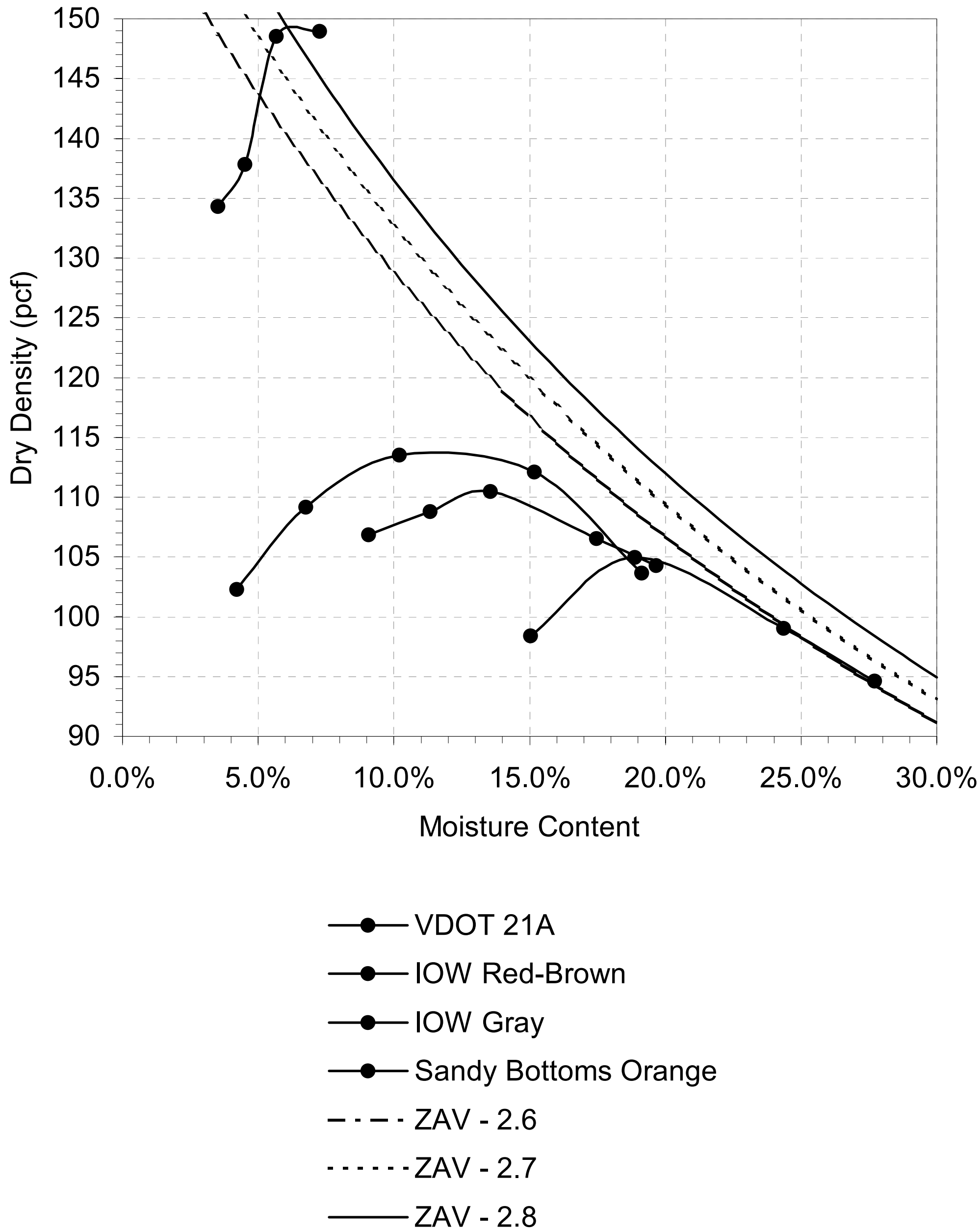
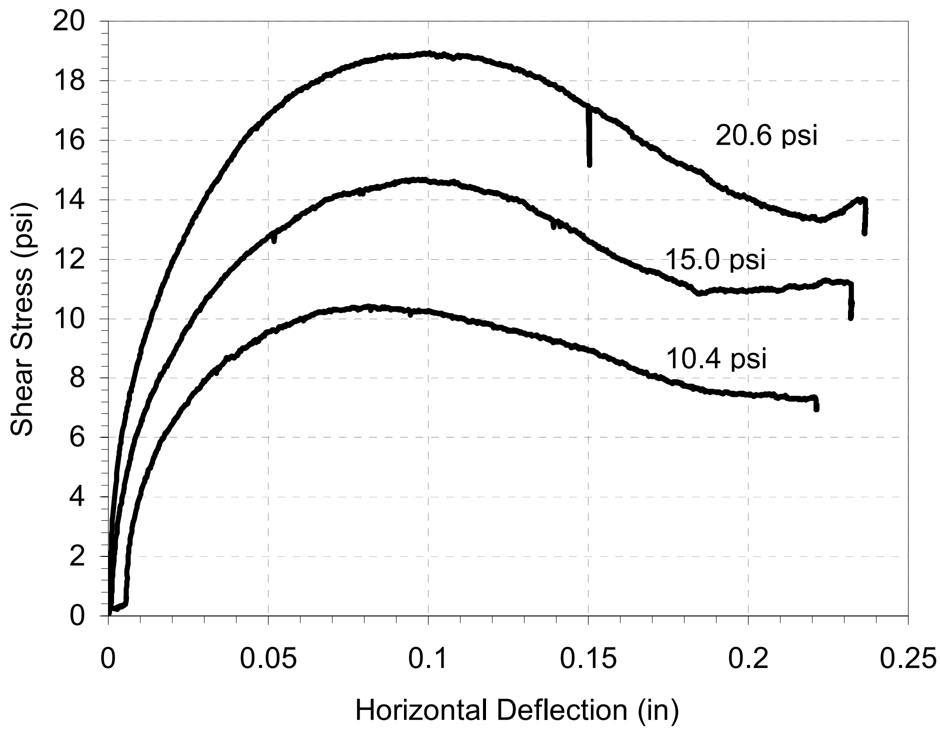


