Stiffness of Soil-Geosynthetic Composite under Small Displacements. II: Experimental Evaluation

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Abstract: While most soil–geosynthetic interaction models have focused on the characterization of failure conditions, little emphasis has been placed on models and parameters suitable for characterizing the stiffness of soil–geosynthetic systems. In the companion paper, a soil–geosynthetic interaction parameter (K_{SGC}) was developed that captures the stiffness of a soil–geosynthetic composite under small displacements. This included validation of the suitability of the assumptions and outcomes of the model for a specific set of materials and testing conditions. This paper presents the results of a comprehensive experimental program that allows the suitability of the model to be generalized for a wider range of materials and testing conditions. An initial test series was conducted using large-scale soil–geosynthetic interaction test equipment to evaluate the repeatability of the experimental results. A comparison of the test results from this series, as well as an assessment of an extensive database on the expected variability of soil and geosynthetic properties, revealed that the coefficient of variation of the model parameters was acceptable and well within the typical range of similar geotechnical and geosynthetic properties. Results from additional test series confirmed the linearity and uniqueness of the relationship between the geosynthetic unit tension squared and corresponding displacements, which are the key features of the proposed model. These tests were conducted under various conditions using different geosynthetic and backfill materials. Results also showed that the constitutive relationships adopted in the model were adequate for the extended range of confining pressures, geosynthetic lengths, geosynthetic types, and backfill soil types adopted in the study. The consistency of the results obtained in the experimental testing program underscores the suitability of the proposed K_{SGC} parameter as a basis for the evaluation of soil–geosynthetic interactions under small displacements. **DOI: 10.1061/(ASCE)GT.1943-5606.0001769.**

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Introduction

The use of geosynthetics has led to important improvements in geotechnical systems such as retaining structures where polymeric inclusions are used to control the development of conditions leading to collapse (e.g., Allen et al. 1992; Zornberg and Arriaga 2003; Zornberg et al. 1997), and in roadway systems where geosynthetics are used to control deformations (e.g., Al-Qadi et al. 2008; Giroud and Han 2004b, a; Perkins 2002; Perkins et al. 2004; Roodi and Zornberg 2012; Zornberg et al. 2012a, b). Characterization of the soil–geosynthetic interaction under limit states has traditionally been the focus of most previously proposed soil–geosynthetic interaction models. Yet the quantification and characterization of the effectiveness of geosynthetics under small displacements, with a focus on deformation control, has been limited at best. Consequently, the geosynthetic properties used to evaluate limit state conditions are well established for the design and analysis of systems where geosynthetics fulfill the reinforcement function (e.g., geosynthetic-reinforced retaining walls). In contrast, the geosynthetic properties used to evaluate deformability and the response under small displacements are still not clearly defined for the design of systems where geosynthetics fulfill a stiffening function (e.g., geosynthetic-stabilized pavements).

An analytical model, referred to as the soil–geosynthetic composite (SGC) model, was introduced in the companion paper (Zornberg et al. 2017), with a focus on the soil–geosynthetic interaction under small displacements. The analytical solution is characterized by a stiffness parameter referred to as the *stiffness of the soil–geosynthetic composite* (K_{SGC}). This parameter represents the slope of a linear relationship defined between the unit tension squared (T^2) and the geosynthetic displacements (u) in a soil–geosynthetic interaction test. That is

$$T(x)^2 = K_{\mathrm{SGC}}.u(x) \tag{1}$$

The companion paper details the assumptions, formulation, and solution of the SGC model. According to this model, K_{SGC} can be expressed as

$$K_{\mathrm{SGC}} = 4\tau_y.J_c \tag{2}$$

where J_c = geosynthetic stiffness under confined conditions; and τ_y = yield interface shear.

The companion paper provided evidence of the validity of the SGC model using the results of a pilot test. Instead, this paper presents the results of a comprehensive experimental evaluation aimed at examining the repeatability of test results as well as the suitability of the constitutive relationships and outcomes of the SGC model for a wide range of test conditions and materials. Specifically, a test series was conducted to evaluate the repeatability of

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the experimental results. In light of the findings from an extensive assessment of the bibliography, also conducted as part of this study, the variability of the model parameters was evaluated based on the typical ranges of variability of similar geotechnical and geosynthetic properties. Subsequent test series were conducted to evaluate the linearity and uniqueness of the relationship between the unit tension squared (T^2) and displacements (u), as well as the adequacy of the constitutive relationships adopted for the geosynthetic and interface shear for varying confining pressures, geosynthetic lengths, geosynthetic types, and backfill soil types.

Tests were conducted using large-scale soil–geosynthetic interaction equipment and involved rigorous testing procedures that were tailored to evaluate the geosynthetic response under small displacements. The experimental results were analyzed following the procedures described by Zornberg et al. (2017) to determine the constitutive parameters of the model [i.e., the confined geosynthetic stiffness (J_c) and the yield interface shear (τ_y)], as well as the stiffness of the soil–geosynthetic composite (K_{SGC}).

Experimental Testing Program

The testing program was designed to evaluate the suitability of the SGC model for varying testing conditions and materials. Interface direct shear and pullout tests have been the most common testing procedures used to evaluate soil–geosynthetic interactions (e.g., Abdi and Zandieh 2014; Sukmak et al. 2015; Weldu et al. 2016; Xiao et al. 2015). However, the mechanisms that are relevant to applications such as roadway base stabilization require appropriate characterization of the initial stiffness of the soil-geosynthetic composite rather than its shear resistance. Accordingly, the tests conducted in this study involved soil–geosynthetic interaction test equipment based on a large-scale pullout testing device, but the focus was on initial displacement measurements. The experimental evaluation involved a large-scale device to minimize the potential impact on the result of the boundary conditions. Instrumentation and testing procedures were specifically tailored to study soil–geosynthetic interactions under small displacements.

Large-Scale Soil–Geosynthetic Interaction Equipment

The large-scale soil–geosynthetic interaction box used in this study consisted of a steel box with inner dimensions of 1,500 mm (length), 600 mm (width), and 300 mm (depth). The box was mounted on steel rollers equipped with heavy-duty locks, which were engaged during tests to restrain the box against lateral movement. The device also included a 75-mm-long sleeve in the front wall of the box to minimize the development of lateral pressures induced by soil movement toward the front wall during testing. As recommended by ASTM D6706, the box was lined on all side walls with smooth geomembrane sheets to minimize friction with the soil. The front portion of the testing equipment included a roller grip clamp bar connected to the box with hinges. The geogrid specimen was inserted through the sleeve and rolled around the clamp bar before being bolted down. The hinge joints of the clamping bar setup provided flexibility to the loading system and facilitated loading of the geogrid parallel to the length of the box. Two hydraulic pistons were fitted on each side of the box, locked into the clamping setup at the front, and operated by needle valves to maintain a constant displacement rate. The roller grip on the clamping system was interlocked with the piston arrangement.

A load cell of 44.5 kN capacity and 0.04 kN resolution was installed in the soil–geosynthetic interaction test equipment as the point of contact between the clamping system and the hydraulic piston setup. To monitor and control frontal displacements, two linear potentiometers (LPs) of 500 mm range and 0.02 mm resolution were installed on the sides of the box to measure displacements of the hydraulic pistons. As illustrated in Fig. 1, five LPs of 120 mm range and 0.004 mm resolution were installed in the back of the box and connected via telltales to five points along the embedded length of geosynthetic specimen. The telltales consisted of cobalt-based alloy wires, 0.41 mm in diameter, which were inserted into high-strength plastic tubes to minimize friction from direct contact with the surrounding soil. An additional telltale was also installed in the front of the embedded length of the geosynthetic to measure frontal displacements. A data acquisition system and associated *LabVIEW* program were used to convert voltages received from the load cell and the LPs, store the digital data, and monitor testing progress in real-time. The *LabVIEW* code was also used to monitor the piston displacement rate, which was kept constant during testing.

The conventional testing procedure using the large-box interaction device was originally developed to characterize the pullout performance of reinforcements under ultimate conditions. This procedure was significantly modified for the present study to allow collection of the data needed for the SGC model. While the conventional testing procedure may be suitable for defining ultimate strength properties, the modifications were implemented to

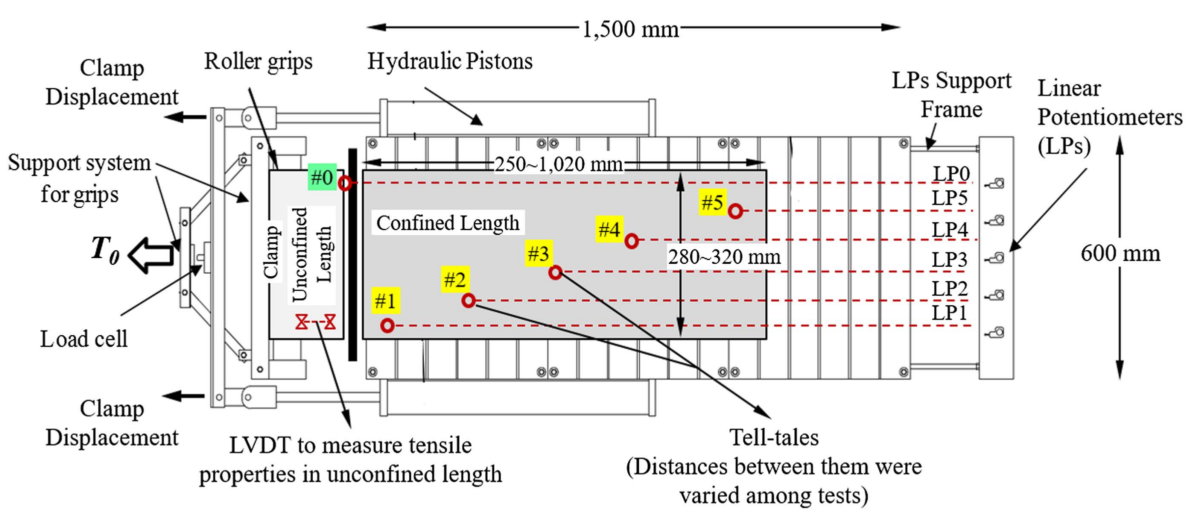

Fig. 1. Schematic view of components and instrumentation in a soil–geosynthetic interaction test

characterize properties relevant under small displacements. Such properties are comparatively more sensitive to sample preparation and testing procedures. Additional details regarding modifications made to the testing procedure are presented by Roodi (2016).

Materials

The backfill material used for several of the tests described in this paper was a clean, poorly graded sand, known as Monterey No. 30 sand. The characteristics of this sand were discussed in the companion paper and also by Zornberg et al. (1998). The sand was placed with a moisture content ranging from 1.5 to 2% and was compacted using hand tampers in four 75-mm-thick layers. The soil unit weight was controlled using the soil mass and the final thickness of each layer after compaction. The dry unit weight of the soil after compaction was measured as 14.94 kN/m^3.

