Implementation of T-Z Analysis Approach to Predict Pullout Test Results

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ABSTRACT

Characterization of soil-geosynthetic interaction is a key aspect in understanding mechanical performance of structures made using geosynthetics (e.g., geosynthetic-stabilized roadways, geosynthetic-reinforced soil walls). Pullout test has been one of the most widely used experimental techniques to characterize this interaction. This study implements t-z analysis approach, originally developed to evaluate load transfer mechanism in deep foundations, to predict pullout test results of geosynthetics. Formulations of the method and step-by-step solution algorithm are presented. Suitability of the method was evaluated by comparisons made between predictions of the t-z analysis approach and experimental pullout test results presented in literature. Specifically, geosynthetic and soil-geosynthetic characterization data presented in four studies with a wide range of soil and geosynthetic types, normal pressures, and test configurations were used in the t-z analysis framework presented in this study to simulate pullout test results. Results from the simulations were then compared to the experimental data presented in the four studies. This comparison underscored suitability of the t-z analysis approach in producing reasonably consistent results with the experimental data.

INTRODUCTION

Geosynthetics have been widely recognized to enhance the mechanical performance of soils. Their use in soil reinforcement applications involved a wide range of transportation infrastructures such as retaining walls (e.g., Allen et al. 1992; Bathurst 2005; Morsy et al. 2017a) and pavements (e.g., Zornberg 2017). In roadway systems, geosynthetic layers have been used to reinforce hot mix asphalt concrete (e.g., Brown et al. 1985) and to stabilize granular base course and subgrade layers (e.g., Al-Qadi et al. 2008; Zornberg et al. 2012a, b). The benefits from the use of geosynthetics in soil reinforcement requires understanding characteristics of (1) geosynthetic material, (2) surrounding soil, and (3) soil-geosynthetic interaction.

Soil-geosynthetic interaction has typically been evaluated using interface direct shear and pullout tests. The two testing methods mobilize the soil-geosynthetic interface shear in two different modes. In direct shear testing, the soil is forced to slide on a stationary geosynthetic, whereas in pullout testing, the geosynthetic is embedded in a stationary confined soil mass and pulled from one end under an increasing tensile load. While both tests have been used to evaluate soil-geosynthetic interaction, pullout test is deemed more appropriate method in applications where elongation of the geosynthetic is expected. Pullout testing has been used to provide better understanding for the behavior of structures designed at limit state conditions, such as reinforced soil walls (e.g., Bathurst 2005; Morsy et al. 2017b) as well as structures designed at working stress conditions, such as stabilized base roadways (e.g., Roodi and Zornberg 2017; Zornberg et al. 2017).

This study implements t-z analysis approach to predict geosynthetic pullout test results. The t-z analysis approach was originally developed to predict load-settlement relationships in deep foundations based on constitutive laws for the load-transfer mechanisms. Formulations of this technique are presented for general constitutive relationships of geosynthetic and soil-geosynthetic interface. A step-by-step iteration algorithm is then presented to solve soil-geosynthetic interaction problem in pullout tests. Suitability of the method was evaluated by comparisons made between predictions of the t-z analysis approach and experimental pullout test results presented in the literature. Specifically, experimental data presented by Alfaro et al. (1995), Yuan (2011), Abdi and Arjomand (2011), and Perkins and Eden (2003) was used to validate suitability of the predictions made by t-z analysis approach.

BACKGROUND ON SOIL-GEOSYNTHETIC INTERFACE MODELS

Interaction between soil and inextensible reinforcements differs from that between soil and extensible reinforcements (e.g. polymeric geosynthetics). Shear stresses generated at the soil-reinforcement interface do not cause axial elongation in inextensible reinforcements. On contrary, extensible reinforcements elongate when they interact with the surrounding soil. That is, the soil-reinforcement interaction is governed by the progressive development of tension in the reinforcement and the mobilization of interface shear along its length.