FIGURE A-3
Direct Shear Test Results – 2.5-inch Diameter Shear Ring
Isle of Wight Materials Co., Inc. – Gray Silty SAND (SM) – 17% Fines



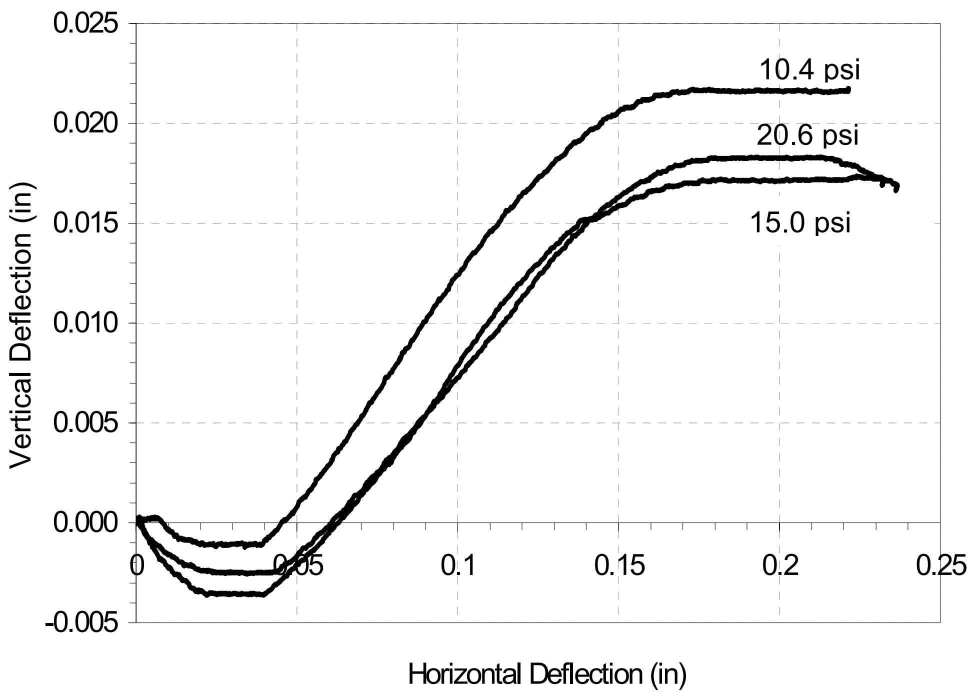


FIGURE A-4
Direct Shear Test Results - 2.5 inch Diameter Shear Ring
Mohr Strength Envelope
Isle of Wight Materials - Gray Silty SAND - 17 % Fines