In addition to Monterey sand, a gravelly backfill material was tested to evaluate the validity of the model assumptions and outcomes for a different backfill soil type. The gravel was collected from Martin Marietta Sand and Gravel Quarry in Garfield, Texas. It involves a clean, river-washed pea gravel with rounded particles. The gravel contains particles passing a 3/8" sieve and includes portions retained in all sieves up to Sieve No. 16. This soil is referred to as AASHTO No. 8 because it conforms to specifications of this class of aggregates as specified in AASHTO M43 (AASHTO 2013) and ASTM D448 (ASTM 2012). The average specific gravity of this soil is 2.65 and is classified as poorly graded gravel (GP) according to the Unified Soil Classification System (USCS) [ASTM D2487 (ASTM 2011)] and as Group A-1-a according to the AASHTO classification system [AASHTO M145 (AASHTO 2012); ASTM D3282 (ASTM 2015)]. The AASHTO No. 8 soil was placed dry and compacted using the same procedure adopted to place the Monterey sand. The dry unit weight of the soil after compaction was measured as 16.24 kN/m^3. The main characteristics of Monterey sand and AASHTO No. 8 soil are summarized in Table 1, and their particle size distribution curves are illustrated in Fig. 2.

Two commercially available geosynthetics, commonly used in applications related to roadway base stabilization, were used in the testing program. They included a biaxial geogrid and a geotextile. Specifications of the geosynthetics are summarized in Table 2. The geogrid product was a polypropylene biaxial geogrid with aperture dimensions of 25×33 mm and a minimum rib thickness of 0.76 mm in both directions. The unconfined tensile modulus of the geogrid at 2 and 5% strains has been reported as 330 and 268 kN/m, respectively, in the cross-machine direction (CD) and as 205 and 170 kN/m, respectively, in the machine direction (MD). The geotextile product was also made of polypropylene. The unconfined tensile modulus of the geotextile product at 2 and 5% strains has been reported as 965 and 760 kN/m, respectively, in the CD and as 700 kN/m in the machine direction (MD). As presented in Table 2, the ultimate tensile strength of the geotextile was reported to be higher than that of the geogrid in both directions. The two geosynthetic products used in this study are referred to as GGPP1 (for the geogrid) and GT (for the geotextile).

Scope of Testing Program

Five series of large-scale soil–geosynthetic interaction tests were designed to evaluate the suitability of the SGC model. As presented in Table 3, Test Series A involved three identical tests aimed at

Table 1. Characteristics of Backfill Materials Used in This Study

Test	Index parameter	Monterey No. 30 sand	AASHTO No. 8 soil	Standard
Soil classification	—	SP	GP	ASTM D2487 (ASTM 2011)
		A-1-b	A-1-a	AASHTO M 145 (AASHTO 2012)
Specific gravity	Specific gravity, G_s	2.655	2.65	ASTM D854 (ASTM 2014)
Grain size distribution	D_{10} (mm)	0.28	4.8	ASTM D422 (ASTM 2007b)
	D_{30} (mm)	0.41	6.1	
	D_{60} (mm)	0.50	7.5	
	C_u	1.8	1.6	
	C_c	1.2	1.0	
Minimum void ratio	e_{min}	0.56	0.54	ASTM D4253 (ASTM 2016a)
Maximum void ratio	e_{max}	0.76	0.73	ASTM D4254 (ASTM 2016b)

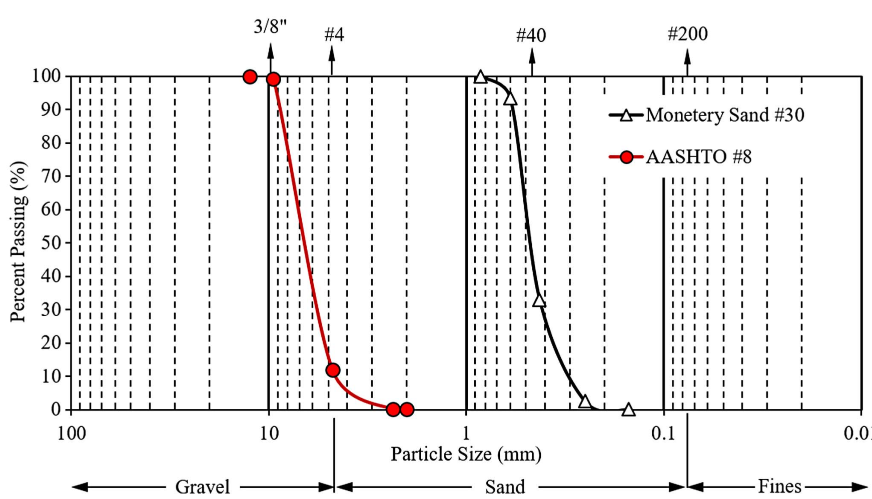

Fig. 2. Gradation curves of backfill materials used in this study

Table 2. Characteristics of Geosynthetic Products Used in This Study

Properties	Geogrid (GGPP1) Machine direction (MD)	Geogrid (GGPP1) Cross-machine direction (CD)	Geotextile (GT) Machine direction (MD)	Geotextile (GT) Cross-machine direction (CD)
Polymer composition	Polypropylene		Polypropylene	
Aperture dimensions (mm)	25	33	—	—
Aperture stability (m-N/deg)	0.32		—	—
Minimum rib thickness (mm)	0.76	0.76	—	—
Rib width (mm)	3.30	3.30	—	—
Aspect ratio of ribs (ratio of the thickness to width)	0.23	0.23	—	—
Tensile modulus at 2% strain (kN/m)	205	330	700	965
Tensile modulus at 5% strain (kN/m)	170	268	700	760
Ultimate tensile strength (kN/m)	12.4	19	70	70

Table 3. Scope of Large-Scale Soil–Geosynthetic Interaction Testing Program for Evaluation of the Suitability of the SGC Model

Test series	Objective	Number of repeats	Backfill soil	Confining pressure [kPa (psi)]	Geosynthetic	Specimen dimensions (mm)	Test direction
Series A (baseline tests)	Evaluation of repeatability of test results	3	Monterey No. 30 sand	21 (3)	GGPP1	320 × 590	CD
Series B	Evaluation of suitability of model for varying confining pressures	1	Monterey No. 30 sand	7 (1) 21 (3) 35 (5)	GGPP1	320 × 590	CD
Series C	Evaluation of suitability of model for varying geosynthetic lengths	1	Monterey No. 30 sand	21 (3)	GGPP1	320 × 250 320 × 590 320 × 1,020	CD
Series D	Evaluation of suitability of model for varying geosynthetic types	1	Monterey No. 30 sand	21 (3)	GGPP1 and GT	320 × 590	CD
Series E	Evaluation of suitability of model for varying backfill soil types	1	Monterey No. 30 sand and AASHTO No. 8 soil	21 (3)	GGPP1	280 × 590	CD

Note: CD = cross-machine direction.

evaluating the repeatability of the test results. This series was conducted using the conditions defined herein as those for the *baseline test*, which involved testing of a geogrid (GGPP1) in the CD under a confining pressure of 21 kPa. The baseline test conditions were selected as being representative of geosynthetics in base stabilization applications, where the response under small displacements governs the performance of the SGC system. In this application, the vertical deflections in the wheel path and the lateral spreading of base course material can be mitigated by the interaction between the base aggregate and the geosynthetic. Moreover, interaction of the base course aggregate with the geosynthetic in the CD has been reported to delay the initiation and propagation of longitudinal cracks induced by environmental loads. The confining pressure of 21 kPa is representative of the normal pressure that geosynthetics are subjected to in a typical stabilized pavement system. The dimensions of the geosynthetic specimens in the baseline test were selected following recommendations in ASTM D6706 (ASTM 2007a). The width of the specimen was selected as 320 mm to allow for approximately 150 mm of clearance on each side of the specimen from the side walls of the box. The embedment length of the specimen was selected as approximately 600 mm to maintain a minimum embedment length-to-width ratio of 2, as suggested in ASTM D6706.

Test Series B to E were designed to evaluate the validity of the SGC model assumptions and outcomes for varying test conditions and materials. In particular, the suitability of the SGC model was evaluated for varying confining pressures, geosynthetic lengths, geosynthetic types, and backfill soil materials. In each test series, the varying parameter was changed while other test conditions were maintained equal to those in the baseline test. Specifically, in Test Series B the confining pressure was varied from 21 to 35 kPa, while in Test Series C the geosynthetic length was varied from about 250 to 1,000 mm. In Test Series D, the geosynthetic type was changed to a geotextile, and in Test Series E the backfill soil was changed to AASHTO No. 8 soil. Baseline test results were compared to the results obtained from interaction tests in each test series. Furthermore, the suitability of the assumptions and outcomes of the model were also evaluated for results obtained from test series. In particular, analysis of the results obtained from each test series involved the following key aspects:

- Evaluation of the linearity of the relationship between unit tension squared (T^2) and telltale displacements (u), which leads to the determination of the stiffness of the soil–geosynthetic composite (K_{SGC}).
- Evaluation of the uniqueness of the K_{SGC} parameter within the embedded length of the geosynthetic, which can be achieved by analyzing the unit tension squared (T^2) versus telltales displacement (u) data measured in the multiple telltales along the embedded length of the geosynthetic.
- Evaluation of the suitability of the constitutive model adopted for the soil–geosynthetic interface shear, resulting in the determination of the yield interface shear (τ_y). This can be achieved by analyzing the frontal unit tension at the times when displacements in the telltales were first triggered versus the location of the telltales, as described by Zornberg et al. (2017).
- Evaluation of the suitability of the constitutive model adopted for the geosynthetic, resulting in the determination of the confined geosynthetic stiffness (J_c). This can be achieved by analyzing the confined unit tension versus corresponding tensile strain data, as described by Zornberg et al. (2017).

Evaluation of Repeatability of Test Results

The experimental results from Test Series A, which aimed at evaluating the repeatability of the experimental data, are discussed in this section. This includes an evaluation of the expected variability of the experimental results.

Expected Variability of the Properties Defined in This Study

The analytical model and testing procedures developed as part of this study led to a new stiffness parameter, K_{SGC}, that combines the confined geosynthetic stiffness and soil–geosynthetic shear properties into a single variable. While variability in the experimental results was expected, a basis was deemed necessary to define what constitutes an acceptable degree of variability. Consequently, as presented in the appendix, an extensive assessment of the bibliography was initially conducted on the various sources of variability in geotechnical and geosynthetic properties. Findings from this assessment were used to establish an acceptable range of variability expected for the experimental parameters reported in this study, which included measurements from soil–geosynthetic interaction tests. More specifically, the focus was on the stiffness of the soil–geosynthetic composite (K_{SGC}), which relies on force and displacement data obtained under small displacements. The box and associated loading systems remained unchanged throughout the entire research, although testing procedures, instrumentation, calibration, and operators changed over time. The same sandy fill was used throughout the testing period. A virgin geosynthetic specimen was used in each test, as specimens were tested under tension and deformed by the end of the tests. When possible, geosynthetic specimens were selected from a single roll, and caution was exercised while cutting specimens from the roll to minimize the variability associated with changes in geometry and aperture size. However, the *inherent manufacturing variability* of geosynthetic specimens was expected to contribute to variability of the test results.

