Analysis of Drilled Piers Used for Slope Stabilization

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This paper presents a technique to evaluate the effects of drilled piers on the stability of slopes. The model allows for construction sequences, interaction between pier displacements and soil movements, remolded areas around the piers, and weaker seams (if present) in the soil profile. Applications of the technique are presented, with emphasis on identifying and optimizing the factors that control the performance of piers used for slope stabilization. Three applications of the use of drilled piers for slope stabilization were investigated: surcharge loading, excavation from a horizontal ground surface, and cut slope stabilization. Several conclusions regarding slope-pier interaction can be drawn. Of prime importance is that the piers must be positioned at a point where relatively large displacements are expected to occur; the magnitude of these displacements determines how much stress will be mobilized against the pier. The balance between cohesive strength mobilized and frictional strength mobilized is important to the performance of the stabilizing piers. Where upward movement is a prime component, the support offered by the piers becomes very indirect, providing added support through retention of confinement. The entire soil-structure interaction that occurs between the piers and soil mass must be considered when evaluating drilled piers for slope stabilization—not simply the added shearing resistance provided by the piers. The redirection of stresses throughout the soil can be either beneficial or detrimental to the final stability. On the basis of the analyses performed to date, the best applications of the piers seem to be in purely cohesive materials under loading conditions that can use the vertical resistance of the piers.

During the past two decades, innovative soil-reinforcement techniques (such as reinforced earth, stone columns, soil anchors, piles, and cast-in-place piers) have been developed to solve many slope stability problems. These techniques use semirigid members that are capable of transferring loads through shear, tension, or compressional resistance from an unstable mass above the failure surface to more stable underlying layers. In Sweden, battered timber piles have been used to increase the stability of slopes in very soft clays by dowelling across potential failure surfaces. Inserting the piles at an angle allows them to act partially in compression rather than to rely totally on their resistance in shear. Steel pipe piles have been used for the same purpose in Japan (*1*). Broms and Wong (*2*) discussed similar applications for piles to support fills, bridge abutments, and deep excavations.

Systems of drilled-in minipiles (such as the reticulated root pile method, which consists of dense clusters of vertical and battered small-diameter piles) have been used in the United States. These systems take advantage of the soil strengthened by the piles to form a barrier that resists movement and transfers stresses downslope (*3*). Stone columns, although relatively new in the United States, have been proven to be quite effective in Europe (*4*, *5*). The relatively high shear strength of the columns increases the shearing resistance along the failure surface, whereas the high modulus of elasticity of the column transfers vertical loads from the surrounding moving soils to the more stable foundation layers. The vertical loads absorbed, in turn, increase the confining stress within the stone column, and thus its shearing resistance.

Large-diameter, cast-in-place, reinforced concrete piers have been used in Europe and the United States to stabilize active landslide areas in stiff clays and shales through dowel action (*6–12*). The diameters of these piers can be as large as 1.5 m (5 ft). In the United States, these piers have often been used side by side or slightly overlapped to form a continuous wall in highway and building foundations. Drilled piers can also be placed discretely (Figure 1) with clear space between the piers, which takes advantage of many of the strong points of

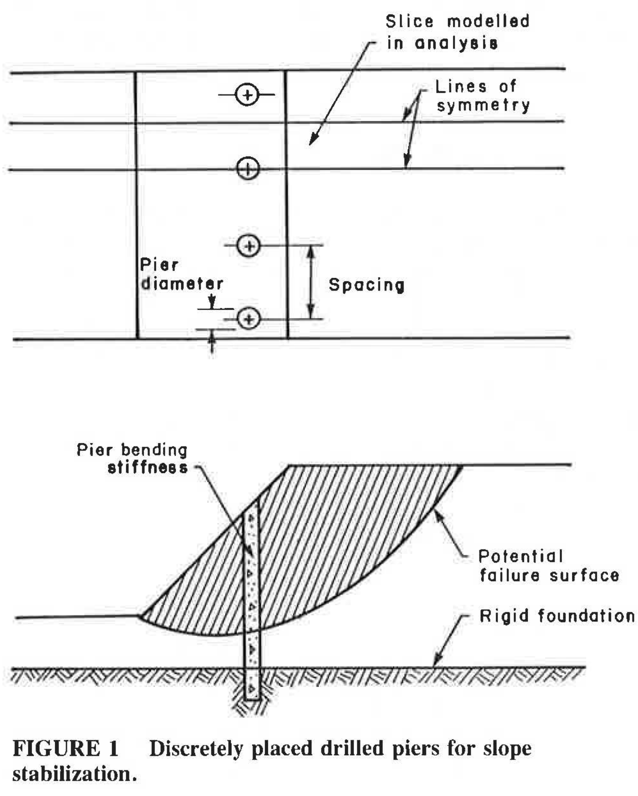

FIGURE 1 Discretely placed drilled piers for slope stabilization.

