Numerical Evaluation of the Pullout Box Method for Studying Soil-Reinforcement Interaction

Zehong Yuan and Koon Meng Chua

Soil-reinforcement interaction properties are fundamental design parameters in the design of reinforced earth structures. Those properties can be determined either from direct shear or pullout tests. The pullout box test was used because it is more versatile than the direct shear test method in that it can be used to study wire meshes, strips, geogrids, geotextiles, and other types of reinforcements in different soils. The internal dimension of the pullout box is 30 in. long, 28 in. wide, and 24 in. deep. A finite-element program called GEOT2D capable of solving both the planar problem and the axisymmetric solid of revolution was developed. The soil-reinforcement system in GEOT2D is represented by isoparametric continuum elements, two-node membrane elements, and four-node interface elements. It is shown that with the nonlinear interface shear parameters obtained from the direct shear test pullout responses can be accurately predicted. Conversely, by matching numerically predicted pullout responses with measured pullout responses, it is possible to obtain nonlinear interface parameters directly from pullout tests, which can in turn be used to study performance of various types of reinforcements in earth structures. Effects of size of the apparatus, the reinforcement stiffness or extensibility, and the gap on pullout responses are discussed. The procedure presented takes into account those variables and thus yields soil-reinforcement interface properties both accurate and consistent for all applications.

In the design of reinforced earth structures, which includes reinforced earth retaining walls and reinforced slopes, the soil-reinforcement interaction properties described by the interface friction angle and cohesion are the fundamental design parameters. Those properties can be obtained either from direct shear tests or from pullout tests.

In recent years the New Mexico State Highway and Transportation Department (NMSHTD) has been considering and using different types of reinforcements in slope stabilizing and earth-retaining structures. During the past 4 years, more than 130 pullout tests have been performed for NMSHTD at the University of New Mexico (UNM). The reinforcing materials tested include wire meshes, geogrids, and geotextiles, and they were tested in concrete sands and different types of clays and silty sands. A substantial data base of pullout responses is available (*1*).

This study is presented in two parts. In the first part the soil-reinforcement interaction properties obtained from direct shear tests can be used in the numerical code to match pullout box test results. In the second part, to obtain soil-reinforcement interface properties from pullout tests accurately, those properties should be back calculated by matching

numerical predictions with test results. The interface properties obtained by using this approach are thus independent of the apparatus configuration.

BACKGROUND

Soil Reinforcement

The concept of reinforced earth essentially involves introducing elements that can take tension into the soil mass and, as a result, increase the stability of the earth structure. This concept is an appreciable one and is shown to be reliable. However, recently and with numerous types of new soil-reinforcing elements being made available and also with more demanding design applications, there is a need to perform more accurate analysis of these reinforced earth structures.

There are two basic types of soil-reinforcing elements: metallic and synthetic. The metallic type includes galvanized steel strips, welded wire mesh, stainless steel, plain steel, and aluminum alloy. Structural plastics were introduced only in recent years. Polyester was used first in the nonwoven form. Other plastics used include polypropylene and polyethylene. These plastic materials are available in forms of geogrids and geotextiles. Geogrids are relatively stiff, netlike materials with large openings (typically ½ to 2 in.) between ribs or bars, whereas geotextiles are textiles in the traditional sense and can be found in woven and nonwoven forms.

Analysis of Reinforced Earth Structures

The conventional methods (*2,3*) for analyzing and designing reinforced earth structures follow the limit equilibrium approach. Basically, the methods require the assumption of the failure surface and then checking for equilibrium based on that surface. The methods are simple and easily understood by practicing engineers. However, the methods cannot be used to predict deformation behavior of reinforced earth structures and are usually very conservative. The other approach involves numerical methods, which include the finite-element method. Numerical methods can be made to predict both failure and deformation of reinforced earth structures more accurately.

The finite-element approach used in studying reinforced earth structures is divided into two methods: composite and discrete. The composite method (*4,5*) treats the behavior of

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University of New Mexico, Albuquerque, N. Mex. 87131.

reinforced earth as a locally homogeneous composite material. The displacement compatibility between soil and reinforcement is inherently assumed. This approach suffers from two major drawbacks: (a) it does not allow direct assessment of the internal and local stability of reinforced earth structures and (b) the discretization of the soil-reinforcement system is somewhat arbitrary. In the discrete approach, the soil-reinforcement system is represented by continuum elements, reinforcement elements, and soil-reinforcement interface elements and can evaluate both internal stability and external stability of reinforced earth structures.

Constitutive Modeling of Soil-Reinforcement System

Modeling of Soils

The soil can be modeled as in other applications by using the hyperbolic stress-strain soil model (6). The tangent modulus (E_t) can be expressed as

$$E_t = \left[1 - \frac{R_f(1 - \sin \phi)(\sigma_1 - \sigma_3)}{2c \cos \phi + 2\sigma_3 \sin \phi} \right]^2 KP_a\left(\frac{\sigma_3}{P_a}\right)^n \quad (1)$$

where

$$
\begin{aligned}
P_a &= \text{atmospheric pressure,} \\
k &= \text{modulus number,} \\
n &= \text{modulus exponent,} \\
R_f &= \text{failure ratio,} \\
c, \phi &= \text{soil cohesion and soil friction angle,} \\
&\quad \text{respectively,} \\
\sigma_1, \sigma_3 &= \text{maximum and minimum principal} \\
&\quad \text{stresses, respectively, and} \\
K, n, R_f, c, \text{ and } \phi &= \text{material parameters.}
\end{aligned}
$$