Since the specimens were mostly selected from the same manufacturing roll and tested in the same box following the same test standard, the variability expected from *repeatability* tests was deemed the most relevant to this study. Furthermore, since the focus of this study was on the stiffness of a soil-geosynthetic composite under small displacements, the coefficient of variation from the *inherent manufacturing variability* of geosynthetic specimens was expected to be consistent with the upper range of values obtained for geosynthetic stiffness in *repeatability* tests reported in the appendix, which exceeds 28%.

Additionally, as the SGC model and experimental procedures developed in this study focused on the interaction between soil and geosynthetic, the expected variability should have been affected not only by the variability associated with each material but also by the variability resulting from their interaction. Since the same soil was used in all tests, the *inherent variability* attributed to soil changes would be small. However, the extent of the variability attributed to the soil–geosynthetic interaction mechanism is unknown.

A relevant contribution to the overall variability of the experimental results in this study is the variability resulting from *measurement errors* (i.e., *bias errors* and *random errors*). The impact of *bias errors* on the experimental results was partially minimized by using the same box and associated accessories; however, changes in testing procedures, instrumentation, calibration, and operators inevitably contributed to uncertainties in the results. *Random errors* were also expected to inevitably affect the experimental results in this study. Owing to the complexity of the testing procedures and data reduction processes, *measurement errors* in the experimental results are expected to be consistent with the variability of results in relatively complex laboratory tests, such as soil shear strength and consolidation tests. Furthermore, since the focus of the model is on stiffness measurement under small displacements and not on the maximum strength or ultimate capacity of the system, associated *measurement errors* were expected to approximate the upper limits of the ranges reported in the appendix for stiffness measurements.

Evaluation of the variability data for soil properties (presented in the appendix) for the described conditions of the experiments suggests that the expected overall variability associated with soil variation and *measurement errors* ranges from as low as 10% to over 30%. The *inherent variability* of the backfill soil was expected to contribute the least and *measurement errors* were expected to contribute the most to this overall variability.

Considering the variability associated with changes in geosynthetic specimens and those associated with changes in soil and testing procedures, the overall variability of the experimental results of this study is expected to exceed 30%. It should be noted that, as in every laboratory testing measurement, *model uncertainty* is present in this overall variability.

Repeatability of Frontal Load Versus Frontal Displacement Data

In Test Series A, three tests were conducted using the baseline test conditions described in Table 3. The testing conditions were identical across the various tests.

Fig. 3 shows the frontal unit tension versus frontal displacement curves for the three tests in Test Series A. In all three, very good agreement was observed in the frontal force–frontal displacement responses, and low variability in the ultimate frontal load was obtained. Specifically, the ultimate interface shear (τ_{ult}) was found to range from 15.1 to 16.6 kN/m^2. However, the focus of the analytical model was on the initial displacements in the embedded portion of geosynthetic specimens, which is discussed next.

Repeatability of the Stiffness of the Soil–Geosynthetic Composite (K_{SGC})

The procedure developed in this study to determine K_{SGC} involved direct measurement of the slope of the relationship between the unit tension squared and telltale displacement data at small displacements. Geosynthetic unit tension at the location of the five telltales was obtained following the procedure described by Zornberg et al. (2017). As presented in Fig. 4, the unit tension squared (T^2) was plotted against the telltale displacements (u), which were directly measured by LPs. It should be noted that only the displacement

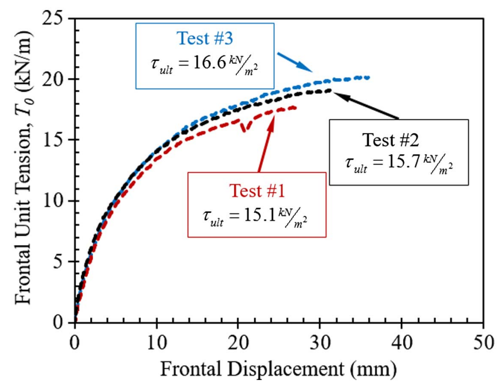

Fig. 3. Frontal unit tension versus frontal displacement data for tests in series A (evaluation of repeatability)

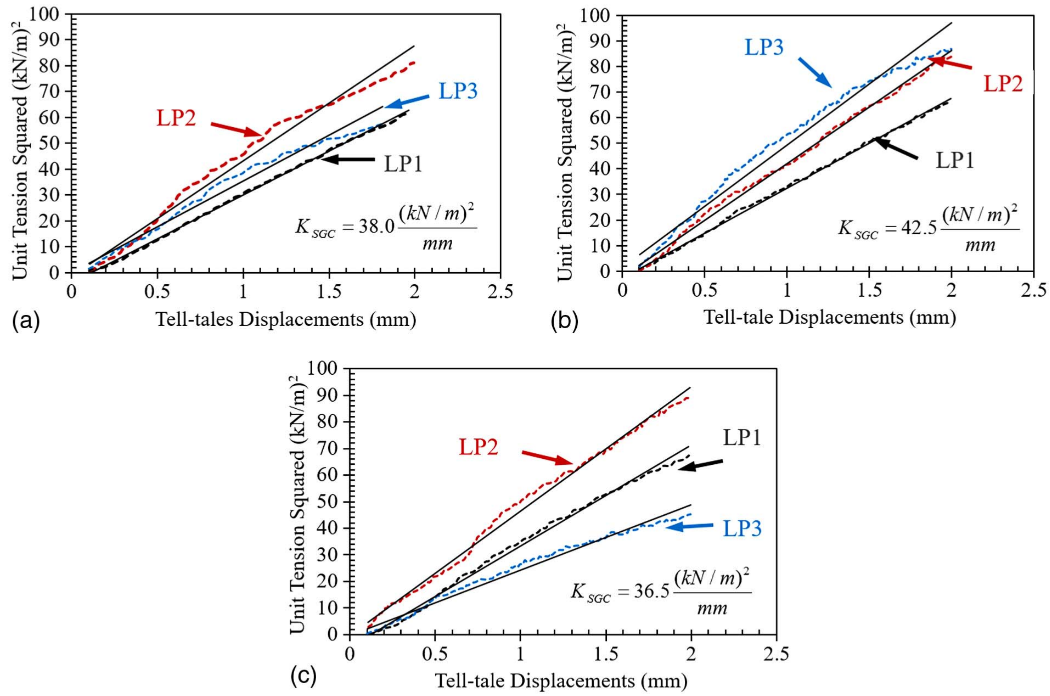

Fig. 4. Experimental evaluation of the repeatability of K_{SGC}: (a) results from Test #1; (b) results from Test #2; (c) results from Test #3

data recorded by LPs 1, 2, and 3 were considered in this analysis. Results from LPs 4 and 5, located near the free end of the geosynthetic, were not used because they could have been affected by the proximity to the geosynthetic boundary.

The linearity of the data presented in Fig. 4 can be clearly observed, confirming the adequacy of the SGC model, which predicts a linear relationship between T^2 and u. The slope of the regression lines corresponds to the K_{SGC} value. According to the SGC model, this slope is expected to be essentially the same at any location within the active length of the specimen.

Fig. 4 also shows the regression lines defined using the T^2 versus u data for the three LPs in the three equivalent tests. The slopes of the regression lines were found to vary to limited extents, although in no particular order among the LPs. However, except for data from LP 3 in Test 3, the slopes defined by data from all LPs lie within a comparatively narrow range. The slope obtained from LP 3 data in Test 3 was found to be somewhat smaller, which may be attributed to low local confinement around the location of Telltale 3 or to problems associated with the telltale's attachment to geosynthetic. The K_{SGC} values reported in Fig. 4 correspond to the average slope among the three regression lines obtained using data from the three LPs.

Repeatability of the Yield Interface Shear (τ_y) and the Confined Geosynthetic Stiffness (J_c)

As detailed by Zornberg et al. (2017), the experimental data obtained in the soil–geosynthetic interaction tests can be used to evaluate the adequacy of the constitutive models adopted for the geosynthetic materials and the soil–geosynthetic interface shear.

The parameter τ_y for the soil–geosynthetic interface shear was obtained by plotting the frontal unit tension corresponding to the time when each telltale was first triggered versus the corresponding location of that telltale. Fig. 5 shows the results for each test in Test Series A. The linearity of the plotted points indicates that a constant interface shear progressively mobilized from the loading front toward the free end of the specimen, confirming the suitability of the adopted constitutive model for the soil–geosynthetic interface shear.

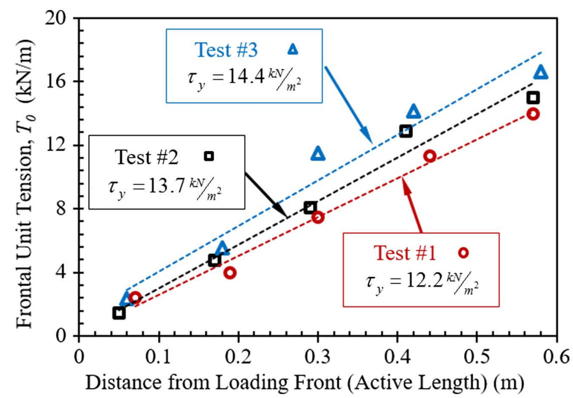

Fig. 5. Experimental evaluation of the repeatability of τ_y

The repeatability of the yield interface shear (τ_y) was evaluated by comparing the slope values obtained for the three tests. The yield interface shear (τ_y) was characterized as half of these slope values. As shown by the data presented in Fig. 5, the yield interface shear values estimated using data from the three tests were found to agree reasonably well. The lowest yield interface shear value was 12.2 kN/m for Test #1, and the highest value was 14.4 kN/m for Test #3. The range of values obtained for the yield interface shear was comparatively narrow, underscoring the good repeatability of the results.

A detailed procedure to define the confined geosynthetic stiffness (J_c) is presented by Zornberg et al. (2017). This stiffness was obtained from the unit tension versus tensile strain data (for comparatively small strain values) within the confined geosynthetic portion. The unconfined tensile properties of each geosynthetic specimen were also obtained by the data recorded by the linear variable differential transformer (LVDT) sensor installed in the unconfined length of the geosynthetic (Fig. 1). Fig. 6 presents the unit tension versus strain data measured directly in the unconfined geosynthetic portion as well as the tensile data predicted by the analytical model using the displacements measured by the LPs connected to the embedded geosynthetic portion. Because comparatively small deformations were obtained in the confined geosynthetic portion, the data obtained here were limited to tensile strains below 1%. The slope of the regression line defined using data from the confined portion characterizes the confined geosynthetic stiffness (J_c). Consistent with the analytical model, J_c was found to be similar to the initial slope defined using data from the unconfined geosynthetic portion.

