DEVELOPMENT AND APPLICATION OF ANCHOR-SOIL INTERFACE MODELS

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ABSTRACT

Understanding the anchor-soil interface behavior is essential to the determination of the anchor pullout capacity and prediction of the deformation of the anchor-reinforced system under working load conditions. Numerous anchor pullout tests under both lab and field conditions have generally yielded extremely high apparent interface strength, which cannot be explained by the conventional interface friction theory between the anchor material and the soil. In this paper, the mechanism and the phenomena of the anchor-soil interaction were studied, and the soil dilatancy due to shearing was regarded as the main factor contributing to the increase of the anchor-soil interface friction. Considering this effect, along with the adoption of a cylindrical shear deformation pattern of the soil, anchor-soil interface models have been developed for hardening and softening behavior, respectively. Using the developed interface model, the forward and backward calculation algorithms have been formulated and applied to predict the anchor performance for the given interface parameters and to determine the interface parameters from the given anchor pullout test, respectively. Furthermore, the prediction of the anchor pullout performance was compared favorably with two hypothetical cases, two laboratory test results, and one field case.

Key words: anchor, deformation, dilatancy, friction, interface, model, pullout, reinforcement (IGC: E4)

INTRODUCTION

The properties of the interface between the soil and the inclusion (e.g., ground anchor, pile, and geosynthetic) play an important role in governing the response of the composite (soil/inclusion) system under the applied loads. The load transfer behavior of the interface in terms of the relationship between shear stress and displacement is particularly important if one needs to accurately understand the deformation behavior of the system under the working load conditions. In the area of ground anchor analysis/design, the attention paid to the interface shear stress-displacement properties has been very limited. In general, the practice (e.g., tieback wall anchors) calls for all of the installed anchors to be proof tested and at least 3–5% of the total number of the installed anchors be subjected to the performance test (Nicholson et al., 1982; Otta et al., 1982; Pfister et al., 1982; Schnabel, 1984). All of these tests, however, only serve the purpose of verifying the design load as well as load-carrying capacity. These test results have not been used in such a way that they can provide information on the deformation field of the system under the working loads.

The majority of research on ground anchors has been on the determination of the anchor pullout capacity under a variety of conditions, such as the embedded depth, the inclination angle, and the types of anchors and soils. In a few cases where the anchor load versus the anchor head displacement has been studied, the interface models used were mostly empirical in nature (Vijayvergiya, 1977; Coyle and Reese, 1966; Coyle and Sulaiman, 1967), or were derived based on the pure shear test results of soils (Su and Fraga, 1968; Kraft et al., 1981).

The existing interface models (Randolph and Wroth, 1978; Kraft et al., 1981) developed for simulating pile behavior under axial load assume that the surrounding soil is subjected to pure shear and under constant confining pressure. This assumption cannot be applied directly to modeling the anchor pullout behavior, as both laboratory tests and field tests indicate significant soil dilatancy during anchor pullout (Su and Fraga, 1988; Yoshihara and Kishida, 1982; Shields et al., 1978). Due to shear dilatancy of the soil, the radial stress can reach as high as 20 times the overburden pressure. Because of the pronounced dilatancy effort, the use of the existing soil-pile interface model to predict ground anchor pullout behavior would be erroneous. A soil-anchor interface model that is capable of taking into account of the effects of the dilatancy and confining pressure is needed.

The objectives of this paper are as follows:

a. Development of soil-anchor interface models that explicitly include the effect of soil dilatancy, the confining pressure acting on the anchor, the relative stiffness between the anchor and the soil, and the size of the influence zone. Furthermore, both 'hardening' and 'softening' types of load transfer...
mechanism at the interface are considered.

b. Presentation of a series of parametric study results to shed insights on the importance of the soil dilatancy, the relative rigidity, and the influence zone size on the anchor pullout behavior.

c. Development of the back-calculation algorithms to extract important soil-anchor interface properties from anchor pullout test results. The proposed back-calculation techniques can be used in practice to allow engineers to achieve more confidence and efficiency in predicting anchor performance.