The values of those material parameters can be determined from conventional triaxial tests. The Mohr-Coulomb failure criterion is assumed, and the deviator stress is limited by

$$(\sigma_1 - \sigma_3)_f = \frac{2c \cos \phi + 2 \sigma_3 \sin \phi}{1 - \sin \phi} \quad (2)$$

Modeling of the Soil-Reinforcement Interface

One method of modeling soil-structure interaction behavior was suggested by Clough and Duncan (7). The expression for the interface shear stiffness is given by

$$k_s = K_i \gamma_w \left(\frac{\sigma_n}{P_a}\right)\left(1 - \frac{R_{fi}|\tau|}{c_i + \sigma_n \tan \delta}\right)^2 \quad (3)$$

where

K_i = interface modulus number,
γ_w = water density,
P_a = atmospheric pressure,
σ_n = normal stress,
τ = interface shear stress,
R_{fi} = failure ratio,
c_i = interface cohesion, and
δ = interface friction angle.

As can be seen, the Mohr-Coulomb failure criterion is incorporated into this hyperbolic equation.

Factors Affecting Pullout Response

A summary of the factors affecting pullout responses observed in the laboratory is given by Juran et al. (8), and the factors include the following:

● Density of soils: The pullout resistance is greater in a dense soil than in a loose soil.

● Confining/normal stress: The amount of dilation in dense soils decreases with increasing confining stress. As a result, the apparent friction angle also decreases.

● Particle interlocking: The degree of particle interlock is defined by the ratio between the grid opening or fiber spacing and average particle size. When this ratio approaches unity, the effective particle interlocking within reinforcement increases and the pullout resistance is also increased.

● Grid orientation: The orientation of grid reinforcements affects the total bearing area, which is the area normal to the direction of pullout.

● Stiffness of the reinforcement: The pullout resistance and thus average soil-reinforcement interface shear stresses are functions of reinforcement stiffness. For low stiffness reinforcements, the interface shear stresses are not uniformly distributed.

● Boundary effects: The effect of the rigid front face on pullout response is not well understood. Johnston (9) performed pullout tests with and without the front face and reported that a lower pullout resistance was generated with a rigid front face. However, the results of similar tests by Hornbeck (10) showed that a flexible front face produced a lower pullout resistance. This disparity suggests that it may be important to determine the appropriate gap size in the soil box to pull the reinforcement through. This issue and the effect of reinforcement stiffness are investigated in this study.

Laboratory Test Methods

As was mentioned earlier, direct shear and pullout tests are two laboratory testing methods to determine soil-reinforcement interface properties. In the direct shear test, the reinforcement is fixed on a plane face, usually onto a wooden block, with soil sliding over that. The pullout test is a more elaborate one. A reinforcement is placed at the mid-depth in a rectangular container full of soil. The reinforcement is pulled through a gap from one side of the box. For reinforcements with larger openings, such as Tensar SR2 geogrids, the shear box method may not be applicable, and a pullout box of a large dimension is more appropriate.

When comparing the results from the two different test methods, Ingold (11) observed that the pullout test gives higher values of the apparent friction angle for metallic and high modulus geogrid reinforcements and lower values for the more extensible geogrids and geotextiles. Also, it has been observed that the apparent friction angle obtained from pullout tests decreases with increasing normal pressure for low tensile modulus reinforcements, whereas the direct shear tests yield an

approximately constant interface friction angle (11,12). Thus, there appears to be a need to consider what is really being measured. In the direct shear test, because the membrane is glued to a rigid block (see Figure 1a) the tensile stress in the reinforcement is small. As a result, measurements from the direct shear test will be more a function of soil-reinforcement interface properties (i.e., interface friction angle and cohesion) and not the reinforcement stiffness. On the other hand, in the pullout test (see Figure 1b) both interface shear stress and tensile stress are mobilized owing to the pullout, and the pullout resistance depends on both interface properties and reinforcement stiffness.

COMPARING PULLOUT TEST RESULTS WITH NUMERICAL SIMULATION

Experimental Study

Pullout Box and Test Procedure

The steel pullout box at UNM was designed and built in 1985 for the NMSHTD and is described in detail by Carney (13). The internal dimension of the pullout box (see Figure 2) is 30 in. long × 28 in. wide × 24 in. deep. The loading system consists of three 20-ton capacity hydraulic jacks, one of which is used to apply the vertical load and the other two for the pullout. Strain-gauge–type load cells are used to measure applied loads. The applied vertical load is transmitted to the soil by a pyramid thick wooden blocks and is assumed to be uniformly distributed in the soil mass before the pullout force is applied. The maximum vertical pressure available is 6,800 psf, which is equivalent to an overburden of 60 ft of soil. The reinforcement is usually pulled at some constant rate after the vertical load is applied, and the pullout distance is measured by dial gauges at the pulling end. The dial readings are then reduced by the amount caused by extension of the exposed end of the reinforcement to obtain the accurate pullout displacement at the front face of the box.
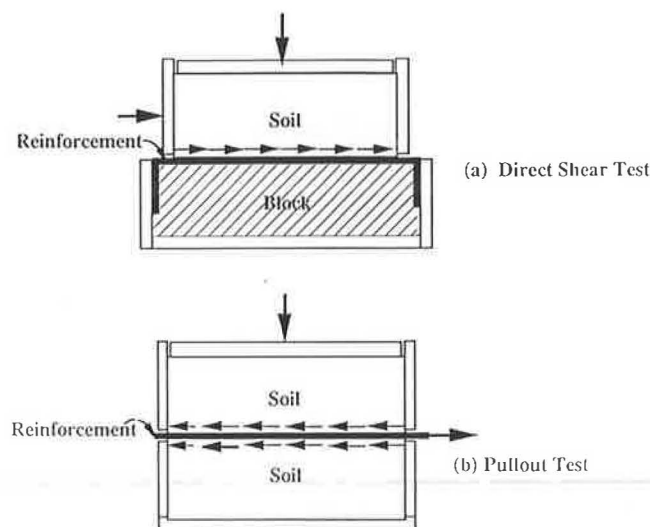

