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Standard Guide for Considerations When Evaluating Direct Shear Results Involving Geosynthetics[1]

This standard is issued under the fixed designation D7702/D7702M; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

1. Scope

1.1 This guide presents a summary of available information related to the evaluation of direct shear test results involving geosynthetic materials.

1.2 This guide is intended to assist designers and users of geosynthetics. This guide is not intended to replace education or experience and should only be used in conjunction with professional judgment. This guide is not intended to represent or replace the standard of care by which the adequacy of a given professional service must be judged, nor should this document be applied without consideration of a project's many unique aspects. Not all aspects of this practice may be applicable in all circumstances. The word "Standard" in the title of this document means only that the document has been approved through the ASTM consensus process.

1.3 This guide is applicable to soil-geosynthetic and geosynthetic-geosynthetic direct shear test results, obtained using either Test Method D5321/D5321M or D6243/D6243M.

1.4 This guide does not address selection of peak or large-displacement shear strength values for design. References on this topic include Thiel (**1**),[2] Gilbert (**2**), Koerner and Bowman (**3**), and Stark and Choi (**4**).

1.5 The values stated in either SI units or inch-pound units are to be regarded separately as standard. The values stated in each system are not necessarily exact equivalents; therefore, to ensure conformance with the standard, each system shall be used independently of the other, and values from the two systems shall not be combined.

1.6 *This standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety, health, and environmental practices and determine the applicability of regulatory limitations prior to use.*

1.7 *This international standard was developed in accordance with internationally recognized principles on standardization established in the Decision on Principles for the Development of International Standards, Guides and Recommendations issued by the World Trade Organization Technical Barriers to Trade (TBT) Committee.*

2. Referenced Documents

2.1 *ASTM Standards:*[3]

D653 Terminology Relating to Soil, Rock, and Contained Fluids
D4439 Terminology for Geosynthetics
D5321/D5321M Test Method for Determining the Shear Strength of Soil-Geosynthetic and Geosynthetic-Geosynthetic Interfaces by Direct Shear
D6243/D6243M Test Method for Determining the Internal and Interface Shear Strength of Geosynthetic Clay Liner by the Direct Shear Method

3. Terminology

3.1 *Definitions*—For definitions of terms relating to soil and rock, refer to Terminology D653. For definitions of terms relating to geosynthetics and GCLs, refer to Terminology D4439.

3.2 *Definitions of Terms Specific to This Standard:*

3.2.1 *adhesion, c_a or c, n*—the y-intercept of the Mohr-Coulomb shear strength envelope; the component of shear strength indicated by the term c_a, in Coulomb's equation, $\tau = c_a + \sigma \tan \delta$.

3.2.2 *failure envelope, n*—curvi-linear line on the shear stress-normal stress plot representing the combination of shear and normal stresses that define a selected shear failure criterion (for example, peak and post-peak). Also referred to as shear strength envelope.

3.2.3 *Mohr-Coulomb friction angle δ, n*—angle of friction of a material or between two materials (degrees), the angle defined by the least-squares, "best-fit" straight line through a

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[2] The boldface numbers in parentheses refer to a list of references at the end of this standard.

[3] For referenced ASTM standards, visit the ASTM website, www.astm.org, or contact ASTM Customer Service at service@astm.org. For *Annual Book of ASTM Standards* volume information, refer to the standard's Document Summary page on the ASTM website.

defined section of the shear strength-normal stress failure envelope; the component of the shear strength indicated by the term δ, in Coulomb's equation, $\tau = c + \sigma \tan \delta$.

3.2.4 *Mohr-Coulomb shear strength envelope, n*—the least-squares, "best-fit" straight line through a defined section of the shear strength-normal stress failure envelope described in the equation $\tau = c_a + \sigma \tan \delta$. The envelope can be described for any chosen shear failure criteria (for example, peak, post-peak, or residual).

3.2.5 *secant friction angle, δ_{sec}, n*—(degrees) the angle defined by a line drawn from the origin to a data point on the shear strength-normal stress failure envelope. Intended to be used only at the shearing normal stress for which it is defined.

3.2.6 *shear strength, τ, n*—the shear force on a given failure plane. In the direct shear test it is always stated in relation to the normal stress acting on the failure plane. Two different types of shear strengths are often estimated and used in standard practice:

3.2.6.1 *peak shear strength, n*—the largest value of shear resistance experienced during the test under a given normal stress.

3.2.6.2 *post-peak shear strength, n*—the minimum, or steady-state value of shear resistance that occurs after the peak shear strength is experienced.

3.2.6.3 *Discussion*—Due to horizontal displacement limitations of many commercially available shear boxes used to determine interface shear strength, the post-peak shear strength is often specified and reported as the value of shear resistance that occurs at 75 mm [3 in.] of displacement. The end user is cautioned that the reported value of post-peak shear strength (regardless how defined) is not necessarily the residual shear strength. In some instances, a post-peak shear strength may not be defined before the limit of horizontal displacement is reached.

4. Significance and Use

4.1 The shear strength of soil-geosynthetic interfaces and geosynthetic-geosynthetic interfaces is a critical design parameter for many civil engineering projects, including, but not limited to: waste containment systems, mining applications, dam designs involving geosynthetics, mechanically stabilized earth structures, reinforced soil slopes, and liquid impoundments. Since geosynthetic interfaces often serve as a weak plane on which sliding may occur, shear strengths of these interfaces are needed to assess the stability of earth materials resting on these interfaces, such as a waste mass or ore body over a lining system or the ability of a final cover to remain on a slope. Accordingly, project-specific shear testing using representative materials under conditions similar to those expected in the field is recommended for final design. Shear strengths of geosynthetic interfaces are obtained by either Test Method D5321/D5321M (geosynthetics) or D6243/D6243M (geosynthetic clay liners). This guide touches upon some of the issues that should be considered when evaluating shear strength data. Because of the large number of potential conditions that could exist, there may be other conditions not identified in this guide that could affect interpretation of the results. The seemingly infinite combinations of soils, geosynthetics, hydration and wetting conditions, normal load distributions, strain rates, creep, pore pressures, etc., will always require individual engineering evaluations by qualified practitioners. Along the same lines, the list of references provided in this guide is not exhaustive, nor are the findings and suggestions of any particular reference meant to be considered conclusive. The references and their related findings are presented herein only as examples available in the literature of the types of considerations that others have found useful when evaluating direct shear test results.

4.2 The figures included in this guide are only examples intended to demonstrate selected concepts related to direct shear testing of geosynthetics. The values shown in the figures may not be representative and should not be used for design purposes. Site-specific and material-specific tests should always be performed.

5. Shear Strength Fundamentals

5.1 Mohr first presented a theory for shear failure, showing that a material experiences failure at a critical combination of normal and shear stress, and not through some maximum normal or shear stress alone. In other words, the shear stress on a given failure plane was shown to be a function of the normal stress acting on that plane (**5**):

$$\tau = f(\sigma) \tag{1}$$

If a series of shear tests at different values of normal stress is performed, and the stress circle corresponding to failure is plotted for each test, at least one point on each circle must represent the normal and shear stress combination associated with failure (**6**). As the number of tests increases, a failure envelope (line tangent to the failure circles) for the material becomes apparent (Fig. 1).

5.2 In general, the failure envelope described by Eq 1 is a curved line for many materials (**5**). For most geotechnical engineering problems, the shear stress on the failure plane is approximated as a linear function of the total or effective normal stress within a selected normal stress range, as shown in Fig. 1. This linear approximation is known as the Mohr-Coulomb shear strength envelope. In the case of total stresses, the Mohr-Coulomb shear strength envelope is expressed as:

$$\tau = c_a + \sigma \ tan \ \delta \tag{2}$$

where:

τ = shear stress,
σ = normal stress,
δ = friction angle (degrees), and
c_a = adhesion.

In the case of effective stresses, the linear failure envelope is:

$$\tau = c_a' + (\sigma - u) \ tan \ \delta \tag{3}$$

or

$$\tau = c_a' + \sigma' \ tan \ \delta'$$

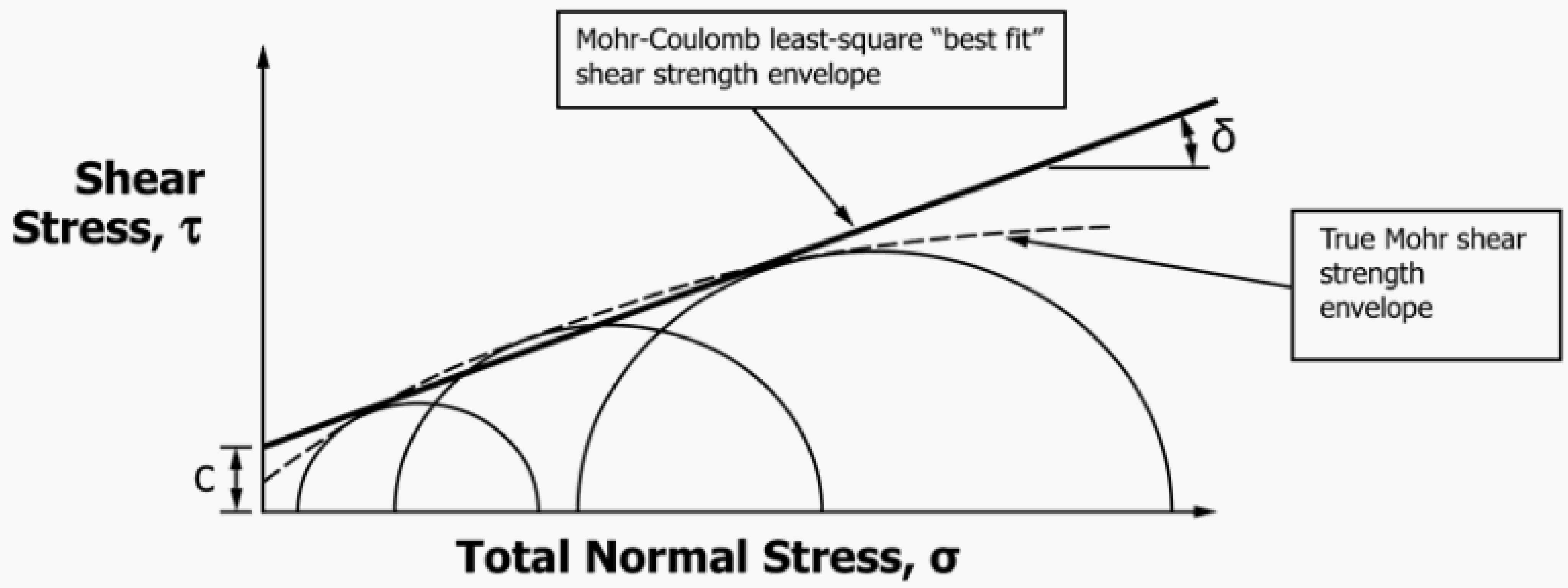

FIG. 1 Curved Mohr Failure Envelope and Equivalent Mohr-Coulomb Linear Representation (from Wright (7))

where:

u' = pore pressure,
σ' = effective normal stress,
δ' = drained friction angle (degrees), and
c_a' = effective stress adhesion.