DEVELOPMENT OF SOIL-ANCHOR INTERFACE MODEL

Generally, when the pullout force is applied to the anchor head, the soil surrounding the anchor will be subjected to shear. Consequently, three stages of soil particle movement can be identified (Morimichi et al., 1988, 1990): (i) the particle's relative movement (shear deformation of soil), (ii) transition of the particle's relative movement to rigid body movement (mixed shear deformation and sliding), (iii) rigid body movement (sliding). Each stage is associated with different interface behavior: stage 1 indicates that only shear deformation is developed in the soil mass; in stage 2 yielding occurs at the interface with the accompanying shear deformation; and stage (iii) indicates the onset of rigid body sliding along the interface. Correspondingly, two types of interface models are adopted, the hardening model (two-stage model) and the softening model (three-stage model), as shown in Fig. 1.

Some basic assumptions in the mathematical derivation of the interface models are summarized as follows: First, the dilatancy of the soil is considered as an average value during the entire range of loading. Second, the shear deformation pattern in the soil surrounding the anchor resembles a concentric cylindrical pattern, as shown in Fig. 2. Thirdly, the work (energy) equation is applied. Lastly, the nonlinear behavior for the soil is represented by a hyperbolic equation. It should be noted that the above basic assumptions are adopted so that a simple mathematical equation can be developed. If desired, some of these assumptions can be relaxed or removed, resulting in a more complex equation.

From the force equilibrium in vertical and radial directions of the soil element shown in Fig. 3(a), one gets:

\[
\frac{\partial}{\partial R}(R\sigma_t) = R \frac{\partial \sigma_r}{\partial X}
\]  

(1)

where

- \( \tau \) = Mobilized shear stress at location \((X, R)\).
- \( \sigma_r \) = Normal stress in the soil mass at location \((X)\), and,
- \( R \) = Radius.

Normally, the change of shear stress along the \( X \)-direction for a small finite change in location \( (dx) \) is very small compared to the change in the radial direction \( (R) \); therefore, the term \( (\partial \sigma_r / \partial X \equiv 0) \), which leads to

\[
\frac{\partial R}{\partial X} = \tan \psi
\]  

(4)

Where:
where:

* $w_e$ = external work done by the applied load
* $w_p$ = energy stored in the soil body due to the radial stress
* $w_s$ = energy stored in the soil body due to shear deformation

The energy equation can be expanded as follows.

$$w_e = \frac{1}{2} \int_{R_0}^{R_1} \left( \frac{E}{G} \right) \tau_0 R_0 \ln \frac{R_1}{R_0}$$

$$w_p = \int_{R_0}^{R_1} \left( \frac{1}{2} \right) \sigma_x \varepsilon_x (2\pi R) dR + \int_{R_0}^{R_1} \sigma_x \varepsilon_x (2\pi R) dR$$

$$= \frac{\pi R_0^3 \tau_0}{2} \ln \frac{R_1}{R_0} \frac{R_1}{R_0} \tan \psi + 2\pi \sigma_0 \int_{R_0}^{R_1} \frac{R_0 \tau_0}{G} \ln \frac{R_1}{R_0} \frac{R_1}{R_0} \tan \psi$$

$$w_s = \int_{R_0}^{R_1} \left( \frac{1}{2} \right) \sigma_x \varepsilon_x (2\pi R) dR$$

where:

* $R_1$ = the radius of the influence zone
* $T$ = the axial force per unit length in the anchor

Substituting Eqs. (9), (10), and (11) in the energy equation Eq. (8), the mobilized maximum interface shear stress can be expressed as follows.

$$\tau_{\max} = \tau_0 \left( 1 + \frac{E}{G} \tan^2 \psi \right) + 2\sigma_0 \tan \psi \left( \frac{R_1}{R_0} - 1 \right)$$

where:

* $\tau_{\max}$ = maximum mobilized interface shear strength
* $E/G = 2(1 + \nu)$, and
* $\nu = $ Poisson's ratio

It should be emphasized, however, that above equations are derived for material with elastic stress-strain behavior. However, the results offer a rational way to analyze the mechanisms of the mobilization of the soil-anchor interface strength. Furthermore, the analysis presented in this work provided good agreement with the tests results.