FIGURE 1 Two types of laboratory testing methods for determining soil-reinforcement interface properties.
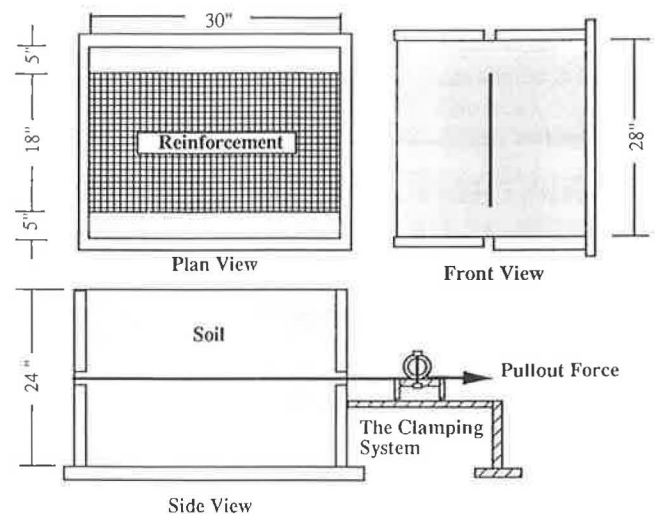

FIGURE 2 Schematic diagram of the pullout box.

Pullout Test Results

Performed pullout tests were performed with Geolon 200 and Tensar SR2 in concrete sand. The sand used was fine to medium, well graded (SW), with a coefficient of uniformity of 5.08. For all the pullout tests the sand was rained from 14-in. height to obtain a uniform dry density of about 108 pcf and a relative density of about 70 percent. Results from conventional triaxial tests indicate that the effective friction angle of the sand is 42 degrees.

Geolon 200, which is a woven geotextile, had a tensile strength of about 100 lb/in. according to uniaxial tensile tests. The stress-strain curve is given in Figure 3. For the pullout tests, the 18-in.-wide geotextile was placed at the middle horizontal section of the pullout box with 1 ft of concrete sand below and 1 ft above. The lateral clearance between the specimen and box was 5 in. on each side (see plan view, Figure 2). The geotextile was wrapped around a 3-in. diameter cylinder at the pulling end, and the cylinder was fixed to a channel beam. The channel beam moves along a track in the horizontal direction (see Figure 2). The other end of geotextile
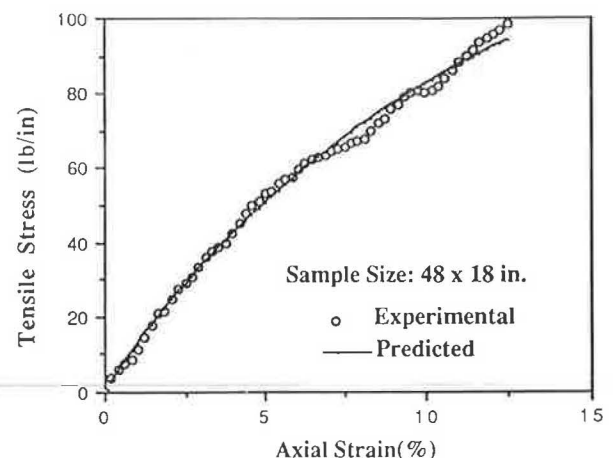

FIGURE 3 Stress-strain relationship of Geolon 200.

is free to move. The geotextile was pulled at a constant rate of about 0.3 in./min after the predetermined vertical load was applied.

The overburden pressures used were 0.71, 1.58, 4.14, and 5.03 psi, respectively. The results in terms of pullout force-displacement curves are presented in Figure 4. Figure 5 presents the average shear strength curve (τ_f-σ_n relation) of the sand-Geolon 200 interface. The average maximum shear stress is defined as the maximum pullout force from each pullout test divided by twice the embedded plan area of the reinforcement. The sand-Geolon 200 interface shear strength envelope obtained from direct shear tests is also indicated in Figure 5 for comparison.