Modeling soil-geosynthetic interaction requires suitable constitutive relationships to be defined for the soil, the geosynthetic, and the soil-geosynthetic interface. The geosynthetic constitutive model relates its unit tension to its tensile strain and can be obtained by tensile testing of the unconfined geosynthetic. Although the unit tension-strain response of geosynthetics is inherently nonlinear, a linear response has often been adopted (e.g., Juran and Chen 1988; Yuan 2011; Roodi 2016). However, nonlinear functions have also been used, including polynomial functions (e.g., Wilson-Fahmy and Koerner 1993; Bergado and Chai 1994) and hyperbolic functions (e.g., Weerasekara and Wijewickreme 2010).

Interface constitutive relationships relate the shear stress mobilized at the soil-geosynthetic interface to the relative displacements that are mobilized between these two materials. Experimental data has been obtained to determine this relationship using interface direct shear tests. Models adopted in previous studies to describe the interface interaction include: linear elastic (e.g., Yuan 2011); linear elastic-perfectly plastic (e.g., Juran and Chen 1988; Bergado and Chai 1994); rigid-perfectly plastic (e.g., Roodi 2016); bilinear and hyperbolic models (e.g., Wilson-Fahmy and Koerner 1993); and more complex nonlinear multiphase models such as elasto-plastic strain hardening and softening models (e.g., Juran and Chen 1988); as well as linear pre-peak followed by nonlinear post-peak response (Weerasekara and Wijewickreme 2010).

The use of different constitutive models and boundary conditions has led to a wide range of formulations to solve the differential equations that result after considering force equilibrium in a soil-geosynthetic system. However, due to the complexity of these formulations, the reported solutions have often involved the use of numerical techniques, including finite elements and finite differences (e.g., Wilson-Fahmy and Koerner 1993; Weerasekara and Wijewickreme 2010).

IMPLEMENTATION OF T-Z ANALYSIS APPROACH

A brief background on t-z analysis approach along with specific formulations and solution algorithm adopted in this study are presented in this section.

Background. The t-z approach is a method that was developed to predict the load-transfer between piles and surrounding soils (Coyle and Reese 1966). This method has been widely used to estimate the load-settlement relationship in axially loaded piles. This approach involves modeling the pile as a series of discrete elements connected axially by nonlinear springs, which represent the axial stiffness of the pile. In addition, these elements are connected through their skin to their surrounding soil by other nonlinear springs (t-z relationship), which represent the resistance of the soil in skin friction. An additional nonlinear spring is assumed at the pile tip, which represent the end-bearing (q-y relationship). Several research studies were conducted and obtained t-z and q-y relationships empirically based on model and full-scale pile load tests (e.g., Coyle and Sulaiman 1967). Subsequent studies proposed general recommendations for estimating t-z and q-y curves (e.g., Vijayvergiya 1977). Theoretical approaches have been developed to relate the t-z and q-y relationships to the soil properties surrounding the pile (e.g., Kraft et al. 1981; Randolph 1994).

Solution algorithm for pullout test data. The general approach adopted in this study to simulate experimental pullout test data involves discretizing the geosynthetic specimen into several segments, adopting an incremental displacement to the free end of the geosynthetic, and satisfying the force equilibrium equation of each segment using an iterative procedure to eventually determine the displacement, strain, interface shear, and load profile along the geosynthetic length. The constitutive model for soil-geosynthetic interface was used to estimate the interface shear resistance mobilized along each segment. This model relates geosynthetic displacement (u) to the interface shear between soil and the geosynthetic (τ):

$$\tau = f(u) \quad (1)$$

The model adopted for geosynthetic material was used to estimate the unit tension developed in the geosynthetic. This model relates geosynthetic unit tension (T) to its tensile strain (ε):

$$T = g(\varepsilon) \quad (2)$$

Specific steps adopted in this study to implement the t-z analysis procedure to simulate pullout test data are as follows:

Step 1: The geosynthetic specimen is discretized into several segments from 1 (at the loading front) to n (at the free end). Definitions used in discretization of the geosynthetic specimen are presented in Figure 1. Geosynthetic nodal displacements, unit tension in the geosynthetic at the nodal locations, interface shear between soil and geosynthetic segments, and tensile strain in the geosynthetic are defined by u, T, τ, and ε, respectively.