The mobilized shear strength of the anchor-soil interface is a function of the basic anchor-soil interface strength, $\tau_0$, the size of the influence zone ($R_1/R_0$), and the dilatancy angle $\psi$. Based on the adopted concentric cylindrical shear deformation pattern, the relative shear displacement at the interface, $u$, can be calculated as:

$$u = \int_{R_0}^{R_1} \frac{\tau}{G} dR$$

A hyperbolic form is adopted to represent the nonlinear stress versus shear strain relationship for this interface. Thus,

$$G = (\tau, \sigma) = G \left( 1 - \frac{\tau R}{\tau_0} \right)^2$$

$$G_i = k p_s \left( \frac{\sigma}{p_s} \right)^n$$

where:
\[ G_i = \text{initial shear modulus} \]
\[ P_s = \text{atmosphere pressure} \]
\[ k, n, R_i = \text{hyperbolic soil model parameters} \]

Thus, the expression of the interface displacement, \( u \), as a function of the mobilized interface shear stress, \( \tau \), can be expressed as:

\[
\begin{align*}
\frac{\tau R_0}{K p_i R_0} R(1 + R_0 R_k R_i, \tan \psi)(1 - R_k R_i f_i) \\
R_G = 2(1 + v) \\
R_k = \frac{R_0}{R_i} \\
R_i = \frac{\tau}{\sigma_0} \\
f_i = \frac{r_0}{\tau_i}
\end{align*}
\] (16) (17) (18) (19) (20)

\section*{Construction of Interface Load Transfer Curve}

For the hardening (two-stage) model, the derived relationship of the interface displacement vs. interface stress (Eq. (16)) can be used directly to construct the interface load transfer curve. Once the maximum mobilized interface strength is reached, the shear stress vs. displacement curve becomes horizontal (refer to Fig. 1). However, for the softening (three stage) model, the post peak portion of the curve cannot be evaluated by the Eq. (16). Several empirical methods have been proposed via an analogy to the direct shear test on soils (Su and Fragaszy, 1988; Kraft et al., 1981). However, the condition of the direct shear test cannot truly represent the conditions that exist at the soil-anchor interface. This is mainly due to the fact in direct shear test the confining stress remains constant while the soil is allowed to dilate or compress; whereas in the anchor-soil interface the constant volume condition is maintained while the confining stress is varied. Thus, for all practical purpose, a straight line connecting between the peak point and the residual state is used to represent the post failure portion. Two parameters representing this post-peak straight line portion are needed: the slope \( (m) \), and the ratio of the residual strength over the maximum strength \( (\tau_{\text{res}}/\tau_{\text{max}}) \), respectively. Kraft et al. (1981) suggested the following values: the strength ratio, \( \tau_{\text{res}}/\tau_{\text{max}} = 0.9 \), and the displacement that occurs from \( \tau_{\text{max}} \) to \( \tau_{\text{res}} = 0.0762 \text{ cm} - 0.127 \text{ cm} \) for sand, and about 0.254 cm for clay. It will be demonstrated later in this work that these parameters can be determined by back calculation technique given the anchor pullout test.