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previously described techniques. Drilled piers can be installed relatively quickly, at a moderate cost, and thus provide an ideal remedial measure to slow or halt the progress of failing slopes (*9*). Similar to drilled pier walls, the discretely placed piers add shearing resistance across potential failure surfaces. However, by proper positioning and spacing, the piers can also absorb vertical driving forces from the slope and transmit these forces directly to stable foundation layers below. This incorporates the design philosophy of transmitting the loads by compression rather than pure shear, as in the cases of stone columns and inclined timber piles. The discrete positioning, rather than continuous placement, provides an increased surface area that interacts with the soil and is more efficient in absorbing vertical forces. Soil arching develops between the piers, as is the case with the smaller root piles, and forms a continuous barrier that limits soil displacement between the piers.

Analysis techniques have been developed to predict lateral soil forces against piers used as reinforcement in slopes (*11–16*). The plastic deformation method developed by Ito and Matsui (*14*) to evaluate the lateral pressures acting on a row of passive piles has been incorporated in limiting equilibrium solutions for slope stability (*17*, *18*). Although these techniques are useful, especially for estimating the lateral loads on the drilled pier reinforcing system, they do not model the behavior of the slope itself and, most specifically, the changes in stress fields along potential failure surfaces. To improve the analysis of this problem, this paper presents a three-dimensional, finite element model of slopes stabilized by drilled piers that are socketed in bedrock. The technique provides displacement, strain, and stress fields within the reinforced slope. In addition, an analysis routine to estimate a limiting equilibrium factor of safety of the slope from the finite element-generated stresses has also been developed. The model and its numerical features are summarized first. Emphasis is then placed on applying the numerical method to relevant highway problems. Conclusions are drawn based on these applications and earlier examples (*19*).

PREVIOUS NUMERICAL ANALYSES

The problem of piers placed at discrete locations in a slope clearly involves three-dimensional effects; however, approximate two-dimensional analyses have proven useful. Rowe and Poulos (*20*) developed a two-dimensional model that makes allowances for the three-dimensional effect of soil flowing through the row of piers. This was accomplished by using separate solutions to analyze the soil movements and pier displacements, then comparing and adjusting their relative values. The study dealt primarily with the effect of the slope movement on the displacement and bending of the piers. A parametric study conducted for three rows of piers placed at the crest of a small slope led to the following conclusions (*20*):

1. Stability increased slowly with pier stiffness, and thus very rigid piers may have to be used to stabilize slopes.
2. Effectiveness was enhanced by restraining the pier tip. However, this restraint, combined with increased stiffness of the piers, greatly increased the bending moments in the piers.
3. Increasing soil stiffness and strength with depth had a positive effect on reducing the bending moments in the piers.

Oakland and Chameau (*19*) performed a simplified finite-element study of drilled piers (modeled as rectangular columns) used to stabilize a slope distressed by a surcharge loading at the crest. Following the recommendations made by Rowe and Poulos (*20*), the piers modeled in the study were of large diameter, very rigid, and firmly socketed into bedrock. The results were as follows:

1. To be most effective, piers should be located at the point of the expected maximum slope movement, as identified from an analysis without piers.
2. Both increasing the diameter and reducing the spacing of the piers had similar effects in reducing the surface displacements.
3. Increasing the stiffness of the pier generally decreased the displacements uniformly over the entire profile.

This simple model (rectangular piers, no slip elements, linear soil model, etc.) was used as a pilot study to establish the feasibility of drilled-pier reinforcing systems and led to the development of the technique discussed here.

PROPOSED MODELING TECHNIQUE

The finite-element program models the interaction between the slope and drilled piers used for stabilization by computing the resistance offered by passive, laterally loaded piers and applying that load to oppose soil movement. The principal features of the modeling procedure are summarized below, as well as the basic components of the associated computer program, SPILES. Additional details of the technique and program can be found elsewhere (*21*).

Finite Elements

Soil Elements

The basic soil element is a three-dimensional, linear strain eight-node isoparametric parallelepiped that is capable of adding a midside node to one or more of its sides, making it expandable to a quadratic strain 20-node isoparametric parallelepiped (Figure 2). The additional midside nodes are necessary to define the circular cross section of the piers and useful in subdividing the mesh in critical areas to improve numerical accuracy. The process of adding midside nodes is achieved by modifying and adding to the basic eight-node shape functions. Cook (*22*) described it for a two-dimensional case, and it is extended here to the three-dimensional case. Variable shape functions were developed to account for midside nodes (*21*). A second-order Gauss quadrature is used to integrate over the shape functions to determine the stiffness coefficients.

