Geocomposite Lamination Strength Design and Testing: A New Approach

Richard Thiel, P.E.[1], and Douglas Gatrell, P.E.[2]

[1]Thiel Engineering, Oregon House, CA, email: richard@rthiel.com
[2]GHD, Farmington Hills, MI, email: Doug.Gatrell@ghd.com

ABSTRACT

Recent project experience that included continuous peel testing across the roll width of geocomposite drainage materials indicates the variability of peel strength is greater than expected. The authors believe that the number of specimens (five) taken to evaluate peel strength across a roll width in accordance with ASTM D7005 is inadequate, and even potentially misleading. This is important because of the relation of peel strength to shear strength and slope stability. This paper presents the basis of a quantitative approach that could be used for specifying and testing for the required peel strength, taking into account manufacturing variability. The method proposes a measurement of the continuous peel profile across the roll width of geocomposite samples. The approach establishes two design criteria: the maximum percentage of total area that would be allowed to become un-bonded during construction, and a probabilistic level of reliability for construction conditions that will meet the first criterion.

BACKGROUND

A drainage geonet geocomposite, as discussed in this paper, consists of a geonet core to which a geotextile is heat-laminated on either one or both sides.[1] The lamination process used to manufacture geocomposites involves heating the geonet surface immediately prior to bringing it into contact with the geotextile(s) via two counter-rotating rollers. The source of heat is either electrically heated wedges or a gas flame. The lamination mechanism is that the geotextile fibers are then pushed by the rollers into the partially molten polyethylene. When the polyethylene cools, the fibers are mechanically locked into the outer surface of geonet. Although the lamination is thermally induced, the actual lamination mechanism can be considered mechanical in nature. The amount and distribution of heat, the temperature of surroundings, air-circulation, cleanliness, and roller pressure can affect the strength and uniformity of bonding between the geotextile and the geonet. Low temperatures and pressures will maintain maximum transmissivity but could result in weak lamination strength. Higher temperatures and pressures will increase lamination strength, but will reduce transmissivity. The adequacy of the thermo-mechanical bond is typically checked with an index peel test conducted in accordance with ASTM D7005, Standard Test Method for Determining the Bond Strength (Ply Adhesion) of Geocomposites.

Thiel and Narejo (2005) described a case history of a geocomposite that had poor lamination strength, which resulted in a slippage during construction (Figure 1). That paper attempted to correlate index peel strength with shear strength, and concluded that on projects where

[1] Note that this discussion is for geonet drainage geocomposites manufactured in North America that have a polyethylene geonet core with heat-bonded polypropylene geotextiles. This discussion does not apply to geocomposite lamination in Australia or other places that use a hot melt adhesive, rather than thermal bonding. Also, some other places in the world may use polyester geotextiles rather than polypropylene. While the approach discussed herein may apply to polyester geotextiles, as well, the issue has not been studied by the authors.

shear strength was critical, a minimum peel-strength specification of 1.0 pound per inch (ppi) [180 N/m] MARV (minimum average roll value) be recommended as an industry standard.

Figure 1 – Geocomposite delamination failure in the field after attempted soil covering.

Recent project experience that included continuous peel testing across the roll width of geocomposite materials has shed new light on the issue of peel strength between geotextiles and geonets that are heat-bonded together. Figure 2 shows a photo of a sample where contiguous 4-inch [101.6 mm] wide specimens were cut in a checkerboard pattern across the roll width. Depending on the roll dimensions, a total of 38-42 samples can typically be cut from a roll. Figure 2 also presents an example of the peel test results, where each specimen test was conducted in accordance with ASTM D7005, plotted across the roll width. This plot is called a "peel strength profile". This particular sample yielded 38 specimens that resulted in an average peel strength of 1.65 ppi [297 N/m] with a standard deviation (SD) (assuming normal distribution of data) of 1.32 ppi [237.6 N/m], not counting the un-bonded edges. It is noteworthy that even though the average value of 1.65 ppi [297 N/m] is soundly above the target specification of 1.0 ppi [180 N/m], 12 of the 38 specimens (32%) are below the target specification. It is also noteworthy that when the standard deviation is close to the average value, and negative values are not allowed, then the data is probably not normally distributed. Indeed, statistical analyses conducted by the second author indicated that most of the peel strength data fits a lognormal distribution, and not a normal distribution.

Thiel (2018) presented examples of peel strength profiles on samples from both sides of ten different rolls on two separate projects, yielding 20 data sets. It is interesting to note that the data sets from both projects produced similar statistics, with the average peel strengths for all samples of about 2.3 ppi (414 N/m) and with SD's of about 1.25 ppi [225 N/m] (assuming the data is normally distributed). The question is: given the high variability of geocomposite peel strengths, *what are the appropriate acceptance criteria for the peel strength profile*? The high variability of the lamination process, which is the subject of this article, was not known by the primary authors until recently.