As discussed above, the parameters required to construct the anchor-soil interface load transfer curves are as follows: (1) the hyperbolic soil model parameters, \( k, n, R_i \), (2) the influence zone radius ratio, \( R_i/R_0 \), and the soil dilatancy angle, \( \psi \), and (3) basic interface strength, \( \tau_0 \) and confining pressure, \( \sigma_0 \) acting on the anchor-soil interface. If the softening model is used, two more parameters are needed: strength ratio, \( \tau_{\text{res}}/\tau_{\text{max}} \), and straight line slope, \( m \). The slope \( m \), as illustrated in Fig. 1, is a representation of how fast the interface strength would reduce from peak strength, \( \tau_{\text{max}} \), to residual strength, \( \tau_{\text{res}} \). Obviously, the model behavior will vary with the different combinations of these parameters, which will be presented in detail in the later sections.

Studies on the pile-soil interfaces has been taken out for many decades, more representative ones is Kraft's theoretical t-z curve (Kraft et al., 1981). However, in their theoretical derivation, an empirical influence zone size and constant interface strength were used. As a result, the model may not be strongly recommended for high anchor capacity generally observed in lab and field tests (Shield et al., 1978). Comparing the developed model with Kraft's equation, it can be observed that when the soil dilatancy is not considered, the interface stress-displacement curves have the same trend. However, once the dilatancy is included, the interface strength predicted by the present model is increased due to the change in the size of the influence zone.

\section*{Numerical Studies}

The developed nonlinear interface model is used to perform a series of parametric studies to gain a better insight into the influences of the various factors contributing to the anchor behavior. The factors investigated include the dilatancy angle, \( \psi \), the size of the influence zone as represented by \( R_i/R_0 \), and the relative rigidity \( \alpha \) defined by the following expression.

\[
\alpha = \frac{\sqrt{E_s A}}{KL/L}
\] (21)

where:
\[ L = \text{anchor bond length} \]
\[ K = \text{average interface modulus for the loading range} \]
\[ A = \text{anchor cross section area, and} \]
\[ E_s = \text{elastic modulus of the anchor} \]

To demonstrate the importance of considering the dilatancy effect on the interface behavior, the dilatancy angle is varied between 5° and 25°, while maintaining other properties constant as follows: \( \tau_{\text{res}} = 27.58 \text{ kPa, } \alpha = 68.95 \text{ kPa, } R_i/R_0 = 20 \), hyperbolic model parameters for the soil: \( k = 500, n = 0.8, R_i = 0.9 \), and the anchor properties: diameter of anchor, \( D = 2.54 \text{ cm} \), Young's modulus of steel used for anchor, \( E_s = 2.0 \times 10^5 \text{ MPa} \).

As shown in Fig. 4, the mobilized interface stress increases dramatically as the dilatancy angle increases from 5° to 25°. The effect of the size of the influence zone, represented by the ratio \( R_i/R_0 \), is shown in Figs. 5 (a) and (b) for the dilatancy angle of 5° and 25°, respectively. In general, an increase of \( R_i/R_0 \) leads to an increase in the mobilized interface shear strength and a decrease in the interface stiffness. However, based on the cases studied, it has been noticed that the change of the interface stiff-
the size of the influence zone ($R_i/R_0$). However, the size of the influence zone has greater effects on the stiffness of the stress-displacement behavior. Theoretically, $R_i/R_0 = 1$ implies a perfectly rigid relative displacement at the interface (or lack of the influence zone); the interface stress versus interface displacement is a vertical line along the $y$-axis in Fig. 6. From the above discussions, it is apparent that any design referring only to the maximum anchor pullout capacity, $\tau_{\text{max}}$, without taking into account the load transfer curve is incomplete, especially when the working load and the related deflection are concerned. Furthermore, when both the dilatancy angle, $\psi$, and the size of the influence zone, $R_i/R_0$, are accounted for, as shown in Fig. 7, the shear stress-displacement relationships are quite different, even though the maximum interface strength is forced to be the same (refer to Fig. 7).

