CHAPTER 4

MATERIALS TESTING PROGRAM METHODS AND ASSUMPTIONS

This chapter provides information to use when conducting or reviewing testing results that will be used in geotechnical and stability analyses for a *waste containment facility* in Ohio. It also addresses selecting appropriate test results for materials and interfaces that will be used for design or construction.

At a minimum, testing of in situ soil materials must occur during the subsurface investigation when preparing to design a *waste containment facility*. Testing of soil materials that will be used for structural fill, recompacted soil layers, and other engineered components can be conducted during the subsurface investigation (recommended) or as *conformance testing* before construction. Testing of the interface shear strengths of geosynthetics and the internal shear strengths of geosynthetic clay liners (GCL), is likely to occur as *conformance testing*. This is due to frequent changes in geosynthetic materials on the market and the time between design and construction. However, designers may want to evaluate their designs against appropriate test results for typical materials that are available. This will allow the designer to evaluate the likelihood that appropriate materials will be available when needed.

It is expected that the appropriate ASTM test methods or other applicable standards will be followed whenever testing of materials is being performed. When using approved test methods, ensure the testing apparatuses and the *specimens* are prepared and used so that the test results are appropriately conservative in representing the field conditions in which the soils and geosynthetics will be used. Common tests used during geotechnical investigations addressed in this chapter are:

For soils;

- Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions (ASTM D 3080),
- Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils (ASTM D 2850),
- Standard Test Method for Unconfined Compressive Strength of Cohesive Soil (ASTM D 2166),
- Standard Test Method for Consolidated-Undrained Triaxial Compression Test for Cohesive Soils (ASTM D 4767), and
- Standard Test Method for One-Dimensional Consolidation Properties of Soils (ASTM D 2435).

For interface testing;

- Standard Test Method for Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method (ASTM D 5321), and

- Standard Test Method for Determining the Internal and Interface Shear Resistance of Geosynthetic Clay Liner by the Direct Shear Method (ASTM D 6243).

GENERAL CRITERIA FOR MODELING SITE CONDITIONS WHEN PREPARING SAMPLES AND RUNNING TESTS

In 1974, Ladd stated, "The results of research have shown that major variations in strength can be caused by *sample* disturbance, strength anisotropy, and strain-rate effects. None of these effects is explicitly included in present design practice. The reason the present methods generally work is that the variations frequently tend to be self-compensating. It is therefore quite possible for the resulting design to be either unsafe or overly conservative, particularly in view of the large scatter often found in triaxial test data." Additional research since then has continued to confirm these findings (e.g., Jamiolkowski, et al, 1985).

Failure planes propagate through the materials and interfaces that exhibit the weakest shear strength at any given loading. The materials and interfaces that are the weakest are likely to change as the normal load and displacement changes. As a result, failure planes may propagate through several different interfaces and materials. At many *waste containment facilities*, a large array of materials and combinations of materials often exist under varying normal loads that need to be evaluated for shear strength. Furthermore, *waste containment facilities* can have widely varying site conditions that may affect the applicability and/or validity of testing results, and the site conditions are likely to change over time. Because of these variables, it is extremely important to ensure that *samples* of soil and construction materials are prepared and tested so that they conservatively represent the expected worst-case field conditions for each facility-specific design.

Factors Affecting the Validity and Accuracy of Soil Shear Strength Testing

The commonly used unconfined compression tests and unconsolidated-undrained triaxial compression tests tend to produce values of *undrained shear strengths* that exceed field values because of the triaxial compression stress condition and the high strain rate used (60%/hr). However, *sample* disturbance, on the other hand, tends to cause lower values of *undrained shear strength* provided that drying of the *sample* is avoided. These effects may compensate each other and yield a reasonable average design shear strength. However, the method is highly empirical and these compensating factors are not controlled or controllable, but in practice, the disturbance effects can be greater than the testing effects and thus the resulting *undrained shear strengths* are often conservative. The situation is further confused by the tendency for *sample* disturbance effects to increase with depth and to obscure shear strength variations in the profile. *Sample* disturbance typically underestimates the *undrained shear strength* of a *sample* from 20 to 50%. Stress-strain anisotropy can cause differences between the *undrained shear strength* obtained by different tests to vary by a factor of 1.5 to 2.5. For triaxial compression tests, each log cycle decrease in strain rate is typically accompanied by a 10 to 15% decrease in *undrained shear strength*. For highly plastic, creep susceptible clays, triaxial compression strength obtained from consolidated *samples* failed at an axial strain rate of 60%/hr (typical for UU triaxial and Unconfined Compression tests) can be 1.2 to 1.3 times the shear strength obtained at 0.5%/hr (typical for CU triaxial tests w/pore water pressure measurement) (Quoted and adapted from Ladd, 1974). The variability discussed by Ladd is largely independent of the triaxial compression test conducted and thus is inherent in the variability of soil material properties and the difficulties experienced during sampling. As a result, variations in values of *undrained shear strength* are still found in testing today (Stark, 2002).

It is important to model failure surface propagation through a composite system at varying normal loads. To do this, the individual failure envelopes of each material and interface in the composite system can be plotted on one shear stress vs. normal stress graph. The weakest compound envelope (see **Figure 4-1**) can then be determined and used for calculating or verifying the stability of the composite system (see Conformance Testing starting on page 4-15 for more details).

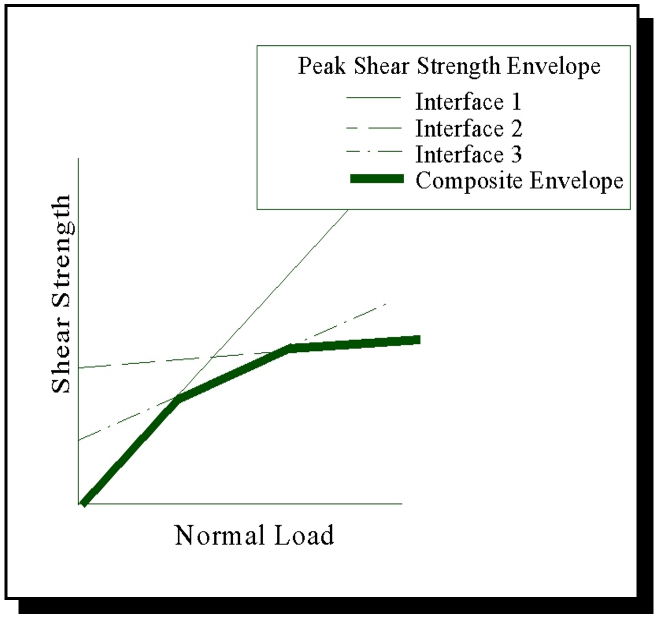

Figure 4-1 Example of a compound *peak shear strength* envelope for a multi-layered engineered component of a *waste containment facility*.

At some facilities, the shear strength of a material cannot be ascertained through laboratory testing. Using empirical relationships then becomes the only alternative. On the rare occasion that this is necessary, the theoretical or empirical correlation that produces the weakest reasonable estimate of the shear strength should be used. For example, when using correlations between liquid limit and shear strength, the highest liquid limit measured that is representative of the *soil unit* should be used to estimate the shear strength, instead of averaging a number of liquid limits from several *samples*.

In situ foundation materials and project-specific materials must be tested for internal and interface shear strengths over the entire range of normal stresses that will be encountered by the materials and interfaces for a given design. The range of normal stresses that need to be evaluated can be extensive, varying from low values at the perimeter of a facility to much higher values under the deepest areas of a facility. For cover systems, this range includes the low normal stresses caused by the cap materials and any additional stresses that may be induced by surface water diversion benches, roads, or other structures constructed above the cover system, and equipment.

If a reasonable expectation exists that at a future time the *waste containment facility* may be expanded in a manner that will increase the normal stresses associated with the facility, then the *responsible party* should ensure that materials and interfaces selected for construction are tested at the higher normal loads. Otherwise, future expansion may be precluded because it will be unknown if the existing materials can maintain stability under the higher normal loads, and the materials that were used may no longer be manufactured or otherwise available for testing.

Shear strength tests are performed by shearing different *specimens* of the same material or interface at three to five different normal loads to develop the failure envelope. For each test, at least one *specimen* should be sheared at a load that is as near as possible, or preferably below, the lowest expected normal stress that will be experienced by the material or interface in the field. One *specimen* should be sheared at a load that is at least 110 percent of the maximum normal stress expected to be experienced by the material or interface in the field. The remaining *specimens* should be sheared at normal loads well distributed between the low and high loads.

Care must be taken to prevent damage or changes to undisturbed *samples* that would invalidate test results. For example:

- Undisturbed *samples* of soil should be sealed in moisture-proof containers immediately after collection.

- During shipping, the *samples* should be protected from vibration, shock, and extreme heat or cold in accordance with ASTM D 4220, "Standard Practices for Preserving and Transporting Soil Samples."

- Preparation of undisturbed *specimens* should be conducted in an environment that will minimize the gain or loss of moisture, disturbances, and changes in cross sections.

The hydration necessary for determining the shear strength of in situ materials is dependant upon site-specific conditions. Any fine-grained material that is currently, or may become, *saturated* in the field should be tested for *undrained shear strength* in a fully *saturated* condition using the UU triaxial compression test. It is typically assumed that fine-grained in situ materials are or will be *saturated*. For rare cases when fine-grained in situ materials are not *saturated* and are unlikely to become *saturated* in the field, an effective stress analysis using *drained shear strength*s may be conducted using the CU triaxial compression test with pore water pressure measurements and the appropriate site-specific range of normal loads.

> "...the shear strength of a given soil is also dependent upon the degree of saturation, which may vary with time in the field. Because of the difficulties encountered in assessing test data from *unsaturated samples*, it is recommended that laboratory test samples be saturated prior to shearing in order to measure the minimum shear strengths. Unsaturated samples should only be tested when it is possible to simulate in the laboratory the exact field saturation (that is matric suction) and loading conditions relevant to the design." (Abramson, et al, pp 270)

The procedures specified in each test method must be followed closely. Other procedures such as setting the rate of the shear stress and the amount of confining stress should be selected carefully to mimic field conditions as much as possible and to avoid obtaining questionable results.