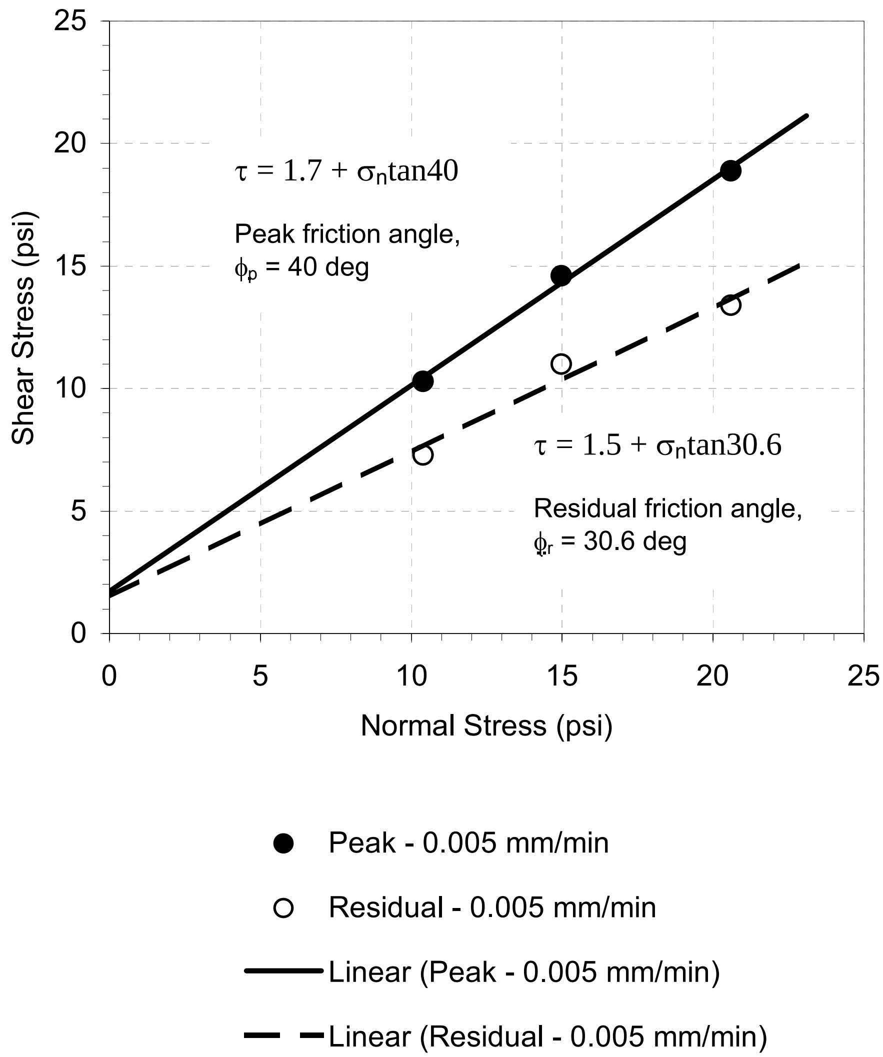
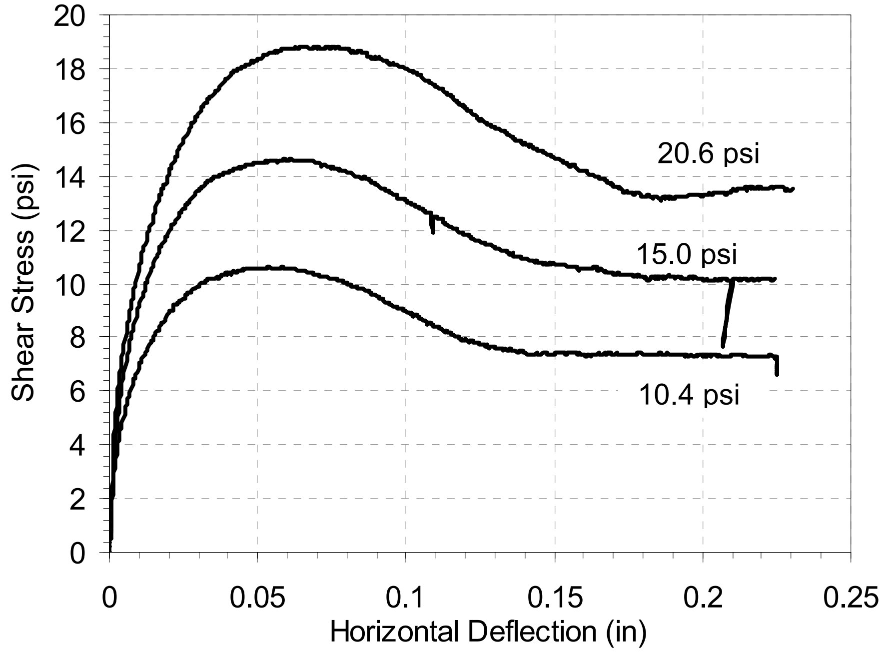