As illustrated in Fig. 6, reasonably good agreement was found among the confined stiffness values obtained from all tests conducted as part of Test Series A. Specifically, the confined stiffness ranged from 608 to 727 kN/m, which was deemed to represent a comparatively small variability for stiffness measurements. In particular, the variability of the confined geosynthetic stiffness was found to be comparatively lower than that of the unconfined stiffness. The confined stiffness values (estimated for strains below 1%) and the unconfined stiffness at 1, 2, 5, and 7% strains are summarized in Table 4 for the three tests. As discussed in the assessment of expected variability, larger variations are expected for stiffness parameters defined at small displacements or strains. This trend is confirmed by inspecting the stiffness values obtained for the unconfined portion of the geosynthetic specimens. Stiffness values in the confined portion of the geosynthetic specimens are defined at significantly lower strain levels than those in the unconfined portion, and, hence, considerably larger variability was expected for this parameter. However, inspection of the stiffness values presented in Table 4 reveals that the variability of the confined stiffness is of the same order of magnitude as that of the unconfined stiffness for strain values below 2%. These results provided additional evidence of the good repeatability of the experimental results used to estimate the confined geosynthetic stiffness (J_c) in the SGC model.

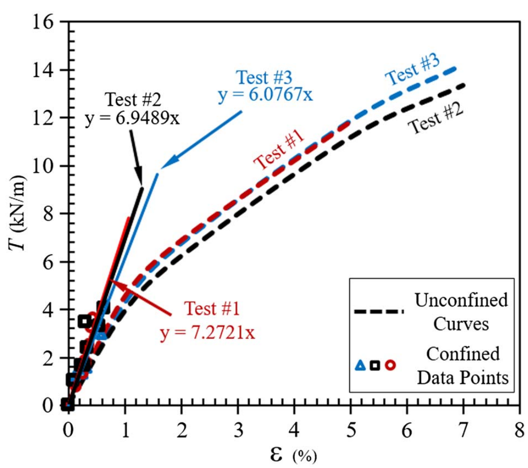

Fig. 6. Experimental evaluation of the repeatability of J_c

Table 4. Summary of Confined and Unconfined Geosynthetic Stiffness in Test Series A (Evaluation of Repeatability)

Property	Test 1	Test 2	Test 3
Confined stiffness (J_c) at 1% strain (kN/m)	727	695	608
Unconfined stiffness at 1% strain (kN/m)	500	391	426
Unconfined stiffness at 2% strain (kN/m)	350	310	337
Unconfined stiffness at 5% strain (kN/m)	240	223	237
Unconfined stiffness at 7% strain (kN/m)	—	190	203

As discussed by Zornberg et al. (2017), τ_y and J_c can also be estimated from the parabolic regression of the telltale displacement data. According to the analytical model, the relationship between telltale displacements and the location of the telltales along the active length of the specimen corresponds to a parabola whose coefficients can be used to define τ_y and J_c. Fitting a parabolic function to the experimental data obtained from telltales, using the least-squares method, allows determination of the τ_y and J_c values. Fig. 7 presents the results obtained following this procedure for the three tests in Test Series A. It should be noted that the fitting was conducted at various levels of the frontal load and for small displacements. The sum of the squares of the errors, S, did not exceed 2.6 mm^2. The confined geosynthetic stiffness (J_c) ranged from 585 to 677 kN/m, while τ_y was found to range from 12.5 to 14.4 $\mathrm{kN/m^2}$. Accordingly, K_{SGC}, estimated using Eq. (2), ranged from 33.6 to 35.2$(\mathrm{kN/m})^2/\mathrm{mm}$, which is within the range of values obtained in Fig. 4 from T^2 versus u plots.

Discussion on the Repeatability of Test Results

Several procedures can be used to characterize the parameters of the SGC model using the experimental data generated from the three tests in Test Series A. Evaluation of the variability of values estimated using the different procedures provides additional insight on the repeatability of the experimental results. Specifically, the K_{SGC} value was estimated using the following three procedures:

- Procedure 1: Direct estimation of K_{SGC} from the unit tension squared (T^2) versus displacement (u) experimental data recorded using each of the telltales
- Procedure 2: Estimation of the constitutive parameters using experimental data as described in the previous section, and using Eq. (2) to define K_{SGC}
- Procedure 3: Estimation of the constitutive parameters obtained from parabolic regression of telltale displacement data, and using Eq. (2) to define K_{SGC}

While the various procedures should lead to the same values for the three parameters, investigation of the consistency among the values predicted by them provides additional insight on the suitability of the analytical model. Table 5 summarizes the values for the three model parameters obtained using the various procedures. Quantification of the variability among these results is presented in Table 6, which summarizes the variation of each parameter among the different procedures and among the equivalent tests.

Inspection of the results presented in Table 6 reveals a reasonably low variability for the three parameters. The lowest coefficient of variation was obtained for the strength parameter of the model, i.e., the yield interface shear, as 7%. The coefficients of variation for the confined geosynthetic stiffness (J_c) and the stiffness of the soil–geosynthetic composite (K_{SGC}) were 9 and 16%, respectively. The comparatively higher variability obtained for K_{SGC} is consistent with the fact that, in line with Eq. (2), this parameter involves a combination (product) of properties that characterize both the geosynthetic stiffness and the soil–geosynthetic interaction. Consequently, the variability of K_{SGC} is affected by uncertainties involved in the determination of both properties and, hence, is expected to be higher than the variability of each individual property.

To further assess the adequacy of the variability obtained for the model parameters, a database of unconfined tensile strength and pullout tests carried out using the same geosynthetic product and soil type at the geosynthetics laboratory at the University of Texas was evaluated. This database included the results of 8 large-scale pullout tests and over 10 unconfined tensile strength tests. The coefficient of variation for the ultimate pullout resistance from

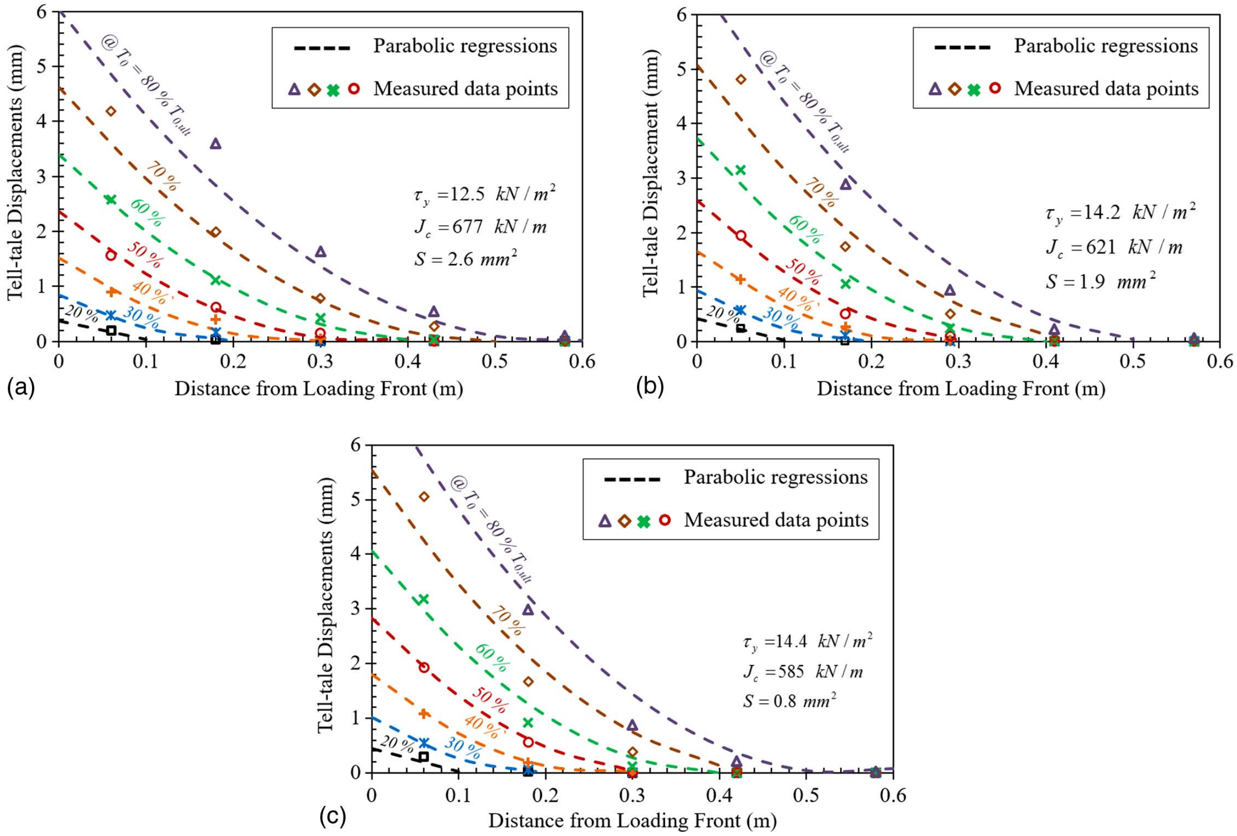

Fig. 7. Evaluation of the repeatability of the experimental data by parabolic regression of telltale displacement data: (a) results from Test #1; (b) results from Test #2; (c) results from Test #3

Table 5. Summary of Estimated Model Parameters in Test Series A (Evaluation of Repeatability)

Procedure	Parameter	Unit	Test 1	Test 2	Test 3
Direct estimation of K_{SGC} from T^2 versus u data	K_{SGC}	$\frac{(\mathrm{kN/m})^2}{\mathrm{mm}}$	38.0	42.5	36.5
Direct estimation of τ_y and J_c	τ_y	$\mathrm{kN/m^2}$	12.2	13.7	14.4
	J_c	kN/m	727	695	608
	$K_{\mathrm{SGC}} = 4\tau_y J_c$	$\frac{(\mathrm{kN/m})^2}{\mathrm{mm}}$	35.5	38.1	34.8
Parabolic regression of telltale displacement data	τ_y	$\mathrm{kN/m^2}$	12.5	14.2	14.4
	J_c	kN/m	677	621	585
	$K_{\mathrm{SGC}} = 4\tau_y J_c$	$\frac{(\mathrm{kN/m})^2}{\mathrm{mm}}$	33.9	35.2	33.6
	S	$\mathrm{mm^2}$	2.6	1.9	0.8

Table 6. Variability of Model Parameters Obtained for Test Series A (Evaluation of Repeatability)

Parameter	Number of data	Minimum	Maximum	Average	Standard deviation	Coefficient of variation (%)
Yield interface shear (τ_y) ($\mathrm{kN/m^2}$)	6	12.2	14.4	13.55	0.96	7
Confined stiffness (J_c) (kN/m)	6	585	727	652	55	9
Stiffness of soil–geosynthetic composite (K_{SGC}) $(\mathrm{kN/m})^2/\mathrm{mm}$	15	34.8	42.5	37.5	6.1	16

repeatability pullout tests was 7% and the coefficient of variation for the ultimate tensile strength was 2% (Table 7). The larger variability in the ultimate pullout resistance can be attributed to the higher expected uncertainty of the soil–geosynthetic interaction. While the coefficient of variation for the ultimate tensile strength is mainly affected by the inherent variability of the geosynthetic in isolation (i.e., without soil involvement), the coefficient of variation for the ultimate pullout resistance is affected by the inherent variability of the soil and of the geosynthetic, as well as the uncertainties from the soil–geosynthetic interaction.

The variability of the geosynthetic tensile modulus was found to be affected by the strain level at which measurements are taken.