NOTE 1—Adhesion, c_a, is commonly associated with interface shear strength results. Cohesion, c, is often associated with internal shear strength results involving soils or GCLs. Mathematically, these terms are the identical; simply the y-intercept of the Mohr-Coulomb shear strength envelope, or in other words, the component of shear strength indicated by the term c_a, in Coulomb's equation, $\tau = c_a + \sigma \tan \delta$.

NOTE 2—The end user is cautioned that some organizations (for example, FHWA (**8**) and AASHTO (**9**), along with state agencies who are using these documents) are currently using the Greek letter Delta (δ) to designate wall-backfill interface friction angle, and the Greek letter Rho (ρ) to designate the interface friction angle between geosynthetics and soil.

5.3 Since most laboratory direct shear tests do not include pore pressure measurements, shear strength results reported by laboratories are normally expressed in terms of total normal stress. For direct shear tests involving geosynthetics, Test Methods D5321/D5321M and D6243/D6243M provide recommendations for shear displacement rates intended to allow dissipation of pore water pressures generated during shearing. Recommended shear rates are 0.2 in./min for geosynthetic (non-GCL) interface tests, 0.04 in./min for geosynthetic/soil (including hydrated GCLs) interface tests (**10**), and 0.004 in./min for hydrated GCL internal shear tests (**11**). However, as shown by Obermeyer et al. (**12**), even slower displacement rates may be needed for GCLs and high-plasticity clay soils to ensure that positive pore pressures do not develop during shearing. If tests involving GCLs or clays are loaded or sheared too quickly, excess pore water pressures could develop, and results may not be representative of field conditions, which are often assumed to be drained. The assumption of drained conditions is reasonable because drainage layers are common in liner systems and because field loading rates are generally slow (**13, 11**). From Eq 3, positive pore pressures that are not allowed to dissipate will decrease the measured shear stress. Tests that are sheared undrained may yield erroneous results similar to those discussed in Section 9. Drained and undrained strengths are not interchangeable from a design perspective.

5.4 Combinations of shear stress and normal stress that fall on the Mohr-Coulomb shear strength envelope indicate that a shear failure will occur. Combinations below the shear strength envelope represent a non-failure state of stress (**14**). A state of stress above the envelope cannot exist, since shear failure would have already occurred.

6. Measurement and Reporting of Shear Strength by Test Methods D5321/D5321M / D6243/D6243M

6.1 The shear resistance between geosynthetics or between a geosynthetic and a soil is determined by placing the geosynthetic and one or more contact surfaces, such as soil, within a direct shear box. A constant normal stress representative of field stresses is applied to the specimen, and a tangential (shear) force is applied to the apparatus so that one section of the box moves in relation to the other section. The shear force is recorded as a function of the shear displacement of the moving section of the shear box.

6.2 The test is run until the shear displacement exceeds 75 mm [3 in.] or other value specified by the user. Note that 75 mm of displacement is the practical upper limit of most direct shear devices.

6.3 The testing laboratory plots the test data as a graph of applied shear force versus shear displacement. The peak shear force and the shear force at the end of the test are identified. The shear displacements associated with these shear forces are also determined. An example set of shear-displacement plots for a typical textured geomembrane/reinforced GCL interface is shown in Fig. 2(a). Typical shear-displacement behavior of geosynthetic interfaces is discussed further in Section 9.

6.4 The shear stresses applied to the specimen for each recorded shear force are calculated by dividing the shear force by the specimen area. For tests in which the area of specimen contact decreases with increased displacement, a corrected area should be calculated, unless other technical interpretation arrangements are made ahead of time between the engineer and the testing laboratory.

6.5 The testing laboratory plots the peak shear stress and post-peak (also known as large displacement) shear stress versus applied normal stress for each test conducted. An example set of shear stress-normal stress plots for a typical textured geomembrane/reinforced GCL interface is shown in Fig. 2(b).

6.6 The testing laboratory then draws a least-squares "best-fit" straight line through the peak shear stress data points, Eq 2.

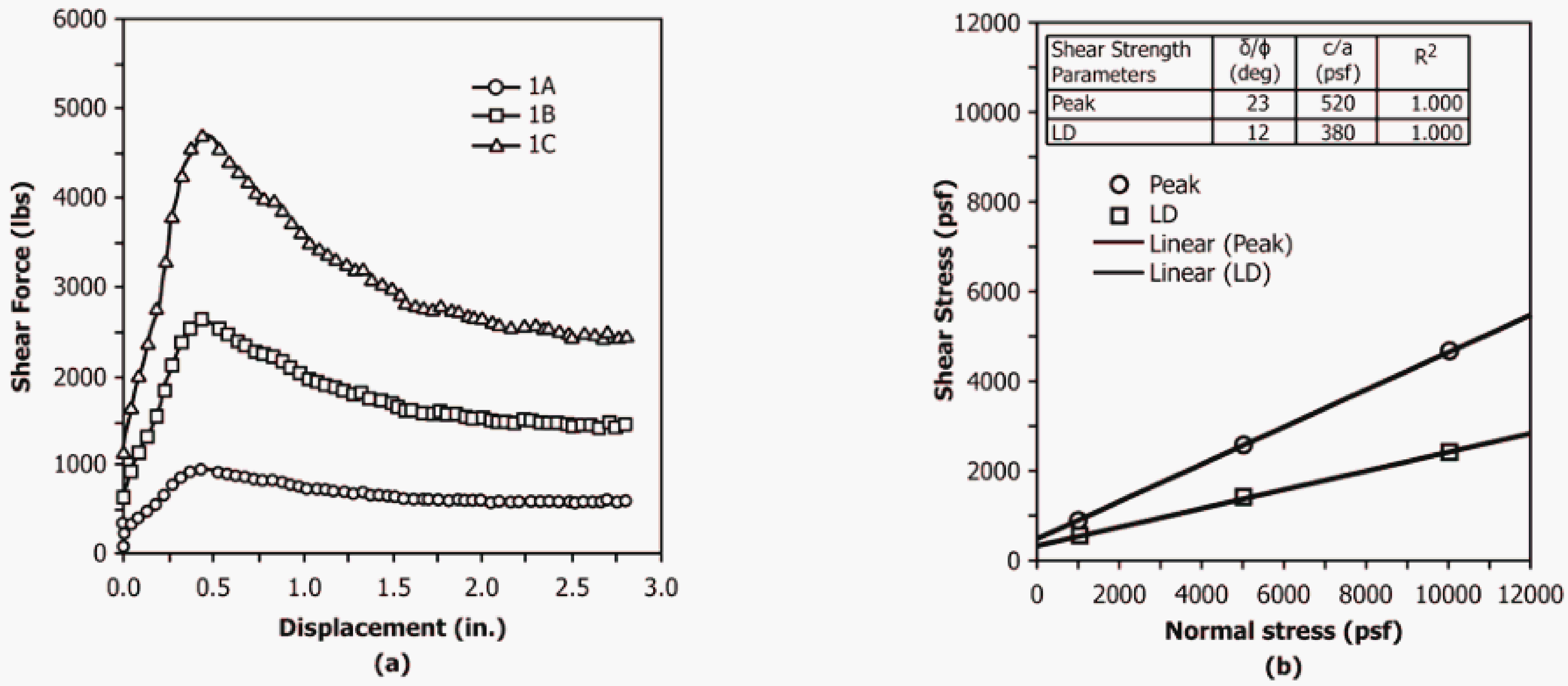

FIG. 2 Typical Shear-Displacement Curves (a) and Peak and Large Displacement Failure Envelopes (b) for a Textured Geomembrane/ Needlepunch-Reinforced GCL Interface

The intercept of the straight line with the y-axis (x = 0) is the adhesion, c_a, for interface strength or cohesion intercept, c, for internal strength. Taking the inverse tangent of the slope of the straight line yields the peak angle of friction, δ_{peak}. The adhesion and Mohr-Coulomb friction angle can be described for any chosen shear failure criteria (peak, post-peak, or residual).

7. Evaluation of the Mohr-Coulomb Failure Envelope

7.1 Traditionally, the laboratory-reported Mohr-Coulomb strength parameters c and δ have been used to assess the stability of slopes containing geosynthetics using limit equilibrium methods. Although Test Methods D5321/D5321M and D6243/D6243M call for the testing laboratory to draw a best-fit line through the shear stress-normal stress data and determine c and δ, it is strongly recommended that the design engineer also evaluate the data to determine the appropriate strength parameters to be used in a slope stability analysis.