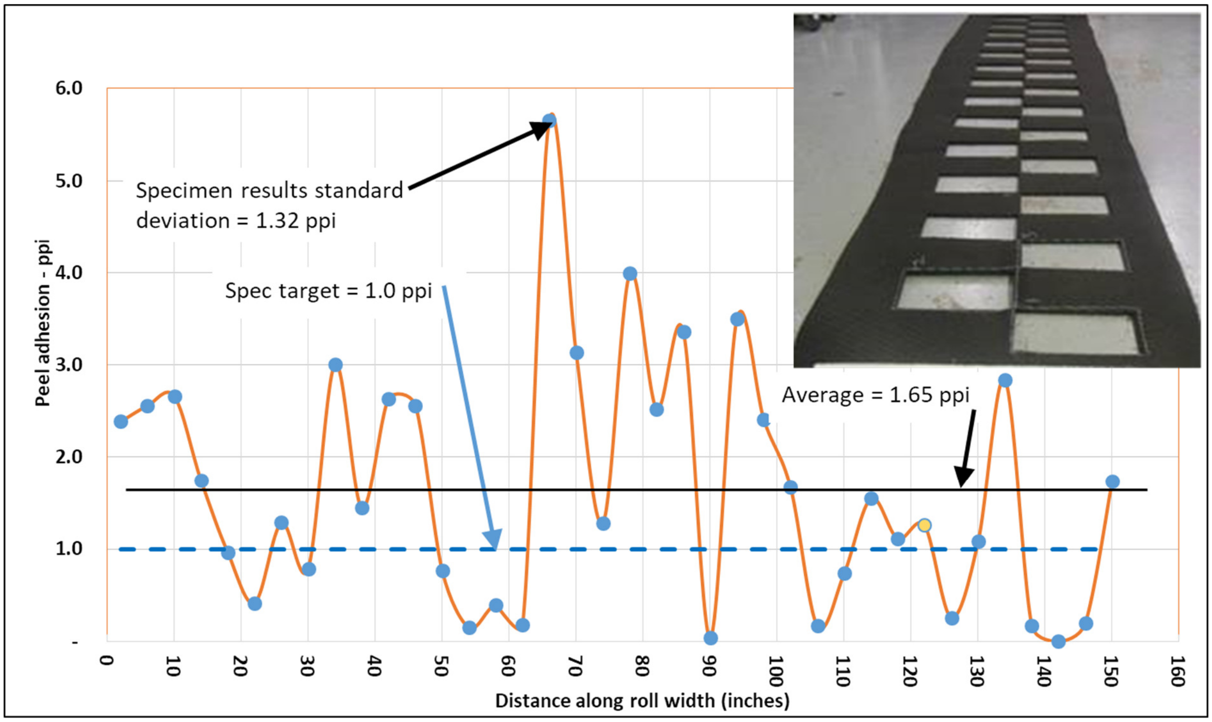

Figure 2 – Example "peel strength profile" sample showing specimen cutouts, and test results for geocomposite across the roll width.

Interestingly, the Thiel and Narejo (2005) article also mentioned that the problematic geocomposite described in the case history had passing peel-test results from both MQC and CQA conformance testing, and none of the extremely poor lamination was picked up in the laboratory testing. While the article did not dwell on this point, there was obviously a significant disconnect between the laboratory test results that indicated well-bonded material versus the reality of the field-deployed material that had numerous large "holidays", where "holiday" is a synonym for "blisters" or "locally unbonded areas within the zone of lamination". This large discrepancy was only fully appreciated after there was a slippage during construction. A similar discrepancy between the results from the standard 5-specimen protocol of ASTM D7005 testing compared with results from a complete peel strength profile was observed for the projects represented in Thiel (2018).

Based on these experiences, the authors believe that the current industry approach for specifying and evaluating geocomposite peel strength *is inadequate* for projects where slope stability is important. *More direct industry guidance needs to be provided* to design practitioners, manufacturers, testing laboratories, and CQA inspectors that takes into account the observed high variability of peel strength in the transverse roll direction. This paper is an attempt to provide an engineering approach for specifying and testing this interface.

Practical Note Concerning Orientation of Weak Zones. While Thiel and Narejo (2005) focused attention on the presence of isolated "blisters" or "holidays" producing weak zones on the geocomposite interface, practical field and manufacturing experience indicates that the concept of “blisters” is usually related to a relatively continuous machine-direction weak zone rather than an isolated spot. While it is possible that isolated blisters do occur, the primary issue is with long continuous weak zones of varying width in the machine direction. In general, there is more inherent variability in the manufacturing technique across the roll width, as opposed to greater uniformity in the machine direction.

Goal. The goal of this paper is specific: what is an adequate specification for the peel strength between the geotextile and geonet components of a geocomposite drainage material to provide reliable slope stability at this interface during and after construction, and how can it be verified? This paper is not a critique of manufacturers. The products that they deliver are ingenious fabrications borne of the attempts to deliver industry solutions using clever manufacturing techniques. As designers and specifiers, we are obliged to understand the limitations of any products that are being used, and provide minimum required specifications that take those limitations into account. What is being newly recognized in this paper is the high level of *variability* of the bonded peel strength, which must be evaluated in the design.

KEY ASSUMPTIONS

The stated goal implies that peel strength is an adequate indicator of reliable shear strength of the geotextile-geonet interface for both construction and long-term conditions. A key assumption of this paper is that peel strength at any location is directly related to the peak bonded shear strength of the geotextile-geonet at that location for a given normal load. This assumption is described in more detail later.

Another key assumption is that the <u>mobilized</u> shear strengths on un-bonded and bonded areas are uniform and not variable for a given normal load, to meet the following conditions:

- The mobilized shear strength on un-bonded areas is assumed equal to the post-peak frictional strength of the geotextile-geonet interface.
- The mobilized shear strength on the bonded areas is uniform and is what is needed to provide an equilibrium force balance. Therefore the mobilized strength of the bonded areas is generally less than the peak strength, except right at the edges of the bonded areas where there is a transition from un-bonded to bonded areas, at which point the mobilized strength is approximately equal to the localized peak strength. These are the locations where the “threshold peel strength” occurs, which is a key concept for the proposed design approach, as described later.

APPROACH

In the case under consideration in this paper, the critical condition occurs when equipment is operating on the slope being covered with soil on top of the geocomposite. Acceleration and deceleration of the equipment causes dynamic forces parallel to the geotextile-geonet interface, which may induce progressive delamination of the bonded interface. Indeed, this potential was recognized by Bachus et al. (2004) in The GSE Drainage Design Manual, which states: “…designers should consider that progressive delamination could occur under aggressive construction conditions with cyclic loads.” This typically occurs during construction in a veneer manner, although the principles discussed in this paper could be extended to dynamic forces caused by earthquakes to both veneer systems and deeper buried systems. This paper is focused on loading during construction. A subject for future papers might be the long-term shear loading that would occur on bottom liners of large piles such as landfills and heap leach pads. The equation for factor of safety (*FS*) against delamination is simple:

$$FS = \frac{\sum R}{\sum D} \tag{1}$$

Where *R* represents *resisting forces*, and *D* represents *driving forces*, over the area of concern. In our case both of these forces would be *shear* forces acting on the geotextile/geonet interface. The forces are discussed herein with units of pounds over a unit area of one square foot (sq ft), thus providing units of *stress* in pounds per square foot (psf) [kPa]. Units of *force in lbs* are equivalent in magnitude to the units of *stress in psf* since they occur over a total assumed unit area of one sq ft (0.093 m^2). The model is a simple localized "infinite slope" model with no accounting for toe buttressing or tensile strength of the geotextile.

Concept of "Threshold Peel Strength". If there are some weak zones in the bonded area due to manufacturing variability, those weak zones might become un-bonded, or delaminated, under the forces caused by construction equipment. We define here the maximum percentage of the total area that is allowed to become un-bonded during construction operations as "$A_{u\text{-}max}$".

A key concept of the approach developed in this paper is that a specific peel strength, called the "threshold" peel strength, $p_{s\text{-}thresh}$, can be associated with $A_{u\text{-}max}$. The logic and assumptions that support this relationship are as follows:

- During the process of construction, the soil and construction equipment are supported at the geotextile-geonet interface by the shear strengths of both the bonded and un-bonded areas. It is assumed that while the *mobilized* shear strengths of the bonded and un-bonded areas are different, they are each uniform in themselves, and not variable. That is, the mobilized shear stress of all un-bonded areas is the same wherever the normal loads are the same, and they are not variable where the normal load is constant. And the same can be said of the bonded areas.
- During construction, weakly bonded areas may progressively delaminate, due to the static and dynamic forces of the equipment loading, and become un-bonded areas. As delamination occurs, the demand for shear resistance on the bonded areas increases. If the increased shear demand exceeds the bonded shear strength in any area, then that area will delaminate and become un-bonded.
- At the point that delamination occurs to the extent that $A_{u\text{-}max}$ is reached, there is a unique value of shear strength required to be uniformly distributed over the remaining bonded area. The peel strength associated with that unique value of required shear strength is defined as the "threshold peel strength", $p_{s\text{-}thresh}$.