Table 7. Variability of Pullout and Tensile Strength Test Results from an Internal Database

Test	Property	Coefficient of variation (%)
Pullout test	Ultimate pullout resistance (16.3, 16.6, 15.7, 14.5, 15.1, 17.9, 15.6, 16.9 kN/m)[b]	7[a]
Wide-width tensile test	Ultimate tensile strength (17.4, 16.8, 16.8, 16.7, 17.1, 15.9, 16.7, 17.1, 16.9, 17.1 kN/m)[b]	2[a]
	Tensile modulus	
	At 1% strain (452, 404, 313, 325, 441, 306, 399, 458, 312, 465 kN/m)[b]	18
	At 2% strain (344, 315, 310, 316, 338, 296, 328, 369, 298, 336 kN/m)[b]	6
	At 5% strain (237, 222, 231, 240, 238, 226, 241, 249, 229, 231 kN/m)[b]	3

[a]From tests conducted on same-lot specimens.
[b]Individual test results.

The coefficient of variation at 1, 2, and 5% strain for the geosynthetic tensile modulus was found to be 18, 6, and 3%, respectively.

The magnitude of the coefficient of variation for the yield interface shear (τ_y) in Table 6 compares well with that of the ultimate pullout resistance in Table 7 because both are strength properties that are affected by soil–geosynthetic interaction mechanisms. The coefficients of variation obtained for the stiffness parameters in Table 6 (i.e., J_c and K_{SGC}) compared well with that of the tensile modulus at 1% strain in Table 7. Since J_c and K_{SGC} are estimated at small displacements and under confinement, comparatively higher coefficients of variation could be expected for these values than for those obtained for the unconfined tensile modulus. However, inspection of the results presented in Tables 6 and 7 indicates that the coefficients of variation for J_c and K_{SGC} are indeed smaller than that for the tensile modulus.

Overall, the variability of the various parameters in the SGC model was found to be adequate because they were found to be consistent with the ranges of variability reported for other geotechnical and geosynthetic parameters used to describe shear strength and stiffness properties.

Evaluation of the Validity of the SGC Model for Varying Test Conditions and Materials

The adequacy of the assumptions of the SGC model was validated by Zornberg et al. (2017) for the materials and test conditions adopted in the pilot test reported in that paper. In this section, the adequacy of the model is evaluated with additional experimental results obtained using different materials and test conditions than those adopted in the pilot study. This included assessment of the validity of the assumptions and outcomes of the model for (1) different confining pressures in Test Series B, (2) different geosynthetic lengths in Test Series C, (3) different geosynthetic types in Test Series D, and (4) different backfill soil types in Test Series E.

Validity of the SGC Model for Varying Confining Pressures

In Test Series B, soil–geosynthetic interaction tests were conducted at confining pressures ranging from 7 to 35 kPa, while the materials and other test conditions remained the same as those in the baseline tests. The range of confining pressures was selected to be representative of conditions in applications involving geosynthetic-stabilized roadways, where the response under small displacements governs the performance. Also, higher confining pressures would result in tensile breakage of the geosynthetic used in this study, before full mobilization of the interface shear.

The results obtained following evaluation of the experimental data from this test series are presented in Fig. 8. As expected, the frontal unit tension versus frontal displacement results [Fig. 8(a)] showed that the ultimate resistance increases with increasing confining pressure. The failure mode under confining pressures of 7 and 21 kPa involved pullout of the geosynthetic specimens, while the specimen broke under tension at a confining pressure of 35 kPa. However, the 35-kPa test still provided adequate data for the analysis of the SGC model prior to breakage.

The suitability of the linear relationship between the unit tension squared (T^2) and telltale displacement (u) data was investigated by evaluation of the results presented in Fig. 8(b). The T^2 versus u data defined the linear relationships for all confining pressures evaluated in the study, confirming the linearity of the model for varying confining pressures. Furthermore, the slopes of the lines obtained from data recorded by various LPs were found to be within a comparatively narrow range for each confining pressure value. This underscored the validity of the uniqueness of the T^2 versus u relationship throughout the active length of the specimen for a given confining pressure. The slopes of the lines characterized the stiffness of the soil–geosynthetic composite (K_{SGC}) in each case. As shown in Fig. 8(b), increasing K_{SGC} values (i.e., increasing slopes) result from increasing confining pressures, which illustrates the impact of confining pressure on K_{SGC}.

Evaluation of the data presented in Fig. 8(c) indicates that, while displacements were triggered for all five telltales under 7 and 21 kPa, only four of them were triggered under a confining pressure of 35 kPa. This indicates that progressive mobilization of interface shear from the loading front toward the free end of the specimen was fully realized under confining pressures of 7 and 21 kPa. However, mobilization under 35 kPa continued only up to (or slightly after) the location of Telltale 4.

The constitutive parameters of the model were estimated from the experimental data presented in Figs. 8(c and d). The yield interface shear (τ_y) was obtained from the slopes of the lines in Fig. 8(c), and the confined geosynthetic stiffness (J_c) was estimated by the slope of the data presented in Fig. 8(d). The reasonably linear relationship defined by the experimental data for the different confining pressures in Figs. 8(c and d) underscores the suitability of the constitutive relationships of the model for varying confining pressures. According to the SGC model, the geosynthetic stiffness in both unconfined and confined conditions is a characteristic of the geosynthetic material, so it is not expected to change with confining pressure. This is consistent with the experimental results shown in Fig. 8(d), because reasonably good agreement was found among the curves defined using results obtained for multiple tests under both unconfined and confined conditions. From the linear regression of the data points along the confined length, J_c was found to range from 678 to 702 kN/m. On the other hand, τ_y, estimated from the slope of the regression lines in Fig. 8(c), increased with increasing confining pressure, as expected.

The constitutive parameters of the model (τ_y and J_c) were also estimated from the parabolic regression of telltale displacement data, and K_{SGC} was also estimated following additional procedures previously described (i.e., Procedures 2 and 3). The results obtained are summarized in Table 8. Inspection of the results presented in this table indicates reasonably good agreement among the outcomes of the various procedures for increasing confining pressures.

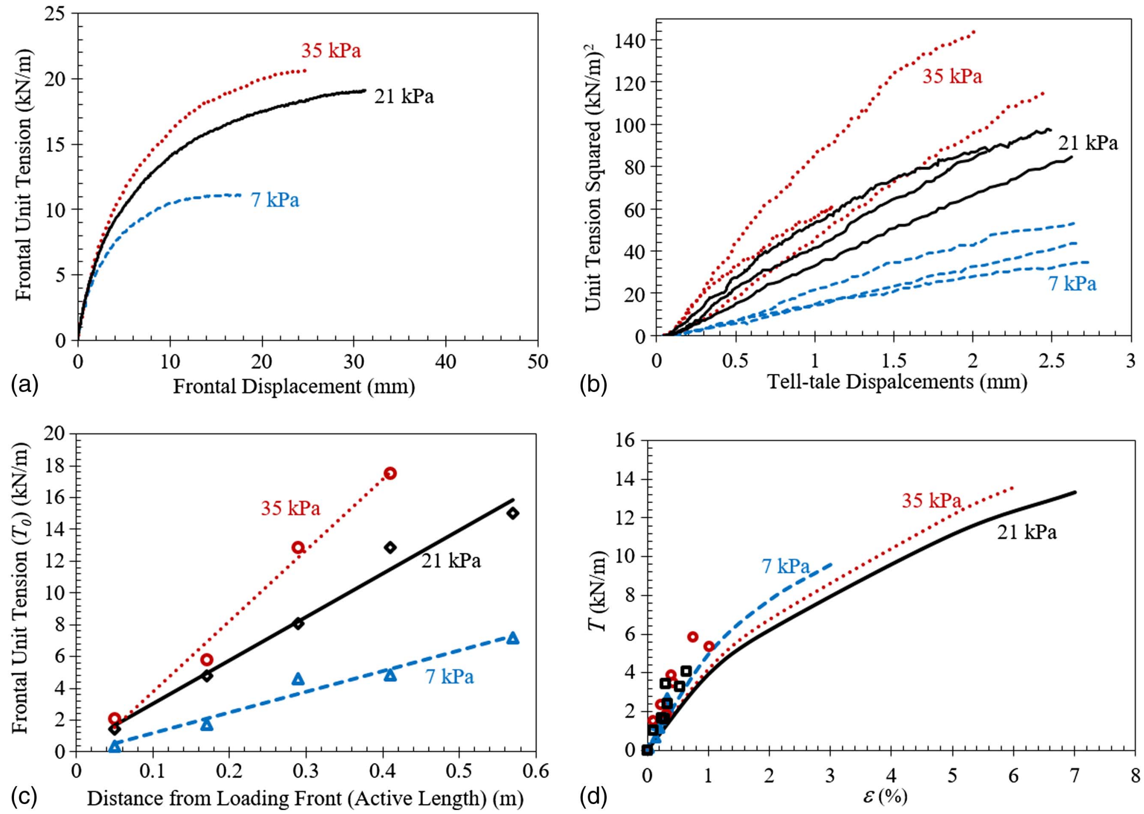

Fig. 8. Experimental results from Test series B (varying confining pressures): (a) frontal load-frontal displacement results; (b) comparison of K_{SGC} results; (c) comparison of τ_y results; (d) comparison of J_c results

Table 8. Summary of Estimated Model Parameters for Varying Materials and Test Conditions

				Test Series B: varying confining pressures (σ)		Test Series C: varying geosynthetic length (L)		Test Series D: geosynthetic type	Test Series E: backfill soil
Procedure	Parameter	Unit	Baseline test[a]	Lower σ (7 kPa)	Higher σ (35 kPa)	Shorter specimen (250 mm)	Longer specimen (1,020 mm)	Geotextile (GT)	AASHTO No. 8 soil
Direct estimation of K_{SGC} from T^2 versus u data	K_{SGC}	$\frac{(\mathrm{kN/m})^2}{\mathrm{mm}}$	42.5	19.4	59.6	39.9	41.9	156	106
Direct estimation of τ_y and J_c	τ_y	kN/m^2	13.7	6.5	22.3	11.0	14.6	19.2	17.6
	J_c	kN/m	695	702	678	804	660	1,838	1,009
	K_{SGC}	$\frac{(\mathrm{kN/m})^2}{\mathrm{mm}}$	38.1	18.2	60.5	35.5	38.6	141.2	71.0
Parabolic regression of telltale displacement data	τ_y	kN/m^2	14.2	7.8	21.8	11.7	14.9	18.2	16.2
	J_c	kN/m	621	603	605	773	630	2,260	1,148
	K_{SGC}	$\frac{(\mathrm{kN/m})^2}{\mathrm{mm}}$	35.2	18.8	52.8	36.2	37.5	164.8	74.2
	S	mm^2	1.9	2.5	4.2	1.2	3.6	1.0	5.0

[a]The baseline test was conducted using a 590-mm-long geogrid (GGPP1) specimen and Monterey sand backfill under a confining pressure of 21 kPa.

Fig. 9 summarizes the experimental results obtained in Test Series B. As illustrated in Fig. 9(a), the yield interface shear increases linearly from 7 to 22 kN/m^2 as confining pressure increases from 7 to 35 kPa. A similar trend is observed for the ultimate frontal load for specimens that failed in pullout mode (i.e., with the exception of the test at 35 kPa, which failed under tension).