REPORTING

The results of all materials testing completed during the design of the *waste containment facility* should be included in the subsurface investigation report. The subsurface investigation report is described in Chapter 3. At a minimum, the following information about materials testing results should be reported to Ohio EPA whenever it is conducted:

- A narrative and tabular summary of the scope, extent, and findings of the materials testing,

- A description of collection and transport procedures for *samples*,

> In addition to the other items included in this chapter, when reporting the results of *conformance testing*, include a comparison of the test results with the requirements contained in rule, the authorizing document, and the assumptions used in the geotechnical and stability analyses, whichever is applicable.

- The test setup parameters and protocols for each test,
- The *specimen* preparation and pre-test characterization used in each test,
- The intermediate data created during each test,
- The results of each test, and
- Any figures, drawings, or references relied upon during the analysis marked to show how they relate to the facility.

STANDARD TEST METHOD FOR DIRECT SHEAR TEST OF SOILS UNDER CONSOLIDATED DRAINED CONDITIONS (ASTM D 3080)

Recommended Uses

The test results from this method are used to assess the shear strength of the material in a field situation where consolidation has occurred under existing normal stresses and no excess pore water pressure is expected to develop during construction or placement of loads on the material. Examples of components that may be tested using this method are granular drainage layers and soils that will be used for structural fill.

> The direct shear device consists of two metal boxes, or "frames," oriented so their open sides face each other. A *specimen* is placed in the direct shear device and consolidated using a normal compressive load representative of field loading conditions. Then one frame is displaced horizontally while the other frame remains at rest. The displacement must be at a constant rate resulting in the ability to measure the shearing force and horizontal displacements during the shearing process.

This test is not usually used when trying to determine the *drained shear strength* of fine-grained cohesive soils, such as in situ foundation soils or recompacted soil liners. Several reasons for this are:

- The consolidation of the *specimen* and the shear rate during testing must be performed very slowly for these types of materials to ensure that the soil *specimen* remains in a *drained condition* during the test. This makes the test inconvenient and often expensive for testing fine-grained cohesive soils.
- The results of this test may not be applicable to fine-grained cohesive in situ foundation soils and recompacted soil layers that will be subjected to high normal loads after they are constructed. This is because the loading experienced by these layers during construction and operations can cause excess pore water pressure to develop.
- During the test, a rotation of principal stresses occurs that may not model field conditions.
- The weakest failure plane through the material may not be identified because the test forces the failure plane to be horizontal through the middle of the *specimen*.

Ohio EPA recommends using triaxial compression testing methods for determining the *drained* and *undrained shear strengths* of fine-grained cohesive soils.

The testing must be continued until a *residual shear strength* is determined or can be conservatively estimated. For slopes that will be permanently loaded with less than 1,440 psf (i.e., final cap), determining the *residual shear strength* may not be necessary. However, it should be carefully considered whether knowing the *residual shear strength* of such a slope will be needed in the future and if it is appropriate for use in current design analysis.

Residual shear strength should be achieved or able to be conservatively estimated once the full displacement of the direct shear device has occurred. As an alternative, especially for designs where it is critical to know the *residual shear strength* of a material, the shear device can be repeatedly returned to zero displacement without disturbing the *specimen*, and the *specimen* can be sheared again at the same normal load. Another alternative is to use a torsion ring shear device to determine *residual shear strength* for soils and many types of interfaces (Stark and Poeppel, 1994).

Data Validation

Numerous parameters exist that can be checked to verify that the test was performed correctly resulting in valid data. Some of these parameters are:

- Adherence to the maximum particle size restrictions of this method. If these size restrictions are not used, then the ASTM method requires that the grain size distribution of the *specimen* be reported with the shear test results.

- Remolded *specimens* may be adequate to assess the shear strength of structural fill and recompacted soil materials. However, to ensure that the results are applicable to the design or construction of the facility, the materials should be remolded to represent the lowest density and highest moisture content specified during construction, and materials should be chosen from the soils expected to exhibit the lowest shear strengths at those specifications.

Exceeding the maximum grain size restrictions of the method may result in erratic and inaccurate test results, due to interference with shear plane development and scale effects created by shearing the larger particles. (ASTM D 3080)

STANDARD TEST METHOD FOR UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST ON COHESIVE SOILS (ASTM D 2850)

Recommended Uses

This test is used to determine the *undrained shear strength* of soil. It is applicable to situations where fine-grained soils will be in a *saturated* condition and loading is expected to take place at a rate that overwhelms the ability of the soil materials to dissipate excess pore water pressure.

If *specimens* are *saturated* at the beginning of this test, it is unlikely that consolidation will take place because the drainage lines are closed, allowing the *undrained shear strength* to be determined. The *undrained shear strength* of several similar *specimens* will be approximately the same at different normal loads, resulting in an

During a triaxial compression test, a cylindrical *specimen* that is wrapped in a membrane is placed into the triaxial chamber, which consists of a top and bottom plate with a stiff walled cylinder in between. A confining pressure using fluid and air is created within the traxial chamber. The *specimen* is then subjected to an axial load until the *specimen* fails. No drainage is allowed to occur during the test.

internal angle of friction of zero. This shear strength measurement should be representative of field conditions that exist when a fine-grained soil material is experiencing excess pore water pressure. Ohio EPA recommends the use of this test when fine-grained soils exist at a facility that are or may become *saturated*.

If *specimens* are partially *saturated* at the beginning of this test, compaction (densification by expelling air) will occur before shearing. The shear strength exhibited by the *specimen* will be different at different normal loads, resulting in an angle of friction greater than zero. The shear strength exhibited by the *specimen* will be applicable only when the soils represented by the *specimen* exist in the field at the same saturation as the *specimen* and are subjected to the same range of normal loads as those used in the test. This is unlikely to occur at most facilities that have in situ fine-grained soils in their foundation. For example, a fine-grained soil *sample* collected in August may have a saturation of 75 percent and exhibit a higher shear strength than the same *sample* if it were collected in April, when it may have a much higher level of saturation. Partially *saturated specimens* should not be used for determining the shear strength of in situ foundation soils using the UU triaxial compression test. This is because the conditions represented by the partially *saturated specimen* are unlikely to represent worst-case conditions that are reasonably expected to occur.

> *Undrained shear strength* testing is appropriate when the field conditions are such that the loading rate allows insufficient time for induced pore water pressures to dissipate, reducing the shear strength of the materials. Accepted practice is to assume in situ clay materials will be *saturated* for the purposes of shear strength testing, unless site investigation provides a conclusive determination that they are not currently *saturated* and will not become *saturated* at any point during construction, operations, or closure of the *waste containment facility*.

Data Validation

A comparison of the pretest density and moisture content vs. the post-test density and moisture content should show that little or no change has occurred, and thus the *specimen* was *saturated* at the start of testing.

It is expected that any given *specimen* of soil will exhibit a similar *undrained shear strength* despite the normal stress used during the test. However, due to variability in the accuracy and precision of the test procedure, Ohio EPA recommends multiple *specimens* of the same soil be sheared at different normal loads as confirmation.

STANDARD TEST METHOD FOR UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOIL (ASTM D 2166)

Recommended Uses

This test is used to obtain a rapid approximation of the *undrained shear strength* for *saturated* fine-grained cohesive soils. It can be conducted on undisturbed, remolded, or compacted *specimens*.

> This test is run by placing a trimmed *specimen* of soil between two platens. The *specimen* is not wrapped or confined in any way. The loading platen is lowered at a constant speed until the *specimen* shears. Both the displacement and the shear force are recorded.

This test is not appropriate for dry or cohesionless soils. If this test is used, the saturation of each *specimen* before beginning the test must be reported.

If the *specimen* is completely *saturated* at the beginning of the test, the results approximate *undrained shear strength* of the *specimen*. If the *specimen* is only partially *saturated*, then the results approximate the total stress analysis, similar to conducting a UU Triaxial Compression test on a partially *saturated specimen*.

ASTM D 2166 is not a substitute for ASTM D 2850. Ohio EPA recommends ASTM D 2850 be used to develop more definitive data regarding *undrained shear strength* of cohesive soils. Because of the speed, low cost, and potential inaccuracy of ASTM D 2166, Ohio EPA recommends using this test as a screening test to identify weak soil layers that should then have *specimens* tested using ASTM D 2850. ASTM D 2166 results can also be used to augment the understanding of the shear strength of cohesive soils at a facility in conjunction with the results of ASTM D 2850. To do this, the soil *specimen* must be *saturated* and a confining membrane should be used around the *specimen*. ASTM D 2850 includes testing at least three *specimens* from each *sample*, thus producing at least three data points at three different normal stresses. ASTM D 2166 involves testing only one *specimen* from each *sample*. As a result, ASTM D 2166 would need to be run three times for each *sample* under the preceding conditions to produce the same number of data points as one test run in accordance with ASTM D 2850.

Data Validation

The saturation level of each *specimen* needs to be known to determine whether the results are approximating *undrained shear strength* or total stress analysis.

No water should be expelled from the *specimen* during trimming or compression. If this occurs, the material must be tested using the UU triaxial compression test.

> The expulsion of water from the *specimen* during compression indicates that consolidation of the *specimen* is occurring. The consolidation will increase the apparent shear strength of the *specimen*, rendering the test results unusable for undrained analyses.

Dry and crumbly soils, fissured or varved materials, silts, peats, and sands cannot be tested with this method.

Multiple tests should be conducted for confirmation of the results.