Establishment of Design Criteria. The essence of the proposed method is to establish two design criteria:

1. Establish a design criterion for the maximum allowable area that is allowed to become un-bonded during construction, denoted as $A_{u\text{-}max}$. This value will be defined as a percentage of the total area.
2. Establish a design criterion for the acceptable risk during construction. This can be defined as a maximum allowable "probability of failure", P_f, where in this case "failure" is defined as the condition when the un-bonded area, A_u , exceeds $A_{u\text{-}max}$. The risk analysis of the probability of failure can also be called a "reliability analysis".

Since the analysis is based on a unit area, then by definition the area that is bonded, A_b, would be $A_b = 1 - A_u$. If the shear strengths of the bonded and un-bonded areas can be estimated for the project-specific loading conditions, then the factor of safety can readily be calculated since the driving forces are relatively well known. The calculations of factors of safety for different conditions are performed as part of the reliability analysis according to the method of Duncan (2000). Note that the design criteria are applied to one side of the geocomposite lamination at a time. The criteria must be repeated for both sides in the case of a double sided geonet geocomposite.

Design Criterion #1: Maximum Allowable Un-bonded Area. The process of manufacturing geocomposites intentionally creates some un-bonded zones along the edges to facilitate seaming. Assuming 6 inches [152.4 mm] of un-bonded width on each side of a 15-ft [4.57 m] wide roll installed with 4-inch [101.6 mm] overlaps, the calculation is 0.67/14.67 = 4.6% of un-bonded area just due to the sides. The calculation is nearly identical for a 14.5-foot [4.42 m] wide roll, resulting in 4.7% of un-bonded area. Nominally, for 6-inch [152.4 mm] wide un-bonded edges and a 4-inch [101.6 mm] seam overlap, we can assume a 5% un-bonded area due to the panel edges.

If there are some weak zones in the bonded area due to manufacturing variability, those weak zones might become un-bonded, or delaminated, under the forces caused by construction equipment. *A key step in the methodology proposed in this paper is to establish a design criterion for the maximum percentage of the total area that is allowed to become un-bonded during construction operations.* This area is designated "$A_{u\text{-}max}$".

The authors believe that it is either unrealistic, or uneconomical, not to allow any delamination past the nominal 5% from the edges, because there are always bound to be a few weak spots in the manufacturing process. So the question is, what is an acceptable amount of delamination?

Bachus et al. (2004) state: "…inadequate or poor quality bonding is indicated by large and/or continuous un-bonded areas in the machine or cross-direction. Small (less than a few inches) localized, random and isolated patches of un-bonded product are acceptable."

Thiel and Narejo (2005) suggested a maximum allowable localized "blister" size on the order of 30 sq ft [2.79 m^2] for critical projects. Assuming a 10-ft [3.05 m] length (approximate contact area of dozer cleats or truck tires) by a 14-ft [4.27 m] wide (ostensibly bonded) area, a 30 sq ft [2.79 m^2] "blister" would represent about 21% of this area, which would be in addition to the nominal 5% area due to the un-bonded edges.

The authors suggest that a maximum allowable post-construction un-bonded area of 25-30% of the total area, which includes the edges, is a reasonable target that could be checked against the design criteria for interface shear stability. Other practitioners will provide their own judgments, or bases for other project-specific values. For example, a more global stability analysis could calculate the prorated interface shear strength for the overall liner system to provide a different determination of what is acceptable for the percentage of unbonded area.

For purposes of this paper, safety and reliability calculations are based on a total un-bonded area of 30%. This means that the final project specification and testing protocol will be directed at the condition of 25% of the intended bonded area to delaminate, allowing that 5% of the total area starts out as edges being intentionally left un-bonded for seaming.

Design Criterion #2: Acceptable Risk. All projects accept some amount of risk, and it is up to the project stakeholders to adopt standards or means by which a project can be accepted as providing the desired level of reliability. Many projects do this by adopting a simple "factor of safety" approach. Other projects perform a probabilistic approach in an attempt to quantify the level of reliability. Some projects look at both factor-of-safety as well as probability of failure. A simplistic mathematical reliability analysis as presented by Duncan (2000) is proposed in this paper as one means by which the issue of geocomposite peel strength can be specified and tested. Four variables are considered in the reliability analysis. Values are estimated for the "most likely value" (MLV), "lowest conceivable value" (LCV), and "highest conceivable value" (HCV) of each variable in accordance with the method described by Duncan (2000), as described later. These four variables are:

- The equipment deceleration rate during construction on the side slopes.
- The relationship between shear-adhesion (psf) and peel strength (ppi), at zero normal load, between the bonded geotextile and geonet.
- The peak frictional shear strength of the un-bonded interface between the geotextile and geonet.
- The post-peak frictional shear strength of the un-bonded interface between the geotextile and geonet.

All other factors going into the analysis, such as slope angle, and static soil and equipment loads, are considered non-variable for a particular analysis.

An acceptance criterion of 98% reliability, as calculated according to the method described by Duncan (2000), is established for the example presented in this paper. This is actually a relatively low bar for many projects because it implies a 2% failure rate (one in 50 chance of failure). However, "failure" in this case is simply defined as the condition when the un-bonded area, A_u , exceeds the value of $A_{u\text{-}max}$. The authors believe this would be acceptable for a small portion of the total area. Other acceptance criteria can be developed on a project-specific basis by the project stakeholders.

Driving Forces. The driving forces would be caused by the weight of the soil, the weight of the construction equipment, and the inertial forces of the construction equipment. The shear transfer of the weight and inertia of the equipment through the contact areas of the equipment with the soil (e.g. tires or tracks) is specific to the type of equipment and drive axles. These forces can be resolved into their perpendicular component vector directions that are normal and tangential to the slope. The tangential component would be summed to provide the total driving force. This is a standard calculation, and an approach for determining these forces under a piece of construction equipment was previously described by Thiel and Narejo (2005). Another excellent discussion of equipment loading on veneer soil slopes is provided by Kerkes (1999).