Pier Elements

The piers are represented by spar elements with four degrees of freedom, consisting of lateral translations and rotations at each end (Figure 3a). The three-dimensional behavior of the piers is developed by assigning the same node number to every

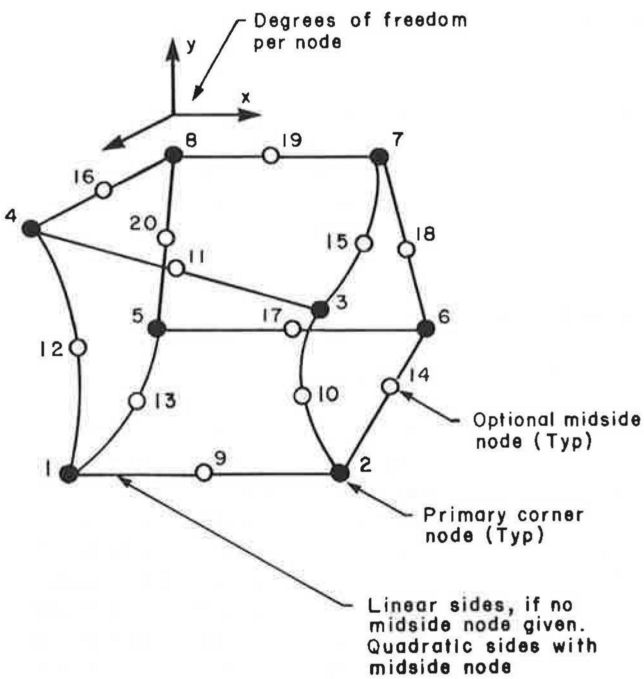

FIGURE 2 Three-dimensional, expandable, soil element.

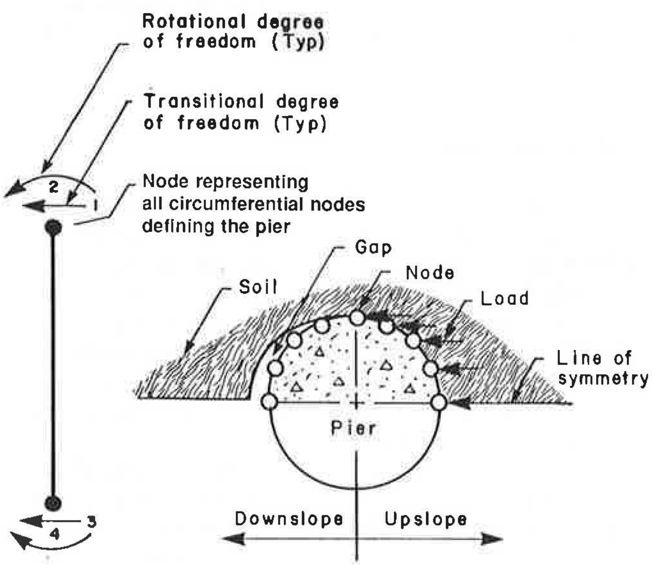

FIGURE 3 Pier elements: (*left*) pier spar element; (*right*) soil/pier nodal interaction.

soil node that defines the interface between pier and soil at a given elevation. The downslope side of the pier is not attached to the soil, which allows a gap to form as the soil separates from the piers (Figure 3b), thus preventing artificial tension forces from developing at the soil-pier interface. Bachus and Barksdale (*23*) noted such a gap when testing the lateral loading resistance of stone columns.

The spar elements chosen to represent the piers allow only for bending and translation of the piers in one direction; no deformation of the cross-sectional shape or length of the pier elements is allowed. The large difference in the elastic modulus between the soil and the concrete forming the piers makes this a realistic limitation that greatly simplifies the computational requirements and reduces the number of nodes required. In addition, the cubic bending characteristics of the spar elements provide a better representation of the pier deformations than could block elements. Furthermore, the aspect ratio of the elements (i.e., element length to width) is no longer a concern.

Slip Elements

Soil-pier interface or weak seams in the soil, or both, can be modeled by slip elements. The thin-layer elements used by Desai and Sargard (*24*) are easily incorporated in the finite element mesh. These thin elements are similar in construction to the soil elements, except that they are proportioned so that the ratio of the shortest to the longest side is between 0.01 and 0.1. They are assigned a small shearing modulus, whereas the modulus against normal displacement remains the same as that of the soil elements. These finite-thickness slip elements have several advantages over the traditional infinitely thin slip elements:

1. Because they have essentially the same characteristics as the soil elements, they are easily incorporated into the finite element solution.
2. They do not suffer from numerical difficulties sometimes associated with traditional slip elements.
3. They not only provide for slippage around the pier, but also provide a means of representing a thin layer of soil disturbed by drilling.

Soil Constitutive Relations

Three simple constitutive models are implemented in the program to model the stress-strain behavior of the soil: linear elastic, Duncan-Chang hyperbolic model with variable modulus of elasticity and constant Poisson's ratio (*25*), and the extended Duncan-Chang model with both variable modulus and Poisson's ratio. The linear elastic model is provided because of its simplicity and economy in making preliminary predictions. The Duncan-Chang nonlinear models, although simple, have been shown to work well under conditions of monotonic loading (*26*); the model parameters can be obtained from triaxial testing, and a wide data base currently exists in the literature. The nonlinear soil models are implemented in the computer program by an initial modulus incremental procedure (*27*). This procedure avoids problems of nonconvergence because the modulus is determined by a closed-form solution from the stresses existing in the elements and does not depend on iteration.