> A *sample* of in situ fine-grained soil has been subjected to overburden stresses from overlying soils and possibly other geologic occurrences prior to retrieving it from the field. When a *sample* is retrieved, the overburden stress is relieved, and the *sample* may also be disturbed. To increase the representativeness of the shear strength obtained from the CU triaxial test, it is important that a *specimen* is sheared under conditions that mimic, as closely as possible, the in situ stresses.

STANDARD TEST METHOD FOR CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST (ASTM D 4767)

Recommended Uses

This test is used to determine the total stress, effective stress, and axial compression of cohesive soils by measuring axial load, axial deformation, and pore-water pressure. This test is to be conducted using *undrained conditions*, while measuring pore water pressure to determine the *drained shear strength* of the *specimen*. The test is applicable to field conditions where soils have been consolidated and are subjected to a change in stress without time for consolidation to recur. To ensure that the test results are applicable to the design of the facility, the test should be run using stress conditions that are similar to the expected worst-case field conditions for the facility. Ohio EPA recommends the use of this test whenever in situ or compacted materials are partially *saturated* and conclusive data shows that it is unlikely that excess pore water pressures will occur during construction of the facility. Ohio EPA also recommends using this test when stability of the *waste containment facility* is being analyzed for the point in time when the pore water pressure in the materials has dissipated (e.g., a staged loading sequence, the point in time after the maximum mass of the facility has been placed and the pore water pressure has dissipated).

Data Validation

For the test results to be meaningful, the over consolidation ratio (OCR) of the *specimen* that existed at the beginning of the test must be known. To accomplish this, the *specimen* must be reconsolidated back to its virgin compression line. For *specimens* that were normally consolidated in situ, the OCR is equal to unity. Therefore, the *specimen* can be sheared after consolidation back to an effective stress greater than that experienced in situ. For *specimens* of overconsolidated in situ materials, the in situ OCR must be calculated from the results of *higher quality data* such as those obtained from oedometer tests. The *specimen* must be reconsolidated back to the virgin consolidation line, and then the effective stress should be reduced to bring the *specimen* back to the in situ OCR. Once the OCR of the *specimen* in the test apparatus matches that of the *sample* in situ, shearing can take place.

The stress history of each *sample* must be carefully investigated to determine how much consolidation must occur to get the *specimen* to return to its virgin compression curve. Usually, *specimens* will need to be consolidated between 1.5 and 4 times the in situ overburden pressure before shearing. For *samples* that were overconsolidated in situ, the apparatus stresses are then reduced so that the OCR in the apparatus is equal to the in situ OCR. The apparatus is set to the normal stress applicable to the design of the facility and to record pore water pressure measurements. The *specimen* is then sheared at a recommended rate of 0.5 percent to 1 percent axial strain/hr.

> Shear testing of quick clays and naturally cemented clays are unlikely to exhibit normalized behavior because the structure of the soil is significantly altered during consolidation to higher stresses. Varved clays may also create difficulties in properly estimating shear strength due to the anisotropy of the soil (Ladd & Foott, 1974). For soils such as these, several different types of shear tests may be necessary, including the direct shear test, to determine the weakest shear plane.

STANDARD TEST METHOD FOR ONE-DIMENSIONAL CONSOLIDATION PROPERTIES OF SOILS (ASTM D 2435)

> The test apparatus consists of a cylindrical dish that contains the *specimen*. A piston is pushed into the dish under a load to compress the *specimen*. The apparatus allows drainage from the *specimen* as it is being consolidated. The displacement is measured during the test.

Recommended Uses

The consolidation (oedometer) test is used to determine the rate of primary compression and *secondary compression* of a soil. This test will provide the effective stress-void ratio (log σ'-e curve), the swelling index (C_s), the compression index (C_c), the preconsolidation pressure (σ_p'), the variation of the consolidation coefficient (C_v) vs. effective stress (σ'), and the *secondary compression* coefficient (C_α). The compressibility (M_v), the permeability coefficient (k)[a] , void ratio vs. effective stress plots, the average degree of consolidation as a function of the time factor [$U(T_v)$] vs. square root of time plots, the void ratio vs. log pressure plots, and the dial reading vs. log time curves should also be reported. The results of this test can be used to evaluate the settlement that is likely to occur under the design loads of a *waste containment facility*.

Data Validation

The test method assumes the following:

- The *specimen* is *saturated* and has isotropic properties (i.e., the *specimen* tested must be representative. The more variation encountered in a *soil unit*, the more *samples* that will need to be tested),

- The compressibility of soil particles and pore water is negligible compared to the compressibility of the soil skeleton,

- The stress-strain relationship is linear throughout the load increment,

- The ratio of soil permeability to soil compressibility is constant throughout the load increment, and

- Darcy's law for flow through porous media applies.

- The void ratio vs log time plot can be used to ensure that the consolidation made a transition from primary to secondary before the next load was added. If no transition is visible in the curve, then check with the lab to find out why subsequent loading was done before the transition into secondary consolidation of the *specimen* had occurred.

- The void ratio vs. log pressure plot can be used to ensure that the void ratio decreased with each new load. If it does not, then this indicates a problem with the test.

If the above assumptions do not apply to the *specimen*, then this test method may not be appropriate for the selected *specimen*.

The test results are strongly affected by the saturation of the *specimen*. Fully *saturated specimens* must be used. The pre-test saturation level of each *specimen* must be determined and reported.

If more than one *compressible layer* exists at a facility, each layer should be tested to be able to calculate the *differential* and *total settlement* for the facility properly. In addition, enough *samples* from each *compressible layer* should be tested to be able to identify lateral and vertical differences in consolidation

[a] Bardet, J., 1997, Experimental Soil Mechanics. Prentice-Hall, New Jersey. pp. 350.

and compressibility parameters. For example, if the facility has a lower glacial till that is partly overlain by an upper lacustrine deposit, both layers should be tested to obtain an understanding of the lateral and vertical variability of their respective consolidation/compressibility parameters.

The range of the applied stress during the test should cover from the lowest to the highest normal stresses expected to be exerted by the facility.

During testing, the load should be changed after the consolidation caused by the current load reaches 100 percent. However, the load may be changed at convenient times if consolidation exceeds 90 percent for the current load. Generally, each load is in place for 24 hours. For some soils, more than 24 hours under each load may be necessary to allow complete consolidation to occur.

To be able to calculate *secondary settlement,* the load should be maintained at each stage for as long as necessary to determine the *secondary compression* coefficient.

> Obtaining the coefficient of *secondary compression* through testing is only necessary for plastic materials. Published literature can be used to estimate *secondary compression* coefficients for non-plastic materials if they are appropriately representative of the non-plastic materials found at the site.

If excavations are to occur during the construction of the facility that will be filled later with water, waste, or other materials; or if the facility will be filled and then cut during construction or operations; one or more rebound cycles will be created within the foundation soils. A description of the loading that identifies the rebound cycles should be evaluated and communicated to the lab. This is so the loads representing the cutting and filling can be included in the testing.

Test results are affected by *sample* disturbance, affecting the preconsolidation pressure most significantly. The *specimen* selection and preparation methods should not disturb the *specimen* any more than is absolutely necessary when collecting and preparing the *specimen* for testing.

STANDARD TEST METHOD FOR DETERMINING THE COEFFICIENT OF SOIL AND GEOSYNTHETIC OR GEOSYNTHETIC AND GEOSYNTHETIC FRICTION BY THE DIRECT SHEAR METHOD (ASTM D 5321)

Recommended Use

> The test is usually run within a "large box" direct shear apparatus. A constant normal stress is applied to the *specimen* while a shear force is applied to the apparatus.

This test is used to determine the shear resistance of a geosynthetic against soil or another geosynthetic. Using site-specific geosynthetic material and remolded or undisturbed *specimens* of soils from the *waste containment facility* is important. Ohio EPA recommends using this test for determining the *peak shear strengths* and *residual shear strengths* for all interfaces with a geosynthetic that are part of the facility design. However, this test should not be used when testing GCL. Instead, use ASTM D 6243 when testing internal or interface shear strength of a GCL. Sometimes, Ohio EPA may require composite systems containing multiple geosynthetic interfaces to be tested to determine which interface or material will be the locus of the failure surface throughout the range of normal stresses expected in the field. This may entail using a direct shear device or other appropriate device to test *specimens* comprising all the layers in a composite system. For example, if all of the *peak shear strengths* for each interface and material are near each other, but a wide range of *residual shear strengths* exists, either the lowest *residual shear strength* measured will need to be used, or *specimens* comprising all the layers in a composite system will need to be tested.

Residual shear strength should be determined or able to be conservatively estimated once the full displacement of the direct shear device has occurred. As an alternative, especially for designs where it is critical to know the *residual shear strength* of a material, the shear device can be repeatedly returned to zero displacement without disturbing the *specimen*, and the *specimen* can be sheared again at the same normal load. Another alternative is to use a torsion ring shear device to determine *residual shear strengths* for many types of interfaces. (Stark and Poeppel, 1994)

The test must be continued until the *residual shear strength* has been determined or can be conservatively estimated. Sometimes, such as for geosynthetics with maximum permanent loads less than 1,440 psf (e.g., final cap systems), determining the *residual shear strength* may not be necessary. However, even here, it should be carefully considered whether knowing the *residual shear strength* of the interfaces is necessary for current or future design needs.

Data Validation

Conformance testing of the internal and interface shear strengths of construction materials must be conducted prior to use to verify that they will provide the shear strengths necessary to meet the stability requirements of the design. Interface testing is often not performed during design testing, but is performed during *conformance testing* due to the length of time from design to construction and the changes that may occur in materials that are available. However, at a minimum, designers should review published literature pertaining to the materials anticipated for use in construction to ensure that it is likely that they can meet the minimum required design shear strengths. If no literature exists, then it is recommended that testing occur during the design phase of a project.

To ensure the appropriateness of this test, it must be set up to represent the expected worst-case field conditions. When testing interfaces between geosynthetics and soils, careful consideration should be given to the following:

- Soils used during the test should be recompacted using the highest moisture content and lowest density specified during construction.