7.2 It is important to note that the reported Mohr-Coulomb parameters only define the shear strength envelope for the range of normal stresses tested. Extrapolation of both friction angle and adhesion outside the range of normal stresses tested may not be representative. Extrapolating the failure envelope below the lowest normal stress tested can overestimate shear strength, since the failure envelopes for many geosynthetic interfaces can curve sharply to the origin. Similarly, extrapolating the failure envelope above the highest normal stress tested can overestimate shear strength, since the failure envelope for many geosynthetic interfaces flattens at high loads (**15**). If some extrapolation is required, a conservative and safe method would be as follows (**16**):

7.2.1 Extrapolation of the shear strength envelope to lower normal loads would go from the result tested at the lowest normal load back through the (0,0) origin.

7.2.2 Extrapolation of the shear strength envelope to high normal loads would go from the result tested at the highest normal load with a horizontal line of constant shear strength.

7.2.3 Any extrapolation of shear strengths with resulting strengths greater than these suggestions cannot be defended by the test results.

7.3 In the sample laboratory report shown in Fig. 2(b), the peak Mohr-Coulomb shear strength envelope, in kPa, is described by: $\tau_{peak} = 24.9 + \sigma \cdot \tan 23°$ [$\tau_{peak} = 520 + \sigma \cdot \tan 23°$, in psf]. The large-displacement Mohr-Coulomb shear strength envelope, in kPa, is described by: $\tau_{LD} = 18.2 + \sigma \cdot \tan 12°$ [$\tau_{LD} = 380 + \sigma \cdot \tan 12°$, in psf]. These expressions are only valid for the range of normal stresses tested; in this example, from 47.9 to 479 kPa [1000 to 10 000 psf].

7.4 As shown in Fig. 3 (based on Blond and Elie (**17**)), the term δ in the "best-fit" Mohr-Coulomb shear strength envelope, $\tau = c + \sigma \tan \delta$, is known as the Mohr-Coulomb friction angle.

7.5 Some testing laboratories also report secant friction angles, δ_{sec}. As shown in Fig. 3, the secant friction angle is defined by a line drawn from the origin to a data point on the shear strength-normal stress envelope. The secant friction angle is only intended for use with the normal stress for which

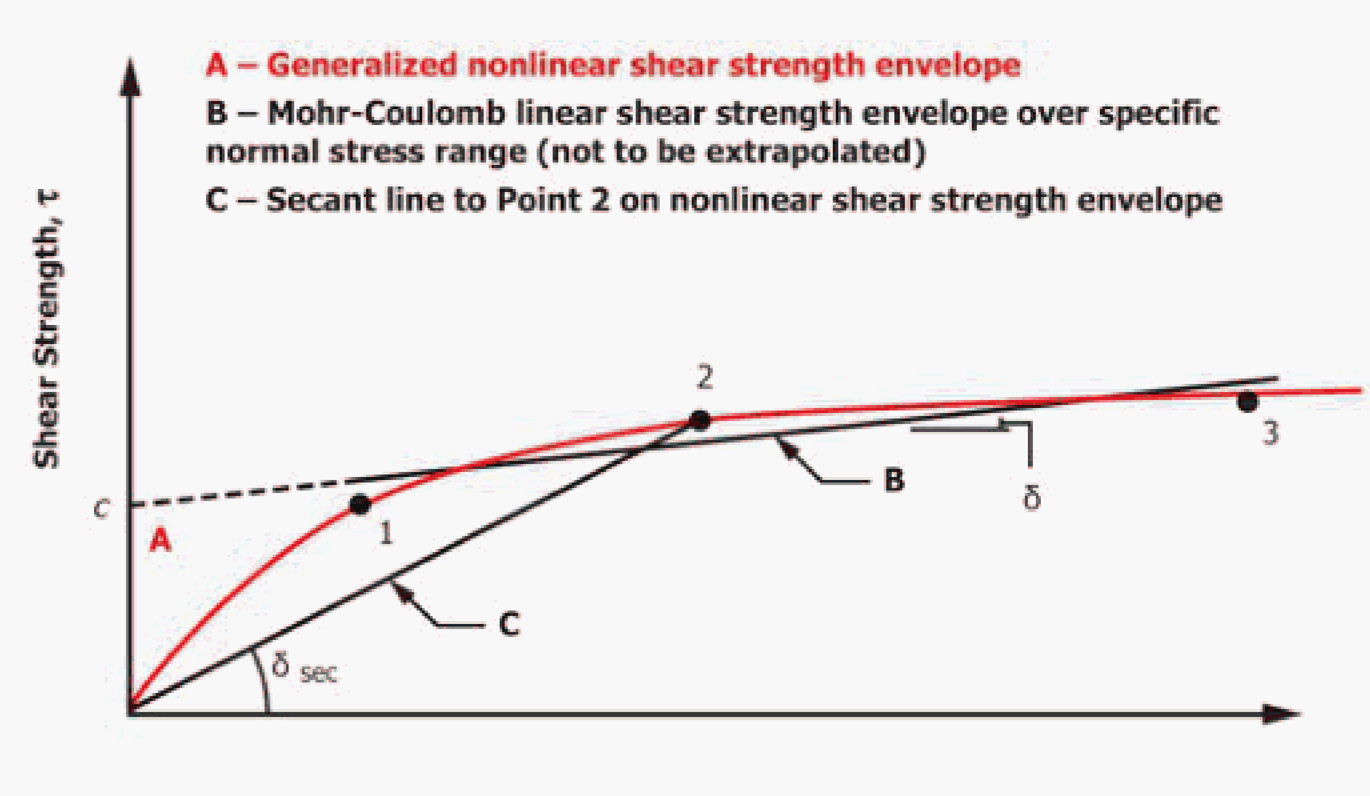

FIG. 3 Friction Angles (based on Fox and Stark (11), and Blond and Elie (17))

it was defined and should not be confused with the Mohr-Coulomb friction angle (**11**). Except for the unique case where $c = 0$ and the shear strength envelope is linear, the secant and Mohr-Coulomb friction angles will be different. (Section 8 discusses how the secant angle can be useful when interpreting shear strength results for a slope stability analysis.)

NOTE 3—Contrary to standard practice, the ISO standard on shear strength properties defines the "angle of friction" as the secant angle, not the Mohr-Coulomb angle.

7.6 In simple cases where the shear strength data is actually linear, the linear failure envelope constructed by the testing laboratory should be an accurate representation of the available shear strength. However, Fox and Stark (**11**) and Giroud et al. (**18**) show that interpretation of the failure envelope may not be as straightforward if the data indicate curved or multilinear failure envelopes. Fox and Stark presented several common models used to characterize GCL shear strength envelopes (Fig. 4), which can also generally apply to many geosynthetic interfaces. Additionally, several studies have shown that many geosynthetic interfaces exhibit nonlinear failure envelopes over a large range of normal stresses, including textured geomembranes/nonwoven geotextiles (**19, 20**), smooth geomembranes/clays (**21**), reinforced GCLs (**22-24**), and textured geomembranes/GCLs (**10, 23**). In such cases, the linear shear strength parameters (c and δ) reported by the laboratory may not be appropriate, or may only be appropriate for a portion of the data. For example, Fig. 5 from Giroud et al. (**18**) shows an example set of geosynthetic shear test results, along with three possible "best-fit" lines through the data set. Line #1 appears to provide a good approximation of shear strength at large normal stress values. However, if considering low normal stresses, Line #1 would greatly overestimate the available shear strength. Use of Line #1 in a slope stability analysis for an application expected to be under low normal stresses would therefore be unconservative. Using Line #2 would accurately depict shear strength for low normal load applications, but would overestimate shear strength at high normal stresses; also an unconservative, and potentially dangerous approach. Line #3, the least-squares regression through all of the data points, may lead to either overly conservative or under conservative estimates, depending on the normal stress considered. Line #3 should only be used for the middle range of data. The intent of this example is to demonstrate that it would be unwise to characterize a nonlinear data set with a single best-fit straight line. To address this difficulty, Giroud proposed the use of a curved, hyperbolic failure envelope to accurately fit the data at all normal stresses.