FIGURE A-5
Direct Shear Test Results – 2.5-inch Diameter Shear Ring
Sandy Bottoms – Orange Silty SAND (SM) – 10% Fines



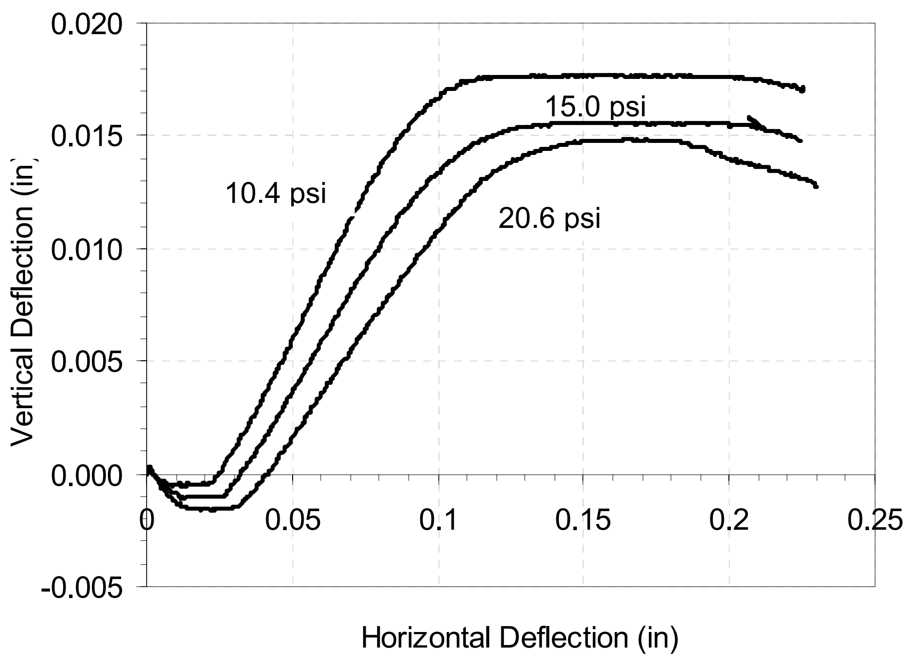


FIGURE A-6
Direct Shear Test Results - 2.5 inch Diameter Shear Ring
Mohr Strength Envelope
Sandy Bottoms – Orange Silty SAND (SM) – 10% Fines

