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Laboratory Pullout Equipment for Testing Soil-Geosynthetic Interface for Reinforced Flexible Pavement Design

Ranjiv Gupta[1], Ph.D., Robert H. Swan Jr.[2], M. ASCE, and Jorge G. Zornberg[3], Ph.D

[1]Engineer, Geosyntec Consultants, Austin, TX (rgupta@geosyntec.com)
[2]Associate Teaching Professor, Drexel University, PA (rswan@coe.drexel.edu)
[3]Professor, University of Texas, Austin, TX (zornberg@mail.utexas.edu)

ABSTRACT:

Pullout tests for geosynthetics were developed to support the design of MSE walls to calculate maximum force required for internal stability calculation. In this case, the stability of the system at ultimate or limit state was of concern and results were reported in terms of coefficient of interaction (Ci) for a given geosynthetic and soil interface. However, to quantify the performance of geosynthetics used in flexible pavements, the soil-geosynthetic interface characteristics at low displacement magnitude is important. An analytical model was proposed by Gupta (2009) to assess the performance of soil-geosynthetic interaction and a new parameter to quantify soil-geosynthetic interaction (K_{SGI}) was defined. This parameter is a function of yield shear stress and confined stiffness of the geosynthetic which can be obtained from a pullout test. The quantification of this new parameter required test equipment which would be able to define the low displacement behavior of soil-geosynthetic interfaces. However, the conventional pullout box available in most laboratories are capable of predicting the ultimate pullout force as required for MSE wall design which require long testing times. To reduce the testing times and to predict the soil-geosynthetic interface characteristics at low displacements, new pullout test equipment was developed which allows testing geosynthetics for reinforced pavement application at low displacement magnitudes. This paper describes the development of new pullout test equipment and discusses the interpretation of test results for a planar geosynthetic specimen to calibrate the proposed parameter based on the analytical model. Finally, the effect of geosynthetic orientation in the pullout equipment on the model parameters and its implication on design of reinforced pavements is discussed.

INTRODUCTION

Design Methodologies for Geosynthetic Reinforced Flexible Pavements

The basic philosophy of flexible pavement systems was originally envisioned by the Romans and it continues to form the basis of flexible pavement design today. This approach involves providing a protection layer over the subgrade, thereby ensuring the serviceability of the pavement under given traffic and environmental loading. Figure 1 shows the evolution of road design methods from the 1930s until today. After the great depression in the 1930s, the CBR Cover Design Method was developed. It required a single input in terms of the California Bearing Ratio (CBR), but it still involved a significant amount of engineering judgment. Following the American Association of State Highway Officials (AASHO) Road Test, which was popularized in the 1960s, a series of design methods were proposed that were more sophisticated than the Cover Design Method and that required a greater number of design inputs. For example, in the 1970's, the linear mechanistic-empirical (M-E) design method was proposed by researchers from South Africa. Since the early 1990s, the focus has shifted to mechanistic-empirical design methods that incorporate features from purely empirical methods to sophisticated finite element non-linear mechanized methods. Attempts have recently been made to incorporate the geosynthetic reinforcement into AASHTO and M-E design methods.

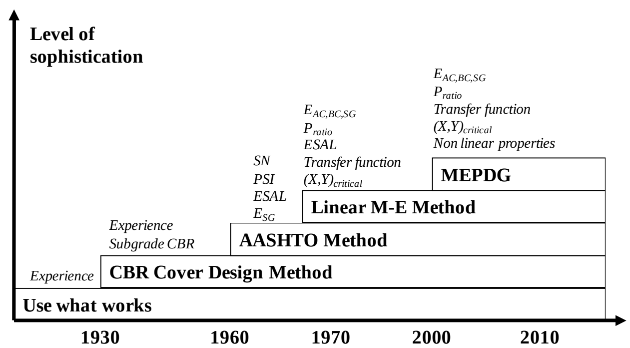


Figure 1: Evolution of pavement design methods (adapted from Reck, 2009)

Performance Data for Geosynthetic Reinforced Flexible Pavements

The methods for collecting pavement performance data are field scale testing, laboratory testing, and numerical simulations. These three methods not only differ widely, but lead to the collection of significantly different performance data as well, as shown in Figure 2. Ultimately, the quality of pavement performance data generated depends on the cost and the method being used for collection (Reck, 2009).

Full-scale tests include field studies and accelerated pavement tests that simulate actual pavement behavior. In both of these cases, the cost of testing is high and fairly limited tests can be done as a consequence. Thus, the test matrix and its scope are generally expanded by undertaking smaller scale laboratory studies or numerical simulations. Laboratory tests are generally cheaper than field tests and can be performed under controlled conditions. However, it is difficult to replicate the true behavior of the system using laboratory tests, which are further limited by the instrumentation used during the given test. For this reason, numerical simulations can be useful in developing models based on field and laboratory tests to perform parametric studies. Thus, these three approaches to data collection can be combined to develop a comprehensive design methodology. This paper focuses on the application of laboratory tests to predicting performance of geosynthetic reinforced pavements.