**ANCHOR PULLOUT PERFORMANCE BY FORWARD CALCULATION**

Based on the above anchor-soil interface model, a numerical iteration procedure is devised to analyze and predict anchor performance. The iteration procedure is as follows: (i) divide the anchor bond length into segments and assume that the anchor-soil interface of each segment is represented by the middle point, (ii) calculate the force and the corresponding displacement of each segment, ensuring that force equilibrium and displacement compatibility are satisfied, (iii) if the calculated anchor head force matches the given externally applied load, go to next load stage; otherwise, adjust the initial condition (anchor tip displacement), and repeat step (ii).

The conditions for the case studied are as follows:
- Anchor parameters: $L = 588.8 \text{ cm}$, $D = 2.54 \text{ cm}$
- Soil parameters: $k = 200$, $n = 0.8$, $R_i = 0.7$, $\psi = 0.25$
- Interface parameters: $\tau_0 = 27580 \text{ Pa}$, $\sigma_0 = 58690 \text{ Pa}$,
\( \psi = 10, \ R/R_0 = 50, \) and

For the softening model: \( \tau_{es}/\tau_{max} = 0.8, \ m = 2.76 \text{ kPa/cm}. \)

The results calculated for the hardening interface (two-stage) model are shown in Figs. 8(a), (b), and (c). As shown in Fig. 8(c), the maximum pullout capacity of the anchor will be developed when the entire bond length of the anchor is at the state of yielding. At this state, the relationship between the applied anchor head force and the anchor head displacement becomes horizontal, as shown in Fig. 8(a). Before reaching this state, the mobilized anchor-soil interface stress and the anchor tension are distributed along the anchor length as shown in Figs. 8(b) and 8(c), respectively. In Fig 8(a), the curve of the pullout force versus anchor head displacement can be broken into a linear portion and a nonlinear portion. The breaking point corresponds to a pullout force around 580 kN.

The calculated results for the softening model (three stage model) are plotted in Figs. 9(a)–(c). Due to the drop of the shear strength after reaching the peak value, the maximum pullout capacity of the anchor depends on a number of factors, such as the ratio of residual strength over the maximum strength, \( \tau_{es}/\tau_{max} \), anchor bond length, \( L_b \), and the slope of interface strength drop, \( m \).

The pattern of the load transfer (the mobilization of the interface stress) has been found to depend greatly on the relative rigidity, \( \alpha \), defined previously. The larger the relative rigidity factor, the smaller the relative elongation of the anchor in the system. Thus, an increase of the relative rigidity factor leads to a more linear pattern of load transfer along the anchor, as illustrated in Fig. 10. When the relative rigidity of the anchor is high enough, uniformly distributed interface stress will be obtained. Further computations showed that when the relative rigidity of the anchor is greater than 1/10, the anchor elongation is negligible; which implies that the anchor can be simply treated as a rigid bar.

**DETERMINATION OF INTERFACE MODEL PARAMETERS BY BACK CALCULATION**

The above section has shown that the provided anchor-soil interface model can predict the anchor performance, provided that the representative anchor-soil interface model parameters are given. The determination of the representative interface model parameters from the traditional soil testing techniques is difficult, due to the fact that numerous factors tend to affect the interface behavior. These factors include the following: (i) a wide range of soil types, (ii) different construction procedures, and (iii) the existence of variety tests and interpretation methods. Furthermore, as elucidated in the model, the main factors affecting the anchor-soil interface behavior also include the dilatancy angle and the interface influence zone size. These model parameters, however, can be extracted from the anchor pullout test results via a back calculation procedure. The advantages of the back calculation procedure are: (i) the parameters obtained are more representative since they are obtained directly from the field test results, (ii) once the interface parameters are
obtained, the performance of the anchor under different design parameters (e. g., anchor length, bond length, etc.) can be satisfactorily evaluated, (iii) in practice, once the anchor performance with different parameters are known, the number of tests in the field can be reduced.

The back-calculation algorithm to determine the anchor-soil interface parameters using the anchor pullout test results is described below. The first step is to calculate the maximum mobilized interface strength through the following equation

$$\tau_{\text{max}} = \frac{P_{\text{sh}}}{\pi D L_b}$$  \hspace{1cm} (22)

Where $P_{\text{sh}}$ is the peak pullout load and $L_b$ is the anchor bond length.