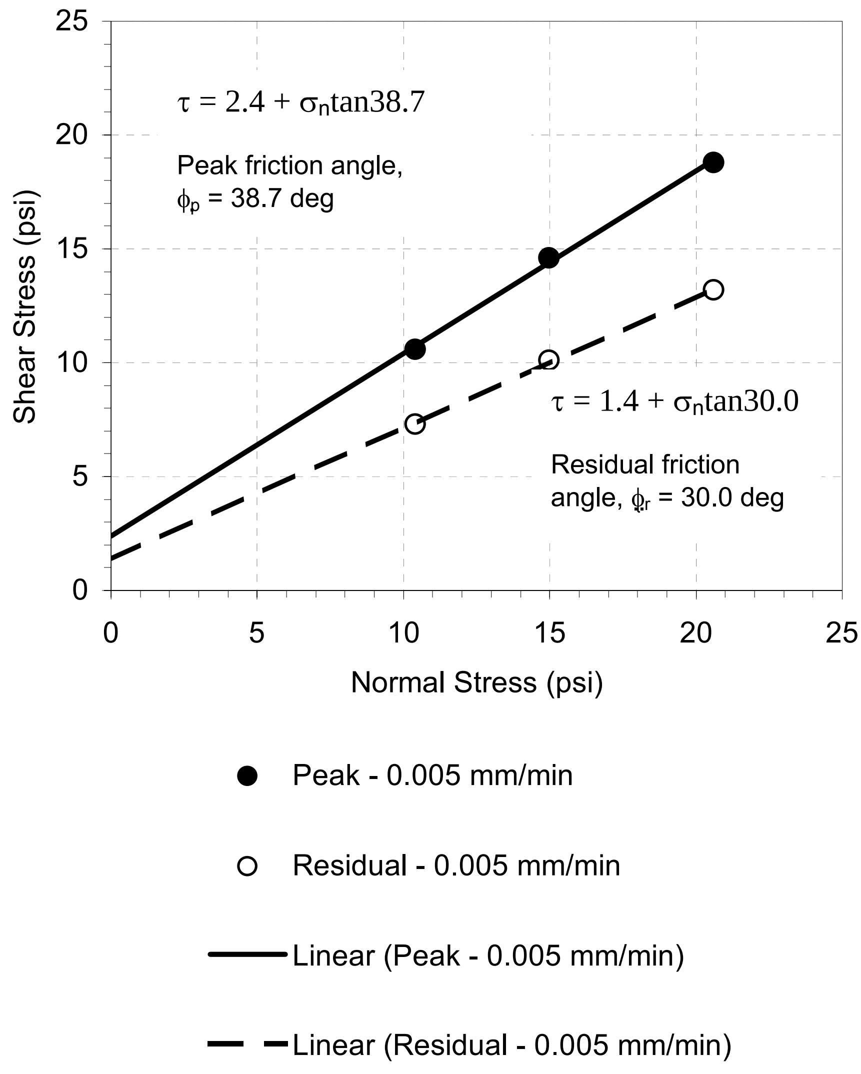
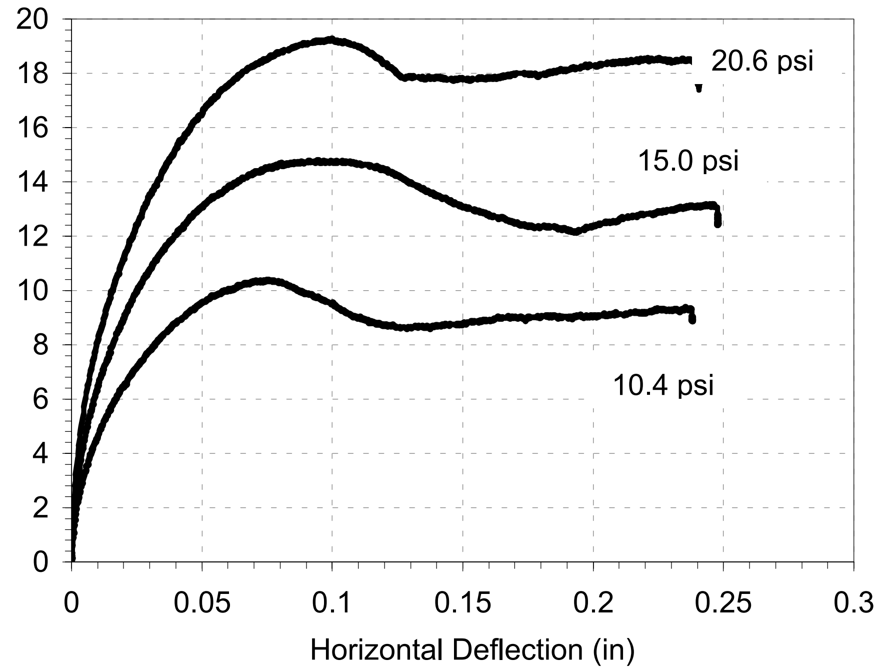