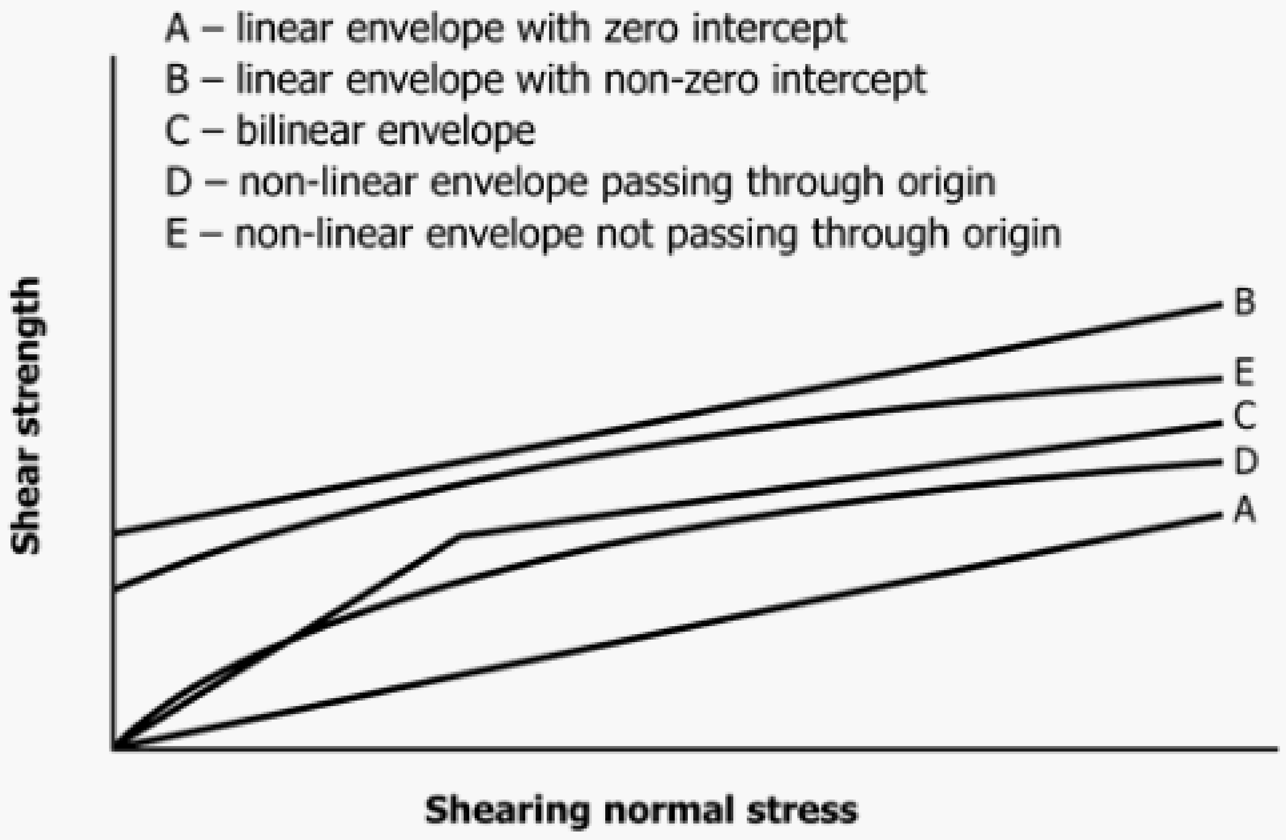

FIG. 4 Typical Failure Envelope Shapes for GCL and Geosynthetic Interfaces (Fox and Stark (11))

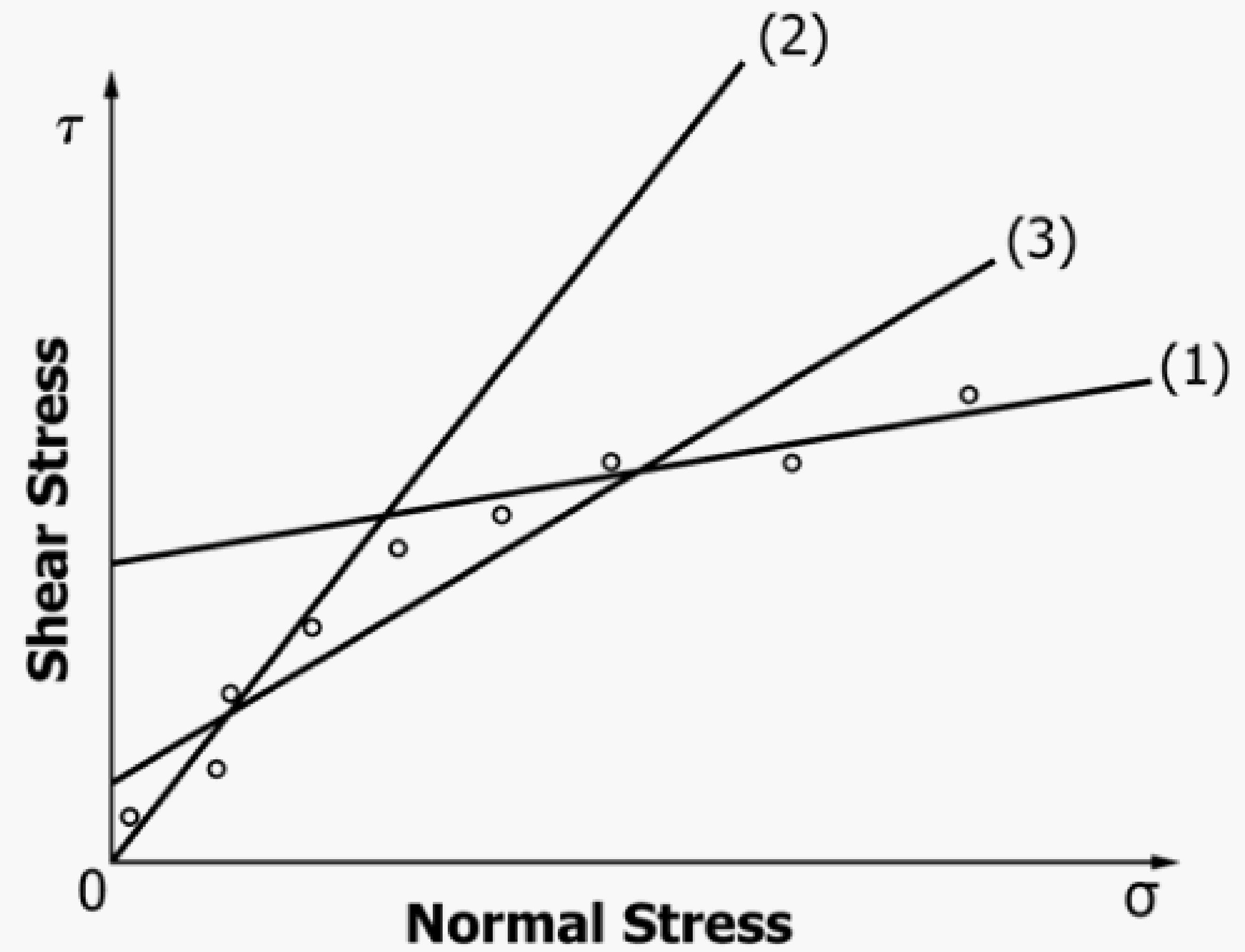

FIG. 5 Linear Approximations of Interface Shear Strength. Best fit straight line for (1) high normal stresses, (2) low normal stresses, and (3) all laboratory data points (Giroud et al. (18))

7.7 It is important to note that Giroud's best-fit Line #3 in Fig. 5, as well as many of the models presented by Fox and Stark in Fig. 4, include a non-zero y-intercept (cohesion or adhesion). Common methods of interpreting cohesion and adhesion are discussed further in Section 8.

8. Interpretation of Cohesion or Adhesion

8.1 As discussed in Section 7, laboratory shear test reports involving geosynthetics often indicate a non-zero y-intercept (cohesion or adhesion). The ultimate decision whether to include the reported cohesion/adhesion in a slope stability analysis rests with the design engineer. In geotechnical engineering practice, interpretation of cohesion in soils is very project-specific. Cohesion values for sands, non-plastic silts, and normally consolidated clays are generally approximated as zero (**5**). Although overconsolidated clays or cemented sands may exhibit cohesion, engineers often choose to ignore this term because it may not be reliable for long-term conditions (**16**). Regarding the interpretation of cohesion/adhesion when geosynthetics are involved, Dixon et al. (**25**) state, "While it is common practice in many applications involving soil to ignore cohesion or adhesion values in design, this approach is not recommended for geosynthetic interfaces. Apparent adhesion values can be considered in design of structures that incorporate interfaces with a true strength at zero normal stress (for example, Velcro type effect between nonwoven needlepunched geotextile and textured geomembranes)." As discussed in the GRI White Paper #11 by Koerner and Koerner (**26**), several geosynthetics and geosynthetic interfaces have been shown to exhibit cohesion or adhesion:

8.1.1 Textured polyethylene geomembranes (HDPE and LLDPE) against geotextiles or soil.

8.1.2 Smooth geomembranes (LLDPE, fPP, EPDM, and PVC) against other geosynthetics or soil.

8.1.3 Drainage geocomposites, where geotextiles are thermally bonded to geonets.

8.1.4 GCL internal shear strength, where needlepunching provides internal reinforcement of the bentonite layer.

8.1.5 Selected geosynthetic-soil interfaces (for example, cohesive soil against a nonwoven geotextile) where the interface friction between the two materials is high enough to force the failure plane into the soil.

8.2 Koerner and Koerner concluded that, "If adhesion is indicated by the linear failure envelope associated with one of these interfaces, its use in a stability analysis can be justified."

8.3 Swan (**27**) provides an example of the potential consequences of indiscriminately ignoring cohesion. Fig. 6 presents the results of two sets of direct shear tests reported by Swan: the first between a smooth polyethylene geomembrane and a site soil, and the second between a textured polyethylene geomembrane and the same site soil. The test results show significant adhesion values for both sets of tests. If one were to only look at the reported friction angles (7° versus 3°), the designer would conclude that the smooth geomembrane/soil interface is far stronger and far less likely to slip than the interface between the textured geomembrane and that same soil. However, if both the reported friction angles and adhesion values are considered together, then one would arrive at an entirely different conclusion: the textured geomembrane/soil interface would be far stronger and far more stable, consistent with intuition and past experience.

8.4 Thiel (**1**) offers another perspective: "If we recognize that the values of c and ϕ are only mathematical tools used to describe shear strength over a given normal load range, we can discount statements that advocate that cohesion be ignored."