Referring again to Figure 5, it can be seen that the maximum average shear stress τ_f obtained from pullout tests approaches a limiting value as the normal pressure σ_n is increased whereas τ_f increases linearly with increasing σ_n in direct shear tests. This is true because the maximum pullout force in any pullout tests should be less than the tensile strength of the reinforcement. The limiting value of τ_f is usually equal
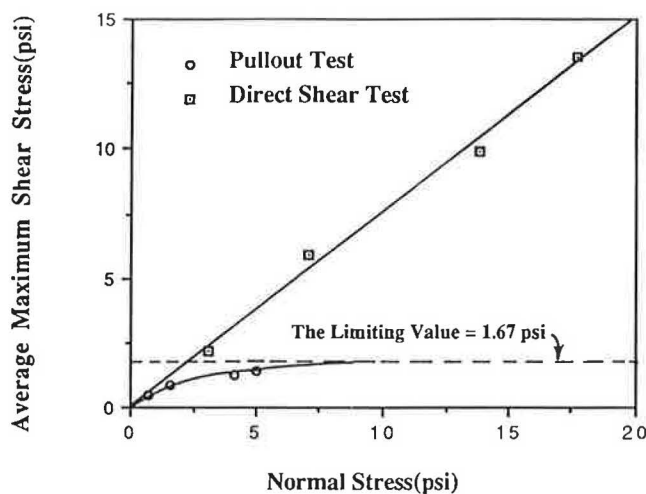
to the tensile strength divided by twice the embedded plan area of the reinforcement. Here, τ_f defined this way is a function of the specimen or the pullout box dimension. The limiting value of τ_f for the sand-Geolon interface with this particular pullout box (30 in. × 28 in. × 24 in.) is only about 1.67 psi, as indicated in Figure 5.

Numerical Simulation

Finite-Element Code: GEOT2D

To model pullout tests and behavior of reinforced earth structures by using the discrete finite-element approach, a finite-element program GEOT2D was developed that models both the axisymmetric volume of revolution and the planner types of reinforced earth structures. This was achieved by using a combination of continuum, membrane, and interface elements. The new axisymmetric interface model and the axisymmetric membrane element model (*14*) were incorporated into the program to model accurately the axisymmetric type of reinforced earth structures. In the laboratory this feature allows the simulation of composite behavior of reinforced soil samples in conventional triaxial conditions. For modeling pullout tests, only the plane strain type of analysis will be performed.

Modeling of the Soil

The hyperbolic stress-strain relation as described in Equation 1 was used to model the sand. The values of parameters K, c, ϕ, n, and R_f were obtained from triaxial tests and are presented in Table 1.

Modeling of Reinforcement

The tensile test was performed with a 48-in. long × 18-in. wide piece of Geolon 200 geotextile. The tensile strength was found to be about 100 lb/in. width at 12.5 percent of axial strain, about 20 percent higher than that reported by the manufacturer that used a 4-in. wide specimen according to ASTM D-1682 (grab test). The larger width is more appropriate because geotextiles are normally not applied in narrow strips, and therefore the edge effects caused by the transverse reinforcement should be considered. The tensile test results of Geolon 200 show the nonlinear stress-strain behavior (see Figure 3). To take into account the nonlinearity, the following hyperbolic relationship was assumed:

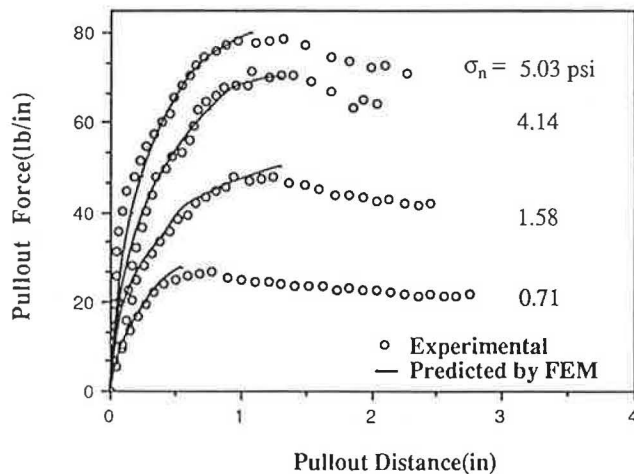$$\sigma = \frac{\varepsilon}{\alpha + \beta\varepsilon} \tag{4}$$

FIGURE 4 Predicted and measured pullout responses of Geolon 200 in sand.

FIGURE 5 Average shear strength curves of sand-Geolon 200 interface.

TABLE 1 HYPERBOLIC PARAMETERS FOR CONCRETE SAND

K	n	R_f	c	ϕ	G	F	d
200.	0.5	0.7	0	42°	0.35	0	0

where

α, β = constants,
σ = tensile stress, and
ε = tensile strain.

It can be seen from Figure 3 that the proposed model fits the test data reasonably well. The values of parameters α and β for Geolon 200 geotextile are 0.00075 in./lb and 0.0049 in./lb, respectively. The tensile modulus of Geolon 200 reduces from an initial value of 1,330 lb/in. at zero strain to about 430 lb/in. at failure (12.5 percent strain), which represents a more than three times variation. At a high strain level, the linear or linear secant tensile modulus approximation will be inaccurate.

Modeling of Soil-Reinforcement Interface

The hyperbolic interface model described by Equation 3 is obtained by incorporating the Mohr-Coulomb failure criterion into the original hyperbolic equation. In the numerical analysis, a small residual interface shear stiffness is assigned to the interface element when the mobilized interface shear stress is greater than the shear strength calculated from the failure criterion. Because the interface is actually an interaction response and not a "material," the "shear strength" and residual interface shear stiffness approach may not be appropriate. In view of this, the original hyperbolic form was followed, but without attempting to incorporate the failure criterion into the model. The interface shear stiffness k_s is directly controlled by the relative displacement between soil and reinforcement

$$k_s = \frac{\partial \tau}{\partial u_r} = \frac{a}{(a + bu_r)^2} \tag{5}$$

where

a = the inverse of the initial shear stiffness,
b = the inverse of the ultimate interface shear stress, and
u_r = the relative displacement between soil and reinforcement.