The initial states of stress of all the elements are required to calculate moduli when a nonlinear soil model is used. Furthermore, for stability calculations it is also necessary to evaluate the final states of stress rather than just stress differences. Although it would be preferable to model the entire formation process of a slope to determine the initial stresses, it does require significant effort and data (often not available) to do so. Although the computer program allows for the input of initial stresses directly (determined from an independent source) or for sequential computations to model a desired slope formation process, the most common procedure is to determine an approximate initial stress field that is generic to all slopes. Approximate initial stresses can be created in this program by building a weightless slope and then instantly "turning on gravity."

Soil/Solution Interaction

The SPILES program is composed of two major finite-element modules. The first module solves for the incremental displacements in the soil on the assumption that the piers are rigid. The second module solves for the pile displacements using the forces obtained at the soil nodes. Both modules are used in each increment of loading. The solution process of each module follows the displacement-based finite-element technique using constrained and unconstrained degrees of freedom. The constrained degrees of freedom representing the pier-soil interface can be temporarily fixed during the solution of the first module, which maintains a specified displacement while the soil stiffness matrix is computed. The two-part solution process of the soil displacements (first module) is as follows:

$$r_u = sk_{uu}\, d_u + sk_{uc}\, d_c \qquad (1a)$$

and, for the forces on the pier to be used in the second module,

$$r_c = sk_{cu}\, d_u + sk_{cc}\, d_c \qquad (1b)$$

where

r_u = forces at unconstrained degrees of freedom (i.e., forces on the soil mass),
r_c = forces at constrained degrees of freedom (i.e., forces on the piers),
d_u = displacements at unconstrained degrees of freedom (i.e., soil movements),
d_c = displacements at constrained degrees of freedom (i.e., pier movements),
sk_{uu} = totally unconstrained stiffness matrix (i.e., equations relating soil degrees of freedom),
$sk_{uc} = sk_{cu}^{-1}$ = partially constrained stiffness matrix (i.e., equations relating soil to pier degrees of freedom), and
sk_{cc} = totally constrained stiffness matrix (i.e., equations relating pier degrees of freedom).

Because the initial pier displacements (the constrained displacements) are known at the beginning of each increment, the first set of equations (1a) can be solved for the unconstrained displacements. The unconstrained displacements are the displacements throughout the soil mass in the particular increment. Once the unconstrained displacements are known, the forces against the pier (the constrained forces) can be calculated from the second set of equations (1b). The constrained (pier) forces are then utilized to compute the new pier displacements in the second module described below.

The forces against the piers determined in the first module are resisted in two ways. First, forces are transferred to the bedrock through cantilever resistance of the piers, and, second, the soil in front (downslope) of the piers resists as they bend. Both these types of resistance are a function of the pier displacements, and both increase as the piers bend. The equation to be solved in the second module is

$$r_c = (a + sk_{cc})d_c \qquad (2)$$

where r, d, sk have been defined above, and a is the pier stiffness matrix. The pier displacements, once computed at constrained degrees of freedom, can be incorporated in the calculation of the new soil element stresses and soil displacements during the next increment.

TYPICAL ANALYSIS

As noted earlier, a simplified version of the SPILES program was used previously to examine the problem of drilled piers used to stabilize a slope subjected to surcharge loading (*17*). The effects of pier position, size, spacing, and stiffness were related to slope movement. That study concluded that with proper positioning, the drilled piers could have a positive effect on slope movement. In order to evaluate further the capability of drilled piers to provide a permanent method of increasing stability and controlling displacement, the example of an excavation is studied first in this paper; it represents the condition of a highway embankment. Although (as will be discussed) this application does not use the piers' capabilities at their best or provide the optimal means of support, it is the best example to clearly illustrate the support mechanism provided by the piers and the characteristics of pier-soil interaction.

Problem Description

The problem considered is similar to the one addressed by Duncan and Dunlop (*28*): a 12.2-m (40-ft) excavation in 18.2 m (60 ft) of soft clay using 1:1.5 side slopes. Movements, stresses, and stability are analyzed without piers first and then with drilled piers installed at the crest of the excavated zone. The bedrock is assumed to be very stiff with respect to the soil and acts as a rigid boundary in which the piers can be socketed. It is located 6.1 m (20 ft) below the final excavation depth, one-half of the slope height.

The finite element mesh used to simulate this problem is shown in Figure 4. The wedge of soil shown slightly elevated in Figure 4 represents the portion of the mesh to be excavated. Excavation is to be conducted in eight increments, each consisting of a horizontal row of elements. To simulate excavation, the average stress in each element excavated is extrapolated to the excavation boundary. The forces on this boundary are computed and applied as upward forces on the new surface boundary. A nonlinear hyperbolic model is used to determine the modulus of elasticity in each soil element during each increment. The parameters of the soil model are given in Table 1. They are representative of a normally consolidated clay with strain hardening stress-strain curve and an undrained shear strength of 38.3 kPa (800 psf). The soil strength is such that the unreinforced slope has an unacceptable factor of safety after completion of the final increment of excavation—the two-dimensional factor of safety (Bishop's method) is 0.95 at the end of the excavation. One row of 1.22-m (4-ft) diameter piers with 2.44-m (8-ft) center-to-center spacings is to be used to reinforce the excavation.