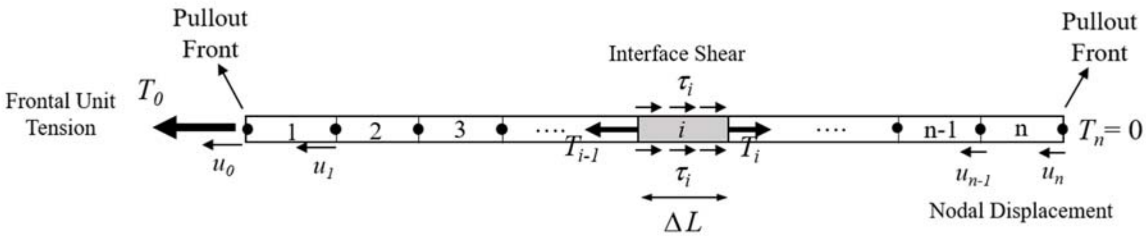

Figure 1. Discretization of geosynthetic along with definition of variables.

Step 2: An incremental displacement, δu, is adopted at the geosynthetic free end (Segment *n*):

$$u_n = \delta u \tag{3}$$

The unit tension at the end of geosynthetic is zero:

$$T_n = 0 \tag{4}$$

Step 3: Displacement at the front of Segment *n* is assumed (u_{n-1}) and the average displacement in Segment *n* ($u_{n(avg)}$) is estimated as follows:

$$u_{n(avg)} = \frac{u_n + u_{n+1}}{2} \tag{5}$$

Step 4: The interface shear along Segment *n* is estimated using the adopted interface shear model:

$$\tau_n = f(u_{n(avg)}) \tag{6}$$

Step 5: Considering the force equilibrium in Segment *n*, unit tension at the front of this segment yields as follows:

$$T_{n-1} = T_n + 2.\Delta L.\tau_n \tag{7}$$

Where ΔL = segment length. The average unit tension in Segment *n* ($T_{n(avg)}$) is then obtained:

$$T_{n(avg)} = \frac{T_n + T_{n-1}}{2} \tag{8}$$

Step 6: Tensile strain in Segment *n* is estimated using the adopted constitutive model for the geosynthetic and elongation of this segment (δ_n) is obtained:

$$\varepsilon_n = g^{-1}(T_{n(avg)}) \tag{9}$$

$$\delta_n = \Delta L.\varepsilon_n \tag{10}$$

Step 7: Displacement at the front of Segment *n* is updated using $u_n$and δ_n:

$$u_{n-1} = u_n + \delta_n \tag{11}$$

The updated displacement at the front of Segment *n*, obtained in this step, is compared with that assumed in Step 3. If the difference between the two was negligible, proceed to the next step. Otherwise, the updated displacement is used in Step 3 and Steps 3 to 7 are iterated to converge.

Step 8: Steps 3 to 7 will be repeated for Segment *n*-1 using the displacement and unit tension obtained at the end of this segment ($u_{n-1}$and T_{n-1}) to eventually determine the displacement and unit tension at the front of this segment. This procedure will then be repeated for Segments *n*-2 to 1 to obtain the displacement, strain, interface shear, and load profile along the geosynthetic length. Eventually, the geosynthetic frontal displacement (u_0) and frontal unit tension (T_0) will be determined.

Step 9: The incremental displacement in Step 1 is increased and the iteration is repeated to determine the updated load and displacement profiles along the geosynthetic length.

VALIDATION OF THE T-Z ANALYSIS APPROACH PREDICTIONS

Suitability of the proposed t-z analysis approach was evaluated by comparison of its predictions with experimental pullout test data presented in the literature. This section details the validation procedure and obtained findings.