The two fundamental hyperbolic parameters a and b are found to be dependent on the normal stress. For sheetlike reinforcements, a and b can be determined easily from the direct shear tests. Four direct shear tests were performed to determine the sand-Geolon 200 interface parameters. The shear stress-displacement curves are presented in Figure 6. The values of a and b corresponding to the different normal stresses σ_n are listed in Table 2. By correlating a and b with σ_n, we obtained the following two relationships:

$$a = 0.0036\sigma_n^{-0.5} \tag{6}$$

$$b = 1.2\sigma_n^{-1.0} \tag{7}$$

and the corresponding coefficient of correlation, R^2, is 0.97 and 0.94, respectively. The forms of expression are chosen to simplify the representation of the numerical interface model. Alternatively, the set of shear stress versus relative displacement curves can be input into the program, and interpolation
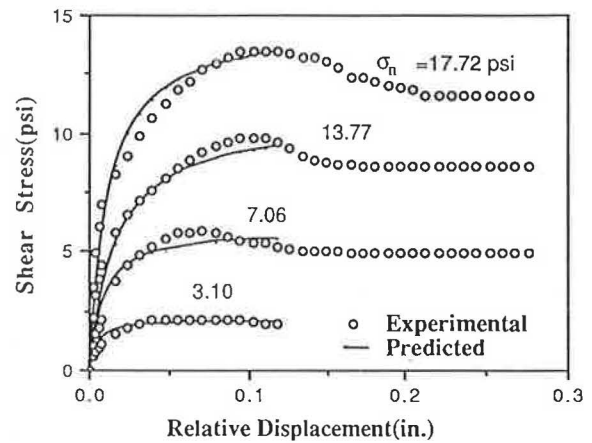

FIGURE 6   Stress-displacement curves of sand-Geolon 200 interface.

TABLE 2   VARIATION OF a and b WITH σ_n FOR SAND-GEOLON 200 INTERFACE

σ_n (psi)	3.10	7.06	13.77	17.22
$a(10^{-3}$ in/lb)	1.98	1.37	1.02	0.76
$b(10^{-3}$ in^2/lb)	463.40	167.62	92.61	68.01

is performed in a subroutine. This simple model can be seen to match the direct shear test results indicated in Figure 6 quite well. The data points beyond the peak shear stresses are neglected because it is inconceivable that the reinforcing material would be allowed to go beyond this level in a design. In any case, as in most laboratory tests, measurements beyond the failure threshold are affected by numerous factors not related to the soil properties.

It is possible that the a and b values are related to c_i and δ. If this is the case, then the "cohesion" will be strictly a function of the surface texture of the material and the soil. However, the "apparent friction angle" will have to include, for instance, the effects of aggregate-reinforcement interlock, grid spacing, lateral-bearing capacity, and soil dilation (and, hence, the soil properties). The aim of the subsequent phase of the study is to attempt to define a and b from the reinforcement characteristics and soil properties by using the pullout test data base at the University of New Mexico.

Simulation of the Pullout Box

The finite-element mesh used to simulate pullout tests is presented in Figure 7. Because the box is made of ½-in.-thick steel plates, it is reasonable to assume that soil particles cannot move in the direction perpendicular to the boundary. The reinforcement, however, is allowed to move freely in the longitudinal direction.

The soil-reinforcement system is represented by four-node isoparametric continuum elements, a two-mode membrane element, and four-node Goodman-type interface elements (15).
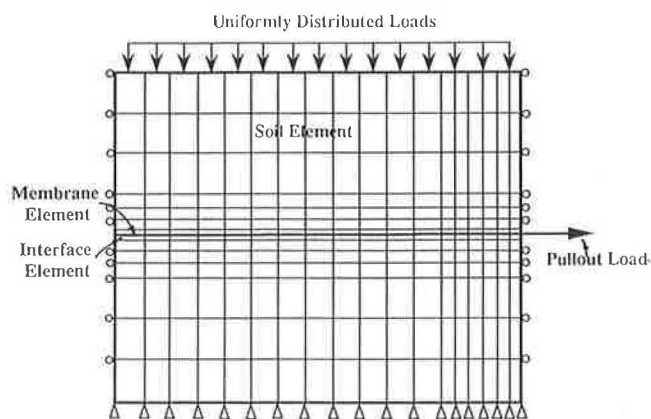
FIGURE 7   Finite-element discretization of pullout tests.

The shear stiffness of the interface element was described in an earlier section. During the pullout test, the applied vertical load is kept constant but the pullout force increases continuously until the peak value is reached. To model this process, the pullout force is applied in small increments.

Results

The *x* displacement fields (the lateral soil movement in the longitudinal direction) for a typical pullout test at 10, 50, and 100 percent of the maximum pullout force of 50 lb/in. width are shown in Figure 8–10. The pullout displacement corresponding to the peak pullout force is 1.28 in. A narrow high displacement band is developed around the reinforcement with the maximum occurring at some distance away from the front face owing to the rigid front face effect. Figure 11 presents the three-dimensional plot of soil movement in the *x* direction.