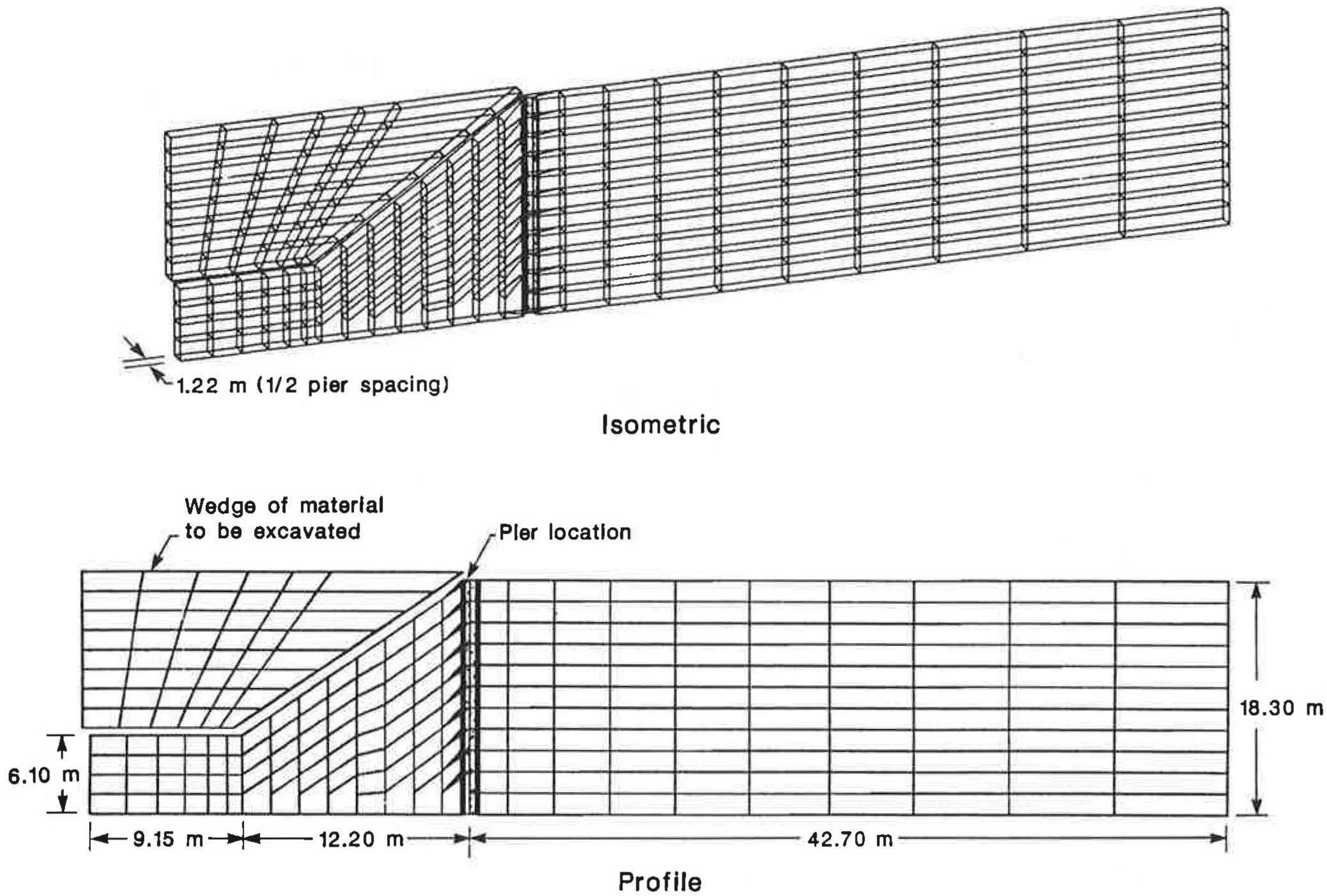

FIGURE 4 Finite element mesh for excavation problem.

TABLE 1 SOIL PARAMETERS FOR EXCAVATION PROBLEM

Type	Parameter	Value
Duncan-Chang for variable modulus of elasticity	Contant, K	47.2
	Constant, n	0.5
	Constant, R_f	1.0
Soil properties	Unit weight, γm	2.1 g/cm^3 (130 pcf)
	Cohesion, C	3900 kg/m^2 (800 psf)
	Friction angle, ϕ	0

Results of Finite-Element Study

Displacements

The nodal displacements for each stage of the excavation were the primary results of this analysis. Although the displacements are not directly applicable to stability analysis, they can give a relative assessment of the stability, and they are useful if the displacement or deformation of nearby existing structures is critical. Plots of the displacement fields can also give insight into the failure mechanism.