The results presented in Fig. 9(b) further underscored the suitability of K_{SGC} to represent soil–geosynthetic composite stiffness properties. While the confined geosynthetic stiffness (J_c) remains essentially constant for varying confining pressures, the K_{SGC} value increases following an approximately linear trend with increasing confining pressures.

Validity of the SGC Model for Varying Geosynthetic Lengths

Test Series C was conducted to evaluate the suitability of the SGC model for varying geosynthetic lengths. Since the SGC model

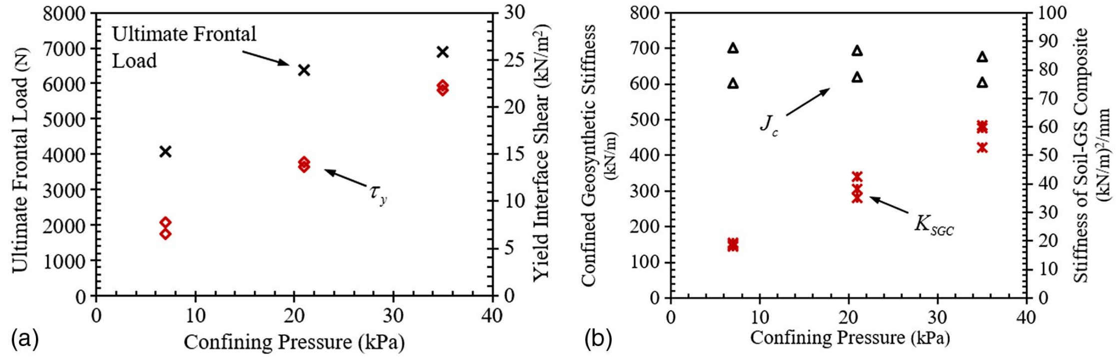

Fig. 9. Comparison of model parameters obtained from Test series B (varying confining pressures): (a) ultimate frontal load and yield interface shear; (b) confined stiffness and the stiffness of soil–geosynthetic composite

relies on identifying the onset of the soil–geosynthetic relative displacement as interface shear progressively mobilizes, the experimental results were not expected to be affected by the geosynthetic length. However, the geosynthetic length should have been long enough to allow progressive mobilization of interface shear. The results from baseline tests conducted using a 590 mm-long specimen were compared to results from tests conducted under the same test conditions, but using 1,020 and 250 mm-long specimens. To avoid the potential impact of boundary on the results, shorter specimens were not used, as progressive mobilization of interface shear may not have been realized for particularly short specimens. Longer specimens were also considered unnecessary because they may reach the tensile strength before mobilizing the interface shear along the additional specimen length.

The results obtained after evaluation of the experimental data from this test series are presented in Fig. 10. Evaluation of the frontal unit tension versus frontal displacement data, presented in Fig. 10(a), indicates that the initial responses of the three tests are in reasonably good agreement. The shorter, 250 mm-long specimen failed in pullout mode at a unit tension of approximately 12 kN/m, while the longer specimens exhibited almost identical responses until the end of the tests. These responses suggest similar trends in progressive mobilization of the interface shear from the loading front toward the free end of the specimen.

The linearity and uniqueness of the unit tension squared (T^2) versus displacement (u) data were validated for varying geosynthetic lengths by evaluating the data presented in Fig. 10(b). The experimental data presented in this figure resulted in reasonably well-defined linear relationships between T^2 and u in all cases. The K_{SGC} values, defined by the slopes of these linear relationships, were found to be within a reasonably narrow range for the data obtained using the different telltales for specimens with different

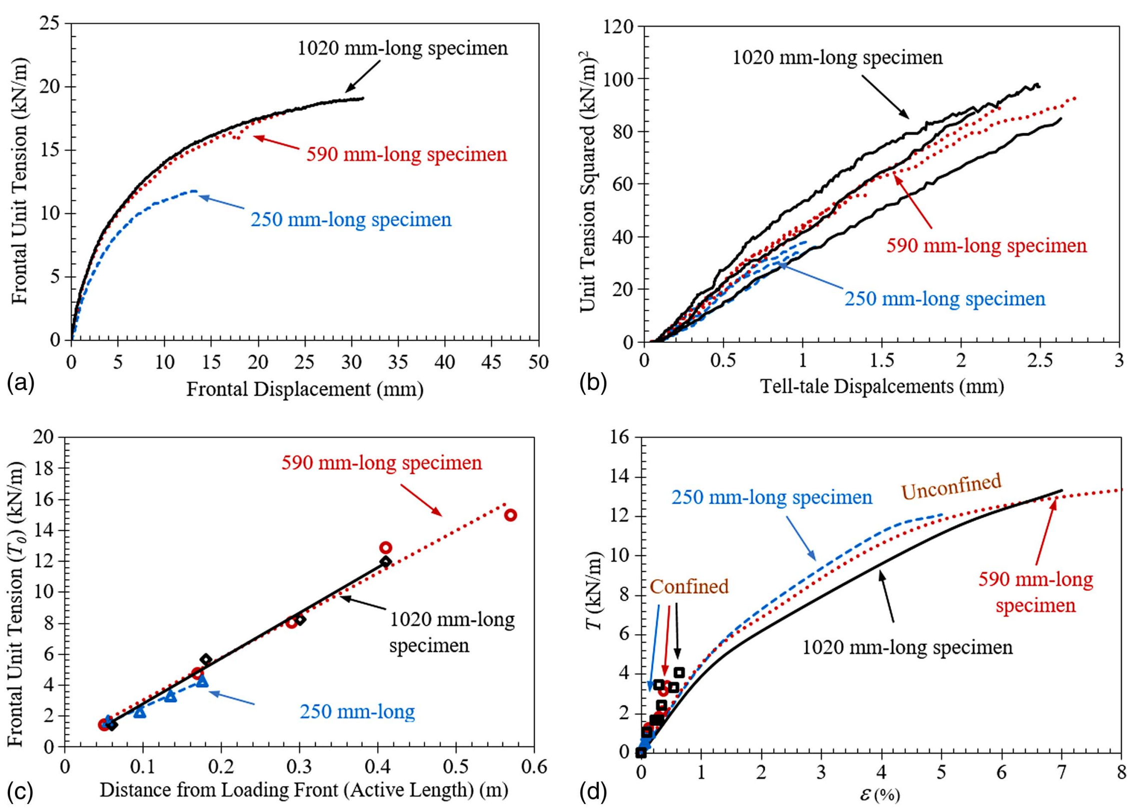

Fig. 10. Experimental results from Test series C (varying geosynthetic lengths): (a) frontal load-frontal displacement results; (b) comparison of K_{SGC} results; (c) comparison of τ_y results; (d) comparison of J_c results

geosynthetic lengths. The stiffness of the soil–geosynthetic composite (K_{SGC}) was also estimated with additional procedures described previously. The results obtained are summarized in Table 8. Reasonably good agreement was found among the results obtained from different procedures.

The frontal unit tension versus telltale location data is presented in Fig. 10(c). The locations of the telltales were different in different geosynthetic specimens because of their different lengths. While all telltales in the shorter, 250 mm-long specimen were equally spaced within its entire length, the telltale locations in the longer specimens extended from approximately 50 mm from the loading front to approximately 580 mm in the 590 mm-long specimen, and approximately 750 mm in the 1,020 mm-long specimen. As presented in Fig. 10(c), the four initial telltales in the two longer specimens were installed at the same distances from the loading front to facilitate comparison between the progressive mobilizations of interface shear. Very good agreement can be observed in Fig. 10(c) among the frontal unit tension versus telltale location data of the different specimens, which indicates a similar pattern of interface shear mobilization independent of the specimen length. Displacements in Telltale #5 within the 1,020 mm–long specimen were not triggered because the geosynthetic broke before the active length reached the location of this telltale. The reasonably straight lines defined in Fig. 10(c) for all tests underscored the suitability of the constitutive model adopted for soil–geosynthetic interface shear for varying geosynthetic lengths. The similarity of the slopes identified in this figure for the three specimens also confirms that the yield interface shear is not affected by the geosynthetic length.

The confined geosynthetic stiffness (J_c) was characterized by the slope of the regression lines from the experimental data obtained from the confined lengths in Fig. 10(d). Similar to K_{SGC} and τ_y, J_c also remained essentially unchanged for varying specimen lengths. Specifically, the slope of the regression lines in Fig. 10(d) was found to range from 660 to 804 kN/m. As summarized in Table 8, the values obtained for J_c from two procedures were found to be in good agreement for the tests in Test Series C.

Validity of the SGC Model for Varying Geosynthetic Types

Test Series D was conducted to evaluate the suitability of the SGC model for geotextiles, which, along with geogrids such as those used in the previous sections, are extensively used in the stabilization of roadways. Although geogrids are manufactured specifically to fulfill reinforcement and stiffening functions, geotextiles have been used in roadway systems to also fulfill additional functions, including separation and drainage. In previous sections, the focus was on evaluation of the validity of the constitutive relationships and outcomes of the SGC model for geogrids. The mechanisms that govern the soil–geosynthetic interaction in geotextiles differ from those in geogrids: geotextiles mobilize only frictional resistance, while geogrids mobilize both frictional and bearing resistance components. The experimental results obtained from this test series were analyzed to evaluate the validity of the constitutive relationships and outcomes of the SGC model for geotextiles.

The characteristics of the geotextile product (GT) used in this test series are presented in Table 2. The geotextile was manufactured using high-tenacity polypropylene yarns with a comparatively higher unconfined stiffness than that of the geogrid product used in the baseline tests. All test conditions were identical to those in the baseline tests. Results obtained from evaluation of the experimental data from this test series, as compared to the baseline tests results, are presented in Fig. 11. In addition, Table 8 summarizes the values estimated for the three model parameters using the previously described procedures.