Experimental test data. The experimental data used in the validation procedure was obtained from results presented in four studies: (1) Alfaro et al. (1995); (2) Yuan (2011); (3) Abdi and Arjomand (2011); and (4) Perkins and Eden (2003). Characteristics of the experimental program in each study are summarized in Table 1. The constitutive relationship for geosynthetic ($T = g(\varepsilon)$) and soil-geosynthetic interface shear ($\tau = f(u)$), presented in each study, were adopted in the t-z analysis approach. Pullout test results were then reproduced for the test configurations in each study.

Table 1. Summary of experimental tests compiled from literature.

Experimental Study	Normal Pressure (kPa)	Soil Type	Geosynthetic Type	Geosynthetic Dimension (mm) (width x length)	Box Size (mm) (width x length x height)
Alfaro et al. (1995)	30	Gravel	Uniaxial Geogrid	440 x 960	600 x 1500 x 400
Yuan (2011)	9.6	Sand	Geotextile	305 x 1220	Not provided
Abdi and Arjomand (2011)	75	Clay	Uniaxial Geogrid	300 x 300	300 x 300 x 200
Perkins and Eden (2003)	35	Crushed Stone	Geotextile	600 x 300	900 x 1250 x 1100

Soil-geosynthetic Interface Shear ($\boldsymbol{\tau = f(u)}$). The soil-geosynthetic interface shear models adopted in the t-z analysis approach was obtained from the information provided in the experimental studies. Specifically, experimental results of the direct shear tests conducted on the interface between soil and geosynthetic were presented in Alfaro et al. (1995), Yuan (2011), and Abdi and Arjomand (2011). This data is reproduced in Figures 2a through 2c, respectively. The interface shear data presented in Alfaro et al. (1995) (Figure 2a) was directly used as the input function for soil-geosynthetic interface shear model ($\tau = f(u)$). The soil-geosynthetic interface shear data presented in Yuan (2011) was represented by a linear-plastic interface shear model described by the ultimate soil-geosynthetic interface shear (τ_{ult}) occurs at a displacement value referred to as u_{ult}. The slope of the linear portion of the interface shear model was represented by K_τ, referred to as the soil-geosynthetic shear stiffness:

$$K_\tau = \frac{\tau_{ult}}{u_{ult}} \tag{12}$$

Considering the data presented in Figure 2b, the ultimate soil-geosynthetic interface shear (τ_{ult}) and its corresponding displacement (u_{ult}) assumed to simulate experimental data presented by Yuan (2011) were 6.9 kPa and 0.9 mm, respectively. The soil-geosynthetic shear stiffness (K_τ,) was then obtained as 7.7 kPa/mm for Yuan (2011).

The interface shear data presented in Abdi and Arjomand (2011) was directly used as the input function for soil-geosynthetic interface shear model ($\tau = f(u)$) (Figure 2c Model (1)). However, further evaluation of the interface shear data presented by Abdi and Arjomand (2011) revealed that the ultimate interface shear (approximately 50 kPa) was not consistent with the ultimate pullout stress (approximately 83 kPa) reported at the same normal pressure. Therefore, a modified soil-geosynthetic interface shear model (referred to as Interface Model (2) in Figure 2c) was also adopted in which the shear stress was modified by a correction factor of 83/50 = 1.66.

As presented in Figure 2d, a linear-plastic soil-geosynthetic interface shear model was adopted by Perkins and Eden (2003). The parameters of this relationship, per suggestion by Perkins and Eden (2003), was adopted as τ_{ult}= 1.15 × 35 = 40.25 kPa, u_{ult}= 1 mm, and K_τ= 40.25 kPa/mm.