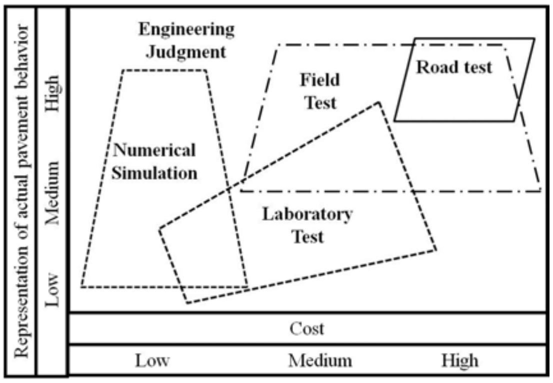


Figure 2: Interrelationship between different facets of pavement design (adapted from Hugo et. al, 1991)

Primary Mechanism Governing Geosynthetic Reinforced Flexible Pavements

The primary mechanism governing the performance of geosynthetic reinforcement in flexible pavements is lateral restraint provided by the geosynthetics (Perkins, 1999) and laboratory tests have been proposed to quantify this mechanism. When the lateral restraint mechanism is mobilized, the geosynthetic develops additional tensile stresses under given loading thereby providing confinement to the surrounding aggregates as shown in Figure 3. This degree of confinement has been attributed to the effect of interface shear provided by geotextiles and dynamic interlocking provided by geogrids when used in the base course layer of the pavement. The primary objective of laboratory tests has been to predict soil-geosynthetic interaction mechanisms in a flexible pavement system either by measuring the index properties of geosynthetics or by replicating the field conditions. Based on the approach adopted, the tests reported in the literature have been divided into two main categories i.e., unconfined and confined tests. In unconfined tests, the geosynthetic properties are generally measured in-air, independent of the site soil. On the other hand, in the confined tests the geosynthetic is placed within the soil and confinement is applied at the interface.

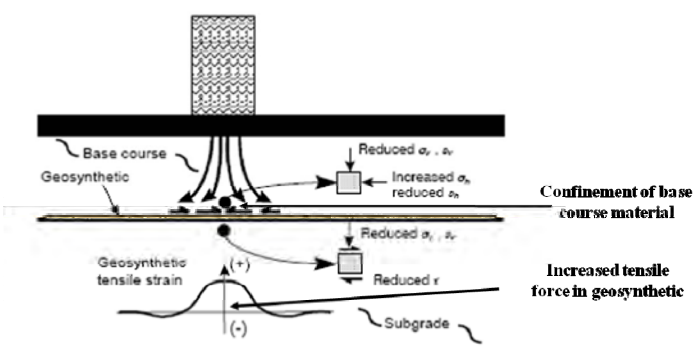


Figure 3: Additional forces due to soil-geosynthetic interaction when a lateral restraint mechanism occurs in a pavement (adapted from Perkins, 1999)

LABORATORY TESTS

The unconfined tests are easy, economical and quick to execute in the laboratory but independent of the site soil thereby making it difficult to replicate field conditions. Moreover, a geosynthetic is loaded under confinement provided by the surrounding aggregate generating a plane-strain multi-axial or isotropic multi-axial rather than a uniaxial state of stress in the field. In other words, the geosynthetic behavior observed in the laboratory from unconfined tests has to be correlated with the field application, which has different loading and boundary conditions. These tests are used as an index but not for the actual design of the geosynthetic reinforced flexible pavements. Therefore, recently confined tests which measure soil and geosynthetic properties together have been recommended for performance based design of geosynthetic reinforced pavements and are the focus of this paper.

Confined Tests for Geosynthetic Reinforced Flexible Pavement Design

Based on a Federal Highway sponsored study regarding the review of the existing confined test for geosynthetics, Elias et al. (1998) concluded that the unconfined response is overly conservative and that the confined response should significantly improve the characterization of geosynthetic materials in engineering applications. Recently, a number of confined tests have been proposed, out of which five tests have specifically focused on characterizing field behavior of geosynthetic reinforced flexible pavements. These tests are the cyclic plate load test, cyclic triaxial test, cyclic pullout test, bending stiffness test and the modified pavement analyzer test. A comparison of the important features of each test is presented in Table 1.

The cyclic plate load test was designed for the purpose of conducting large scale laboratory experiments on reinforced and unreinforced pavement sections (Al-Qadi et al., 1994; Cancelli et al., 1996; Miura et al., 1990; and Perkins, 1999). Based on the studies conducted using the cyclic plate load test, Traffic Benefit Ratio (TBR) ranging from 1 to 70 and Base Course Reduction (BCR) ranging from 20% to 50% were obtained for test sections consisting of geotextiles and geogrids (Hsieh and Mao, 2005).