The contour of principal stress ratio (σ_1/σ_3) at maximum pullout force is presented in Figure 12. The higher stress ratios are found near the face of the pullout box and indicate the
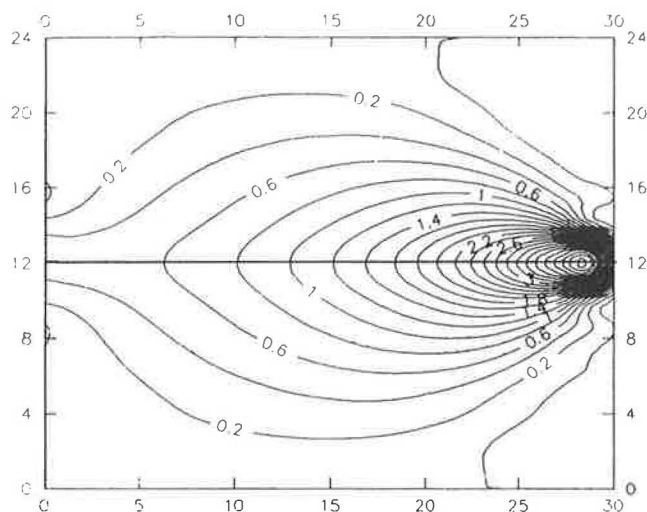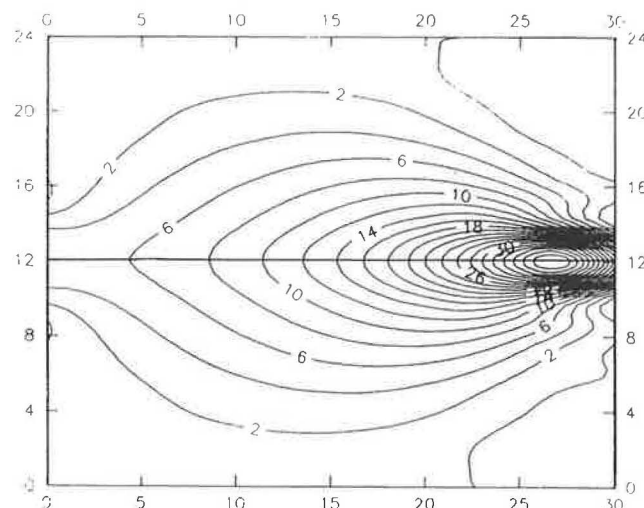

FIGURE 9   Contours of *x*-displacement at 50 percent of maximum pullout force $(10^{-3}$ in.).
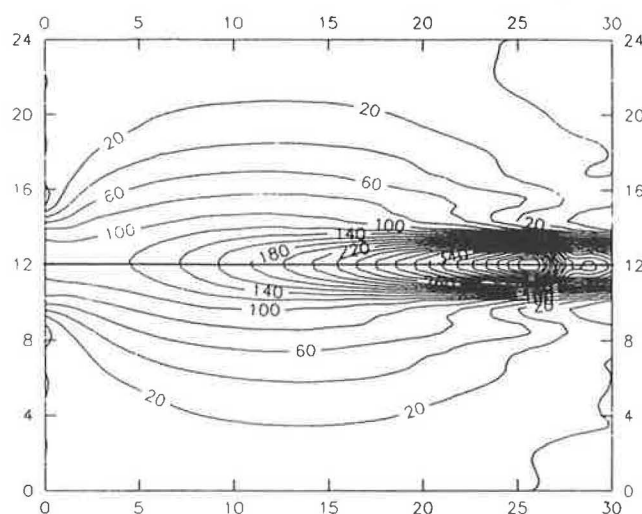

FIGURE 10   Contours of *x*-displacement at 100 percent of maximum pullout force $(10^{-3}$ in.).

failure zone. Figure 13 presents the tensile force distributions along the reinforcement at different pullout load levels for a typical pullout test. The tensile force decreases from the pullout end to the front end in a nonlinear manner, which indicates a nonuniform distribution of shear stresses along the reinforcement.

Referring again to Figure 4, the predicted pullout response matches the measured response of the four pullout tests quite well and suggests that the interface parameters *a* and *b*, obtained from direct shear tests as was described earlier, are essentially the same as those in the pullout tests. Thus it appears that the difference between soil-reinforcement interaction parameters obtained from direct shear and pullout tests reported by other researchers (5–7) may be caused by the interpretation approach rather than by testing methods. If the interface properties are obtained as shown here, then it will be possible to accurately predict responses of reinforced earth structures in the form of the pullout test as well as in the field.
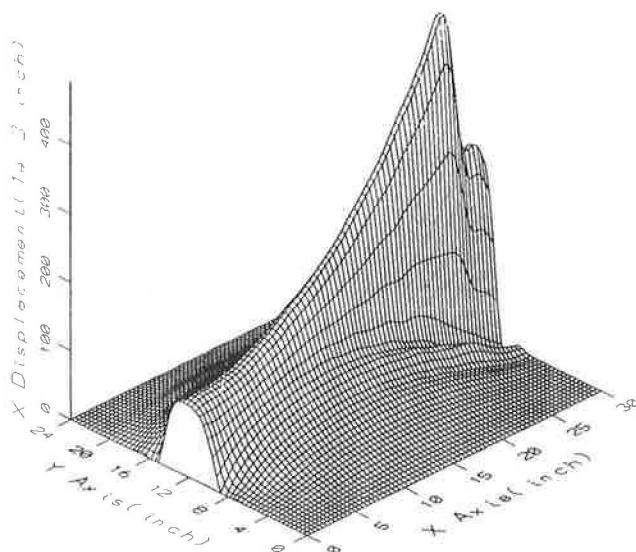

FIGURE 8   Contours of *x*-displacement at 10 percent of maximum pullout force $(10^{-3}$ in.).