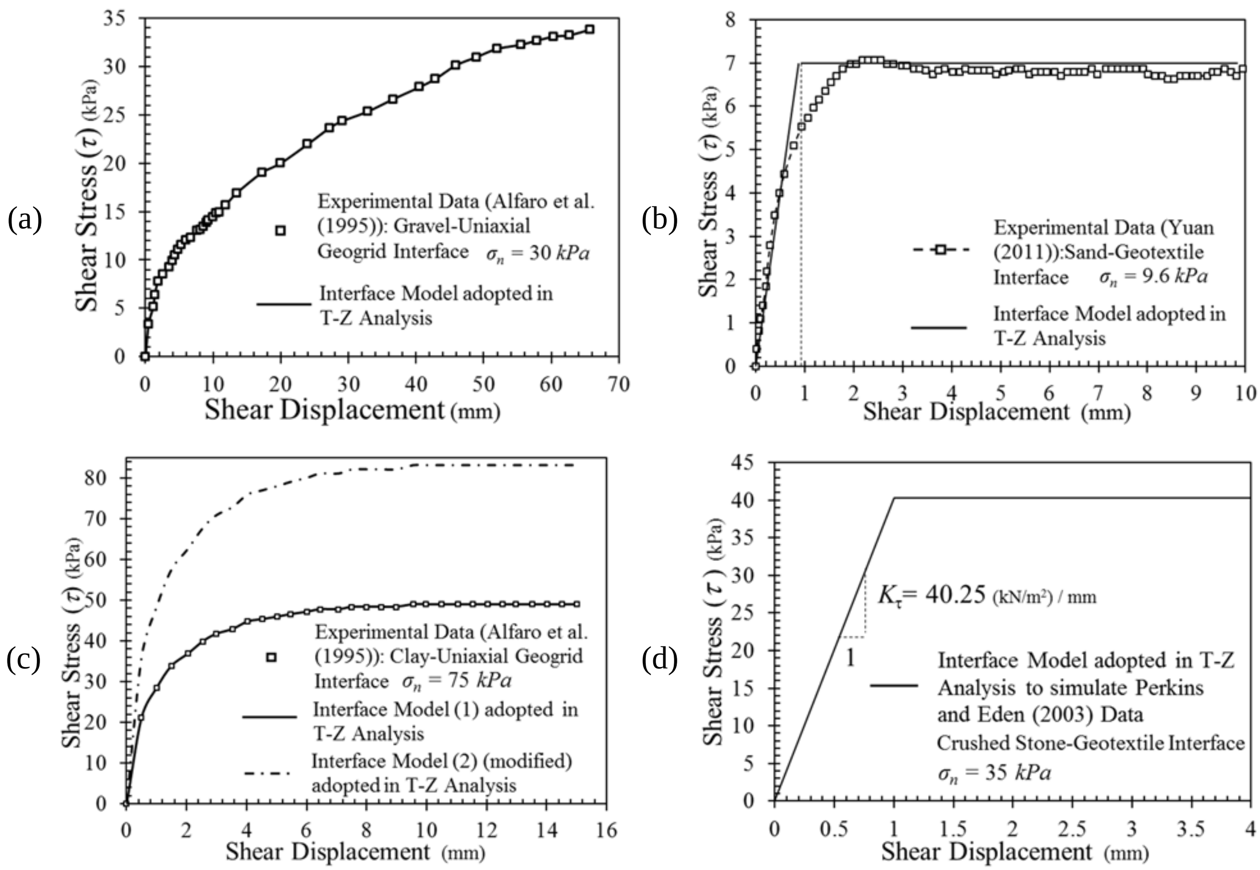

Figure 2. Interface shear models adopted in T-Z analysis approach versus experimental interface shear data in each study: (a) Alfaro et al. (1995); (b) Yuan (2011); (c) Abdi and Arjomand (2011); (d) Perkins and Eden (2003)

Geosynthetic Tensile Characteristics ($T = g(\varepsilon)$). The geosynthetic material properties adopted in this study was obtained using information provided in the experimental studies (Figures 3a through 3d). As reproduced in Figures 3b and 3d, Yuan (2011) and Perkins and Eden (2003) conducted tensile tests on their geotextile specimens and obtained unit tension (T) versus tensile strains (ε) relationship. This data was directly adopted in the t-z analysis approach as the input function for geosynthetic material model ($T = g(\varepsilon)$). Perkins and Eden (2003) presented two sets of experimental data corresponding to slow and fast monotonic tensile load tests. Correspondingly, two geosynthetic models, referred to as Geosynthetic Model (1) and (2), were adopted in the t-z analysis approach (Figure 3d).