The effectiveness of the piers was illustrated by comparing the displacement fields of the two cases studied (i.e., without and with piers) at selected excavation stages and particularly after the last stage (Figure 5). The displacement plots clearly show how the piers act as a barrier, almost eliminating the horizontal component of displacements behind the row of piers. This barrier also significantly reduces basal heave represented by vertical (upward) displacement vectors in front of the piers. In fact, in the case of the reinforced slope, the vertical (upward) displacement represents essentially the rebounding of the unloaded soil rather than horizontal translation or sloughing, as occurs in the unreinforced slopes.

In addition to analyzing the final configurations of displacement, it is also useful to monitor the progression of displacements during the excavation. For example, in the case of the unreinforced slope, the point of maximum displacement on the slope surface moves downward as the excavation progresses and is approximately level with the bottom of the excavation for each stage. The magnitude of these maximum displacements does not grow proportionally with the excavation, but rather increases very rapidly during the last increments. The deepening of the zone of influence of the movement is also apparent as the displacement vectors at each point above the crest continually bend downward with each increment.

The most dramatic change in the displacement pattern for the unreinforced case occurred during the sixth increment, in which the surface displacement vector at the level of the excavation bottom (vector A in Figure 5) almost doubled in length,

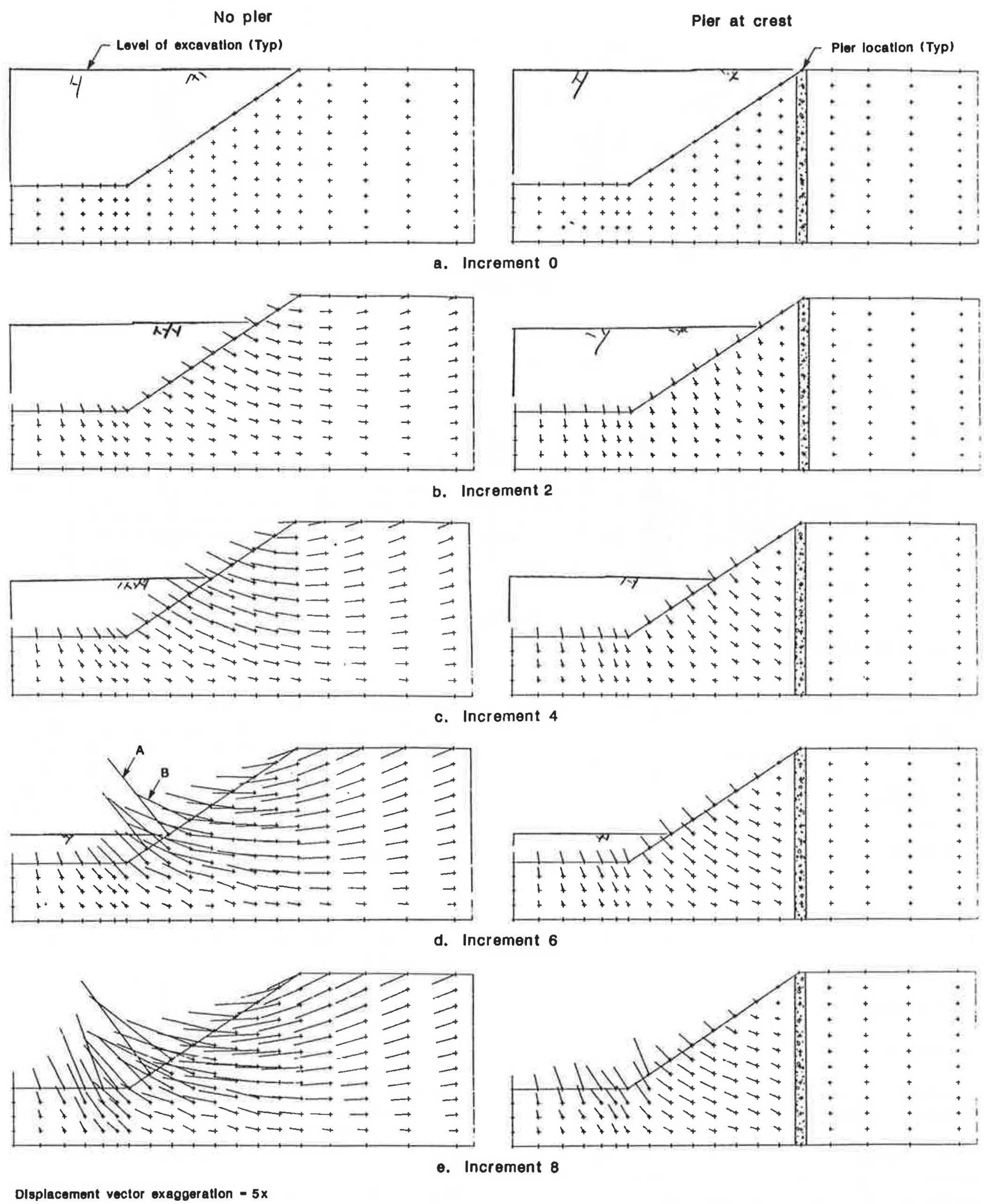

FIGURE 5 Soil displacements for selected stages of excavation.

primarily in an upward direction, whereas the surface nodal displacement vector just above the bottom (vector *B* in Figure 5) turned sharply downward. In contrast, the progression of the magnitude of the displacement for the case with pier reinforcement did not change significantly during excavation (i.e., the magnitudes increased almost proportionally with the number of excavation increments). No significant alteration in the displacement pattern could be readily identified and thus a state of sloughing did not occur.