- The soil selected should represent soils with the lowest internal shear strength of the soils that will be placed during construction.

Shear strength tests of interfaces with a geomembrane should be conducted fully wetted. This is performed by following the ASTM recommendation for submerging the soil *specimen* before shearing or using a spray bottle to wet the interface thoroughly.

Interfaces with the top of a flexible membrane liner (FML) become wetted in the field either from precipitation or from the liquids contained by the unit. Interfaces with the bottom of an FML become wetted in the field from condensation and from consolidation water.

Samples of geosynthetics used for testing interface shear strength should be selected from the geosynthetic rolls that will be used at the facility or from rolls that represent the materials that will be used at the facility. Materials are considered representative if they are from the same manufacturer, use the same raw materials, use the same manufacturing process, and have the same manufacturing specifications.

STANDARD TEST METHOD FOR DETERMINING THE INTERNAL AND INTERFACE SHEAR RESISTANCE OF GEOSYNTHETIC CLAY LINER BY THE DIRECT SHEAR METHOD (ASTM D 6243)

Recommended Use

The test is usually run within a "large box" direct shear apparatus. A constant normal stress is applied to the *specimen* while a shear force is applied to the apparatus.

This test is used to determine the shear resistance of a GCL against soil or a geosynthetic. It is also used to determine the internal shear strength of a GCL. Site-specific GCL, geosynthetic materials, and undisturbed *specimens* of soils or *specimens* of soils from the facility remolded using construction specifications and then hydrated to mimic field conditions must be used. Ohio EPA recommends using this test for determining the *peak shear strengths* and *residual shear strengths* of interfaces with GCL, and for determining the internal *peak shear strength* and *residual shear strength* of a GCL.

The test must be continued until *residual shear strength* has been determined or can be conservatively estimated.

Data Validation

Residual shear strength should be determined or able to be conservatively estimated once the full displacement of the direct shear device has occurred. As an alternative, especially for designs where it is critical to know the *residual shear strength* of a material, the shear device can be repeatedly returned to zero displacement without disturbing the *specimen*, and the *specimen* can be sheared again at the same normal load. Another alternative method, such as torsion ring shear, can also be considered for determining *residual shear strengths*. (Stark and Poeppel, 1994)

The test must be set up and performed to represent the expected worst-case field conditions that will be experienced by the GCL. When testing GCL internal or interface shear strength, careful consideration should be given to the following:

- The soil selected should represent soils with the lowest internal shear strength of the soils that the GCL will be placed in contact with during construction and should be recompacted using the highest moisture content and lowest density specified during construction.

- *Samples* of geosynthetics that will create interfaces with the GCL should be selected from rolls of materials that are representative of the materials that will be used at the facility. Materials are considered representative if they are from actual rolls that will be used during construction. They are also considered representative *samples* if they are collected from rolls that are from the same manufacturer, use the same raw materials, use the same manufacturing process, have the same manufacturing specifications, and are selected from rolls that will create the weakest interfaces.

- *Samples* of GCLs should be selected from rolls of materials that are representative of the materials that will be used at the facility. Materials are considered representative if they are from actual rolls that will be used during construction. They are also considered representative *samples* if they are collected from rolls that are from the same manufacturer, use the same raw materials, use the same manufacturing process, have the same manufacturing specifications, and are selected from rolls that will create the weakest interfaces or the weakest internal shear strength. If needle punched GCL is selected for testing, the test *specimen* should have a peel strength similar to the lowest peel strength sold by the manufacturer (15 pounds with ASTM D 4632 or 2.5 ppi with ASTM D 6496 is the typical minimum average roll value accepted in the United States) or the lowest peel strength specified for use during construction at the facility. An example of this would be choosing *samples* of needle punched GCL from a roll created just before replacing the needles.

> An accelerated hydration procedure can be used to reduce the in-device time for GCL *specimens* to reach hydration time (Fox *et al.* 1998a). According to this method, a GCL *specimen* is hydrated outside of the shearing device for two days under a very low normal stress (. 1 kPa) by adding just enough water to reach the expected final hydration water content (estimated from previous tests). The *specimen* is then placed in the shearing device and hydrated with free access to water for two additional days under the desired (normal seating load) $\sigma_{n,h}$. Most GCL *specimens* attain equilibrium in less than 24 hours using this procedure (Fox *et al.* 1998a, Triplett and Fox 2001) (Fox et al. 2004).

! The hydrating of GCL test *specimens* should be preformed by submerging the GCL *specimen* at a normal seating load approximately equivalent to the initial load placed on the GCL in the field (e.g., 0.8 psi for a one foot drainage layer with a 120 pcf gravel). ASTM D 6243 requires that the swelling of the *specimen* come to equilibrium before beginning to load the test *specimen*. A GCL can be considered fully hydrated when swelling has slowed to less than a five percent change in thickness in twelve hours (Gilbert et al., 1997).

! The loading of GCL test *specimens* from the hydration normal stress to the shearing normal stress should be performed in a manner that allows time for the *specimens* to consolidate. If insufficient time is allowed between loading increments, bentonite will extrude from the *specimen*. If insufficient time is allowed for the final load to consolidate, excess pore pressures will remain in the *specimen* at the start of shearing. These improper loading procedures will produce inaccurate results. A normal stress increment of no more than 50% every half-day (e.g., 0.8 psi, 1.2 psi, 1.8 psi...) has resulted in successful consolidation. If bentonite extrudes from the *specimen* during loading, the test should be repeated with a lower normal stress increment.

! The rate of shear displacement for shear strength tests of interfaces with a GCL should be slow enough so that insignificant excess pore water pressure exists at failure. However, the rate of shear displacement should not exceed 1.0 millimeters per minute (mm/min) until the shear box traverses its maximum length.

! Most studies indicate that internal shear strength increases with increasing displacement rate, although some key studies have produced contradictory results. Until this issue is resolved, a maximum displacement rate of 0.1 mm/min is recommended for GCL internal shear tests. It should be noted that some data sets indicate that an even slower displacement rate is necessary. More research is needed on this issue (Fox et al., 2004).

! A failed GCL or GCL interface test *specimen* should be inspected after shearing to assess the surface(s) on which failure occurred and the general nature of the failure. Unusual distortion or tearing of the *specimen* should be recorded and may indicate problems with the gripping system. The condition of the geosynthetics at the end clamps (if present) should also be recorded. Evidence of high tensile forces at the clamps, such as tearing or necking of the geosynthetics, is an indication that progressive failure probably occurred during the test. Depending on the extent of localized distress, such a test may be invalid and may need to be repeated using an improved gripping system (Fox et al., 2004).

CONFORMANCE TESTING

Conformance testing is conducted on materials that will be used for constructing a *waste containment facility*. *Conformance testing* is used to verify that the materials being used during construction will exhibit the internal and/or interface shear strengths necessary to provide the minimum required factors of safety approved by Ohio EPA. The shear strengths of in situ foundation and construction materials must be verified by comparing the results of the *conformance testing* with the shear strengths specified in the authorizing document as follows:

- In situ foundation soils must be thoroughly tested during the subsurface investigation. Additional testing during construction should not be needed, unless in situ materials are encountered during excavation that may exhibit weaker shear strengths than the values used during the stability analyses (see previous sections of this chapter and Chapter 3 for more information about investigating and testing in situ foundation materials).

- Materials that will be used for structural fill or recompacted soil layers (RSL) will need to be tested during the subsurface investigation (recommended) or during *conformance testing*. These types of materials must be tested using the lowest density and highest moisture content specified for use during construction. The results of two or more complete tests of each type of material being used for structural fill and RSL are needed. If the tests confirm that the materials will exhibit shear strengths that exceed the minimums specified in the authorizing documents, then the materials should not need to be tested again unless construction specifications change, or materials are encountered that may exhibit weaker shear strengths than those already tested (see previous sections of this chapter and Chapter 3 for more information about investigating and testing structural fill and RSL materials).

- Geosynthetic materials, including GCLs, need to be tested for interface shear strength (GCLs also need to be tested for internal shear strength) during *conformance testing*. A minimum of two complete shear tests must be conducted of each interface (as well as internal shear strength of each GCL) before the material is used for the first time at a facility. After that, one complete test must be conducted before each construction project (see previous sections of this chapter for more information regarding testing geosynthetic interfaces and internal shear strengths of GCLs).

The conformance test data for drained and undrained internal shear strengths, interface *peak shear strengths,* and interface *residual shear strengths* should be used to create compound nonlinear shear strength envelopes with each envelope starting at the origin. The type of shear strength (i.e., drained/undrained, peak/residual) used to compare to the specifications in the authorizing document must be the same type of shear strength that was assumed during the stability analyses. The type of shear strengths used during the stability analysis will typically be as follows:

- *Peak shear strengths* may be used for interfaces with a geosynthetic on slopes of 5 percent or less or slopes that will never be loaded with more than 1,440 psf. This allows the use of *peak shear strength*, if appropriate, for most *facility bottoms* during deep-seated failure analysis. This also allows *peak shear strengths* to be used, if appropriate, for shallow analysis of most final caps, granular drainage layers, and *protective layers* on *internal slopes* prior to the time waste has been placed.

- *Residual shear strengths* are required for interfaces with a geosynthetic on slopes greater than 5 percent that will be loaded with more than 1,440 psf. This requires the use of *residual shear strengths* during deep-seated failure analysis for all interfaces that are on *internal slopes*.

- Internal *peak shear strengths* may be used for reinforced GCL, as long as the internal *peak shear strength* of the GCL exceeds the *peak shear strength* of at least one of the interfaces with the GCL.

- Internal and interface *residual shear strengths* are required for unreinforced GCL.