Resisting Forces. The resisting forces would be provided by the shear strength of the geotextile/geonet interface. The strength of this interface would be the sum of contributions from the bonded and un-bonded areas. For purposes of the analysis, the following assumptions are made:

- The mobilized shear stress on bonded surfaces is uniform in all of the areas that are bonded. The area that is bonded is denoted A_b.
- Areas of the geocomposite that have a peak shear-adhesion strength less than the assumed mobilized shear stress are assumed to delaminate, and are assigned a shear strength equal to the post-peak strength of a geotextile-geonet interface. The area that is un-bonded is denoted A_u.
- The total resisting force (R_T) at the internal geotextile-geonet interface would be the sum of mobilized shear force on the bonded interface area (R_b), plus the mobilized shear force on the un-bonded interface area (R_u), which can be written as:

$$R_T = R_b + R_u \tag{2}$$

If the analysis is performed on a unit-area basis, then we can also state that:

$$A_u = 1 - A_b \tag{3}$$

Shear Strength of the Bonded Geotextile-Geonet Interface. Data for the shear strength of the bonded geotextile-geonet interface as a function of the peel strength and normal load is scarce, and is typically only indirectly tested as a byproduct of some other type of interface testing. Limited data published by Thiel and Narejo (2005) has led the authors to use the following relationship for the bonded shear strength, S_b, in the normal load range of zero to 5,000 psf [240 kPa]:

$$S_b = (p_s \cdot c_a) + N \tan \varphi_b \tag{4}$$

Where S_b is the peak bonded shear strength (psf); p_s is the peel strength (ppi); c_a is the bonded adhesion at zero normal load (psf); N is the normal stress (psf); and φ_b is the peak friction angle of the un-bonded interface. This relationship was derived as follows:

- The term "tan φ_b" represents the baseline peak frictional strength for the un-bonded condition. A data set provided by Koerner and Narejo (2005) indicates that the friction for this interface ranges from 16.7 to 26.6 degrees. Testing data provided by TRI Laboratories, as reported in Thiel and Narejo (2005) indicates that the frictional component of a bonded interface might range from 25.5 to 34.5 degrees. Based on evaluation of the data spread in the references, the authors estimates the "tan φ_b" values of LCV, MLV, and HCV for this interface as 0.25, 0.49, and 0.67, representing friction angles of 14, 26, and 34 degrees, respectively.
- The relationship between the bonded adhesion, c_a, and the peel strength, p_s, was discussed by Thiel and Narejo (2005). In that reference they determined the MLV to be 417 psf [20.02 kPa] of adhesion for each ppi of peel strength. A re-review of that data by the authors suggests LCV and HCV values to be 233 psf/ppi [0.062 kPa/(N/m)] and 750 psf/ppi [0.2 kPa/(N/m)], respectively.

The relationship described above for the peak bonded shear strength should be considered provisional and subject to additional testing and verification. It may also be product-specific. However at this time it is the best information available to the authors.

The resisting force due to the bonded area, R_b, is a product of the mobilized shear stress on the bonded area and the area that is bonded, A_b :

$$R_b = S_b \times A_b \tag{5}$$

Shear Strength of the Un-Bonded Geotextile-Geonet Interface. The un-bonded shear resistance, R_u, is taken as the post-peak strength between the geotextile and geonet, which has been measured by various sources (proprietary testing laboratory database; GRI; and pers. comm. with Dhani Narejo). A range of results from 10 to 21 degrees friction has been measured. Certainly the lower end of this range has been viscerally experienced by many practitioners in the field when they inadvertently stepped on an un-bonded edge and found themselves sliding as if on an ice skate. Based on this information the authors will use a MLV for this interface of 14 degrees friction, a LCV of 8 degrees friction[2], and a HCV of 21 degrees friction. The force R_u (lb) would then be calculated as the product of the un-bonded area (A_u), the normal force (N), and the tangent of the post-peak friction angle (φ_u):

$$R_u = A_u \cdot N \tan \varphi_u \tag{6}$$

STEP-BY-STEP APPROACH

1. Determine normal loading, N, and driving shear force, D, based on project-specific geometry and equipment.
2. Perform continuous peel profile testing on selected samples, which should yield approximately 40 specimen data points per sample.
3. Define $A_{u\text{-}max}$ (e.g. $A_{u\text{-}max}$ = 30% total, comprised of 25% of the original bonded area, plus 5% due to edges).
4. Establish the threshold peel strength, $p_{s\text{-}thresh}$, at the 25th percentile (i.e. 75% of the specimens must be stronger than a given value for the case that $A_{u\text{-}max}$ = 30%) by direct inspection of the data and the cutoff point at the 25th percentile.
5. Calculate MLV's for R_b, R_u, and D.
6. Calculate FS_{MLV}.
7. In accordance with the method described by Duncan (2000):
    a. Calculate FS-values using plus and minus one standard deviation for each of the four variables in the analysis. From these results, calculate the standard deviation and coefficient of variation of the factor of safety.
    b. Calculate the lognormal reliability index, whose result is the z-value in a standard normal distribution that can be looked up in a standard statistical table (or found using the built-in NORMSDIST function in Excel) to determine the reliability value.

[2] The assumed LCV of 8 degrees is 2 degrees lower than the reported range. This judgment decision was made by the author based on similar interface test results between geomembranes and geotextiles.

EXAMPLE CALCULATION

Given:

- A haul truck operates on a 3:1 slope on a 3-ft [0.91 m] thick soil cover. One set of the center-rear tires carries a weight of 53,700 lbs [24,357.91 kg] that is transmitted through the soil and acts over an effective area of 105.1 sq ft [9.76 m^2] at the level of the geocomposite, resulting in normal and shear stresses of 798 psf [38.3 kPa] and 266 psf [12.77 kPa], respectively. The LCV, MLV, and HCV decelerations are estimated at 0.1g, 0.3g, and 0.5g, yielding tangential dynamic forces of 51.1 psf [2.45 kPa], 153.3 psf [7.36 kPa], and 255.5 psf [12.26 kPa], respectively.
- The project design criteria require that $A_{u\text{-}max} \leq 30\%$ at the end of construction, with a 98% reliability. Assuming that 5% of the total area is initially un-bonded along the edges to facilitate seaming, a maximum of 25% of the remaining bonded area is allowed to delaminate as a result of construction forces.
- A continuous peel profile test is performed on a sample yields 39 specimens, with results plotted below.

Find: The expected level of reliability (or its inverse, P_f) that $A_u < 30\%$ at the end of construction for roll (#40) that was tested.