The cyclic triaxial test was used to measure the ability of the soil to endure the shearing stresses induced in it due to cyclic loading (ASTM D5311, 2004). The resilient modulus, M_r, of the soil aggregates computed using this test was used as an input in the mechanistic empirical design (NCHRP Project 1-28A, 2000). The above test was modified by Perkins et al. (2004), to obtain the change in resilient modulus and permanent deformation behavior due to the addition of a geosynthetic to the aggregate layer of the pavement.

Cyclic pullout test were conducted by Cuelho and Perkins (2005) by modifying the standard pullout test (ASTM D6706) to resemble the loading protocol used in a cyclic triaxial test Based on test results obtained, a parameter known as geosynthetic-soil resilient interface shear stiffness (G_i) was defined to describe the reinforcement-aggregate interaction under cyclic loads.

Bending stiffness test was developed by Sprague et al. (2004) as a small scale index test procedure for predicting the behavior of different geosynthetics used as reinforcing material in pavements. The test apparatus was a modified version of the multi-axial tension test for geomembrane (ASTM D 5617) property named as bending stiffness (BS) was obtained from test results and was defined as the ratio of the deviator stress (σ_d) to the recoverable deformation (Δ_r).

Han et al. (2008) proposed a test method to use asphalt pavement analyzer (APA) to evaluate the benefit of geosynthetic reinforcement in the base course layer of the pavement. Besides evaluating TBR for the given sections, the authors proposed a parameter known as rut reduction ratio (RRR), which was defined as the ratio of the rut of the reinforced base to that of the unreinforced base at the same service life (8000 cycles).

Table 1: Features of Confined Tests for Geosynthetic Reinforced Pavements

Features	Cyclic plate load test	Cyclic triaxial test	Cyclic pullout test	Bending stiffness test	Modified asphalt pavement analyzer
Loading type	Cyclic	Cyclic	Cyclic	Cyclic	Moving wheel
Design property	TBR	M_r	G_i	BS	RRR
Suitable design method	AASHTO	M-E	M-E	AASHTO	AASHTO
Ease of running test	Difficult	Difficult	Moderate	Moderate	Easy
Control section	Yes	Yes	No	Yes	Yes
Repeatability of test results	-	No	No	No	Yes
Ability to distinguish among various geosynthetics	-	No	No	No	Yes

In general, these tests provide actual quantification of the soil-geosynthetic interaction behavior but are expensive and time consuming to run. The tests report the performance in the form of reduced deflections (TBR, BS, and RRR) or increased interaction or confinement modulus (M_r and G_i) which could be used as an input in design methods with ratios for AASHTO design and resilient modulus for M-E design of flexible pavements. However, the main drawback with the current confined testing methods was that the design parameters were sensitive to small changes in cyclic load levels used to simulate field conditions, which led to variability in low displacement measurements, thereby making it difficult to reproduce them for a given geosynthetic and also difficult to distinguish between the performances of various geosynthetics. Therefore, it was decided to conduct a confined test with monotonic loading in order to reduce the variability in test results and still allow for the realistic measurement of the interface mechanisms.

Confined Monotonic Loading Tests

The two common soil-geosynthetic interface tests which use monotonic loading are modified direct shear test and pullout test as shown in Figure 4. Both the tests involve placing a geosynthetic between the required soils and moving the assembly at constant rate of displacement. While in the direct shear box the top soil layer is moved relative to the clamped geosynthetic, on the other hand, in the pullout test the geosynthetic is moved relative to the soil. This principle difference in these two test methods mobilizes contrasting mechanisms at the soil-geosynthetic interface. In the direct shear test, the primary mechanism is the mobilized interface friction as the goal to characterize the interface shear strength between the soil and geosynthetic at the peak displacement. However, in the pullout test due to movement of the geosynthetic relative to the soil, tensile stresses are developed in it along with interaction mechanism at the interface in terms of shear for geotextiles and interlocking for geogrids. Therefore, for application of laboratory test to reinforcement pavement design where lateral restraint between soil and geosynthetic is to be quantified, pullout tests were considered more suitable than a modified direct shear test.

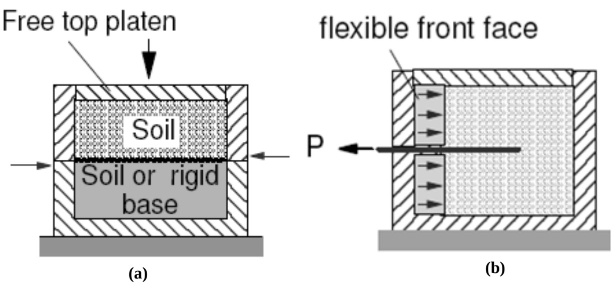


Figure 4: Soil-geosynthetic interface test (a) Modified direct shear test (b) Pullout test (adapted from Palmeria, 2008)

For the current research, it was decided to conduct monotonic load pullout tests with a focus on characterizing the soil-geosynthetic interaction at low displacement magnitudes. Although these pullout tests did not simulate the exact traffic load condition, they reproduced the similar interface mechanisms between the soil and geosynthetic as in real pavements. An analytical model was proposed to predict the confined load-strain characteristics of soil-geosynthetic systems at these low displacement magnitudes (Gupta, 2009) from results obtained from pullout tests. Thus, a new performance-based test method that has all the necessary features in the form of a pullout test was proposed to effectively evaluate the geosynthetic-soil confinement.