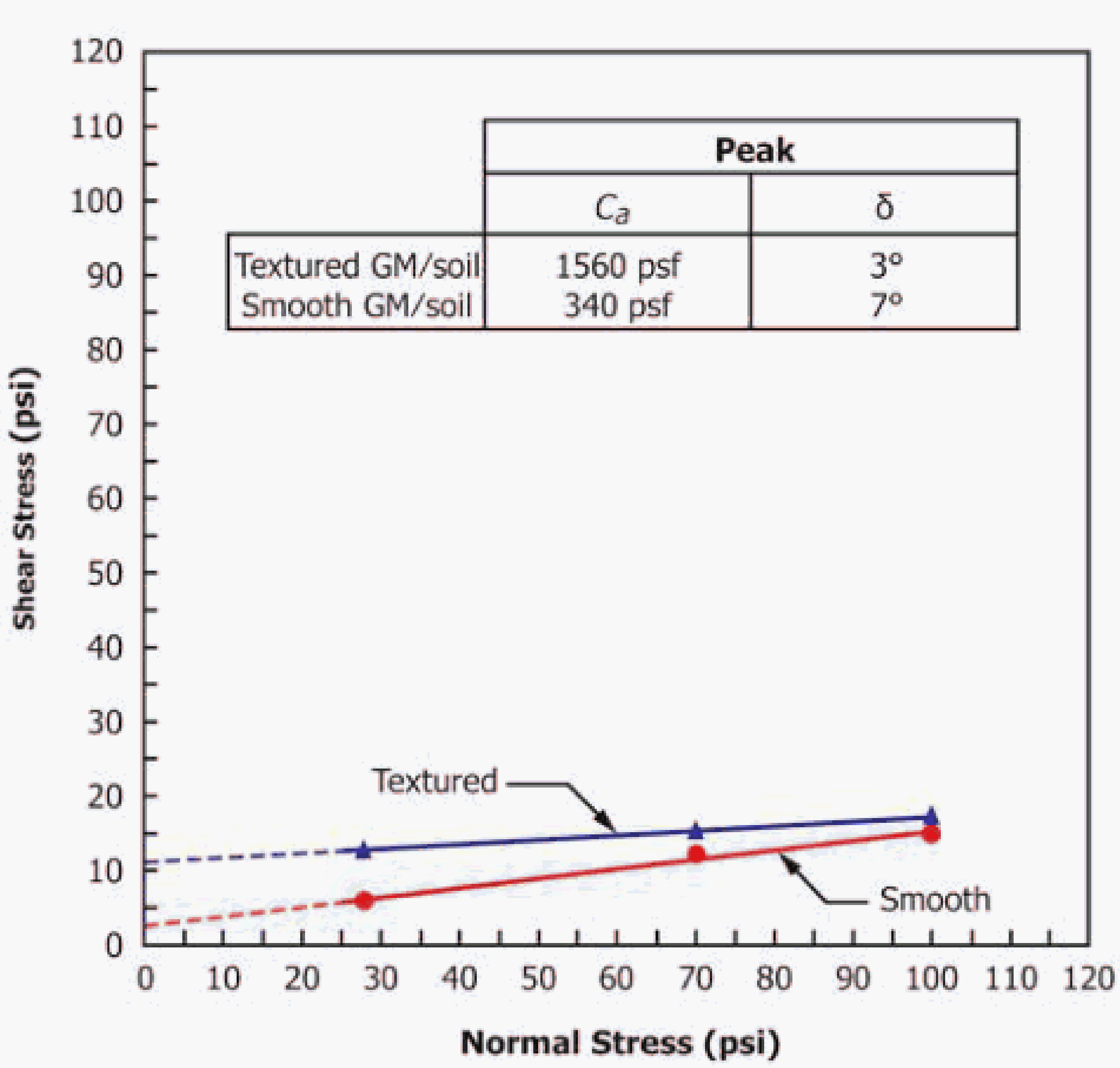

FIG. 6 Interface Shear Results For Two Geomembrane/Soil Interfaces (from Swan (27))

As discussed in 7.7, since the Mohr-Coulomb failure envelope is just a linear representation of data that is oftentimes nonlinear, it would be natural to expect a non-zero y-intercept. Although the cohesion value may not have a true physical meaning for the particular interface tested, its use for design in these cases would still be justified. This concept is shown graphically in Fig. 7, from Giroud et al. (**18**). In this figure, the material's true failure envelope follows a hyperbolic shape, which curves sharply to the origin at low normal stresses. Drawing a best-fit line through the data beyond 20 kPa produces a significant cohesion or adhesion. Disregarding the cohesion would be conservative, perhaps overly conservative to the point that no two materials would be able to meet shear strength requirements.

8.5 Dixon et al. (**25**) raise the question of negative cohesion values. Negative cohesion results have been occasionally reported, the likely result of forcing a best-fit line through limited test data representative of a nonlinear envelope. The state of the practice in these situations is to either force the failure envelope through the origin (resulting in a decreased friction angle), or to re-test.

8.6 Dixon et al. (**25**) also bring the concept of variability into the discussion. There is inherent variability in direct shear tests, due to variability in soils and geosynthetics, as well as equipment calibration and measurement errors. Ramsey and Youngblood (**28**) cite data from the Geosynthetic Accreditation Institute-Laboratory Accreditation Program (GAI-LAP) which shows that direct shear testing protocol produces variation in excess of 15 %. Variations between the measured shear strength value and the actual value will affect both the slope (tan δ) and the intercept (c) of the failure envelope. However, due to the large cost and time commitment associated with shear tests, multiple (for example, replicate) shear tests at each normal stress are rarely performed to enable a statistical analysis of the measured strength uncertainty.

8.7 For the various reasons listed above, Marr (**29**) discourages the common practice of specifying only a δ value for a particular geosynthetic interface ("the material shall have a friction angle greater than ... degrees"). As discussed, many geosynthetic interfaces either have a shear strength consisting of both cohesive and frictional components, or have a nonlinear failure envelope that cannot be described with a friction

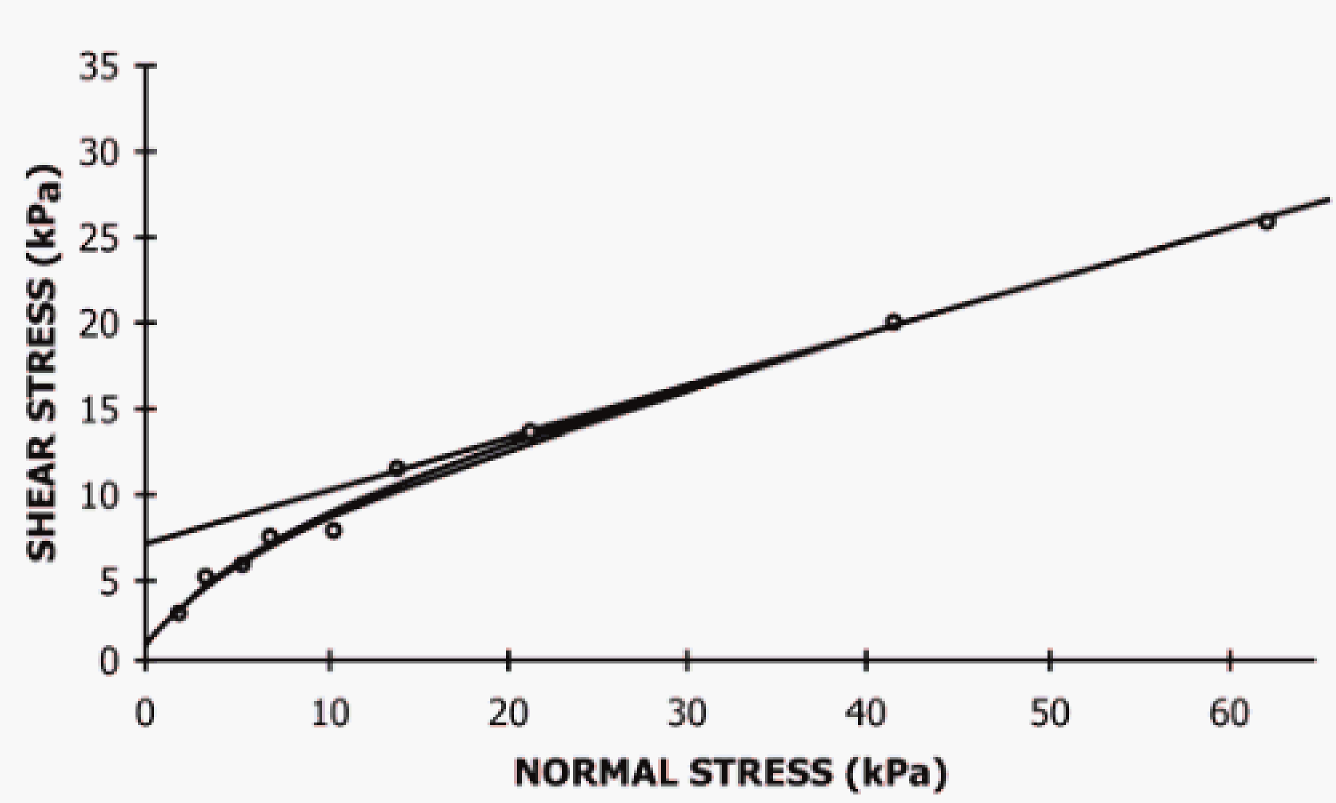

FIG. 7 Linear Approximation of Nonlinear Laboratory Test Results (from Giroud et al. (18))

angle alone. To avoid misinterpretation, Marr offers the following specification as an alternative: "The material shall have a strength greater than that represented by a strength envelope defined by a cohesion of … psf and a friction angle of … degrees when tested under the prescribed conditions over a normal stress range of … to … psf." Another alternative for addressing this limitation is to consider total shear stress, as discussed below.

8.8 Perhaps a more straightforward approach to specifying geosynthetic shear strength requirements, especially for curved failure envelopes, is to require total shear strength values at discrete normal stress values. By specifying total shear strength (in kPa or psf), both cohesive and frictional components are implicitly included. This approach allows one to avoid specifying discrete *c* and δ values, and therefore avoid the complications mentioned above (**26**). According to Fox and Stark (**11**), slope stability software programs may allow a user to enter as many as 20 combinations of shear and normal stress to describe a failure envelope. According to Thiel (**1**), if the slope stability program only allows linear shear strength envelopes, the shear strength can be discretized into a series of straight-line approximations for different normal load ranges.

8.9 A variation of the total shear strength approach involves the use of secant friction angles, introduced in 7.6. Some testing laboratories also report strength results in terms of a secant friction angle, δ_{sec}, defined as:

$$\delta_{sec} = tan^{-1}\left(\frac{\tau}{\sigma}\right) \quad (4)$$

Therefore, one could define a failure envelope as a series of discrete secant friction angles for different normal stresses, as shown in Fig. 8 (**30**). This would be effectively the same as specifying total shear strengths at discrete normal stress values, as described in 8.8.