FIGURE 11 Three-dimensional display of x-displacement at maximum pullout load (10^{-3} in.).
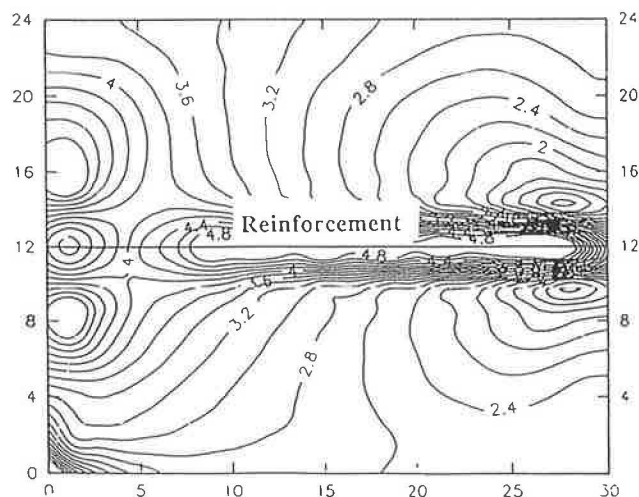

FIGURE 12 Contours of principal stress ratio at maximum pullout load.
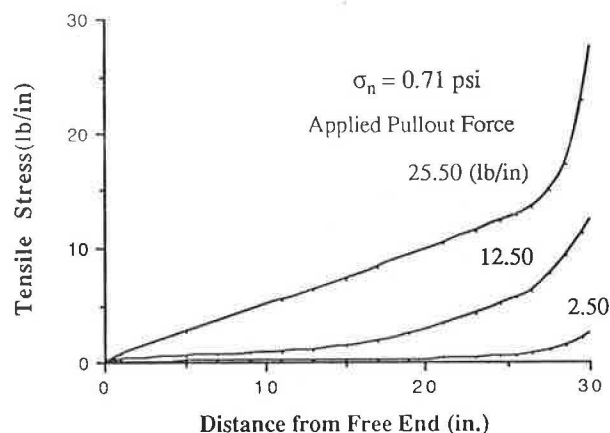

FIGURE 13 Predicted tensile stress distribution along Geolon 200.

OBTAINING INTERFACE PROPERTIES FROM PULLOUT TESTS

The interface properties of sheetlike reinforcements such as Geolon 200 can be obtained from the direct shear tests without difficulty. However, it is difficult to perform direct shear tests to obtain the interface properties for certain types of reinforcements, such as geogrids and wire meshes. For those types of reinforcements, the pullout testing method is more appropriate. The problem associated with the pullout test is how to obtain interface parameters from the pullout results and at the same time consider the nonuniform distribution of interface shear stresses and factors affecting the pullout results. This can be done by matching the measured pullout response numerically. The procedure for obtaining interface parameters from the pullout tests is as follows:

- Determine properties of soil and reinforcement and assign some reasonable values to interface parameters a and b;
- Use the interaction parameters as input and perform the numerical simulation by using GEOT2D or a similar numerical code;
- Compare the numerical pullout response with the measured response; and
- Change interface parameters until the measured pullout response is matched.

Interface Properties

This procedure was used to obtain nonlinear interface parameters a and b between Tensar SR2 geogrid and concrete sand from pullout test results. The stress-strain parameters α and β, obtained from a tensile test, are 0.00023 in./lb and 0.0024 in./lb, respectively. The soil is the same sand used in the study described earlier. Figure 14 presents a reasonably good match of the predicted and the measured pullout response of Tensar SR2 geogrid in the sand. The back-calculated a and b values at different overburden pressures σ_n obtained by the procedure are given in Table 3. By correlating a and b with σ_n, these two parameters can be expressed as functions of σ_n, which are also given in Table 3.
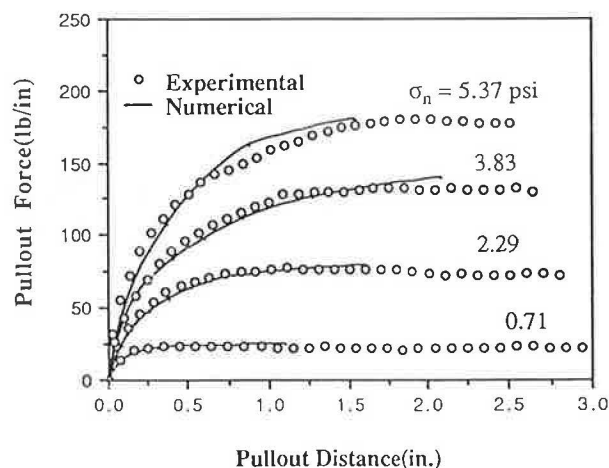

FIGURE 14 Comparison of experimental and numerical results of SR2 in sand.