PULLOUT TESTING OF GEOSYNTHETICS

A reinforced soil mass is somewhat analogous to reinforced concrete in that the mechanical properties of the mass are improved by reinforcement inclusions placed parallel to the principal strain direction to compensate for soils' lack of tensile resistance (Elias et al., 2001). The improvement in the tensile resistance of the system results from the interaction between the reinforcement and the soil. When the reinforcements are distributed regularly throughout the soil mass, stress transfer between the soil and reinforcement takes place continuously along the reinforcement thereby improving the characteristics of the composite system. The two main mechanisms by which stress transfer between soil and reinforcement occurs is either friction or passive resistance depending on the reinforcement geometry. The friction mechanism is developed when there is a relative shear displacement corresponding to shear stresses between the soil and the reinforcement surface. On the other hand, the passive resistance mechanism is developed due to bearing type of stresses occurring on the transverse reinforcement surface which is normal to the direction of soil and reinforcement movement.

Geotextiles and geogrids are two commonly used geosynthetic types for pavement reinforcement applications. Pullout tests are relevant for the study of the soil-reinforcement interaction characteristics of both these geosynthetics. Pullout resistance of geotextile reinforcement is provided mainly by frictional resistance along the soil-geotextile interface as shown in Figures 5a and 5b. On the other hand, the pullout resistance of a geogrid is the result of not only its soil frictional resistance but also the coupled effect of tensile strength of longitudinal ribs and passive bearing resistance provided by its transverse members, as shown in Figures 5c and 5d. Tensile stresses are mobilized in the longitudinal reinforcing elements when they cross shear planes developed due to soil extension.

Pullout resistance of the reinforcement is mobilized through one or a combination of the two basic soil-reinforcement interaction mechanisms. The compositional characteristics of the geosynthetics such as its type, geometry, configuration and those of confining soil such as its grain size distribution and void ratio have significant effect on the results obtained from a pullout test. The measured pullout resistance is influenced by the details of testing equipment and procedures. A thorough understanding is thus required to properly quantify the above effect while interpreting the test results obtained from a pullout test. A discussion on the method proposed to interpret the pullout test results is provided in next section.

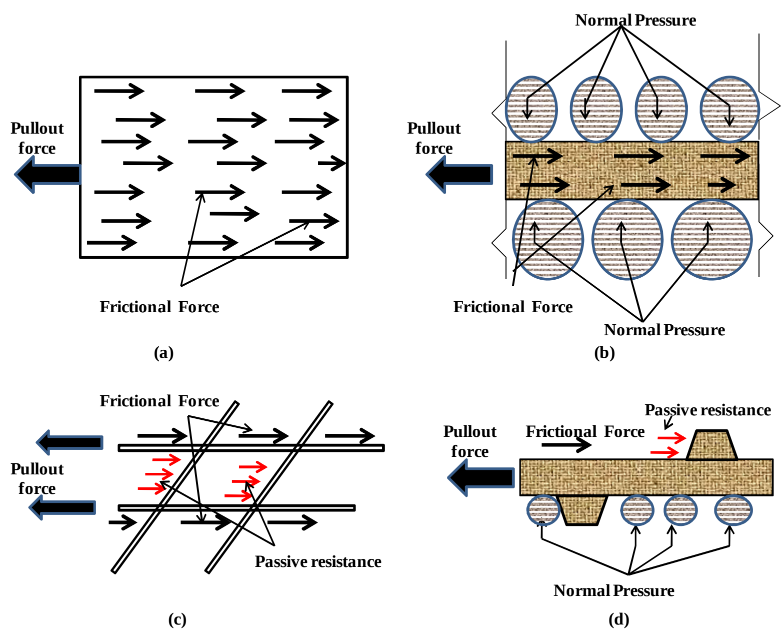


Figure 5: Soil reinforcement interaction mechanisms when geosynthetic is subjected to pullout force (a) cross- section of geotextile specimen (b) forces on geotextile (c) cross-section of geogrid specimen (d) forces on geogrid (adapted from Elias et al., 2001)