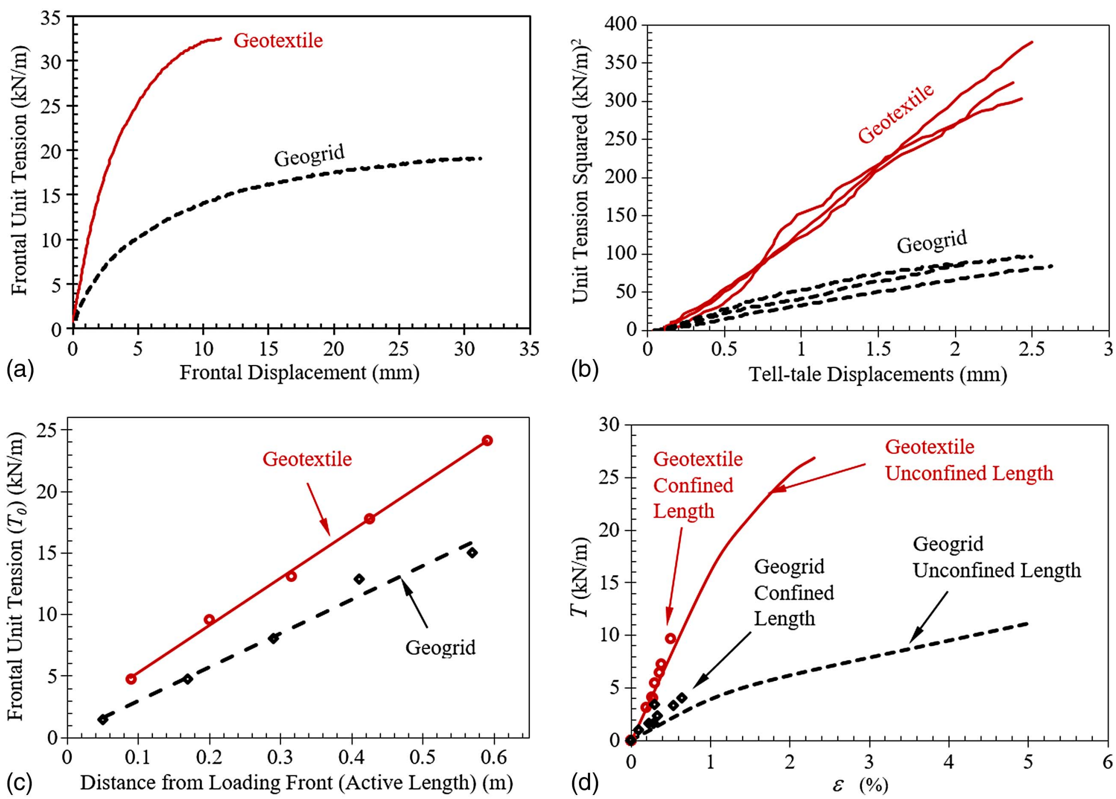

Fig. 11. Experimental results from Test series D (varying geosynthetic types): (a) frontal load-frontal displacement results; (b) comparison of K_{SGC} results; (c) comparison of τ_y results; (d) comparison of J_c results

As shown in Fig. 11(a), the test conducted using the geotextile specimen reached a higher ultimate frontal unit tension at a comparatively smaller frontal displacement than in the baseline test conducted using the geogrid specimen. The ultimate frontal unit tension for the geotextile and the geogrid specimens were 33.0 and 19.1 kN/m, respectively, whereas the frontal displacements at the ultimate frontal unit tension were approximately 11 mm for the geotextile and 31 mm for the geogrid. While the geogrid mobilized both passive and frictional resistance mechanisms over a relatively large displacement range, the geotextile mobilized only frictional resistance over a comparatively smaller displacement range.

Evaluation of the experimental data presented in Fig. 11(b) confirms the suitability of the linear relationship between the unit tension squared (T^2) and telltale displacements (u) for the geotextile. The linear relationships defined by the displacements recorded at different telltale locations were in reasonably good agreement, also confirming the uniqueness of the linear relationship between T^2 and u for the case of geotextiles. The K_{SGC} values, defined by the slopes of these linear relationships, were higher for the test conducted using the geotextile than for the test conducted using the geogrid. Specifically, the average K_{SGC} value was approximately 156 $(\mathrm{kN/m})^2/\mathrm{mm}$ for the test conducted using the geotextile, while it was 43 $(\mathrm{kN/m})^2/\mathrm{mm}$ for the test conducted using the geogrid. A higher K_{SGC} value was expected for the geotextile, as both the yield interface shear and the confined geosynthetic stiffness were higher in the test conducted using the geotextile.

Evaluation of the data presented in Fig. 11(c) confirms the previous observation, based on the results in Fig. 11(a), regarding a comparatively higher yield interface shear for the geotextile test. The experimental data presented in Fig. 11(c) corresponds to the frontal unit tension at the times when displacements at the telltales were first triggered versus the distances of the telltales from the loading front. The comparatively higher slope obtained using data from the geotextile test indicates a higher yield interface shear (τ_y) for the soil-geotextile interaction. The linearity of the relationship defined by this data confirms the adequacy of the constitutive model adopted for the soil–geosynthetic interface shear for the geotextile.

The comparatively higher confined stiffness (J_c) obtained for the geotextile as compared to that for the geogrid can be assessed by evaluating the experimental data presented in Fig. 11(d), which shows the unit tension versus tensile strain data in the confined and unconfined portions of the geogrid and of the geotextile. The suitability of the constitutive model for the geotextile can be validated from evaluation of the data presented for the confined length of the geotextile, which defines a linear relationship. The slope of this line, which defines J_c, is similar to the initial slope of the curve obtained from the data in the unconfined portion of the geotextile, and is higher than the slope from the geogrid test.

The values obtained for τ_y, J_c, and K_{SGC} from the test conducted using the geotextile are summarized in Table 8. Inspection of the data presented in this table indicates reasonably good agreement among the results obtained for the three parameters using different procedures. Overall, the constitutive relationships and outcomes of the analytical model were also found to be suitable for geotextiles, such as the specimen tested in this study.

Validity of the SGC Model for Varying Backfill Soil Types

In Test Series E soil–geosynthetic interaction tests were conducted using a gravel backfill, referred to as AASHTO No. 8 soil characteristics of which are provided in Table 1 and Fig. 2. A confining pressure of 21 kPa and the baseline geosynthetic (i.e., GGPP1) were used for the test conducted using the gravel. The geosynthetic was tested in the cross-machine direction and the embedded portion of the specimen was 280 mm wide and 590 mm long. The backfill soil was placed and compacted following the same procedures used for the baseline tests using Monterey sand. Fig. 12 presents the results obtained from Test Series E, as compared to the results of the baseline tests conducted with the same geogrid and testing conditions, but using Monterey sand. As shown in Fig. 12(a), while the shapes of the frontal unit tension versus frontal displacement curves were slightly different, the ultimate frontal unit tensions were reasonably similar.

The data presented in Fig. 12(b) shows the unit tension squared (T^2) versus telltale displacement (u) data for both tests. The reasonably linear relationship defined in the test conducted using AASHTO No. 8 soil underscored the suitability of the linear approximation between the unit tension squared and displacements. Comparatively narrow range of the slopes for the lines, which define K_{SGC}, obtained in the test conducted using AASHTO No. 8 soil underscored the uniqueness of K_{SGC} throughout the active length of the geosynthetic. Comparison of the linear relationships obtained in the test conducted using AASHTO No. 8 soil to those obtained for the Monterey sand test reflects the different levels of interaction between sandy soil and geosynthetic and that between gravelly soil and geosynthetic. In particular, the SGC model characterized AASHTO No. 8 soil-geogrid interaction with a K_{SGC} that ranged from 100 to 112 $(\mathrm{kN/m})^2/\mathrm{mm}$, and the Monterey sand-geogrid interaction with a K_{SGC} that ranged from 34 to 44 $(\mathrm{kN/m})^2/\mathrm{mm}$.

The suitability of the constitutive relationships assumed in the model for the gravelly backfill was validated by evaluation of the data presented in Figs. 12(c and d). Fig. 12(c) presents the frontal unit tension at the times when displacements were first triggered in each telltale versus the telltale locations. The linearity of the relationship defined by this data underscored the suitability of the constitutive model adopted for the soil–geosynthetic interface shear. The yield interface shear (τ_y), estimated by half of the slopes for these lines, was 17.6 and 13.7 $\mathrm{kN/m}^2$ for AASHTO No. 8 soil and Monterey sand tests, respectively. This reflects a comparatively stronger interaction between the gravel and the geogrid than between the sand and the geogrid.

The unit tension versus tensile strain data in the confined and unconfined portions of the geosynthetic specimens is presented in Fig. 12(d). Since the geogrid specimens used in the two tests were from different rolls, the unit tension versus strain data was found to be slightly different between the two tests. Accordingly, the J_c values, defined by the slope of the linear relationship fitted to the data obtained from the confined length of the specimens, were also found to be slightly different between the two tests. As predicted by the SGC model, the J_c values were similar to the initial slope values of the unit tension versus tensile strain data in the unconfined lengths of the specimens. The model parameters estimated for the test conducted using AASHTO No. 8 soil are summarized in Table 8.

Summary and Conclusions

A soil–geosynthetic interaction parameter, referred to as K_{SGC}, which captures the stiffness of a soil–geosynthetic composite under small displacements, was identified in the companion paper (Zornberg et al. 2017). This parameter was obtained from a closed-form analytical solution to the equilibrium differential equation that results after assuming: (1) a linear constitutive relationship for the geosynthetic, characterized by the confined geosynthetic stiffness (J_c); and (2) a rigid-perfectly plastic constitutive relationship for

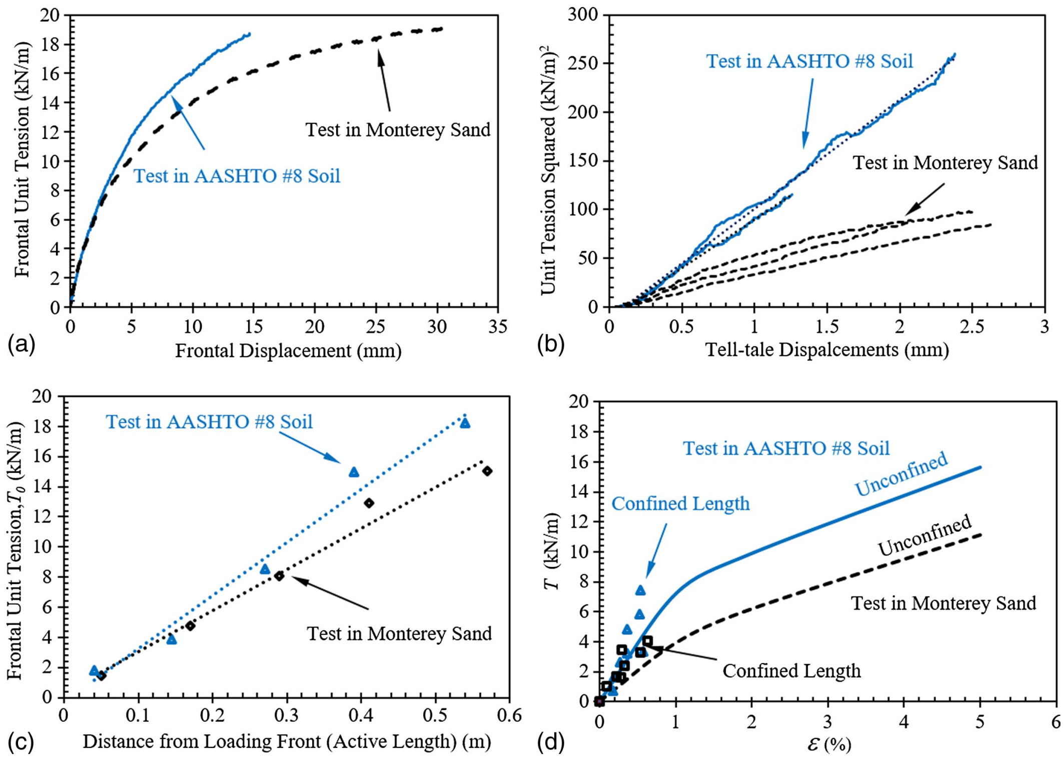

Fig. 12. Experimental results from Test series E (varying backfill soil types): (a) frontal load-frontal displacement results; (b) comparison of K_{SGC} results; (c) comparison of τ_y results; (d) comparison of J_c results

the soil–geosynthetic interface shear that is defined by the yield interface shear (τ_y). Zornberg et al. (2017) detailed the assumptions and solution of the SGC model. In this paper, the repeatability of the parameters predicted by the model was evaluated in light of expected variability, as defined after an extensive assessment of the typical ranges of variability for soil and geosynthetic properties. Additionally, the suitability of the proposed model was evaluated by analyzing results from a comprehensive testing program conducted using large-scale soil–geosynthetic interaction equipment. The constitutive relationships and outcomes of the model were evaluated for a range of materials and test conditions. Specifically, the SGC model was evaluated for varying confining pressures, geosynthetic lengths, geosynthetic types, and backfill soil types.