8.10 In summary, although Test Methods D5321/D5321M and D6243/D6243M call for the testing laboratory to draw a best-fit line through the shear stress-normal stress test results and report values of *c* and δ, it is strongly recommended that the design engineer also evaluate the data to determine the appropriate strength parameters to be used in a slope stability analysis. Many geosynthetic interfaces either have a shear strength consisting of both cohesive and frictional components, or have a nonlinear failure envelope that cannot be described with a friction angle alone.

9. Evaluation of Shear-Displacement Curves

9.1 As discussed in 6.3, the testing laboratory report should include a plot of shear force versus displacement for the materials tested, with the peak shear force and shear force at the end of the test (for example, post-peak or large displacement) identified. It is recommended that the design engineer also review the shear-displacement curves to gain an understanding of the failure mode and to assess the shear test quality (**11**). Soils and geosynthetic interfaces can exhibit two types of stress-strain behavior: brittle and ductile (Fig. 9).

9.2 *Brittle Failure*—Materials that exhibit brittle stress-strain behavior will show a distinct peak shear strength at low displacements, with a decrease in shear resistance at larger displacements until a noticeably lower residual strength is reached (**14**). Brittle shear-displacement behavior (also known as strain-softening or post-peak strength reduction) is characteristic of stiff clays, dense sands, and clays compacted dry of optimum moisture content. Several mechanisms may be responsible for post-peak strength reduction in soils, including clay particle reorientation at the failure plane and soil dilation. Geosynthetic interfaces that exhibit brittle shear-displacement behavior include:

9.2.1 Textured geomembrane interfaces with nonwoven geotextiles, GCLs, or drainage geocomposites. According to Li and Gilbert (**20**), the post-peak strength reduction is caused primarily by polishing, the loss of geomembrane roughness, and asperity with increasing shear displacement.

9.2.2 Reinforced GCLs. According to Marr (**29**), Gilbert et al. (**13**), and Fox et al. (**22**), when testing the internal shear strength of a needlepunch-reinforced GCL, post-peak strength reduction is caused by rupture and/or pullout of needlepunched fibers, and reorientation of bentonite clay with increasing shear displacement.

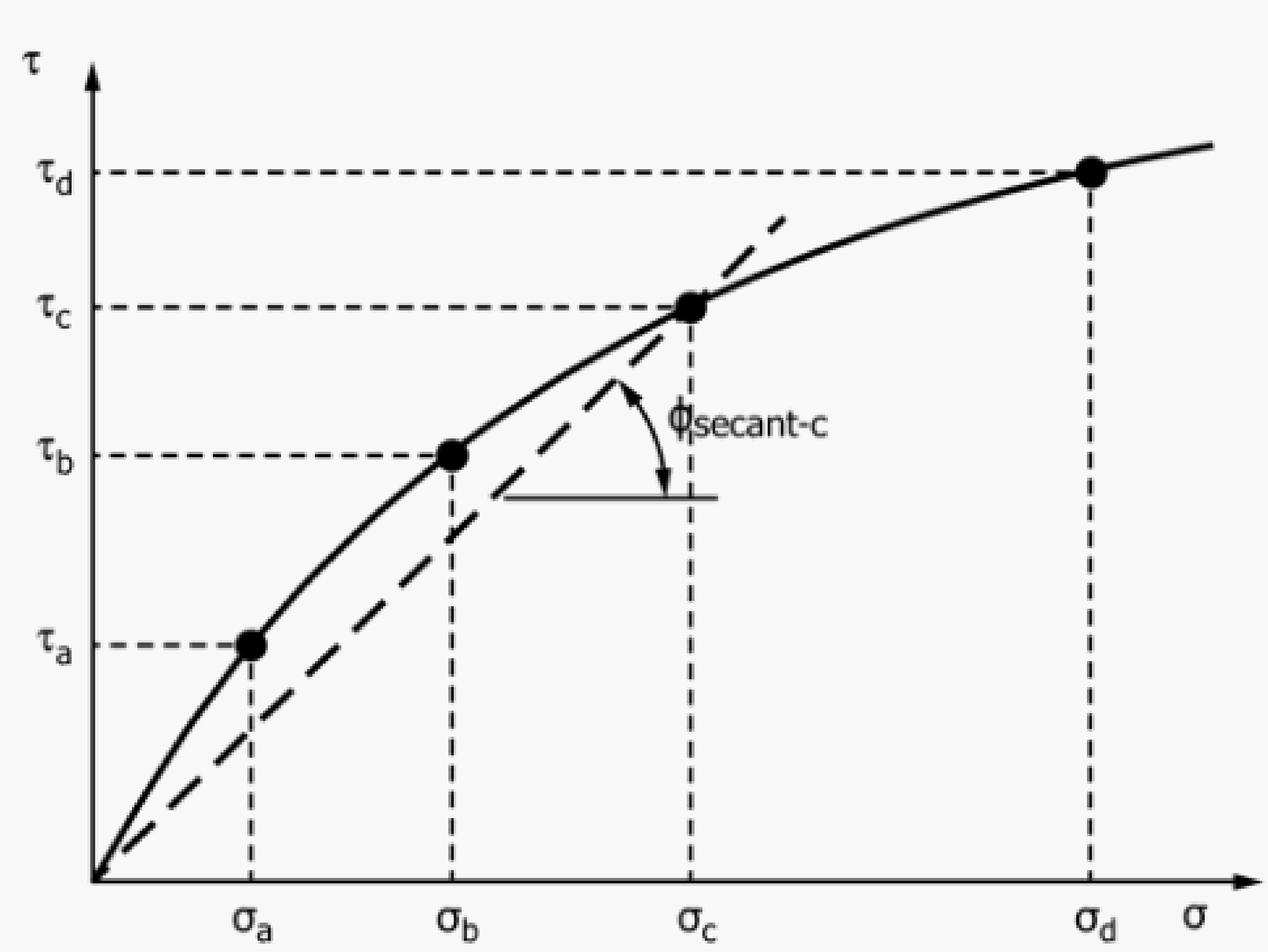

FIG. 8 Construction of a Curved Strength Envelope using Discrete Secant Friction Angles (USACOE ( ))

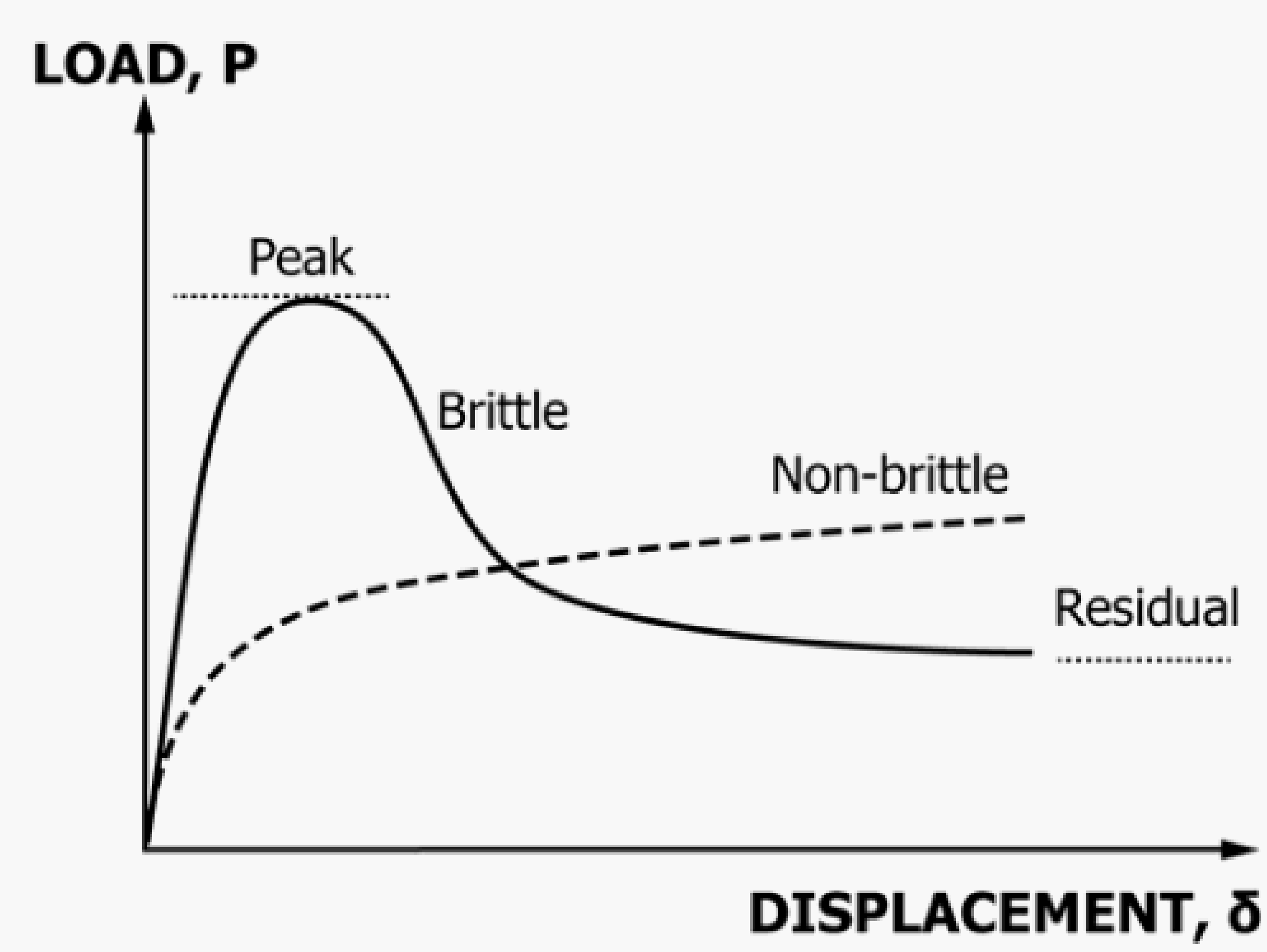

FIG. 9 Shear-Displacement Curves for Brittle and Ductile Materials (Abramson et al. (14))

9.2.3 Geosynthetic interfaces with stiff clays or dense sands, where the failure plane is forced into the "brittle" soil layer.