- *Drained* or *undrained shear strengths*, as appropriate, are required to be used for foundation and construction soil materials. When a slope is underlain by a material that may develop excess pore water pressure during loading, the static factor of safety must be determined using the *undrained shear strength* of the foundation materials. The *undrained shear strengths* must be determined by shear strength testing of site-specific, undisturbed *samples* of all materials that may develop excess pore water pressure.

Residual shear strengths may have been substituted for *peak shear strengths*, especially for interfaces, during the stability analyses. This is done when there is reason to believe that the design, installation, or operation of a facility is likely to cause enough shear displacement within a material or interface that a post-peak shear strength will be mobilized (see **Figure f-2** on page xiv). If this assumption was used, then *residual shear strengths* derived from corresponding materials during *conformance testing* must be used instead of the *peak shear strengths*.

During stability analyses, a composite liner or composite cap system is often modeled as one layer using a linear shear strength envelope, adjusting the strength during modeling until the minimum required factors of safety are provided. To simplify comparison of the *conformance testing* results to the minimum shear strengths specified by the authorizing documents, a compound nonlinear shear strength envelope can be created for an individual material, interface, or system containing multiple interfaces and layers. Determining which shear stresses to plot when creating a compound nonlinear envelope depends upon the type of shear strength envelope being created as follows:

- For compound nonlinear *peak shear strength* envelopes, select the lowest *peak shear strength* measured for any material or interface at each tested normal compressive stress to define the envelope,

- For compound nonlinear *residual shear strength* envelopes, select the *residual shear strength* associated with the lowest *peak shear strength* exhibited by an interface or material at each tested normal compressive stress to define the envelope,

- For compound nonlinear *drained shear strength* envelopes, select the lowest *drained shear strength* measured at each tested normal compressive stress to define the envelope.

- Compound nonlinear *undrained shear strength* envelopes should not be used, select the lowest representative *undrained shear strength* measured for each material regardless of normal compressive stress.

Compound nonlinear shear strength envelopes can be helpful for describing the shear strength of a material and interface when:

- Several complete interface friction tests of the same interface are conducted, resulting in multiple shear stress values for each normal compressive stress used during the testing. The compound nonlinear shear strength envelope can be used, in this case, to represent the minimum expected shear strength that will be exhibited in the field by that one interface when subjected to the range of normal compressive stresses used during testing.

- A composite system (e.g., a composite liner/leachate collection system, or composite cap system) has each interface and material tested for shear strength multiple times, resulting in multiple shear stress values at each normal compressive stress used during the testing. The compound nonlinear shear strength envelope can be used, in this case, to represent the minimum expected shear strength that will be exhibited by the entire composite system in the field when subjected to the range of normal compressive stresses used during testing.

- A soil material to be used for structural fill, RSL, or an in situ material is tested several times resulting in multiple shear stress values at each normal compressive stress used during the test. The compound nonlinear shear strength envelope can be used, in this case, to represent the minimum expected shear strength that will be exhibited by the soil material in the field when subjected to the range of normal compressive stresses used during testing.

An example methodology for creating compound nonlinear shear strength envelopes can be found starting on page 4-18.

Sometimes, Ohio EPA may require composite systems using multiple materials and having multiple interfaces with geosynthetics to be tested to determine which interface or material will be the locus of the failure surface throughout the range of normal stresses expected in the field. This may entail using a direct shear device or other appropriate device to test *specimens* comprising all the layers in a composite system. For example, if all of the *peak shear strengths* for each interface and material are near each other, but a wide range of *residual shear strengths* exists, either the lowest *residual shear strength* measured will need to be used, or *specimens* comprising all the layers in a composite system will need to be tested.

Developing Compound Nonlinear Shear Strength Envelopes - Example Methodology

A stabilization plan for heavy metal contaminated soil at several locations on a property has been approved by Ohio EPA, DERR as part of a negotiated settlement. The plan includes a CERCLA retention unit. The unit will hold a maximum of 30 feet of stabilized soils. It has 3:1 *internal slopes* and 4:1 external slopes. The approved composite liner system includes four (4) feet of $1x10^{-7}$ cm/sec RSL and is overlain with 60 mils thick textured high density polyethylene (THDPE). The drainage layer includes a geocomposite with a one-foot thick *protective layer* of #57 gravel on top. **Figure 4-2** on page 4-20, **Figure 4-3** on page 4-21, and **Figure 4-4** on page 4-22 show the results of the interface shear strength testing of the three interfaces at 1000 psf, 2000 psf, and 4000 psf normal compressive stress. The graphs show the lowest *peak shear strength* for each interface selected from the results of multiple tests of each interface. The 1000-psf test represents the normal compressive stress of about seven (7) feet of stabilized waste (@130 pcf). The 4000-psf test represents 110% of the normal compressive stress of the weight per square foot of the waste at its deepest point. To ensure that the full range of normal compressive stresses experienced in the field are included, another set of interface tests should have been run for each interface at a smaller normal compressive stress to represent one foot or less of the waste. This would be particularly important if these interfaces were also to occur in the composite cap system. Fortunately for this site, the shear stress from 0 psf to 1000 psf can be adequately estimated by connecting a line from the origin to the shear stress measured at 1000 psf for each interface (see **Figure 4-5** page 4-23 and **Figure 4-7** on page 4-25).

Compound Nonlinear Peak Shear Strength Envelopes

This methodology is appropriate when using *peak shear strengths*. It is used for composite systems comprising multiple layers and interfaces (e.g., composite liners and caps). It also applies when developing a nonlinear shear strength envelope for a single material or a single interface tested several times with varying results at each normal compressive stress. In this example, a compound nonlinear *peak shear strength* envelope will be created from the test results shown on **Figure 4-2** on page 4-20, **Figure 4-3** on page 4-21, and **Figure 4-4** on page 4-22. **Figure 4-5** on page 4-23 shows the non-linear shear strength envelopes for three interfaces, and was created by taking the lowest peak shear stress measured from multiple tests of each interface at each normal compressive stress and plotting the points on a graph showing shear stress on the y-axis and normal compressive stress on the x-axis. The data points used to create **Figure 4-5** are found in Table 4.

To create a compound nonlinear shear strength envelope, select the lowest peak shear stress measured for any interface or material in the composite system at each normal compressive stress (see highlighted values in Table 4). Next, plot the selected peak shear stress values vs. the corresponding normal compressive stress values to produce a graph showing the compound nonlinear *peak shear strength* envelope. The shear stress of the system below the lowest normal compressive stress tested is estimated by connecting a line from the origin to the peak shear stress measured at the lowest normal compressive stress. The *peak shear strength* used when modeling the composite system is then plotted on the graph to verify that the entire nonlinear *peak shear strength* envelope plots above it (see **Figure 4-6** on page 4-24).

Table 4. An example of the lowest peak shear stress measured for three interfaces from a composite liner system at three different normal compressive stresses (data points obtained from **Figure 4-2** on page 4-20, **Figure 4-3** on page 4-21, and **Figure 4-4** on page 4-22). The highlight marks the interface with the lowest peak shear stress at each normal compressive stress.

Interface	Peak Shear Stress (psf)		
	1000 psf Normal Compressive Stress	**2000 psf NCS**	**4000 psf NCS**
RSL vs. THDPE	782	1042	2371
THDPE vs. Geocomposite	465	1450	2040
Geocomposite vs. Protective Layer	568	1013	2354

Compound Nonlinear Residual Shear Strength Envelopes

This methodology applies to any composite system comprising multiple layers and interfaces (e.g., composite liners and caps). It also applies when developing a nonlinear *residual shear strength* envelope for a single material or interface tested several times with varying results at each normal compressive stress. The process for developing a compound nonlinear *residual shear strength* envelope is the same as the process for developing the compound nonlinear *peak shear strength* envelope with one exception. When creating the compound nonlinear *residual shear strength* envelope, instead of choosing the lowest *peak shear strength* at each normal compressive stress to plot, choose the residual shear stress associated with the lowest peak shear stress at each normal compressive stress (see highlighted values in Table 5).

Notice that in Table 5, for a normal compressive stress of 2000 psf, the residual shear stress of 984 psf was selected rather than the lowest residual shear stress of 614 psf. This is because 984 psf is the residual shear stress associated with the interface that has the lowest peak shear stress. To create a compound nonlinear *residual shear strength* envelope, use the selected residual shear stresses and the associated normal compressive stresses (see highlighted values in Table 5) to plot shear stress values vs. normal compressive stress values. To ensure that the full range of normal compressive stresses to be experienced in the field are included, another set of interface tests should have been run for each interface at a smaller normal compressive stress to represent one foot or less of the waste. This would be particularly important if these interfaces were to also occur in the composite cap system. To estimate the shear stress below the lowest normal compressive stress used during testing, connect a line from the origin to the residual shear stress measured at the lowest normal compressive stress used during the testing. The *residual shear strength* used when modeling the composite system is then plotted on the graph to verify that the entire nonlinear *residual shear strength* envelope plots above it (see **Figure 4-8** on page 4-26).

Table 5. Examples of the lowest residual shear stresses measured from multiple tests of three interfaces from a composite liner system at three different normal compressive stresses (data points obtained from **Figure 4-2** on page 4-20, **Figure 4-3** on page 4-21, and **Figure 4-4** on page 4-22). The highlight marks the interface with the residual shear stress associated with the lowest peak shear stress at each normal compressive stress.