FIGURE A-7

Direct Shear Test Results – 2.5-inch Diameter Shear Ring

Isle of Wight Materials Co., Inc. – Red Brown Silty SAND (SM) – 24% Fines



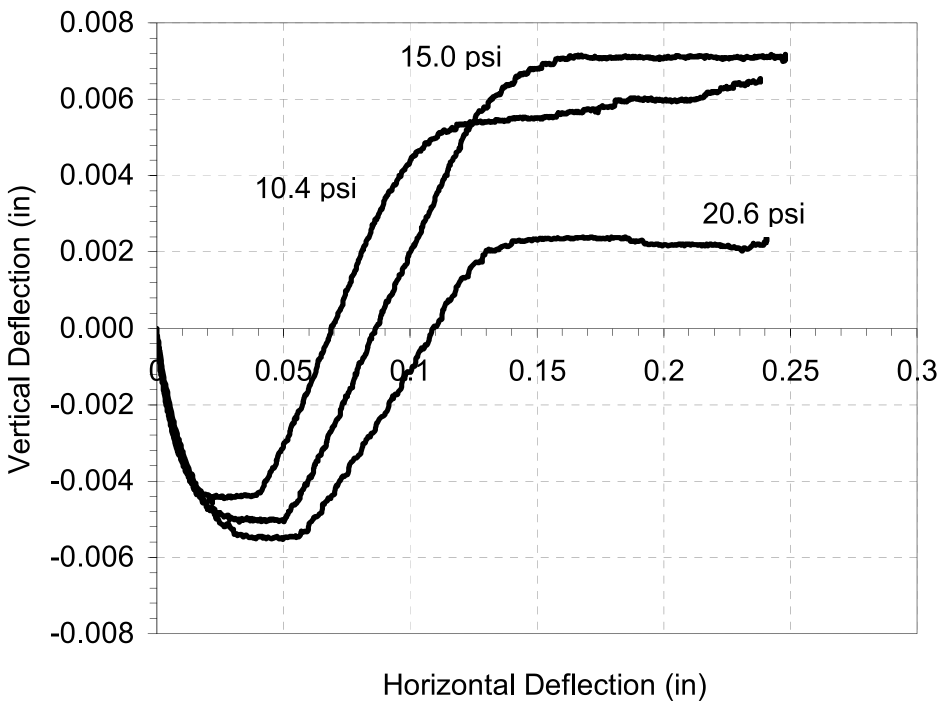
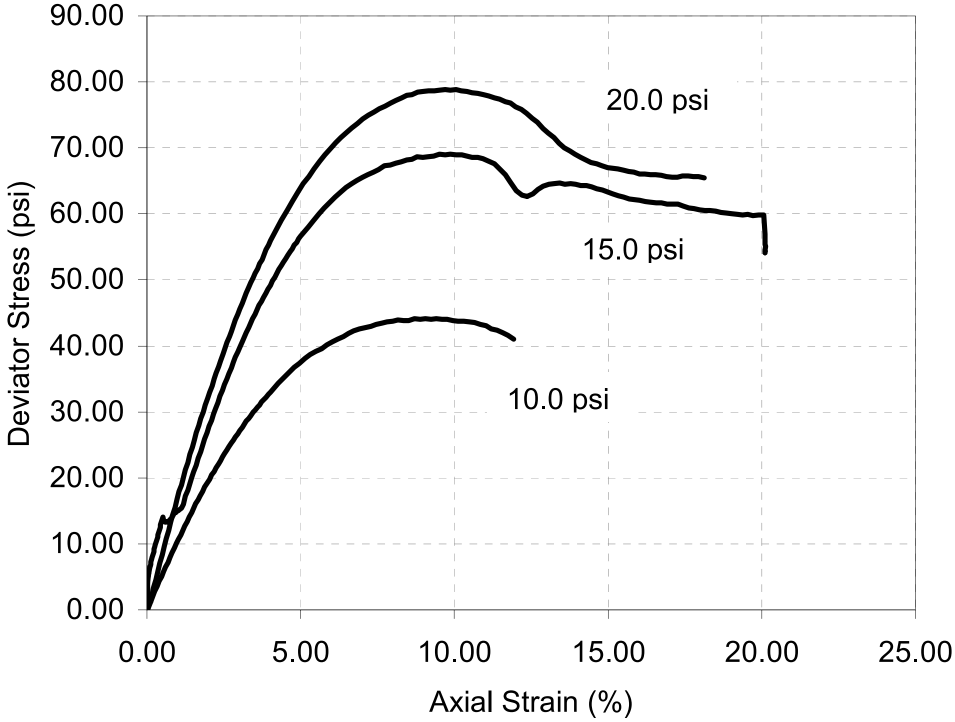


FIGURE A-8
ICD Triaxial Test Results - 1.4 inch Diameter Specimen
Isle of Wight Materials Co. - Red Brown Silty SAND (SM) - 24% Fines