Once the maximum interface strength is obtained, the relationship between the dilatancy angle and the influence zone ratio is computed. A representative set of results from such an interrelationship is shown in Fig. 11. The value of basic interface strength ($\tau_0$), indicated in Fig. 11, is referred to as the friction on the interface between the soil and anchor, expressed as:

$$\tau_0 = \mu \sigma_0$$  \hspace{1cm} (23)

Where ($\mu$) is the interface basic friction factor, and ($\sigma_0$) is the confining pressure acting on the anchor-soil interface.

For each possible combination of these two parameters, a forward calculation is used to calculate the corresponding anchor load-displacement behavior. The calculated results are compared with the actual load test measurements. The most suitable interface model parameters are determined from the best fit between the measured (actual) and calculated load-displacement curves.

For the softening (three stage) model, only the residual strength of the anchor-soil interface can be obtained from a complete anchor pullout test. In this case, the determination of the maximum interface strength cannot be obtained directly. Consequently, a iteration procedure is
applied, in which, the mobilized maximum interface strength to range from $\tau_{\text{res}}$ to $2\tau_{\text{max}}$, where $\tau_{\text{res}}$, $\tau_{\text{max}}$ are the residual interface strength and nominal maximum strength calculated as follows, respectively,

$$
\tau_{\text{res}} = \frac{P_{\text{res}}}{\pi D L_b}
$$

$$
\tau_{\text{max}} = \frac{P_{\text{max}}}{\pi D L_b}
$$

where:
- $P_{\text{max}}$ = maximum applied load
- $P_{\text{res}}$ = residual test load
- $D$ = anchor diameter
- $L_b$ = anchor bond length

Two hypothetical problems using the hardening model and softening model, respectively, are presented to demonstrate the back calculation algorithm. The conditions for the soil and anchor for these two cases:

- Anchor parameters: $L = L_b = 609.6$ cm, $D = 2.54$ cm
- Soil parameters: $k = 150$, $n = 0.9$, $R_f = 0.7$, $v = 0.25$
- Interface parameters: $\tau_0 = 27.6$ kPa, $\sigma_0 = 68.9$ kPa

Based on the above parameters, the interface parameters, dilatancy, $\psi$, and influence zone size, $R_f/R_0$ were determined via using the back calculation procedure. The corresponding interface behavior for these two models are shown in Fig. 12(a) and Fig. 13(a), respectively. Using these calculated parameters, the anchor pull-out force-displacement were obtained from the forward calculation technique. The calculated results are compared with test results in Figs. 12(b) and 13(b) for the hardening and softening model, respectively. It can be seen that the interface parameters determined by the developed back calculation algorithm will yield anchor pull-out performance close to the given test results for both hardening and softening model. It is noticed that only two interface parameters are used in the back calculation: dilatancy angle and influence radius ratio. The basic interface strength $\tau_0$ is considered to be fairly constant. Whereas, the overburden pressure or the grouting pressure normally determines the confining pressure ($\sigma_0$) acting on the anchor-soil interface. On the other hand, previous works reported that the magnitude of the $\mu$ might ranges from 0.3 to more than 1.0 (Vitton, 1991; Little John, 1991). However, the value of $\mu$ has been suggested to be in the range (0.4 to 0.8). Therefore, in the following sensitivity analysis, the $\mu$ was varied from 0.3 to 1.1, while the other parameters were constant as follows:

- (i) Anchor parameters $D = 2.54$ cm, $L = 609.6$ cm, $E_A = 2.0 \times 10^5$ MPa;
- (ii) Interface parameters $R_f/R_0 = 80$, $\psi = 10^\circ$,
- (iii) Hyperbolic model parameters for soil $k = 200$, $n = 0.8$, $R_f = 0.95$.