Coefficient of Interaction from Pullout Tests

Large scale pullout tests are conducted to estimate the extent of interaction between the soil and reinforcement to determine project specific properties. A simplified approach is followed to interpret pullout test results, which lumps all the interaction mechanisms together and is given as follows:

$$P_{\max} = 2 \cdot L_e . C_i \cdot \sigma'_{vo} . \tan\phi' \qquad (1)$$

where C_i is the coefficient of interaction based on ultimate pullout resistance, P_{max} from the pullout test on geosynthetic of length L_e subjected to effective vertical stress σ'_{vo} at the interface and ϕ' is the effective friction angle of the confining material. The above equation can also be written as:

$$C_i = \frac{\dfrac{P_{\max}}{2.L_e.\sigma'_{vo}}}{\tan\phi'} = \frac{\tan\delta'}{\tan\phi'} \qquad (2)$$

where δ' is the apparent angle of interaction between the geosynthetic and the confined material and is given as:

$$\tan\delta' = \frac{P_{\max}}{2.L_e.\sigma'_{vo}} \qquad (3)$$

This conventional analysis is based on limit equilibrium approach and focuses on ultimate loading conditions which occur under large displacements. Furthermore, this method is incapable of taking the effect of geosynthetic geometry, length, extensibility, and the amount of soil confinement into account while predicting its performance of various geosynthetics. For pavement design, the quantification of soil-geosynthetic interface stiffness at comparatively low displacements is critical. The above test is sensitive to boundary conditions and test apparatus used such that the measured soil-geosynthetic behavior at the regime of low displacement is usually not reliable. Therefore, a better analysis technique and testing equipment was proposed by Gupta, 2009 to capture the soil-geosynthetic behavior at low displacement magnitudes reliably.

Soil-Geosynthetic Interaction Model

Analytical model was obtained by solving differential equation (Perkins and Cuelho, 1999) governing soil-geosynthetic behavior in a pullout test. Equation 4 is a second-order differential equation which relates the displacement $w_r(x)$ with the shear stress $\tau(x)$ developed at the soil-geosynthetic interface in terms of confined stiffness J_c, for geosynthetic element of length ∂x in the pullout test. The detailed treatment for developing the solution to the given differential equation is provided in Gupta, 2009 and summarized here:

$$\frac{\partial^2 w_r(x)}{\partial x^2} = \frac{2.\tau(x)}{J_c} \quad (4)$$

The solution for governing differential equation of the pullout test involves two coefficients. The first coefficient can be computed by using the force boundary condition at the pullout end of the geosynthetic. The second coefficient is computed using the incremental distance travelled by increase in frontal pullout force through the confined geosynthetic specimen length.

$$F(x)^2 = (4.\tau_y.J_c).w(x) \quad (5)$$

This is the governing equation for the soil-geosynthetic interaction in the pullout test at each point on the geosynthetic. It suggests that the displacement at a point is related to square of the force at that point through a parabolic relation and the constant is given by Equation 5. The force and displacement at any given point x throughout the geosynthetic can be related by model parameters i.e., yield shear stress, τ_y and confined stiffness J_c of the soil-geosynthetic system. The above model parameters can be lumped into a single constant, called coefficient of soil geosynthetic interaction (K_{SGI}) which can be directly estimated using the pullout test and is given as,

$$K_{SGI} = 4.\tau_y.J_c \quad (6)$$

Then equation 5 can be written as:

$$F(x)^2 = K_{SGI}.w(x) \quad (7)$$

This model was used to predict the soil-geosynthetic interface stiffness at low displacements magnitude. This stiffness values can be used as an index to compare the performance among various geosynthetics. The pullout equipment to meet the testing requirements of the proposed model are explained in the next section.

PULLOUT TESTING EQUIPMENT

The schematic layout of the large-scale pullout testing equipment developed as part of this study is shown in Figure 6. The design changes from the conventional pullout box were: (1) a reaction frame system with wooden boards and air cylinders used for applying normal pressure on the specimen (Figure 6a); (2) the roller grips and its support trolley designed to avoid stress concentration at the geosynthetic reinforcement (Figure 6b). In addition to above features, the modified pullout box has the two hydraulic pistons used to apply pullout force on the specimen and five LVDT's along the support frame to measure the specimen displacements.