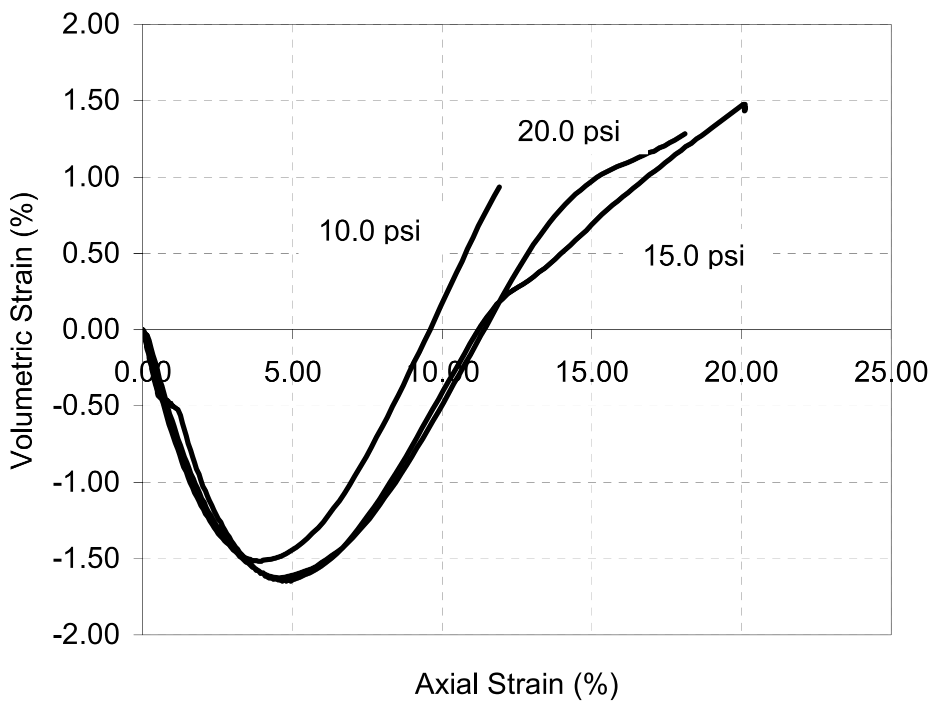


FIGURE A-9
Direct Shear Test Results - 2.5 inch Diameter Shear Ring
Mohr Strength Envelope
Isle of Wight Materials Co. - Red Brown Silty SAND (SM) - 24% Fines

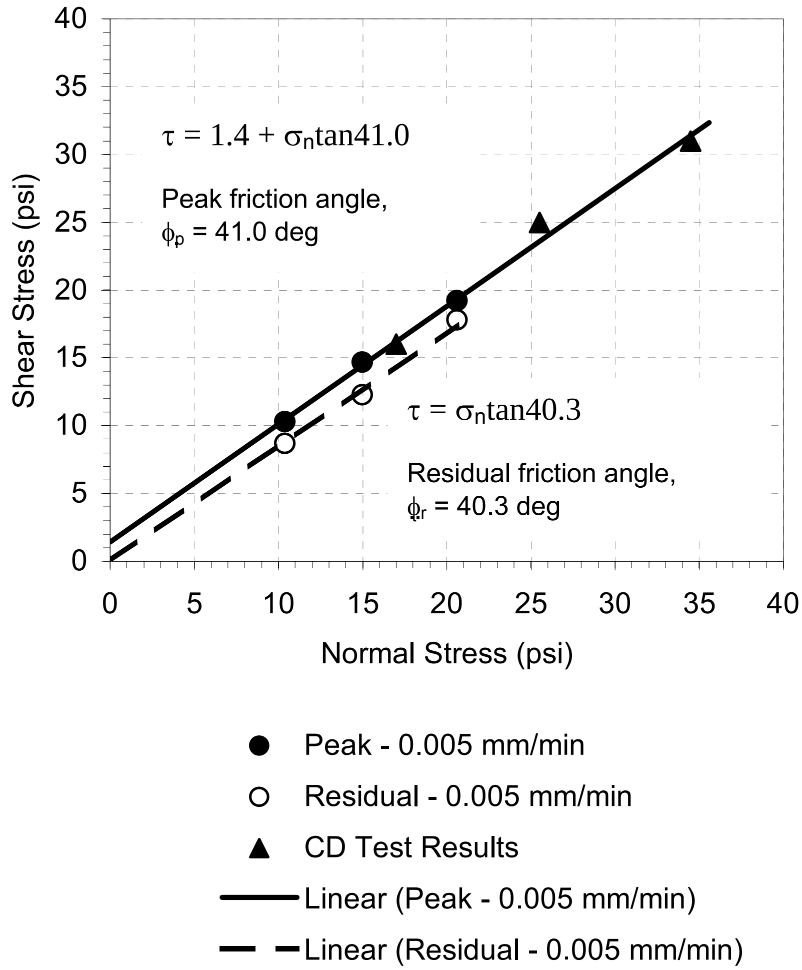
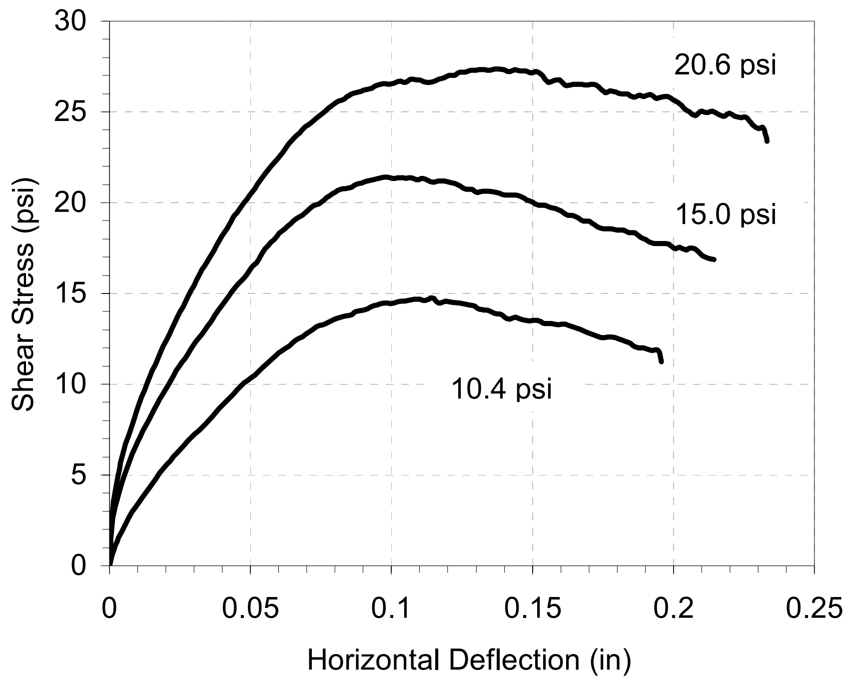