Stresses

Element stresses were calculated from the nodal displacements on an elemental basis at each of the eight Gauss points and then averaged over the element. The Gauss point values or the average values, or both, can be included in the output.

Unlike the situation of surcharge loading (*19*), where increases in the vertical stress state (i.e., the direct cause of instability) could easily be absorbed by skin friction in the piers, this excavation example did not take full advantage of the vertical support that the piers could provide. For this reason, the vertical stress fields (not shown) were not significantly reduced by pier reinforcement. Similarly, the piers did not have a large effect on the horizontal stresses. The shearing stresses at the completion of excavation, however, were significantly higher for the case of the unreinforced slope, as shown in Figure 6.

The development of the shearing stresses was an important factor in the evaluation of the slope stability. The initial shearing stresses on horizontal planes (null for the half-space) were largely preserved (i.e., remained zero) behind the pier row in the reinforced case (Figure 6b). Under the slope itself (below the pier row), within the region where a shallow failure surface was likely to develop, the shearing stresses were also reduced when compared with the unreinforced case, especially in the vicinity of the toe where failure was likely to initiate. In addition, there was a reduction in the shearing stresses along the interface of the soil and foundation material.

Nodal Loads on the Piers

Through the equations of equilibrium, the elemental nodal loads could be calculated from the elemental stresses. A summation of all of the elemental nodal loads yielded the global nodal load field, and thus a summation of the nodal loads on elements with constrained degrees of freedom gives the forces against the piers. The nodal loads against the piers at the end of excavation are shown in Figure 7. These loads can be used in the structural analysis of the pier.

LIMITING EQUILIBRIUM ANALYSIS

In addition to providing data on slope movement and stresses, an assessment of the overall stability (i.e., factor of safety) of the slope was made using a two-dimensional limiting equilibrium analysis. The stress output from the finite-element analysis was used to make limiting equilibrium calculations (*21*) with a factor of safety defined as the sum of the resisting stresses (strength) over the sum of the shearing stresses along a discretized circular or log spiral failure surface. A grid-pattern-searching technique (two-dimensional mesh) was used to locate the most likely failure surface.

The most likely circular failure surfaces for each increment of the excavation problem are shown in Figures 8a and 8b for

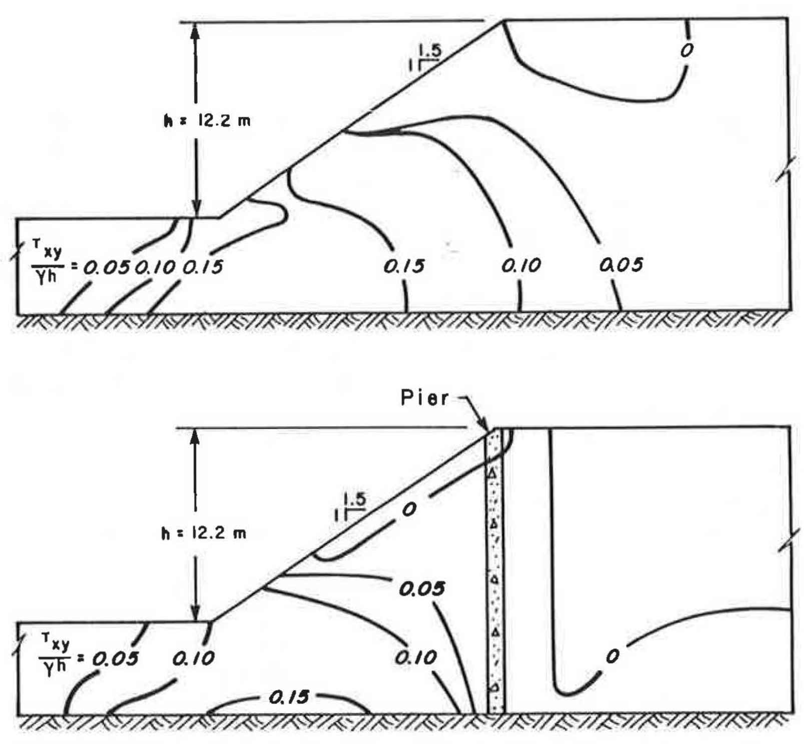

FIGURE 6 Normalized shearing stresses: (*top*) without pier reinforcement; (*bottom*) pier reinforcement at crest.