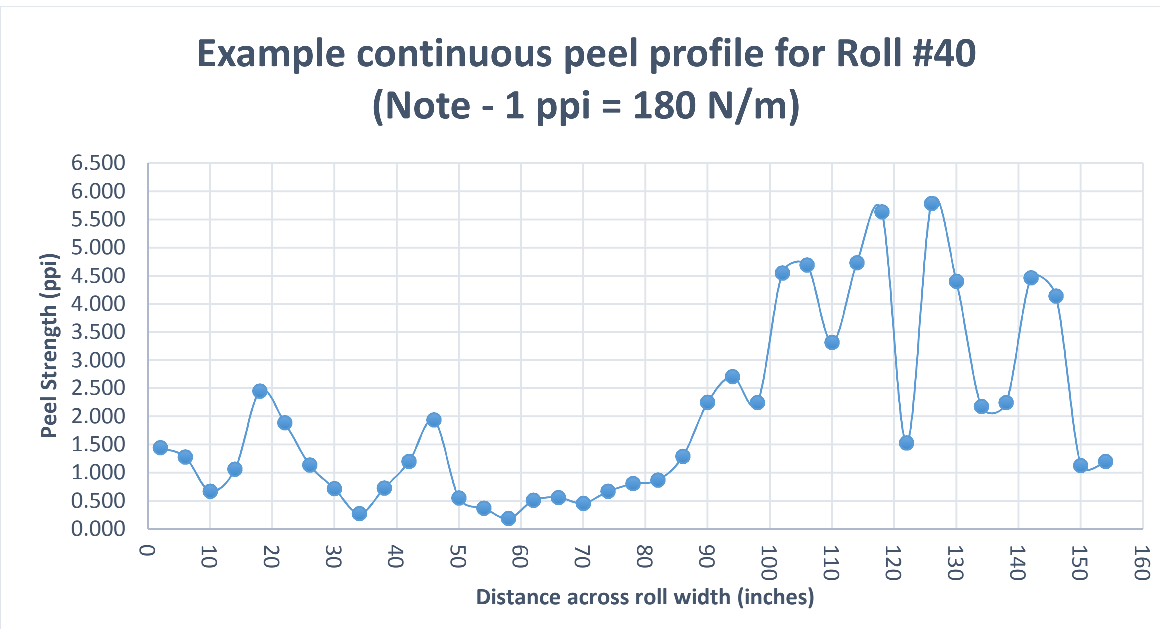

Figure 3 – Peel strength profiles from one side of roll.

Solution:

1. Determine the threshold peel strength, $p_{s\text{-}thresh}$, at the 25th percentile of the area for Roll #40 based on the continuous peel profile test results. The peel results are sorted and plotted against the percentile of the cumulative area from lowest to highest peel values, as shown Figure 4, where each of the 39 specimens is assumed to represent 2.56% of the bonded

area. Interpolation of the data at the 25th percentile indicates that for roll sampled, the value of $p_{s\text{-}thresh}$ is estimated to be 0.705 ppi [126.9 N/m].

2. Calculate the driving force, D, for the MLV condition as the sum of the static and dynamic soil and equipment loads:

$$D_{(MLV)} = 266 + 153.3 = 419.3 \text{ psf [20.13 kPa]}$$

3. Calculate R_b, R_u, and R_T, for the MLV condition:

$$R_{b(MLV)} = [(p_{s-thresh} \cdot 417) + N \tan \varphi_b](1 - A_u) =$$
$$[(0.705 \cdot 417) + (798 \cdot 0.49)](1 - 0.3) = 479.5 \text{ psf [23.02 kPa]}$$
$$R_{u(MLV)} = A_u \cdot N \tan 14 = 0.3 \cdot 798 \tan 14 = 59.7 \text{ psf [2.87 kPa]}$$
$$R_{T(MLV)} = R_b + R_u = 479.5 + 59.7 = 539.2 \text{ psf [25.88 kPa]}$$

4. Calculate $FS_{(MLV)}$:

$$FS_{(MLV)} = {R_{T(MLV)}}/{D_{(MLV)}} = {539.2}/{419.3} = 1.286$$

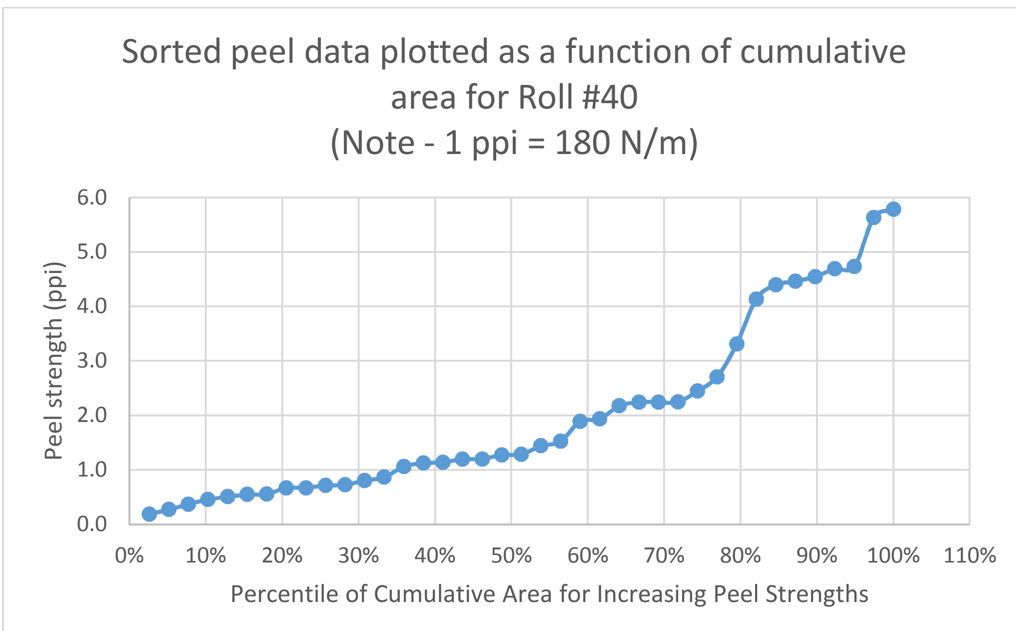

Figure 4 – Sorted peel strength data from example Roll #40.