Interface	**(Peak) and Residual Shear Stress (psf)**		
	1000 psf Normal Compressive Stress	**2000 psf Normal Compressive Stress**	**4000 psf Normal Compressive Stress**
RSL vs. THDPE			
Peak	(782)	(1042)	(2371)
Residual	684	1003	2320
THDPE vs. Geocomposite			
Peak	(465)	(1450)	(2040)
Residual	270	614	1187
Geocomposite vs. Protective Layer			
Peak	(568)	(1013)	(2354)
Residual	555	984	2300

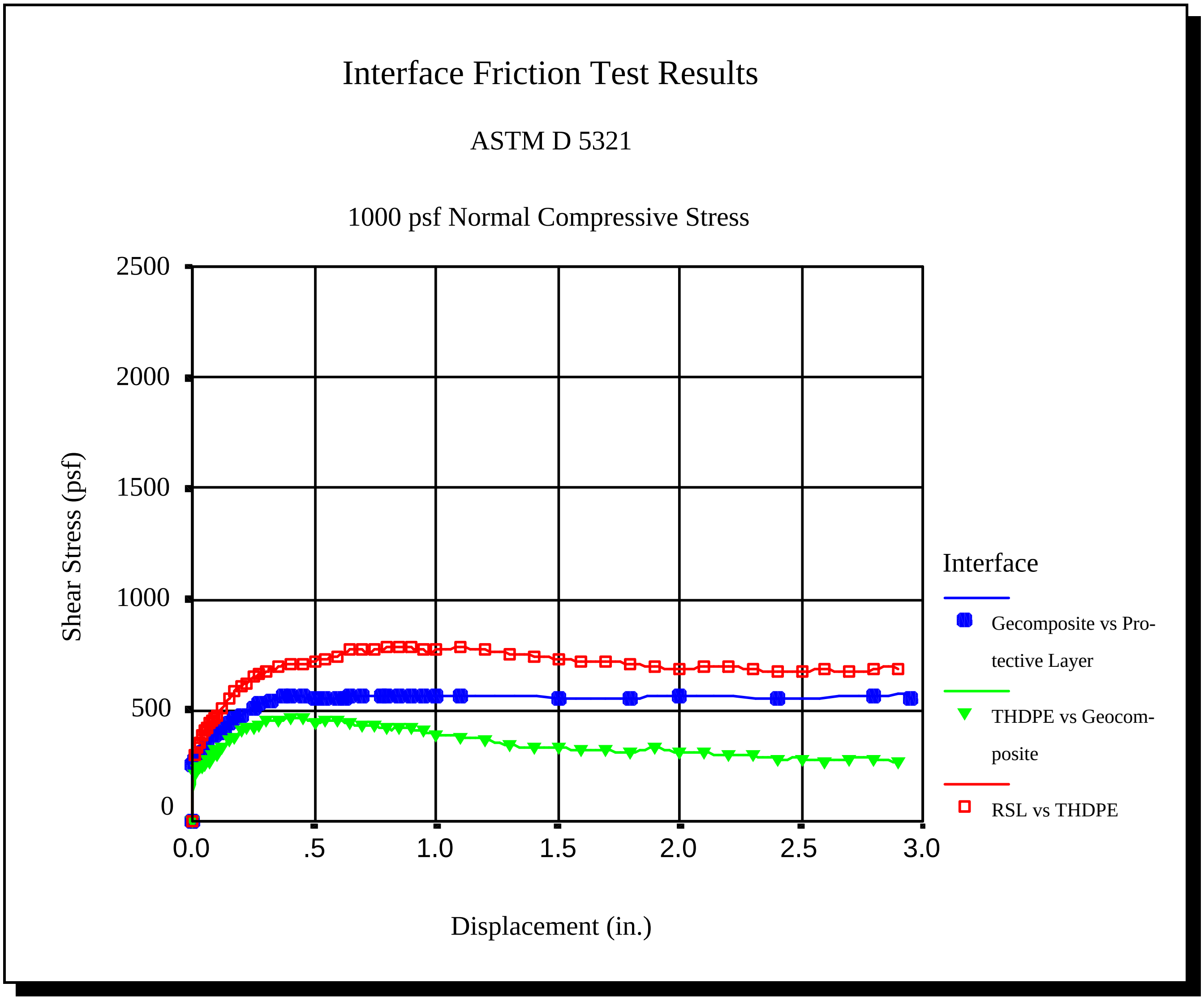

Figure 4-2 An example of interface friction test results for three interfaces of a composite liner system at 1000 psf normal compressive stress. Multiple tests of each interface were conducted. This graph shows only the results of the test for each interface that resulted in the lowest peak shear stress at this normal compressive stress.

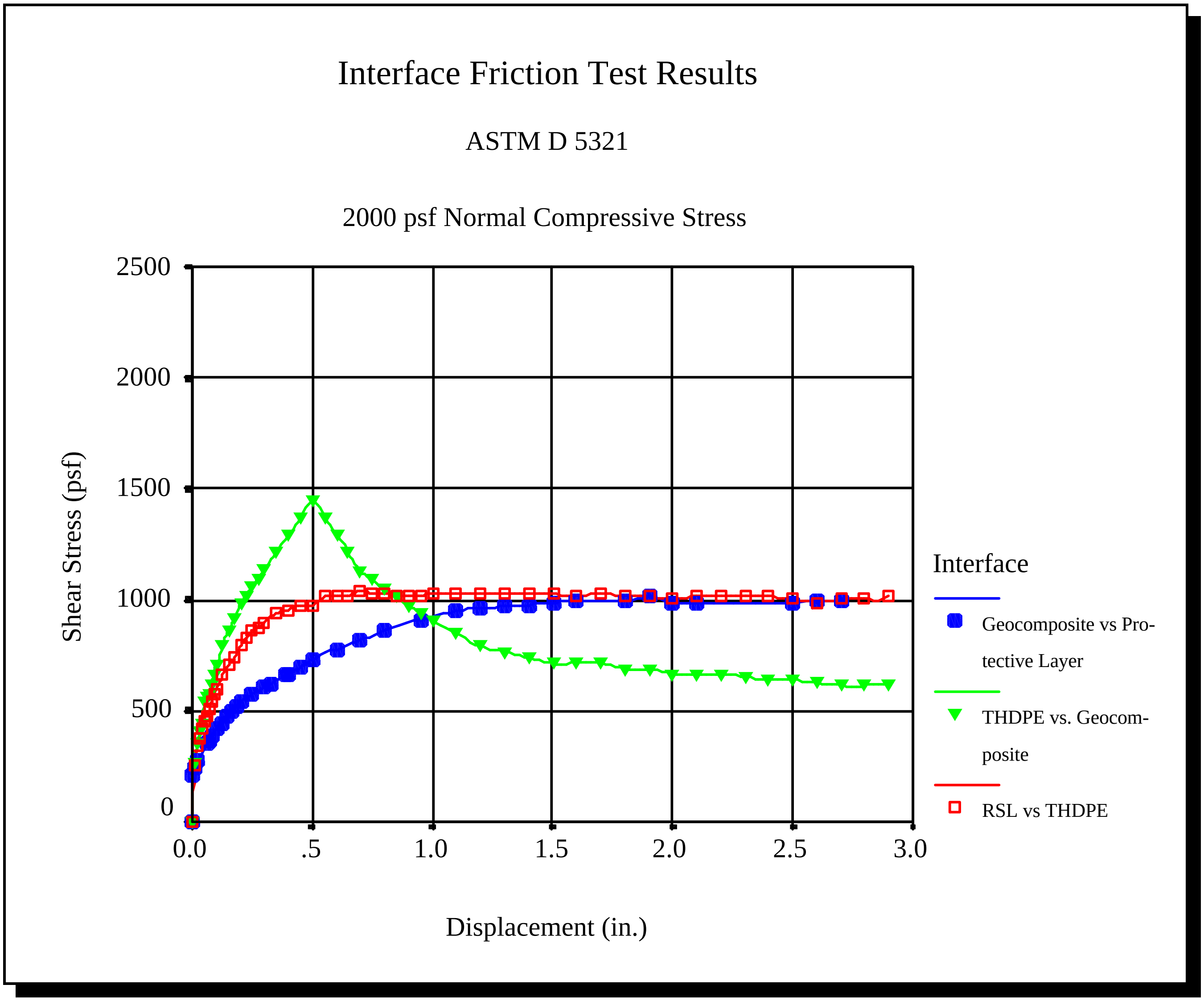

Figure 4-3 An example of interface friction test results for three interfaces of a composite liner system at 2000 psf normal compressive stress. Multiple tests of each interface were conducted. This graph shows only the results of the test for each interface that resulted in the lowest peak shear stress at this normal compressive stress.