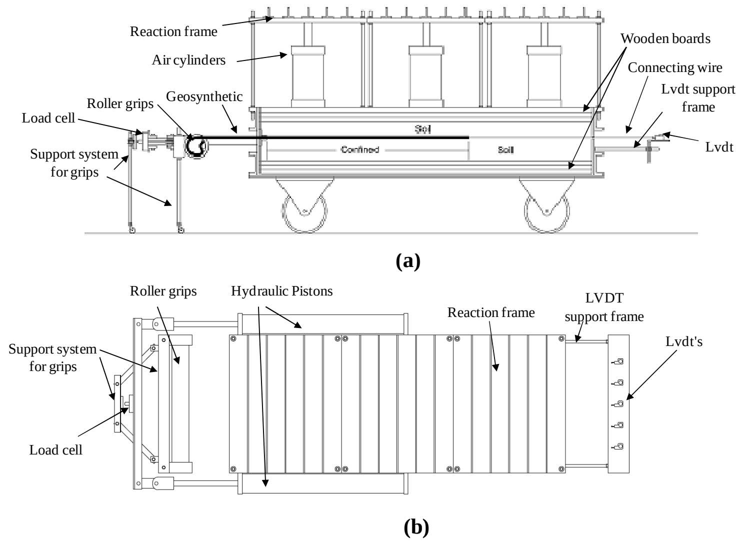


Figure 6: Large scale pullout testing equipment: (a) Side view; (b) Top view

The modified pullout box system with all the accessories is shown in Figure 7. The reaction frame system was found to reliable means of applying uniform normal pressure on top of the geosynthetic specimen. Roller grips were found to be suitable means of clamping different kinds of geosynthetics used during the test. Further, the use of new electric pump enabled better control over the rate of testing which could be independently controlled using the flow valve attached to it. Displacement transducers were attached to new system enabling faster data acquisition. The above changes led to reduction in test preparation time, better control over test procedure thereby providing repeatability among similar tests and reducing variability in test conditions for different geosynthetics. Overall, these modifications led to better equipment design capable of accurately characterizing low displacement soil-geosynthetic interface properties in the pullout box to be used for analysis using the proposed analytical model.

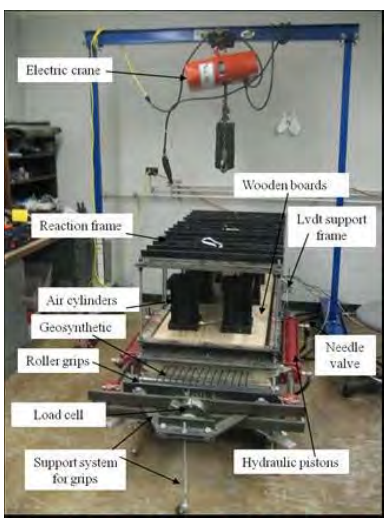
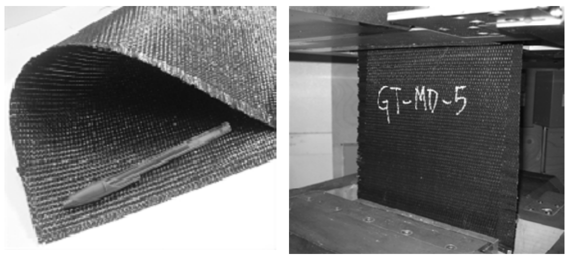


Figure 7: Large scale pullout testing equipment

TEST RESULTS

The material used in the baseline pullout test was geotextile (GT). It is a polypropylene woven geotextile manufactured by Mirafi and branded as HP-570 (Figure 8). The average tensile stiffness from wide width tensile test (ASTM D4595 (2011) obtained at rate of testing of 1 %/min for the two directions of the geotextile was 618 kN/m and 825 kN/m. The test results indicated that the geotextile was stiffer in cross-machine (XD) direction than machine direction (MD).



(a) (b)

Figure 8: Geosynthetic used for baseline tests (a) Geotextile (G3); (b) specimen used in wide-width tensile test

The geotextile (GT) specimen was prepared in the MD direction with dimensions of 0.6m length and 0.45m width as per the guidelines described in ASTM D6706 (2013) for conducting a pullout test. Five LVDT's were used at the horizontal spacing of 100 mm, 200 mm, 300 mm, 450 mm and 600 mm from the front end of the specimen as shown in Figure 9. The advantage of having five LVDT's was that the displacement profile throughout the length of the geosynthetic could be monitored. Furthermore, the readings from three LVDT's could be used to calibrate the model parameters and the other two LVDT's could be used to verify the model predictions.

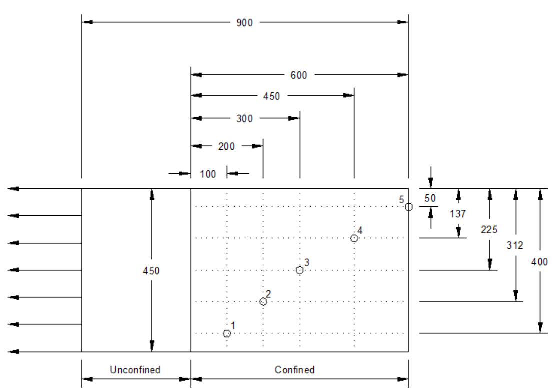


Figure 9: Location of LVDT's on geosynthetic specimen for test 1 with dimensions of 0.6m confined length and 0.45m width (All dimensions in millimeters)

Monterey No. 30 sand was used as the confining soil. The normal pressure applied at the top of the specimen was 21 kPa (3psi). The displacement rate of testing was set to 1 mm/min. The value of frontal pullout force (Fp) for displacement measured at location of five LVDT's was obtained from the given test as shown in Figure 10. The maximum pullout force value for the given test was 14.5 kN/m.