For simplicity, only the hardening interface model is used in this sensitivity analysis. The calculated results are shown in Figs. 14(a), and 14(b) for the interface shear stress-displacement, and anchor head force-displacement, respectively. In both cases, the change does not
seem to greatly influence the calculated results. For practical purposes, it is reasonable to choose the value of basic friction factor between 0.5 and 0.9.

**COMPARISON BETWEEN PREDICTIONS AND LAB TEST RESULTS**

In this section, the laboratory chamber test results obtained by Vitton (1991) are used to evaluate further the capability of the back-calculation and forward prediction algorithm of the developed interface models.

The results of a group of a 1.27 cm diameter rebars with 91.44 cm embedment under the confining pressure with 34.5 kPa, 68.9 kPa, 103.0 kPa are used in this comparison study. The soil used in the test is 20–30 loose Ottawa sand, with $k = 100$, $n = 1.2$, $R_f = 0.7$, and $v = 0.25$.

First, the results of the pullout test under the confining pressure of 34.5 kPa are used to determine the anchor-soil interface parameters via the back calculation algorithm. Next, using these interface parameters as input, the anchor performance is predicted for the cases where the confining pressures are 68.9 kPa and 103.0 kPa. The comparisons between the predictions and the test results are shown in Figs. 15(a) and 15(b), for 68.9 kPa and 103.0 kPa confining pressure, respectively. It can be seen that the overall agreement between the predictions and test measurements is good, especially when the applied anchor load is small. On the other hand, the difference becomes more pronounced with an increase of the applied load. At the final state, the predicted values overestimated the test results; whereas, for a relatively small displacement the predicted values compared well to the measured. The main reason for the discrepancies may be the difference of treatment of the boundary conditions. In the tests, the boundary condition is basically a flexible one with a constant confining pressure. As a result, the effect of increasing confining pressure during pullout will not occur in that test chamber. However, in the field condition and in the anchor-soil model, the rigid displacement boundary condition allows for the confining pressure to change due to the shear action, which will enhance.

![Fig. 14. Sensitivity study of the basic interface strength (hardening model)](image)

![Fig. 15. Comparison between prediction by the present model and anchor tests: (a) test under confining pressure of 10 psi (69.0 Kpa) (Vitton, 1991), (b) test under confining pressure of 15 psi (103 Kpa) (Vitton, 1991)](image)
the interface strength and interface rigidity as well. Therefore, the higher capacity and smaller displacement predicted by the model are to be expected. Meanwhile, the comparisons indicate that the effect of boundary condition becomes more pronounced when the initial confining pressure is higher.

FIELD CASE STUDY

To demonstrate the application of the proposed algorithm in the actual field work, an experimental work done by Ludwig (1984), more specifically Tieback 622-1 is adopted as a field case study.

Tieback 622-1 was a straight-shafted ground anchor installed at Charlotte, North Carolina. Soil conditions at the site are summarized in Table 1. The tieback is 18.3 m long, 32 mm in diameter threaded bar with an ultimate strength of 835 KN, and grouted inside a 65 mm corrugated plastic tube. The bonded length of tieback was covered with a smooth plastic tube. The installation of the tieback was done using a 305 mm diameter, continuous-flight, and hollow stem auger. The drill hole was grouted to the surface. Grout pressure generally did not exceed 1035 kPa.

Instrumentation used in the study included the Hitec HBW-35-250-6-3VR weldable, electrical resistance strain gages. Both zero shift and drift were taken into account by Ludwig (1984) when reducing the data. In addition to strain gages, extensometers were used to provide the measurement of the performance of the grout column above the anchor of the straight-shafted tiebacks.

In facilitating the computations of the algorithm, a computer-based program has been developed by Liang and Feng 1997. An interactive feature of this computer program allows the user to input the required parameters easily. In analyzing the current case, the back-calculation method was used to obtain the load-transfer (t-z) curves along the shaft-soil interface. The following steps were followed.