9.3 *Ductile Failure*—Materials that exhibit ductile, or non-brittle, stress-strain behavior will show very little reduction in strength as displacement increases beyond the peak. Ductile shear-displacement behavior is characteristic of most soft clays, loose sands, and clays compacted wet of optimum moisture content. Geosynthetic interfaces that exhibit ductile shear displacement behavior include:

9.3.1 Smooth geomembrane interfaces with other geosynthetics or soils.

9.3.2 Nonwoven geotextile interfaces with other geotextiles, GCLs, or drainage geocomposites.

9.3.3 Geosynthetic interfaces with soft clays or loose sands, where the failure plane is forced into the "ductile" soil layer (includes undrained tests involving clayey soils).

9.4 With these general relationships in mind, Bove (**31**) suggests that one can use the shape of the shear-displacement curve as a quick indication of the failure mode. For example, consider Fig. 10, which shows a series of shear-displacement curves associated with the interface between a fully hydrated reinforced GCL and a site soil, tested at high normal stresses. In this example, the shapes of shear-displacement Curves 1A and 1B suggest a ductile failure, indicative of sliding between the woven geotextile side of the GCL and the soil subgrade. Shear-displacement Curve 1C, on the other hand, which was obtained at the highest normal stress (958 kPa or 20 000 psf), has a distinct peak, followed by a sharp reduction in strength. This shape suggests a brittle failure, such as rupturing and/or pullout of the needlepunched reinforcing fibers in the GCL. Examination of the sheared samples confirmed that partial internal failure of the GCL had occurred during Test 1C. (Reinforced GCLs subjected to such high normal stresses can be specified with a higher peel strength to increase the amount of needlepunch reinforcement and reduce the potential for internal failure.) Other failure modes that may be apparent from the shear-displacement curves include "rolling" of loose sand or gravel at the interface (resulting in no distinct peak or residual behavior) and embedding of a geosynthetic in the adjacent soil (resulting in a distinct peak but no uniform residual behavior) (**31**).

NOTE 4—The slight "bumps" in the shear-displacement curves at small displacements (<0.1 in.) in Fig. 10 are commonly seen in shear tests, and are believed to represent sliding of the superstrate or substrate at the beginning of shearing.

9.5 Fox and Stark (**11**) recommend that the shear-displacement curves also be used to assess the quality of shear test results. Good quality shear-displacement relationships should exhibit smooth, continuous transitions from the start of loading to peak shear strength and then to large displacement shear strength. The appearance of double peaks, discontinuities, and large undulations suggest problems may have occurred during shearing, and that a repeat test may be warranted. The shear-displacement curves can provide an indication of slippage along gripping surfaces during testing. For example, if tests involving brittle interfaces where distinct peaks and sharp post-peak drop-offs are expected (for example, internal strength of GCLs, textured geomembrane interfaces) instead show unusually wide peaks or little post-peak strength reduction, there may have been slippage along the specimen gripping surfaces during shearing (**11, 32**). As shown in 11, slippage along the gripping surfaces can yield inaccurate results that would predict lower peak shear strengths and higher large displacement shear strengths. Suspected problems can be verified by inspecting the failed samples, as discussed in Section 11.

10. Comparison of Shear Results to Historical Data

10.1 Shear results can also be evaluated by comparing them with past test results obtained with similar materials under

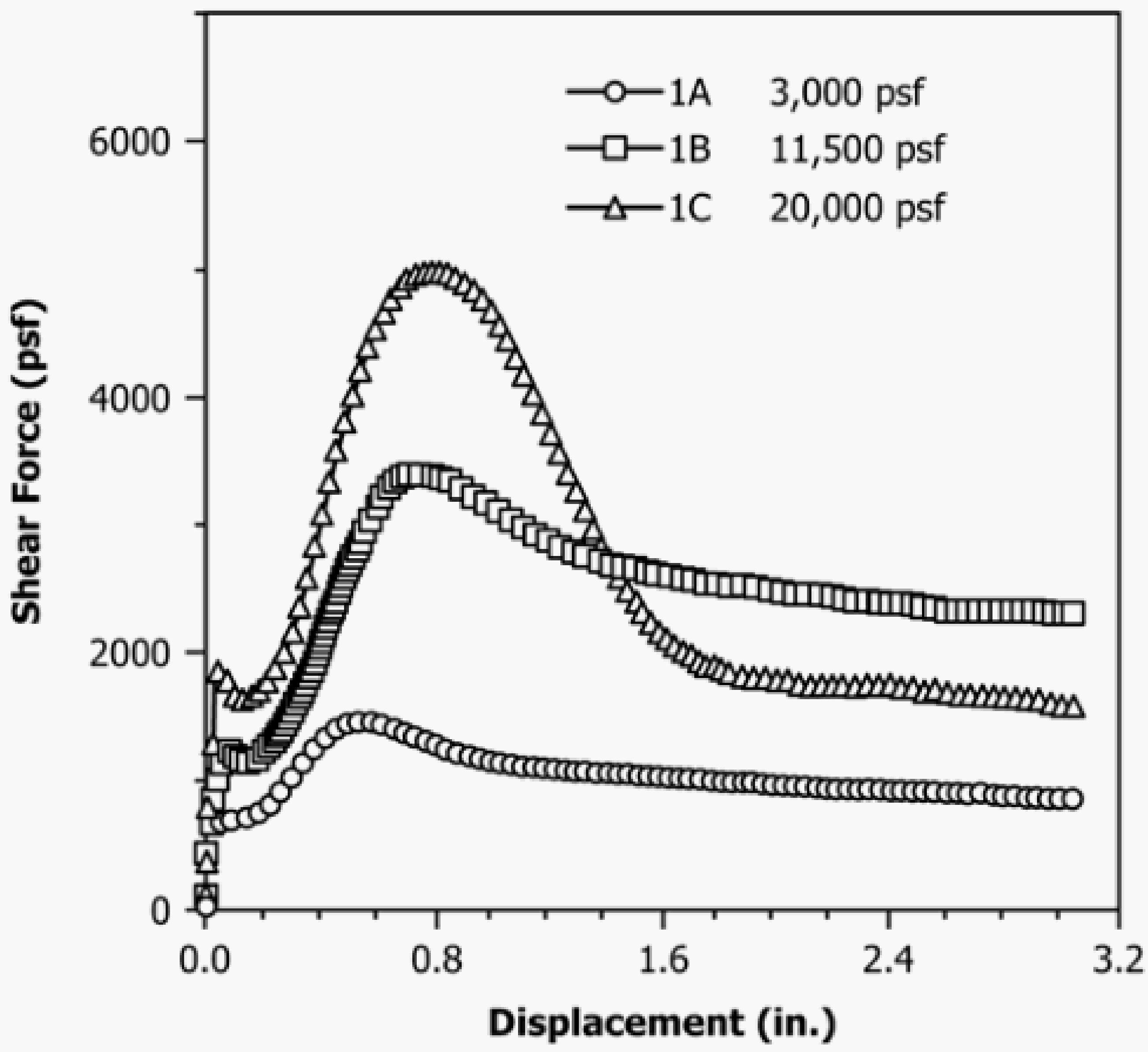

FIG. 10 Example Shear-Displacement Curves for a GCL/Soil Interface Shear Test at High Normal Stresses (Curves 1A and 1B indicate sliding failure between the GCL and soil subgrade; Curve 1C indicates GCL internal failure)