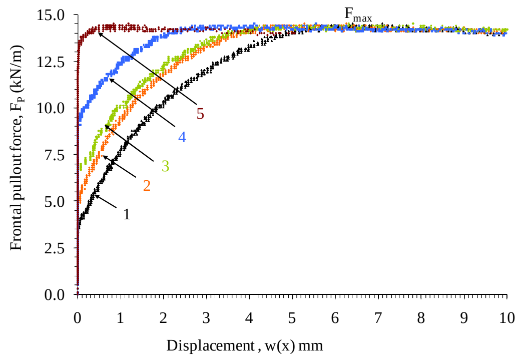


Figure 10: Frontal pullout force vs. displacement curve for each LVDT

After completion of the test, the data was analyzed to obtain the two parameters of yield shear stress (τ_y) and confined stiffness (J_c) proposed in the analytical model. The yield shear stress (τ_y) was obtained graphically as shown in Figure 11a. This plot helped to determine the magnitude of frontal pullout force when each LVDT just started to move ($F_{p,t1}$ through $F_{p,t5}$). That is, the magnitude of frontal pullout force corresponding to the active length of reinforcement was defined. Then, these values were plotted against the location of each LVDT on the geosynthetic specimen as shown in Figure 11b.

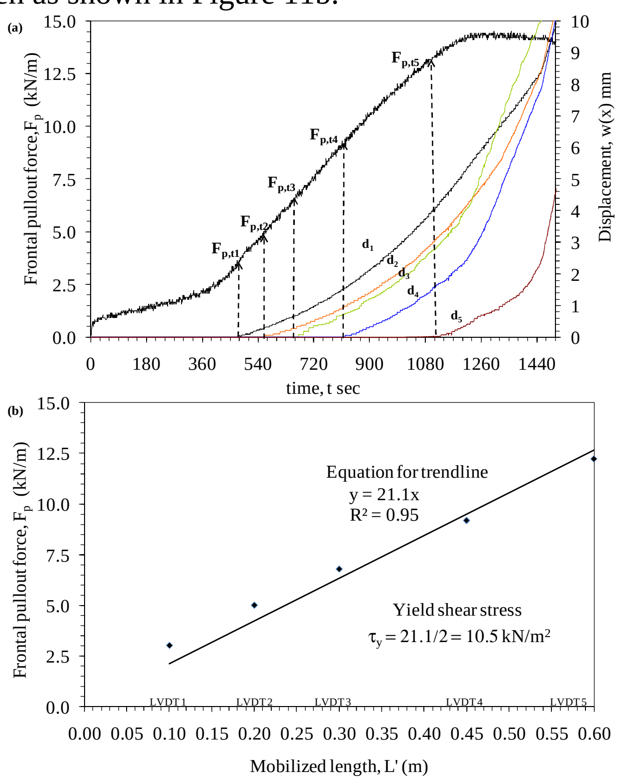


Figure 11: Computation of yield shear stress parameter graphically (a) Frontal pullout force and displacement as function of time from start of test; (b) Frontal pullout force vs. active length of the reinforcement

The line was then fitted through these points and its slope was determined. Since shear stress is mobilized on top and bottom of the specimen, the value of shear stress acting on the surface area of geosynthetic was half of the value obtained from slope of the curve. The yield shear stress (τ_y) for the given soil-geosynthetic system was calculated as 10.5 kN/m^2. To compute the value of confined stiffness (J_c), it was necessary to obtain the confined force and displacement response at each LVDT. Therefore, after computing the yield shear stress τ_y, the confined force F(x) at LVDT point x_i, for a given frontal pullout force Fp was estimated. The confined force F(x) and displacement w(x) at each of the five LVDT points is shown in Figure 12a. The LVDT's 2 and 3 which are in the middle of the geosynthetic specimen were least influenced by the boundary conditions and had similar confined force and displacement response. This trend was as hypothesized in the development of the analytical model, where it was suggested that the confined force and displacement response is unique for a given soil-geosynthetic system.

The next step in the analysis involved determining the magnitude of the confined stiffness (J_c) for the given system. It could be estimated graphically by plotting the square of the confined force at a point vs. displacement at that point as shown in Figure 12b. Then, the slope of this curve directly gives the value of constant K_{SGI} (24.9(kN/m)^2/mm). The average value of confined stiffness based on the data obtained from LVDT 2 (at 1 mm displacement) was estimated at 590 kN/m.