The main findings drawn from evaluation of the experimental data obtained to assess the SGC model are as follows:

- The experimental procedures developed to obtain the SGC model parameters were found to result in repeatable parameter values. This is based on experimental results obtained from tests conducted using the same geosynthetic product, embedment length, confining pressure, and backfill soil. Specifically, the coefficients of variation for the predicted values of K_{SGC} and the constitutive parameters of the model were found to range from 7 to 16%.
- Following an extensive assessment of reported data on the expected variability of soil and geosynthetic properties, a typical range of variability for similar properties and test conditions to those used in this study was identified as approximately 30%. Accordingly, the variability of the K_{SGC} parameter evaluated in this study was found to be acceptable and well within the typical range of that of relevant geotechnical and geosynthetic properties.
- The constitutive relationships and outcomes of the SGC model were found to be suitable for varying confining pressures, as evaluated from the results of large-scale soil–geosynthetic interaction tests conducted using identical test configurations, but under different confining pressures. Overall, the linearity and uniqueness of the relationship between the unit tension squared (T^2) and displacements (u) were found to be unaffected by confining pressure.
- The suitability of the model assumptions and outcomes was found to be unaffected by geosynthetic length. Specifically, the unit tension squared (T^2) versus displacement (u) data was found to be linear and unique along the active length of geosynthetic specimens of varying lengths. The K_{SGC} value and constitutive parameters of the model remained essentially unchanged for geosynthetic specimens of varying lengths.
- The assumptions and predictions of the SGC model were found to be adequate for geotextile products. Specifically, as for the case of geogrids, the unit tension squared (T^2) versus displacement (u) data was found to be linear and unique along the geotextile specimen tested in this study. In addition, the rigid-perfectly plastic constitutive relationship for the soil–geosynthetic interface shear and the linear constitutive relationship for geosynthetic materials were found to be adequate for the case of geotextiles.
- The linear relationship between the unit tension squared (T^2) and displacements (u) was found to be valid for different backfill soils. This relationship was found to be unique along the embedded length of the geosynthetic. The suitability of the constitutive relationships adopted in the model was also confirmed for both sandy and gravely soils.

Overall, the results of this study indicate that the K_{SGC} parameter defined to characterize the confined stiffness of a soil–geosynthetic composite can be obtained experimentally using procedures that are repeatable and for multiple soil and geosynthetic materials as well as for varying testing conditions.

Appendix: Summary of Reported Variability for Relevant Soil and Geosynthetic Properties

This Appendix presents the main findings from an extensive assessment of the bibliography, conducted as part of this study, on sources and expected ranges of variability for relevant soil and geosynthetic properties (Roodi 2016). The evaluation was conducted to define a reasonable range of variability to be expected for the experimental parameters of the SGC model.

The variability reported in the literature of the soil properties associated with three sources of *site variance*, *bias error*, and *random error* was analyzed. Typical values of reported variability were grouped into four classes: (1) index soil properties; (2) soil strength properties; (3) soil deformability and water flow parameters; and (4) field testing parameters. Parameters in each category were grouped as presented in Table 9. This table summarizes the approximate range and average for the coefficients of variation (CV) for three cases:

1. *Site Variance* $[CV(\xi)]$, which results from inherent soil variability at a site.
2. *Measurement Error* $[CV(\alpha + \delta)]$, which involves variations arising exclusively from testing procedures, i.e., uncertainties associated with *bias errors* and *random errors*.
3. *Random Error* $[CV(\delta)]$, which represents deviation from real value caused by lack of *precision* in the testing procedure.

It should be noted that *random error* is inevitable in all testing procedures and cannot easily be minimized. *Random errors* are cumulative effects of an unknown number of small errors simultaneously affecting a measurement, but cannot be independently identified or avoided. It should also be noted that *model uncertainty* is present in all measurements, which is particularly complicated to determine when the property is defined as the result of a specified test.

Inspection of the data presented in Table 9 indicates that density and thickness measurements exhibit the lowest variation among all properties. The largest variability of index properties was found for Plasticity Index (PI) and Liquidity Index (LI), which are determined by two separate laboratory measurements, and for D_r when is determined indirectly from SPT results. Strength parameters were found to be more variable when they are measured for fine grained soils. The average coefficient of *inherent variability* of cohesion and friction angle measurements for fine grained soils was found to be, respectively, 35 and 20%, whereas this coefficient was 10% for measurement of friction angle in coarse grained soils. Contribution of *measurement errors* to the *overall variability* was found to be considerably larger in strength parameters than in index properties. The larger contribution of *measurement errors* to the *overall variability* in strength parameters than in index properties can be attributed to comparatively complex testing equipment and procedures involved in measurement of strength parameters as well as comparatively complicated models used to analyze the test data.

Evaluation of the limited data reported on *random errors* in laboratory testing indicates that these errors would be comparatively easier to determine in less complex testing procedures such as Atterberg limit tests, specific gravity measurements, and compaction tests. The extent of *random errors* in these tests was found to be in a narrow range of $CV(\delta) = 0.2$–3%. Wider variation for *random errors* $[CV(\delta)]$ would be expected for more complex testing procedures and models that are involved in laboratory measurement of strength and stiffness properties.

An assessment of available literature was also conducted to characterize the variability reported for geosynthetic properties and soil–geosynthetic interaction properties. This information was found to be comparatively more limited than that for soil properties. However, a significant database of variability data was identified on the interface shear strength between different types of geosynthetics and between geosynthetics and soil. This may be attributed to the number of failures reported in landfill lining systems, which may have involved one or more of the interfaces between geosynthetics and soil layers (e.g., Brink et al. 1999; Jones and Dixon 2003; Koerner and Soong 2000; Mazzucato et al. 1999).

Internal and interface shear strength variability data reported in the literature has been summarized and evaluated by McCartney et al. (2004) for interfaces involving geosynthetic clay liners (GCL) and by Dixon et al. (2006) for various geosynthetic-geosynthetic and geosynthetic-soil interfaces. McCartney et al. (2004) evaluated the results of over 800 GCL internal and GCL–geomembrane (GM) interface large-scale direct shear tests conducted in a single independent laboratory between 1992 and 2003. Therefore, the variability data they reported consisted of *inter-product* test results as well as *repeatability* data in cases the same products were tested. On the other hand, Dixon et al. (2006) combined variability data

Table 9. Typical Range and Average Variability for Soil Properties

Property group	Parameter	Site variance $CV(\xi)\%$ Range (approximate)	Site variance $CV(\xi)\%$ Average (approximate)	Measurement error $CV(\alpha+\delta)\%$ Range (approximate)	Measurement error $CV(\alpha+\delta)\%$ Average (approximate)	Random error $CV(\delta)\%$ Range (approximate)	Random error $CV(\delta)\%$ Average (approximate)
Index properties	Low variability (density, MDD, thickness)	0–20	10	1–4	2	0.2–1[a]	0.5[a]
	Moderate variability [w_n, w_L, w_P, OMC, D_r (direct estimation)]	5–60	20	5–20	10	1–3[a]	2[a]
	High variability [PI, LI, D_r (indirect estimation)]	10–80	50	5–55	30	No data	
Strength parameters	Cohesion	5–100	35	5–70	25	No data	
	ϕ and Tan(ϕ) (Fine grained soils)	5–60	20	5–30	15		
	ϕ and Tan(ϕ) (Coarse grained soils)	2–20	10	2–20	10		
Deformability and water flow parameters	Compression indexes	20–50	30	10[b]		No data	
	Coefficient of consolidation	10–100	50	32[b]			
	Hydraulic conductivity	5–300	200	22[b]			
Field measurements	SPT	20–90	45	15–100	40	12–15	12
	Other tests	Wide variation		10–25	15	5–15	10

[a]Little data was available.
[b]Only one data point was available.

Table 10. Typical Range of Variability for Geosynthetic Properties

Reference	Property	Interproduct or interlaboratory tests	Repeatability tests	
McCartney et al. (2004)	GM/GCL or GCL internal interface shear	30–70	2–10	Same-lot specimens
			14–36	Different lot specimens
Dixon et al. (2006)	Smooth HDPE GM/NW GT interface shear	18–32		—
	Textured HDPE GM/NW GT interface shear	15–32		—
	Smooth HDPE GM/coarse soil interface shear	10–30		—
	Textured HDPE GM/coarse soil	5–25	8–28	Same-lot specimens
	NW GT/coarse soil interface shear	8–45		—
	Smooth HDPE GM/fine soil interface shear	Wide variation		—
	Textured HDPE GM/fine soil interface shear	Wide variation		—
Tensar (2012)	Ultimate tensile strength (GG)	—	2	Same-lot specimens
	Tensile modulus (GG)	—	14–28	@ 1% strain
			7–13	@ 2% strain
			4–5	@ 5% strain

Note: Coarse soil = primarily sand and gravel; fine soil = primarily silt and clay; GCL = geosynthetic clay liner; GG = geogrid; GM = geomembrane; HDPE = high density polyethylene; inter-product/Inter-laboratory tests = different product types or different laboratories; NW GT = nonwoven geotextile; repeatability tests = same type of product in a single laboratory with one device and operator.

from 76 sources, including *interproduct* test results from the literatures, from an internal database, and from *interlaboratory* and *repeatability* testing programs. The main findings of the two studies are summarized in Table 10 in the form of the coefficient of variation associated with each case.

An additional variability dataset presented in Table 10 involves the data obtained from a Tensar Co. quality control testing report (Tensar 2012). This dataset involves results from wide-width tensile tests conducted on same-lot specimens. The coefficient of variation for this repeatability dataset was calculated for the ultimate tensile strength and the tensile modulus. Although the coefficient of variation for tensile strength was found to be very small (2%), comparatively larger coefficients were found for the tensile modulus. In particular, the coefficient of variation was found to be particularly high for modulus values defined at low strains. While the coefficient of variation of the modulus at 5% strain was found to range from 4 to 5%, this coefficient ranged from 14 to 28% when the modulus was defined at 1% strain. It can be concluded that comparatively larger variations should be expected for stiffness parameters (e.g., modulus) than for strength parameters (e.g., ultimate tensile strength), and that even larger variations should be expected when the stiffness property is measured at comparatively small displacements.

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