5. To estimate the reliability that the geocomposite product will not delaminate more than the stated design criteria, perform the following calculations in accordance with the procedure described by Duncan (2000):
   a. Estimate the standard deviation, σ, for each of the variables that were identified for this analysis. Barring the availability of large data sets or previously published values, the standard deviation of any variable can be estimated using the "three-sigma rule" discussed by Duncan (2000), where:

$$\sigma = \frac{HCV - LCV}{6} \tag{7}$$

Using this approach, the following standard deviations were estimated for the variables used in this analysis:

i. For the bonded adhesion, c_a: $\sigma_{ca} = \frac{750-233}{6} = 86$ psf [4.13 kPa]

ii. For the un-bonded peak frictional strength coefficient, tan φ_b: $\sigma_{(\tan\varphi b)} = \frac{0.67-0.25}{6} = 0.07$ psf [0.0034 kPa]

iii. For the un-bonded post-peak friction angle coefficient, tan φ_u: $\sigma_{(\tan\varphi u)} = \frac{0.384-0.141}{6} = 0.041$ psf [0.002 kPa]

iv. For the equipment deceleration rate, a: $\sigma_a = \frac{0.05+0.5}{6} = 0.075$ psf [0.0036 kPa]

b. In accordance with the method presented by Duncan (2000), calculate *FS* values for the conditions where each of the variables are plus and minus one standard deviation from their MLVs. The calculations, which are not shown here but are similar to the ones shown above, provide the following results:

	Bonded adhesion "c_a"		$\tan(\varphi_b)$ [Peak friction]		Equip Acceleration *a*		$\tan(\varphi_u)$ [Resid friction]	
F_{MLV}	FS for $-\sigma_{ca}$	FS for $+\sigma_{ca}$	FS for $-\sigma_{(\tan(\varphi b))}$	FS for $+\sigma_{(\tan(\varphi b))}$	FS-σ	FS+σ	FS-σ	FS+σ
1.286	1.185	1.388	1.193	1.379	1.416	1.178	1.263	1.309

c. Estimate the standard deviation (σ_F) and the coefficient of variation (V_F) of the factor of safety using the following formulas referenced by Duncan (2000):

$$\sigma_F = \sqrt{\left(\frac{\Delta FS_1}{2}\right)^2 + \left(\frac{\Delta FS_2}{2}\right)^2 + \left(\frac{\Delta FS_3}{2}\right)^2 + \left(\frac{\Delta FS_4}{2}\right)^2} \quad (8)$$

$$V_F = \frac{\sigma_F}{FS_{MLV}} \quad (9)$$

The calculations for these parameters result in the following values for this example (work not shown): $\sigma_F = 0.1833$ and $V_F = 0.1425$.

d. Calculate the lognormal reliability index, β_{LN}, using the following formula recommended by Duncan (2000):

$$\beta_{LN} = \frac{\ln\left(\frac{FS_{MLV}}{\sqrt{1+V_F^2}}\right)}{\sqrt{\ln(1+V_F^2)}} = \frac{\ln\left(\frac{1.286}{\sqrt{1+0.1425^2}}\right)}{\sqrt{\ln(1+0.1425^2)}} = 1.703 \quad (10)$$

e. Calculate the reliability, *r*, using a normal distribution lookup table, or the built-in function NORMSDIST in Excel, using β_{LN} as the argument for this function. The resulting answer, which for this example is 0.956 (95.6%), is the reliability. For this example, 95.6% is less than the design criterion of 98%, so the sample would fall short of the specification. The probability of failure is one minus the reliability, or $P_f = 1.0 - 0.956 = 0.044$, or 4.4% (1 in 23 chance of failure).

6. The above evaluation would be performed for every sample that is tested.

QUESTION OF OVERALL STABILITY EVALUATION FOR UNBONDED AREAS ON SINGLE VERSUS DOUBLE SIDED GEOCOMPOSITE.

In liner systems for environmental covers and bottom liners, the critical weak interface for slope stability typically follows one of the geosynthetic interfaces. If the plane of greatest weakness shifts from one interface to another then it would be required to include the tensile strength of the intervening geosynthetic layer(s) as part of the resisting force. For the consideration of a double-sided geonet geocomposite, which is the subject of this paper, there could be randomly located unbonded areas on both sides of the geonet that could be critical slip interfaces, and for the critical slip surface to cross from one side of the geonet to the other it is the tensile strength of the geonet core that would need to be added to the resisting forces. A complete consideration of this nuance is beyond the scope of this paper, and could be the subject of a future study, but a few general thoughts are provided in the paragraph below.

Depending on the product being considered, the tensile strength of common bi-planar geonets might be on the order of 1,000 lbs [453.59 kg] per foot of width. This tensile strength would be comparable to approximately two linear feet of bonded interface shear strength for a material with 1 ppi [180 N/m] peel strength under a typical veneer cover system on the order of 2-3 feet [0.61m to 0.91 m] thick. The authors of this paper believe that there is adequate conservatism built into the overall design assumptions and specifications to assume that critical shear interface would be inhibited from jumping from one side to the geonet to the other due to the tensile strength of the geonet for most veneer cover situations where the veneer thickness is less than 3 feet [0.91 m] thick. For thicker cover soil layers, or bottom liner situations, the designer may wish to reduce the allowable unbonded area on each side of a double sided geonet geocomposite product to account for critical weak zone shifts from one side of the product to the other.

EXAMPLE OF PROPOSED SPECIFICATION BASED ON CALCULATION RESULTS

The calculations presented above are easily programmed into an Excel spreadsheet. Once the formulas are set up in the spreadsheet, the threshold peel value, $p_{s\text{-}thresh}$, can be iteratively changed to find the limiting value at which the desired reliability is exactly achieved. For the example presented above, it can be found that a $p_{s\text{-}thresh}$ value of 0.81 ppi [145.8 N/m] will exactly provide the desired reliability of 98% in accordance with the design approach described herein. As a matter of interest, the associated FS_{MLV} = 1.36, which is reasonable for a construction condition. It is easy to prove that if the veneer system is stable while under construction, it is much more reliable, with much greater *FS*, under static soil loads. Since geocomposite peel strength is often certified and reported to the nearest 0.1 ppi [18 N/m], a proposed specification for the example project conditions could be formulated as follows:

> "The geocomposite drainage layer material shall consist of nominal 14- to 15-foot [4.27 m to 4.57 m] wide rolls of a geonet core having nonwoven needle-punched geotextiles heat bonded to both sides, with 4-inch [101.6 mm] to 8-inch [203.2 mm] (nominally 6-inch [152.4 mm]) of the geotextile un-bonded along both sides of the geonet for purposes of seaming. The heat bonded lamination between the geotextile and geonet shall have a uniform ply-adhesion (peel) strength over the remainder of the panel area, with a minimum

of “blisters”, “holidays”, or other weakly bonded areas. This specification requires a minimum 25th-percentile threshold peel strength = 0.8 pounds-per-inch (ppi) [144 N/m] based on continuous-specimen profile testing across the roll width. The specimens shall be tested in accordance with ASTM D7005, with the test method modified to allow for continuous four-inch wide specimens taken in a checkerboard pattern across the entire roll width. For example, if the initial roll width is 14.5 feet [4.42 m] and has 6-inch [152.4 mm] un-bonded edges, then a minimum of 40 four-inch [101.6 mm] wide specimens should be taken across the width of the roll for one test. This specification requires that 30 of the 40 specimens would be required to have a peel strength greater than or equal to 0.8 ppi [144 N/m] on both sides.”