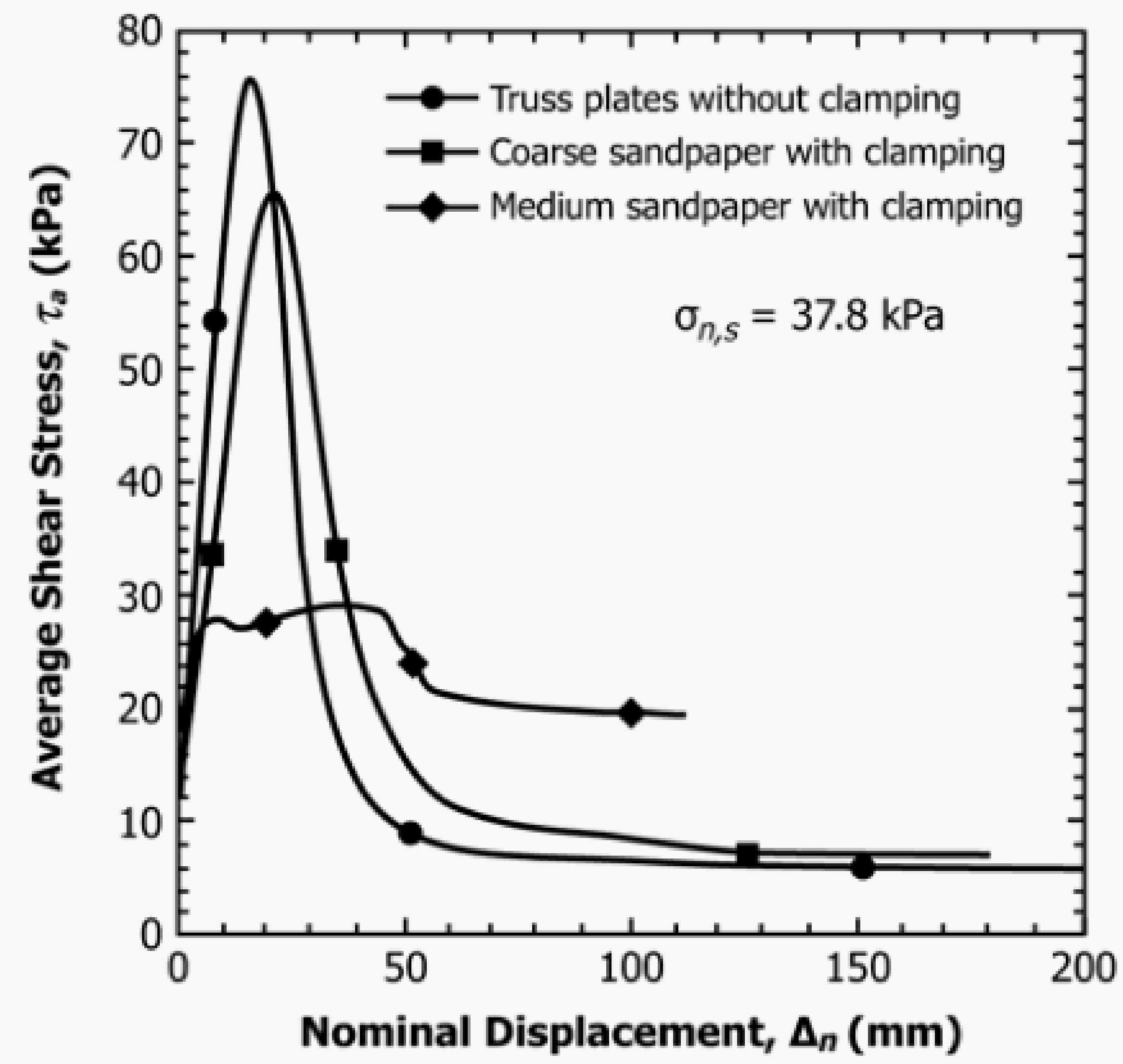

FIG. 11 Effect of Specimen Gripping/Clamping System on Measured Stress-Displacement Relationships for Internal Shear of a Needlepunched GCL (Fox and Stark (11))

similar test conditions. Sources of geosynthetic shear data in the literature include: (**19**, **13**, **22**, **10**, **24**, **33**, **23**, **34**). In addition, many geosynthetic manufacturers maintain databases of past shear strength results involving their products.

10.2 However, engineers are cautioned that historic shear strength results are in no way guaranteed minimum values that will be achieved on every project, under all conditions. There are many variables that can influence shear strength results, including:

10.2.1 *Normal Stress*—As discussed in 7.7, over a wide range of normal stress, failure envelopes for many geosynthetic interfaces are curved, meaning that shear strength parameters obtained at one particular range of normal stresses will not be appropriate for all normal stresses.

10.2.2 *Geomembrane Type*—Hillman and Stark (**35**) showed that smooth flexible geomembranes, such as PVC, are expected to have greater interface friction than smooth HDPE geomembranes. When compared to textured geomembranes, smooth geomembrane interfaces are expected to have lower peak shear strengths, but smaller decreases in post-peak strength.

10.2.3 *Geomembrane Texturing*—In order to improve friction between geomembranes and adjacent soils or geosynthetics, geomembranes can be manufactured with surface texturing. In North America, the two most commonly used geomembrane texturing processes are co-extruded texturing and embossed texturing. Testing performed by Hebeler et al. (**36**) on geomembrane/nonwoven geotextile interfaces over a large range of normal stresses (0.4 to 312 kPa) showed differences in behavior between the two types of surface texturing as a function of normal stress. At low normal loads, the embossed texturing did not exhibit as much of a "Velcro," or hook-and-loop, effect with a nonwoven geotextile when compared with the co-extruded textured geomembrane. However, at high normal loads, the embossed texturing showed higher shear strength with the nonwoven geotextile when compared with the co-extruded texturing. Blond and Elie (**17**) showed that for interfaces involving co-extruded textured geomembranes, the key factor influencing interface shear strength was asperity height, with an optimum value of 20 mils. However, McCartney et al. (**37**) found that co-extruded geomembrane asperity height was highly variable.

10.2.4 *GCL Type*—Fox et al. (**22**) demonstrated that reinforced GCLs had much greater peak internal shear strengths than unreinforced GCLs. The residual (large displacement) shear strength values appeared independent of product type, and were consistent with past test data for fully hydrated sodium bentonite.

10.2.5 *GCL Moisture Content*—Several researchers, including Shan and Daniel (**15**) and Fox and Stark (**11**) discuss the dramatic effect of moisture content on the internal shear strength of GCLs. Because hydrated bentonite can extrude through the geotextile component of the GCL, interface shear strength is also influenced by moisture content (**38**).

10.2.6 *GCL Hydration and Consolidation*—Fox et al. (**22**) showed that if GCL samples are sheared before they are fully consolidated, positive pore pressures may develop within the bentonite, likely resulting in lower measured internal shear strengths. Fox and Stark (**11**) showed that if consolidation loads are applied suddenly to a hydrated GCL specimen, bentonite can extrude through the supporting geotextiles and smear onto adjacent materials, creating a slippery interface.

10.2.7 *Soil Moisture Content and Density*—Stark and Poeppel (**21**) found that for geomembrane/clay interfaces, shear strength increases with decreasing clay plasticity and water content. In research into the behavior of mechanically stabilized earth structures, Khoury et al. (**39**) found that the peak shear strength of a soil-geotextile interface increased with soil suction.

10.2.8 *Geotextile Type*—Stark et al. (**19**) found that geotextile mass per unit area, polymer composition, fiber type, fabric style, and calendaring can influence the peak shear strength between textured geomembrane/nonwoven geotextile interface.

10.2.9 *Displacement Rate*—As shown by Obermeyer et al. (**12**), very slow displacement rates may be needed when shearing GCLs and high-plasticity clay soils to ensure that positive pore pressures do not develop during shearing, and that conditions are truly drained. If tests involving GCLs or clays are sheared too quickly, test conditions could be undrained, allowing excess pore water pressures to develop, and likely producing erroneous or unrepresentative results.

10.2.10 *Wetting the Interface*—As shown in a case study by Thiel (**40**), wetting of geosynthetic interfaces, especially those involving smooth geomembranes, can result in critical differences in measured shear strength compared to the same interfaces tested without first wetting the interface. Since many geomembrane installations in the field result in condensation water on the bottom surface of the geomembrane before it is covered with soil, spraying the interface with water before assembly is recommended, unless explicitly otherwise directed by the engineer.

11. Post-Test Sample Inspection for Indication of Failure Mode or Test Problem

11.1 Test Methods D5321/D5321M and D6243/D6243M recommend that, at the end of each shear test, the specimen should be removed and inspected carefully to identify the surface on which failure occurred and the general nature of the failure. For tests involving GCLs or soils, final water contents should be taken after shearing to assess the level and uniformity of hydration that was achieved. Additionally, the design engineer may request that the testing laboratory return the sheared geosynthetic specimens along with the test data for closer evaluation. Indications of the failure mode could include (**31**):

11.1.1 Abrasion or "polishing" of geomembrane surfaces.

11.1.2 Combing or tearing of nonwoven geotextiles.

11.1.3 Signs of soil "plowing," where geosynthetics on one portion of the shear box plow into the soil in the other half.

11.1.4 Bentonite extrusion.

11.1.5 Signs of GCL internal failure, including fiber pullout or rupture of reinforced GCL.

11.1.6 Embedding of geonet or soil particles into geomembrane.

11.1.7 Signs of failure within the soil substrate, such as a thick film/layer of soil retained on the surface of the adjacent geosynthetic.

11.1.8 Signs of drainage geocomposite internal failure, such as delamination of fabric from the geonet or damaged geonet ribs.

11.2 Indications of a test problem include elongation or tearing at the clamps, indications of the nonuniform distribution of the normal stress, or specimen damage at a location other than the intended shear surface, which may indicate that the result should be discarded and the test repeated (**22**).

12. Evaluation of Multi-Interface Test Results

12.1 Increasing the gap between the upper and lower halves of the direct shear box can allow for testing multiple interfaces at one time. These tests are also known as "sandwich" or "floating" tests, Marr (**29**). The advantage of multi-interface tests is that, if performed properly, they will yield peak and large displacement shear strength for the entire liner system in just one test (**11**). However, drawbacks of multi-interface tests include:

12.1.1 Shear strength parameters are only obtained for the failure surface, and not for the other materials or interfaces, some of which may have moved and may have been close to failure (**11**).

12.1.2 Different interfaces will be the critical failure surface at different normal loads, and the discernment of this may be difficult to interpret with multi-interface tests (**16**).

12.1.3 The shear displacement measured during a multi-interface test is the cumulative displacement of all the interfaces and not that of the failure surface alone (**11**).

12.1.4 One of the problems with the direct-shear test method is that the normal force is not evenly applied, and the stress has been shown to vary significantly over the area being sheared. This situation is even further exacerbated when soil exists in the bottom box because its deformations are nonuniform between the center and the sides. The resulting nonuniform normal stress distribution is already problematic for single interfaces. Having multiple interfaces can significantly compound this problem because different interfaces may perform differently under different normal stresses (**16**).

12.1.5 Multiple interfaces can have wrinkling and bunching occur during deformation, which can complicate data interpretation (**16**).

12.1.6 Having soils or GCL as a larger area (typically in the bottom box) will often create problems with "plowing." This is already a challenge with a single interface, and its occurrence may further complicate data interpretation of multi-interface tests (**16**).

12.1.7 If any of the multi-interface elements moves with the top box during the test, it is imperative to check that they do not experience any interference with elements of the bottom box (**16**).

13. Keywords

13.1 direct shear; GCL; interface shear strength; internal shear stress; Mohr-Coulomb friction angle; Mohr-Coulomb shear strength envelope; performance test; secant friction angle

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