TABLE 3   VARIATION OF *a* AND *b* FOR SAND-SR2 INTERFACE

σ_n (psi)	0.71	2.29	3.83	5.37
$a(10^{-3}$ in/lb)	3.92	1.61	1.18	0.98
$b(10^{-3}$ in^2/lb)	3.00	1.20	0.72	0.50

$$a = 0.0030\ \sigma_n^{-0.7}$$

$$b = 2.67\ \sigma_n^{-1.0}$$

Factors Influencing the Pullout Response

Extensibility

To investigate the effects of reinforcement extensibility or stiffness on pullout responses, a series of pullout tests was simulated by fixing interface parameters *a* and *b* and by varying reinforcement stiffness k_a. The reinforcement was assumed to be linearly elastic to simplify this investigation. Figure 15 shows predicted pullout responses with different k_a. From Figure 15 a higher k_a will result in a larger pullout resistance for a given pullout displacement.

Gap Size of the Pullout Box

The gap effect was investigated by simulating pullout tests of SR2 in concrete sand in the pullout box with different gap sizes. The results, given in Figure 16, indicate that the pullout resistance will increase as the gap size is made smaller. This is because the soil arches around the opening and thus provides more shear support to the reinforcement near the face of the front wall. The results indicate that this factor should be taken into account when a pullout box is designed. However, if the numerical approach presented here is followed, then the actual gap size can be simulated and, hence, be included in determining the pullout response.
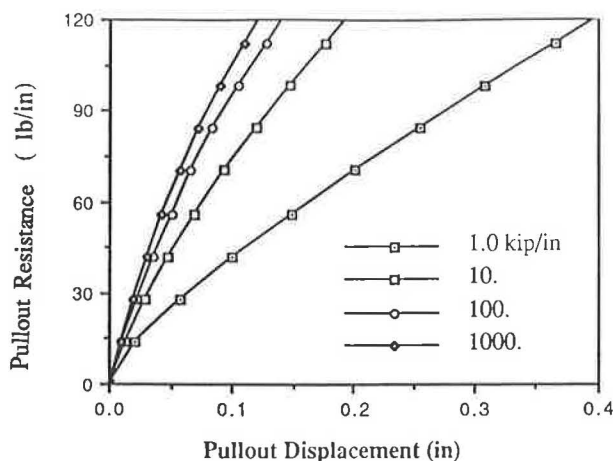

FIGURE 15   Predicted effect of reinforcement stiffness on pullout resistance.
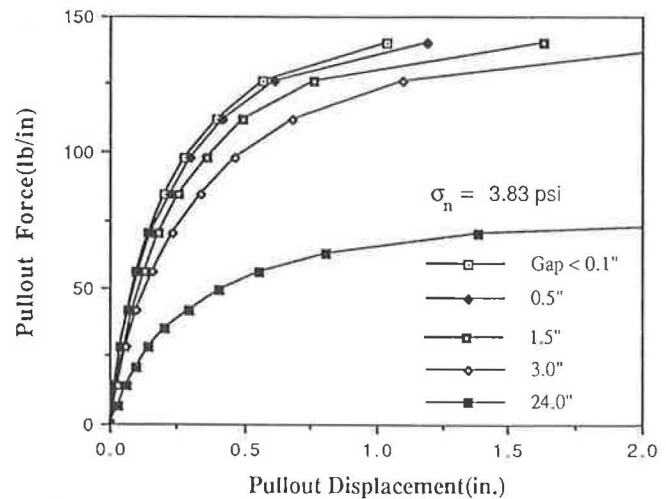

FIGURE 16   Predicted effect of gap size on pullout resistance.

CONCLUSIONS

Soil-reinforcement interface behavior can be obtained from the pullout test by using the test results in conjunction with the GEOT2D finite-element program. The pullout test is more versatile than the direct shear test and can be used to study wire meshes, strips, geogrids, geotextiles, and other types of reinforcements in different soils. The computer program GEOT2D can accurately predict pullout responses in pullout tests by using the nonlinear interface properties obtained from direct shear tests, which indicate that interaction parameters from the two testing methods are basically the same for sheetlike reinforcements.

The following conclusions were made regarding pullout resistance measured in the laboratory:

1. Pullout resistance measured directly from pullout tests should be used with caution because the response is a total response that depends on the configuration of the pullout box or the specimen dimension.

2. In pullout tests, soil-reinforcement interface shear stresses are not uniformly distributed along reinforcements. Average shear stresses and thus apparent interface parameters are functions of the specimen dimension.

3. The direct shear testing method can be easily used to evaluate interaction properties between soil and sheetlike reinforcements, such as Geolon 200, but is not appropriate for gridlike reinforcements, such as Tensar SR2.

4. A stiffer reinforcement will result in a larger pullout resistance for a given pullout displacement.

5. The pullout resistance appears to increase with a small gap size. It is suggested that the gap effect should be considered when comparing results of different test apparatus.

Finally, it should be realized that average or apparent soil-reinforcement interaction properties obtained from the direct and the pullout testing methods are different. This is mainly because of different methods of interpretation rather than by testing methods. However, the procedure presented here will take into account the different test configurations, soils, and

reinforcements to determine soil-reinforcement interface characteristic.

ACKNOWLEDGMENT

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