MANUFACTURER RESPONSE

A variation of the proposed specification was used on two projects in 2018. One was a 13.5 acre [5.46 ha] bottom liner project for a landfill (Site X); the other was a 20-acre [8.09 ha] project for a waste pile cap (Site Y). Both projects also had relatively demanding transmissivity requirements. The intent of the specifications was to achieve a minimum peel strength bond of 0.8 ppi [144 N/m] across the roll width. The manufacturer for Site Y produced material peel strengths for Side “A” of the geocomposite shown in Figure 5.

Figure 6 presents a distillation of the data at the average adhesion values at the 25, 50, 75, and 100 percentages for Side “A” and “B”, respectively. The 25th percentile clearly exceeds the peel specification of 0.8 ppi [144 N/m] by a factor of 6. Additionally, the peel strength escalates to high end values of approximately 12 ppi [2,160 N/m]. This demonstrates the overcompensation in manufacturing in order to meet the specification. This overcompensation also creates a constructability issue in the field when preparing net-to-net connections for drainage at the butt seams; additional labor and materials are required to remove the geotextile components due to the considerably higher peel strength.

What is of particular note in the actual experience is that the manufacturer provided material with a peel strength that averaged 6 times the requested value for Site Y using continuous sampling data from 24 rolls. Additionally, the average standard deviation was 2.8 ppi [504 N/m] and 2.7 ppi [486 N/m] for side “A” and “B”, respectively. Similar results were experience at Site X.

Clearly the manufacturing for these two projects significantly overshot the mark with respect to lamination strength. Examination of the data indicates that even given the inherent variability in the lamination process, the manufacturing parameters could have been safely backed off significantly to produce a lower peel strength that still would have easily met the specifications.

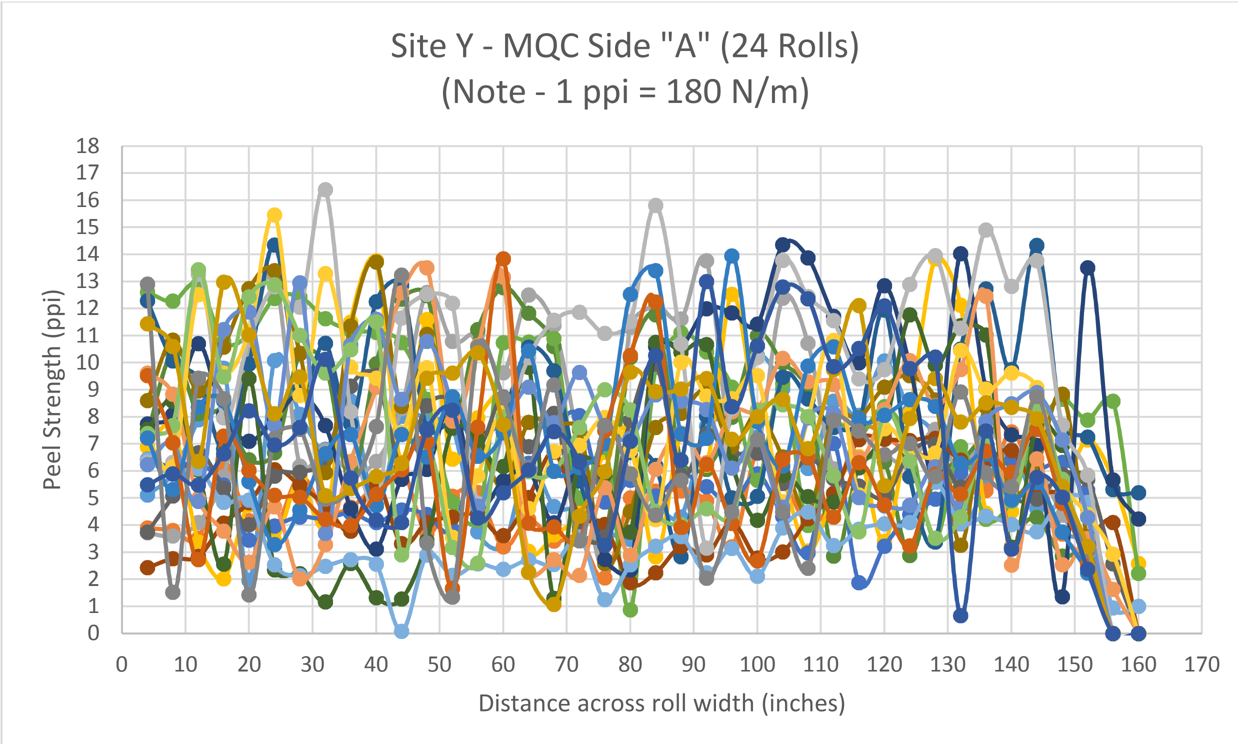

Figure 5 – Peel strength data from actual project using example specification.

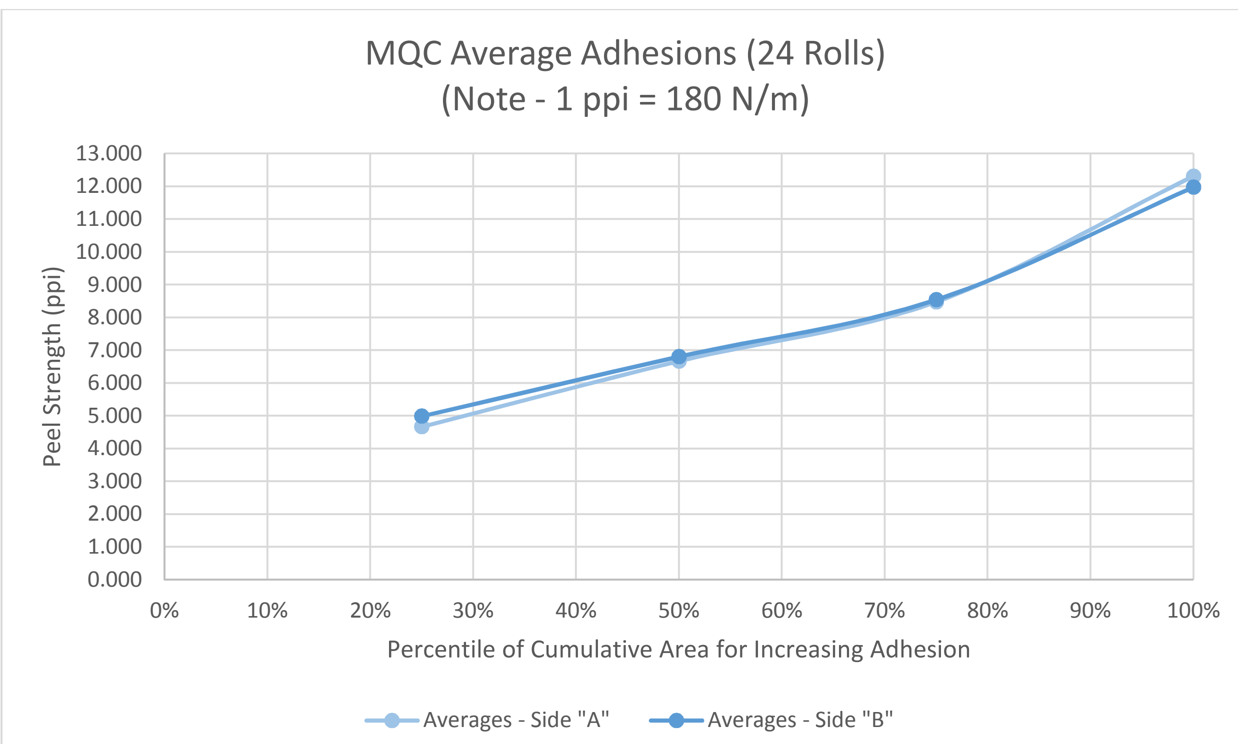

Figure 6 – Sorted peel strength data from actual project that used example specification.