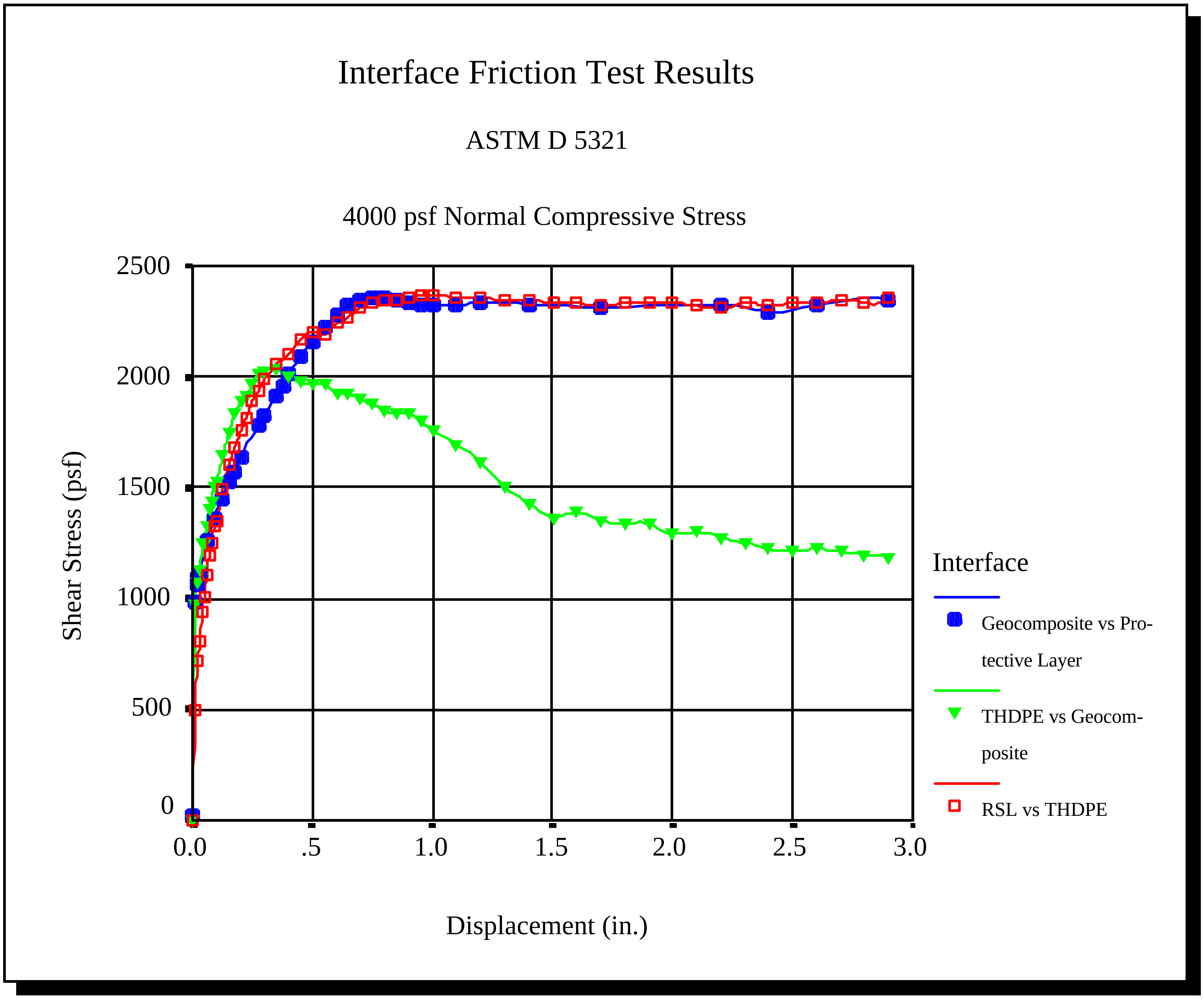

Figure 4-4 An example of interface friction test results for three interfaces of a composite liner system at 4000 psf normal compressive stress. Multiple tests of each interface were conducted. This graph shows only the results of the test for each interface that resulted in the lowest peak shear stress at this normal compressive stress.

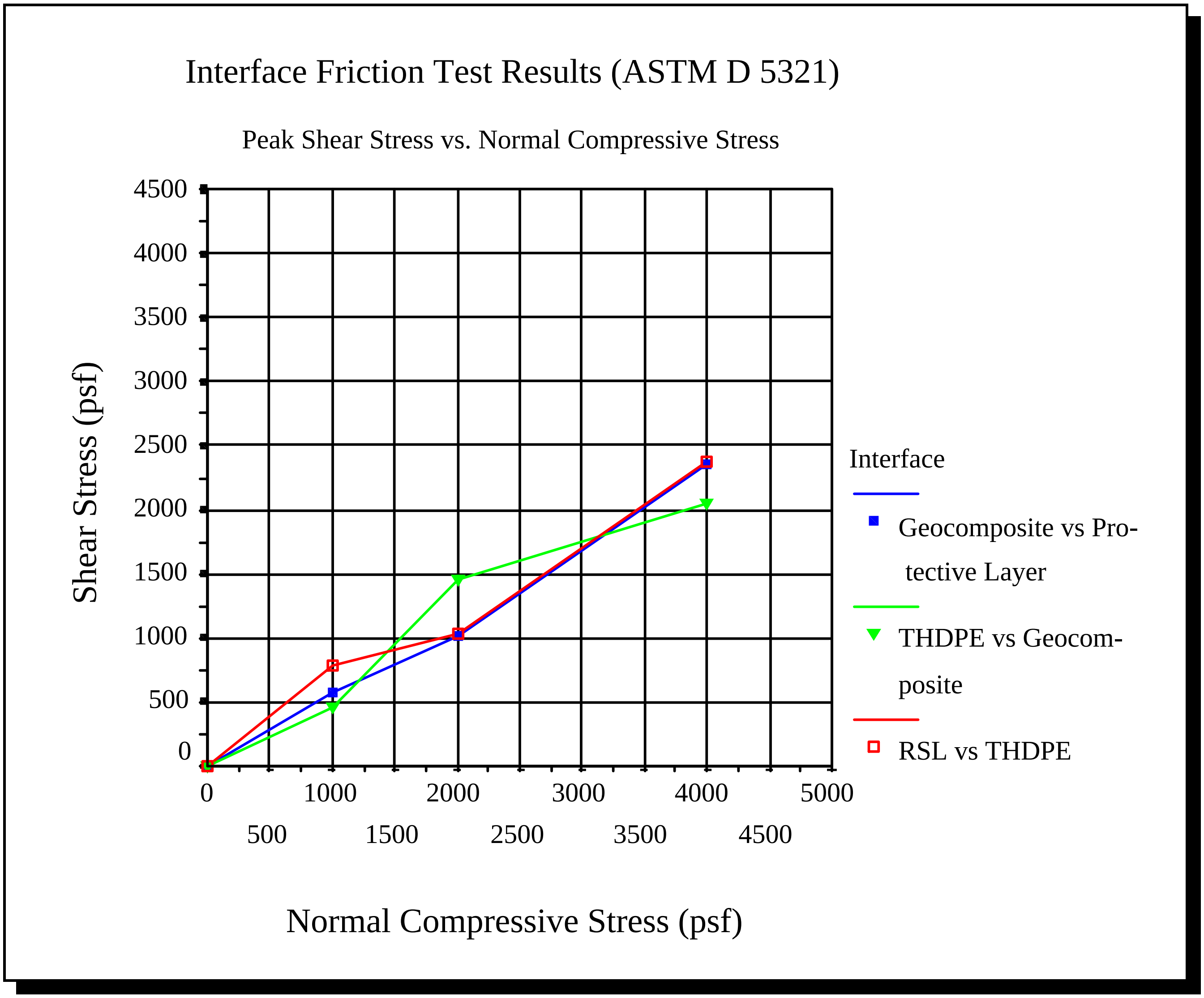

Figure 4-5 An example of individual nonlinear *peak shear strength* envelopes derived from the lowest peak shear testing data at each normal compressive stress for each of three interfaces in a composite system. The shear stress below 1000 psf normal compressive stress was estimated by drawing a line from the origin to the shear stress at 1000 psf normal compressive stress for each interface. If normal compressive loads greater than 4000 psf are expected at the facility, then additional testing at higher normal compressive loads will be necessary.

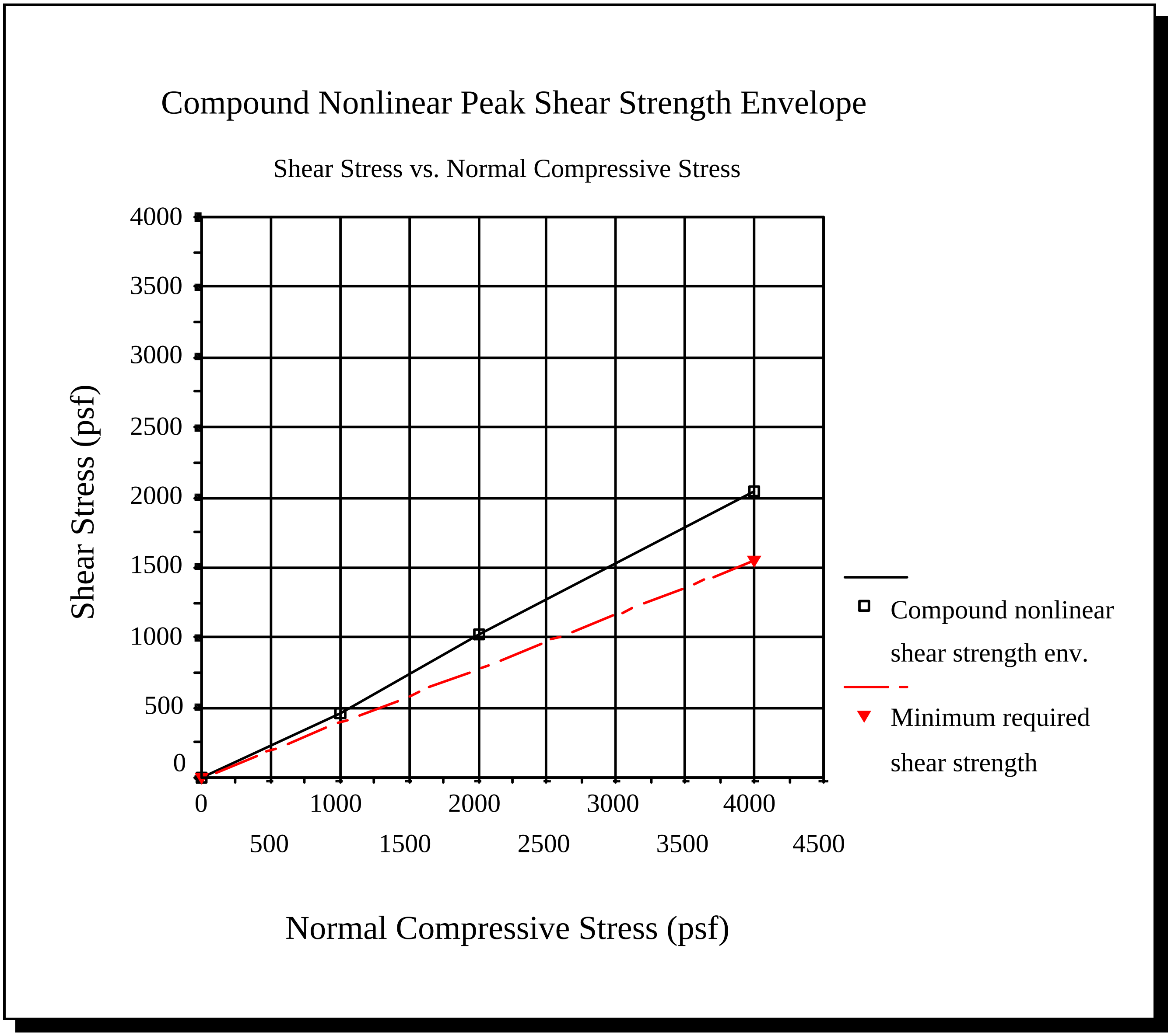

Figure 4-6 An example of a compound nonlinear *peak shear strength* envelope created from the individual nonlinear *peak shear strength* envelopes of three interfaces of a composite system. When the *peak shear strength* envelope is compared to the minimum *peak shear strength* specified in the authorizing document, it can be seen that the composite system exhibits enough *peak shear strength* at all normal compressive stresses expected at the facility, and thus the minimum required *peak shear strength* is exceeded. This ensures that all the tested materials can be used during construction of composite systems when *peak shear strength* conditions are expected. If normal compressive loads greater than 4000 psf are expected at the facility, then additional testing at higher normal compressive loads will be necessary.