Step1: Select the interface model type (hardening or softening model) and calculation method (back or forward). In this case, the hardening model with backward calculation scheme was selected.

Step2: Input the required parameters. The parameters for the present case are as follows:

- Anchor parameters: Length = 1498.6 cm, Diameter = 3.49 cm, drillhole diameter = 30.48 cm.
- Soil parameters: hyperbolic soil parameters $k = 100$, $n = 0.9$, $R_f = 0.8$, Poisson ratio = 0.2.
- Interface parameters: soil-grout interface basic strength = 8.96 kPa, confining pressure = 206.85 kPa, tieback modulus = 2.0 $\times 10^7$ MPa.

The user of this program needs to specify the number of segments to be used to divide the bonded length in the numerical calculations. Typically, 20 to 30 segments are quite satisfactory.

Step3: Input load-displacement curve measured at the anchor head from the pullout tests. Table 2 summarizes the actual load-displacement data for Tieback 622-1. It should be noted that the time-dependent displacement was not considered in the present study.

The back-calculation generates the interface load transfer (t-z curves) using the above input information. The output of the current analysis is shown and compared to measured curve in Fig. 16.

Once the back-calculated load transfer curve is obtained, then the user can predict the load-displacement curve at the anchor head and along the anchor via the forward calculation option provided in the developed program. Figure 17 compares the predicted and measured load-displacement response of Tieback 622-1. Whereas, Fig. 18 shows the distribution of the load transfer along the anchor length under different loads. The proposed calculation algorithm seems to be able to capture the load transfer behavior quite good.

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<tr>
<th>Table 1. Soil data for Tieback 622-1 (Ludwig, 1984)</th>
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<td><strong>Soil properties</strong></td>
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<td><strong>Properties</strong></td>
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<td>Degree of saturation</td>
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<th>Table 2. Load-displacement measurements for Tieback 622-1 (Ludwig, 1984)</th>
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<td><strong>Applied load at anchor head (KN)</strong></td>
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Fig. 16. Comparison of the interface model between field test by Ludwig (1984) and analysis

Fig. 17. Comparison of the pullout behavior between field test by Ludwig (1984) and analysis

Fig. 18. Comparison of the interface shear force distributions between field test by Ludwig (1984) and analysis

DISCUSSION

The field case study and the previous comparisons with laboratory pullout tests were geared towards uniform soil deposits, in which the load transfer curve ($t-z$) can be considered to be the same for the entire anchor length. In many actual cases, however, a layered soil deposit is likely to be encountered. The proposed algorithm is theoretically applicable to multi-layered soil that it allows for more than one load transfer curve to be determined via the backward calculation approach in the developed program. One of the drawbacks, though, is that the more $t-z$ curves specified in the program, the more difficult it is for the program to attain a convergent solution.

The emphasis of the current modeling representation was placed on the interface of the straight shaft anchor, with a typically small grouting pressure. The failure mode associated with this type of anchor during the pullout test will be along the shaft-soil interface. Other types of failure modes, such as cone-shaped or wedge are possible in other anchor types. The failure modes are not considered in the current model.

CONCLUSIONS

The following conclusions can be drawn from this paper.

1. The mobilized interface shear strength is strongly dependent upon the soil dilatancy angle and the influence zone. In general, an increase of either of them will lead to an increase of the mobilized shear strength. The dilatancy angle nevertheless exerts more significant effect.

2. The rigidity of the anchor-soil interface increases drastically with an increase of the dilatancy angle, but decreases with an increase of the influence zone size. However, the latter factor is negligible for the dilatancy angle from $5^\circ$ to $25^\circ$.

3. The distribution of the mobilized anchor force along the anchor bonded length is strongly dependent upon the relative rigidity of the anchor. For the case where the relative rigidity factor is larger than 1/10, the distribution is linear.

4. By using the back calculation technique, the important interface model parameters can be determined accurately, which can then be used to predict the performance of the anchors with different design lengths.
REFERENCES