The experience of 2018, in conjunction with interviews with the manufacturers, has shown that there is a high degree of uncertainty in controlling the strength of the heat laminated interface during manufacturing. While this has likely always been the case, it has never been openly acknowledged in the industry. A high degree of uncertainty in the manufacturing process will translate directly to uncertainty in design performance. It is imperative that designers understand this uncertainty and incorporate it into their design specifications. The whole point of performing engineering calculations and utilizing factors of safety and reliability parameters is to take issues such as this into account.

When the design uncertainty was removed by requiring "full disclosure" in the form of a continuous peel profile test upon which the specification was based, the impact of the manufacturing uncertainty was revealed by the great extent by which the manufactured material overshot the specification, at extra cost. The authors suspect that if the proposed methodology becomes more mainstream, manufacturers will become more tuned in to their own capabilities and will be able to deliver to a tighter tolerance.

CONCLUSIONS

The authors conclude that the current method of evaluating peel strength of geocomposites by taking 5 specimens across roll width in accordance with ASTM D7005 is inadequate and may be misleading because of the very high variability in the manufacturing process. The approach presented herein for specifying and testing the required average shear strength of a geocomposite based on a *continuous* peel profile is considered a significant improvement and should be adopted, until modified through further testing and evaluation, by the industry for the following reasons:

- The method directly measures the manufacturing variability across the roll width.
- The method rewards higher levels of uniformity and control in the manufacturing process by only having a requirement on the lower bound threshold value of peel strength.
- The method accounts for worst case construction loading conditions in an objective manner and reduces dependencies on visual inspections that may not be as reliable.
- The amount of additional testing required for continuous peel testing compared to the current industry standard of 5 specimens per test is reasonable for CQA programs, and the amount of additional data evaluation is negligible, given the benefit of the information.

Note that the testing requirement is really aimed at the engineer/owner to perform. Manufacturers may not be ready to regularly perform such extensive peel testing as part of their QC process, mainly because of the amount of time one single test of doing continuous specimens will take. Many rolls of material will already have been manufactured by the time the test result is ready. Therefore, manufacturers will have to devise their own approaches for performing QC so that they can be assured that the product that they are delivering will be able to pass the engineer's conformance testing program.

The authors believe that this approach will provide an appropriate level of reliability for specifying and testing this critical interface, will allow competitive manufacturing pricing, and will reduce material rejections and controversy.

It is incumbent on designers to address the issue of lamination strength variability in their design approach and specifications more directly. Slope failures due to this interface have occurred in the industry, and now the root cause has been identified, and a design and testing approach have been presented. Certainly the design and testing approach presented in this paper are open to discussion, improvement and modification.

RECOMMENDATIONS

Until further research and evaluations suggest otherwise, the authors recommend that designers require, and that ASTM adopts, the following modifications to ASTM D7005 (Standard Test Method for Determining the Bond Strength (Ply Adhesion) of Geocomposites):

- Samples cut for *CQA conformance testing* shall always be the entire roll width, and typically be 3 feet [0.91 m] long in the machine direction.
- The width of the un-bonded zone on the edges of the rolls should be measured and reported.
- For the bonded zone, 4-inch [101.6 m] wide specimens should be taken contiguously across the roll width (in a checkerboard pattern).
- Test all of the specimens in accordance with the ASTM D7005 test method, and report the results for each specimen, the 25th percentile value, the median (50th percentile) value, the mean value, any other percentile value requested by the specifier, and the normal standard deviation.

For CQA conformance testing, the author recommends a baseline testing frequency, using complete peel profile testing, on an initial basis of once per 50,000 sq ft [4,645.15 m^2]. The CQA testing could follow a "method of attributes" approach that allows a reduction in testing frequency for all-passing tests, and an increase in the testing frequency if there are failures.

The test requirements for MQC production are open for discussion. Manufacturers will have to decide what level of testing they need to do to know that their products will meet the conformance testing.

In addition to specifying the product peel and testing requirements described earlier in this paper, designers should also specify the following:

- The maximum allowable size of the construction equipment allowed on the slope should be specified. The minimum allowable soil thickness being spread over the geocomposite should be 12 inches [0.305 m], and it is recommended that the spreading equipment be no larger than a Caterpillar® D6 with LGP tracks. Smaller equipment will produce less potential for delamination, and should be considered for slopes steeper than 33% (3:1). For larger soil hauling equipment, greater soil cover thickness is beneficial because it will spread out the equipment loads further. The number of drive wheels on the haul truck equipment will determine the maximum loads in that case.
- Materials should generally be pushed up from the bottom of the slope, and pushing from top-down discouraged except under special circumstances approved by the engineer and where field tests are performed. Avoiding pushing or hard-stopping in the downslope

direction will reduce potential inertial forces. Equipment should be operated carefully with no sharp turns.

The last opportunity to identify weakness in the geocomposite bonding is during field deployment. At this time, field inspectors should observe the following:

- Look for obvious signs of non-laminated areas in the form of blisters as the material is being deployed. If blisters are observed, more detailed inspection of those areas should be performed by pushing on the material with the soles of boots, particularly to investigate if there is a general tendency for a machine-direction defect. The size of any blisters should be recorded and reported to the engineer.
- For every 5th panel, and more frequently if weak spots are identified, the ends of the panels should be tested by the field inspector by hand-pulling the geotextile away from the geonet (on both sides of double-sided product). Weak areas, which generally run in the machine-direction, are generally readily identified in this way and, if discovered, should be reported to the engineer and more thoroughly evaluated through additional sampling and laboratory testing.
- If panels are found to contain weakly bonded areas in excess of what is deemed acceptable by the engineer, those panels must be removed and a more detailed investigatory program undertaken that searches for the extent of the problem. It is also important to involve the manufacturer to try to understand the reason for the problem, which will be helpful in determining how much of the shipment might need to be rejected.

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