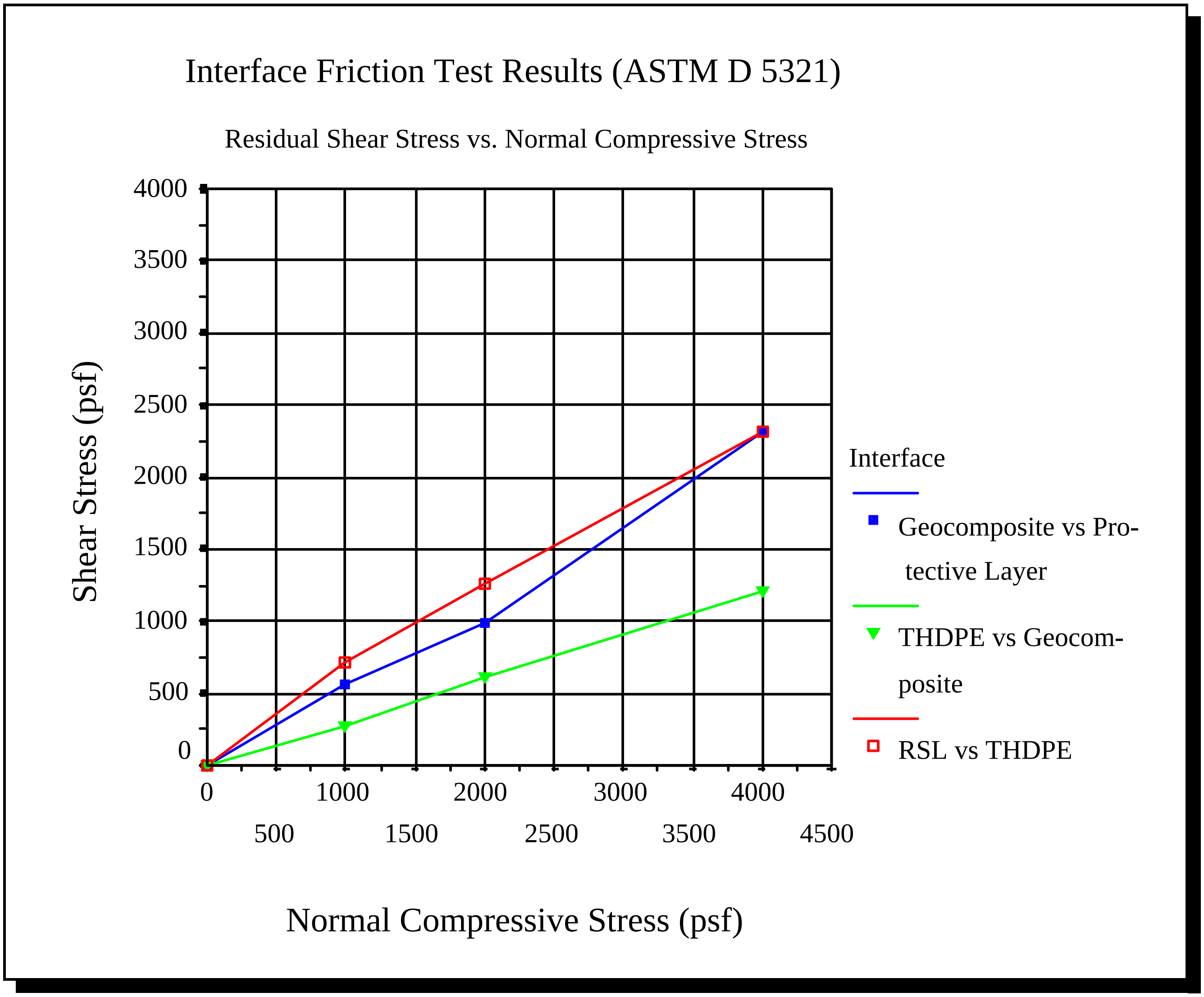

Figure 4-7 An example of individual nonlinear *residual shear strength* envelopes derived from the lowest residual shear testing data at each normal compressive stress for each of three interfaces in a composite system. The shear stress below 1000 psf normal compressive stress was estimated by drawing a line from the origin to the shear stress at 1000 psf normal compressive stress for each interface. If normal compressive loads greater than 4000 psf are expected at the facility, then additional testing at higher normal compressive loads will be necessary.

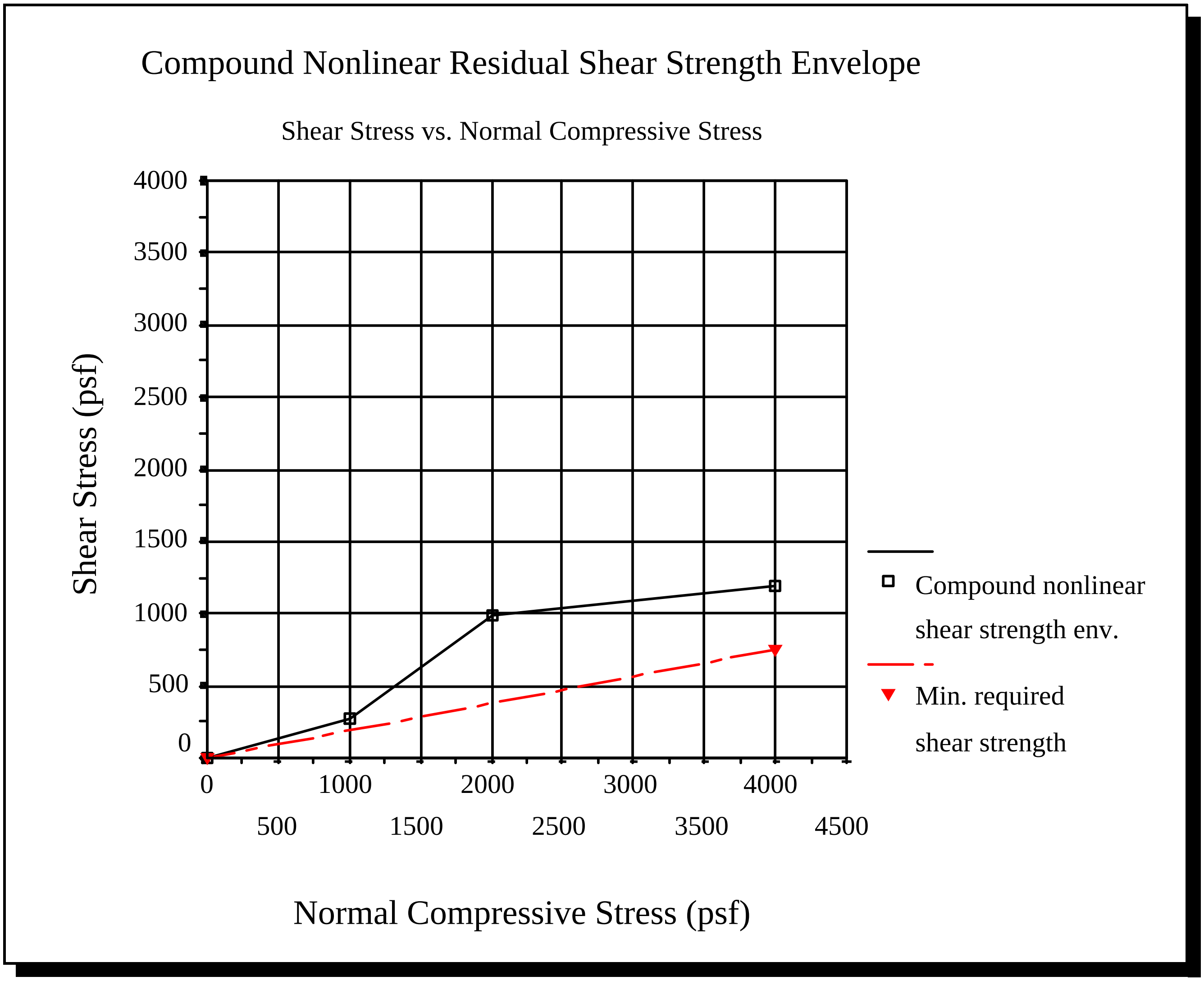

Figure 4-8 An example of a compound nonlinear *residual shear strength* envelope created from the individual nonlinear *residual shear strength* envelopes of three interfaces of a composite system. When the *residual shear strength* envelope is compared to the minimum *residual shear strength* specified in the authorizing document, it can be seen that the composite system exhibits enough *residual shear strength* at all normal compressive stresses expected at the facility that, and thus minimum required *residual shear strength* is exceeded. This ensures that all the tested materials can be used during construction of composite systems when *residual shear strength* conditions are expected. If normal compressive loads greater than 4000 psf are expected at the facility, then additional testing at higher normal compressive loads will be necessary.

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