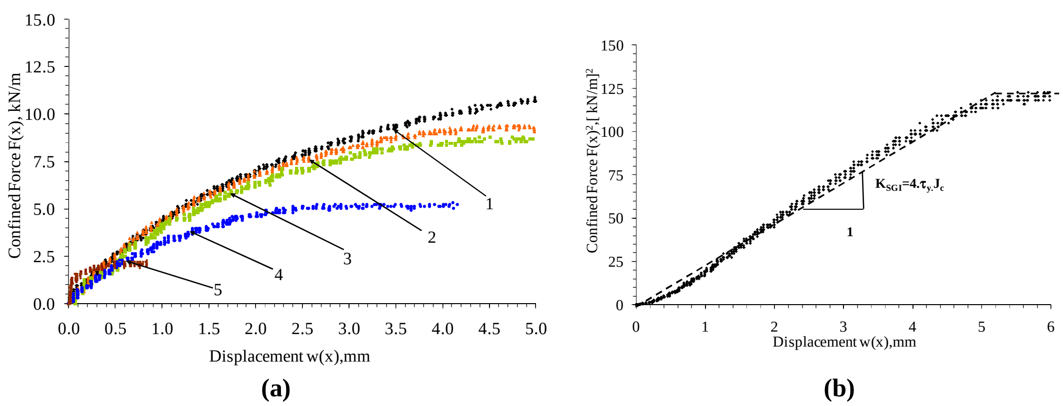


Figure 12: Interpretation of Pullout tests (a) Confined force vs. displacement curve for baseline test; (b) Estimating K_{SGI} graphically

Effect of Geosynthetic Orientation

The geosynthetic evaluated as part of this study has two principal manufactured directions i.e., longitudinal or machine and transverse or cross-machine direction. The testing direction of a geosynthetic in the pullout test is one in which force is applied, similar to conducting a tensile test on it. Therefore one pullout test was run to evaluate the interaction properties of change in orientation by pulling the specimen in the longitudinal (or machine) direction.

The effect of change in specimen direction was quantified by conducting a test similar to the baseline test, but reversing the principal directions of the specimen to the XD direction. The specimen was prepared for confined length of 0.6m and width of 0.45m and then subjected to a normal pressure of 21 kPa. The frontal pullout force values corresponding to displacements for the five LVDT locations were obtained. The comparison for the frontal pullout force values obtained from LVDT 2 for tests in the MD and XD are shown in Figure 13a. The maximum pullout force value obtained in test I-8 was 18 kN/m as compared to value of 14.5 kN/m obtained for test I-1. Furthermore, the yield shear stress value was obtained for this tests based on LVDT 2 movement was 16 kN/m as shown in Figure 13b. Finally, using the yield shear stress value, the K_{SGI} value for the test was computed as shown in Figure 13c. The value of K_{SGI} was lower in test 2 than that obtained in Test 1 as the surface area of stronger longitudinal elements was reduced when the specimen was tested in the machine direction.

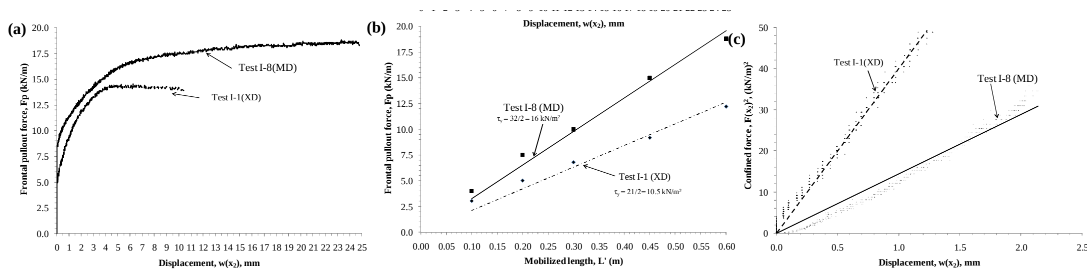


Figure 13: Comparison of tests (MD and XD) conducted to evaluate effect of specimen direction on parameters: (a) Maximum pullout force (b) Yield shear stress (c) K_{SGI}

SUMMARY AND CONCLUSIONS

The tests were conducted to calibrate the proposed model using the new pullout equipment. A standardized procedure was established for conducting the pullout test for geosynthetics and the data obtained from these tests was interpreted to obtain model parameters τ_y and J_c. The value of soil-geosynthetic interaction coefficient (K_{SGI}) was calculated based on model parameters for each test and it was shown that the proposed constant was able to quantify the low displacement interaction behavior of the geosynthetics evaluated in this test program.

The effect of orientation of specimen was evaluated by conducting the test similar to the baseline test but reversing the principal specimen direction. It was found that the geosynthetic orientation perpendicular to the pullout force direction led to about 25 percent change in the interaction capability of the given geosynthetic. Therefore, for uniaxial or biaxial oriented geosynthetics used during construction of roadways; the geosynthetic layers should be oriented such that the principal tire loading direction is in-line with longitudinal geosynthetic elements to obtain maximum reinforcement benefit.

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