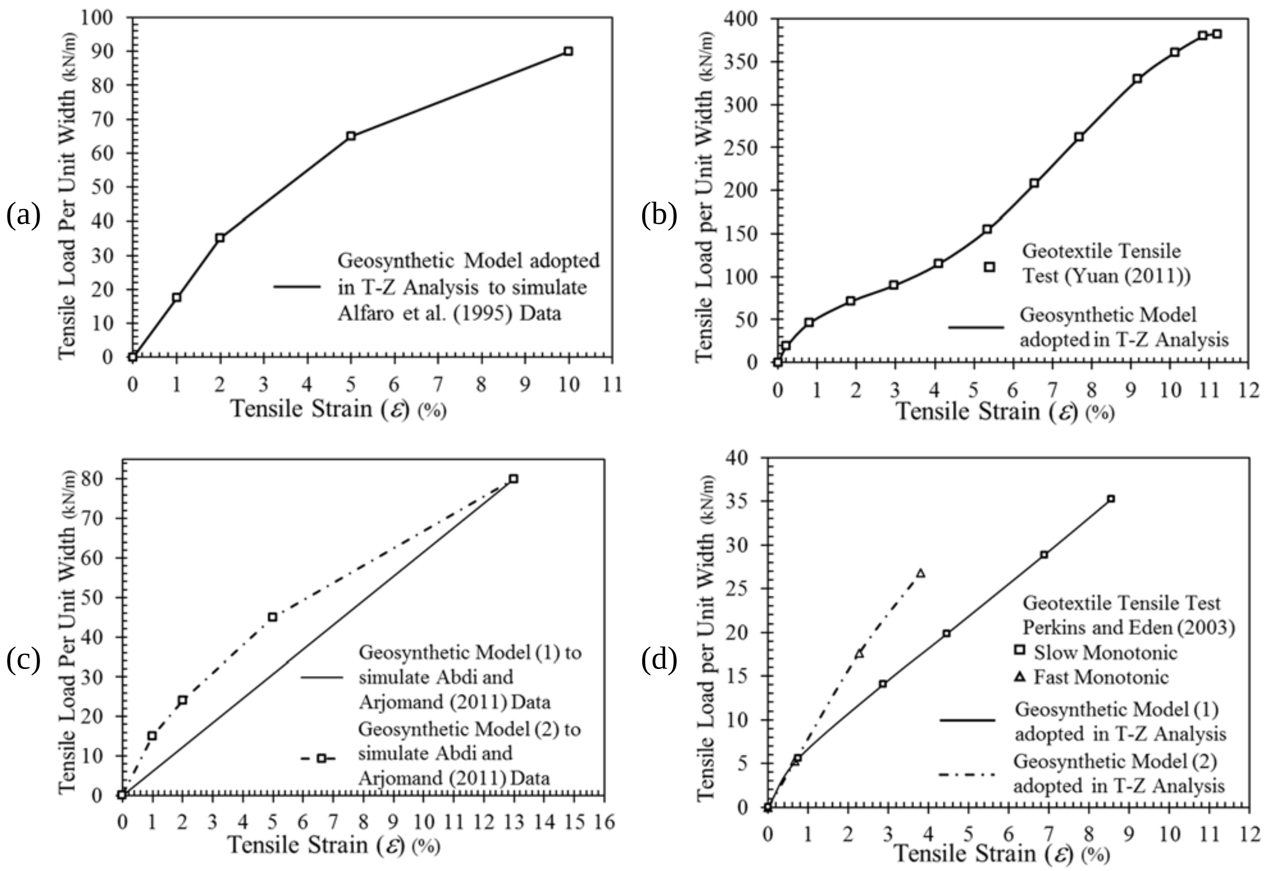

Figure 3. Geosynthetic models adopted in t-z analysis compared to test data: (a) Alfaro et al. (1995); (b) Yuan (2011); (c) Abdi and Arjomand (2011); (d) Perkins and Eden (2003)

Alfaro et al. (1995) provided the name of the uniaxial geogrid tested in their experimental program. Typical characteristics for this product were obtained from information provided by the manufacturer. Specifically, tensile strength of this product at 1, 2, 5, and 10 % tensile strain was estimated as 17.5, 35, 65, and 90 kN/m, respectively. The geosynthetic constitutive model was then generated using these data points and assuming linear relationships between them (Figure 3a).