FIGURE A-10
Direct Shear Test Results – 2.5-inch Diameter Shear Ring
Sisson & Ryan Quarries – VDOT No. 21A Stone – 13% Fines



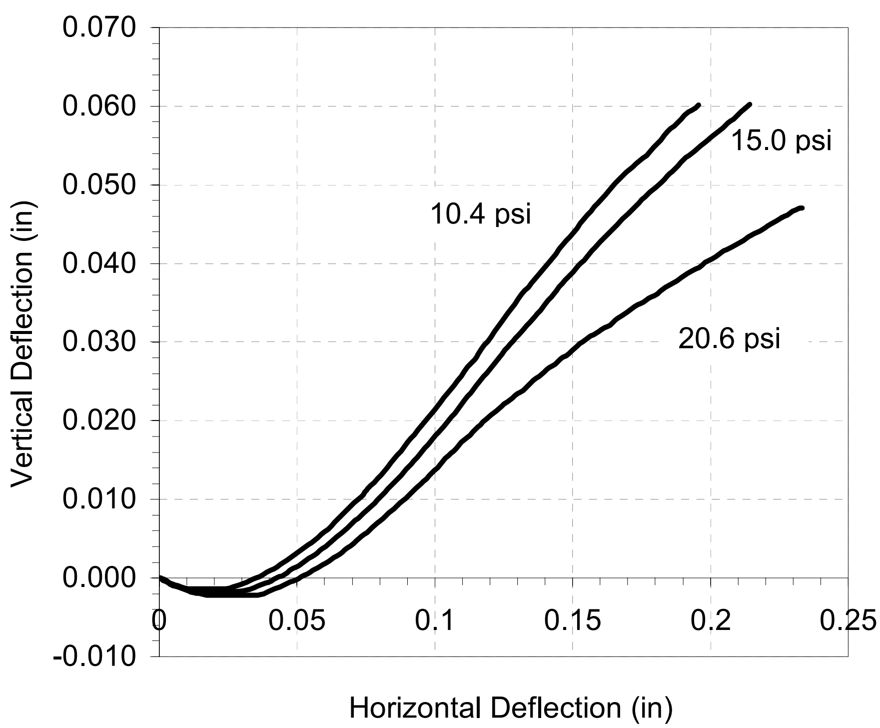


FIGURE A-11
Direct Shear Test Results - 2.5 inch Diameter Shear Ring
Mohr Strength Envelope
Sisson & Ryan Quarries – VDOT No. 21A Stone – 13% Fines