Abdi and Arjomand (2011) provided the ultimate longitudinal tensile strength and corresponding strain for the uniaxial geogrid used in their experimental program as 80 kN/m and 13%, respectively. Considering this data, a linear unit tension-strain relationship was adopted in the t-z analysis approach to simulate the geosynthetic material properties (Geosynthetic Model (1) in Figure 3c). However, as the stiffness of the geosynthetic is expected to be higher at small strain as compared to that in large strains, a modified geosynthetic model was also adopted (Geosynthetic Model (2) in Figure 3c). In this modified model, typical ratios between the stiffness of the geosynthetic at 10 % tensile strain to that at comparatively smaller strains were used to obtain modified tensile strength at 1, 2, and 5 % strain.

Pullout test results. Results of the simulations obtained using t-z analysis approach as compared to the experimental pullout data in each study are shown in Figures 4a through 4d. The horizontal axis of the plots corresponds to the frontal pullout displacement and the vertical axis represents frontal unit tension. In cases that more than one interface shear and/or geosynthetic models were assumed, simulations were conducted using all relevant combinations of interface shear and geosynthetic models.

Evaluation of the data presented in this figure indicates reasonably good agreement between the simulated and experimental data when suitable interface shear and geosynthetic models were used. Perhaps the most favorable comparison was found between simulated results in this study and experimental data presented by Yuan (2011) (Figure 4b). This can be attributed to the geosynthetic and interface shear models used in this case. As in Yuan's study both geosynthetic tensile strength and soil-geosynthetic interface shear tests were conducted using specific soil and geosynthetic materials tested, the constitutive models used in the t-z analysis of Yuan's data were comparatively more realistic than those used to simulate other studies.

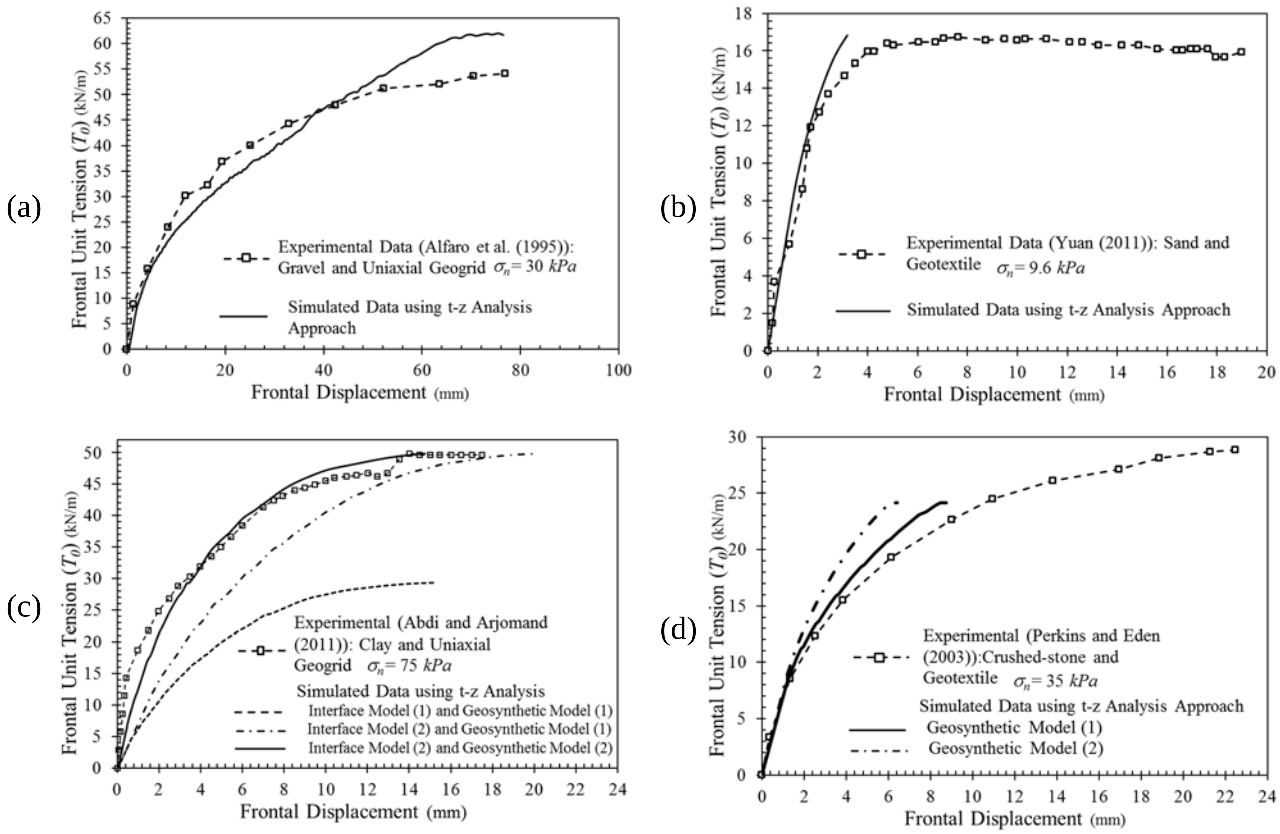

Figure 4. Simulated versus experimental pullout test data: (a) Alfaro et al. (1995); (b) Yuan (2011); (c) Abdi and Arjomand (2011); (d) Perkins and Eden (2003).

As presented in Figure 4c, t-z analysis approach was adopted using three cases to simulate experimental pullout data presented by Abdi and Arjomand (2011). The simulation conducted using Geosynthetic Model (1) and Interface Model (1) resulted poor consistency between the simulated and experimental data. As previously mentioned, this was expected because the ultimate interface shear in Geosynthetic Model (1) did not match the ultimate pullout stress. The simulation conducted using Geosynthetic Model (1) and the modified interface shear (Interface Model (2)) resulted in significantly better consistency between the experimental and the simulated data. The most favorable comparison, however, was found when the stiffness of the geosynthetic was modified at small strains (simulation conducted using Geosynthetic Model (2) and Interface Model (2) in Figure 4c).

Comparison of the simulated and experimental data in Perkins and Eden (2003), presented in Figure 4d, further underscored significance of using proper models in simulation. The simulation conducted using slow tensile load data (Geosynthetic Model (1)) resulted in comparatively better consistency with the experimental data than the simulation conducted using fast tensile load data (Geosynthetic Model (2)). This can be explained by that the strain rate in the slow tensile test was comparatively closer to the strain rate of the geosynthetic in the pullout test. However, as the ultimate interface shear used in the interface model, suggested by Perkins and Eden (2003), did not match the ultimate pullout stress, simulation data in both cases did not reach the ultimate pullout resistance.

SUMMARY AND CONCLUSIONS

Soil-geosynthetic interaction in pullout test was studied using a solution algorithm that implemented t-z analysis approach. The simulated results were compared to experimental data compiled from four different studies conducted using equipment of different scales, under various normal pressures, and using different fill materials and geosynthetic types. This allowed suitability of the proposed solution algorithm in predicting pullout results be evaluated for a wide range of soil and geosynthetic types, normal pressures, and test configurations. It was concluded that the precision of both the geosynthetic tensile behavior and soil-geosynthetic interface shear behavior constitutive models has significant effect on the predicted pullout results. That is, accurate knowledge of these behaviors is important to produce pullout test results

Overall, findings from this study suggest that the solution algorithm adopted using t-z analysis approach can provide reasonably good estimation of pullout test results. Consistency of the estimations made by t-z analysis approach with the experimental results was found to be well within the typical consistency levels presented between experimental results and comparatively more sophisticated methods (e.g., Finite Element Analysis).

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