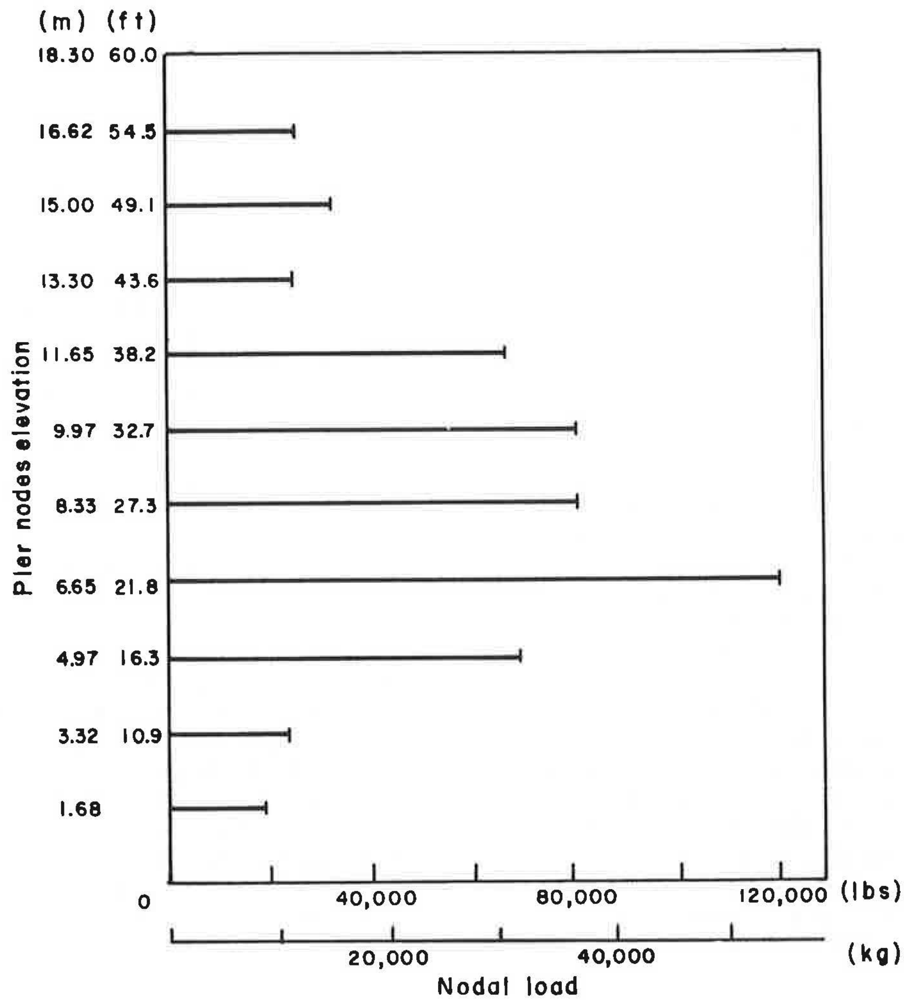

FIGURE 7 Nodal loads against pier.

the cases without and with a pier, respectively. The piers tended to reduce the volume of material involved in a potential slide. This reduction increased with excavation depth, as indicated by the location of the centers of the failure surfaces. In the case of a slope without a pier, these centers moved up, whereas they moved down for the case of a slope reinforced by piers, thus shortening the arc radius. A plot of the factor of safety with respect to the elevation of the bottom of the excavation is given in Figure 9. Note that the factor of safety for the case with a pier was less (although very high) after the first increment than that without a pier. This was due to the disturbed layer modeled around each pier, which provided a weak seam along the failure surface.

The pier reinforcement improved the stability from a factor of safety of 1.0 without reinforcement to approximately 1.7 at the end of excavation. Although this increase was significant, it was not as dramatic as had been seen in other examples (*21*). A review of the soil movements and stresses showed this case not to have used the piers to their fullest capacity. The direction of soil movement at the level of the pier was primarily horizontal, and thus the piers did not provide significant vertical support of the soil, which would have utilized the axial capacity of the piers. Under conditions of downward forces acting on the slope, such as for surcharge loading, this component was of great benefit. In conclusion, although the piers significantly increased the factor of safety of a slope during excavation, their greatest benefit under this loading condition was to control displacements and prevent bottom heave. Better reinforcement of the slope might have been achieved by positioning the piers lower on the slope. At lower positions, the piers could have absorbed additional stresses from the larger movements. However, the loads in the piers would become primarily horizontal and the structural demand on the piers would significantly increase unless tiebacks were used (*21*).

CUT SLOPE STABILITY

This example represents a practical application of the pier system that can be used to explore further the mechanics of drilled-pier reinforcement. The problem considered is a 1:2 slope with a height of 9.1 m (30 ft) and a depth to bedrock under the toe of 3.7 m (12 ft). Instability would initiate, as the slope is to be cut to a 1:1 slope for widening purposes. Reinforcement, consisting of 1.2-m (4-ft) diameter piers with

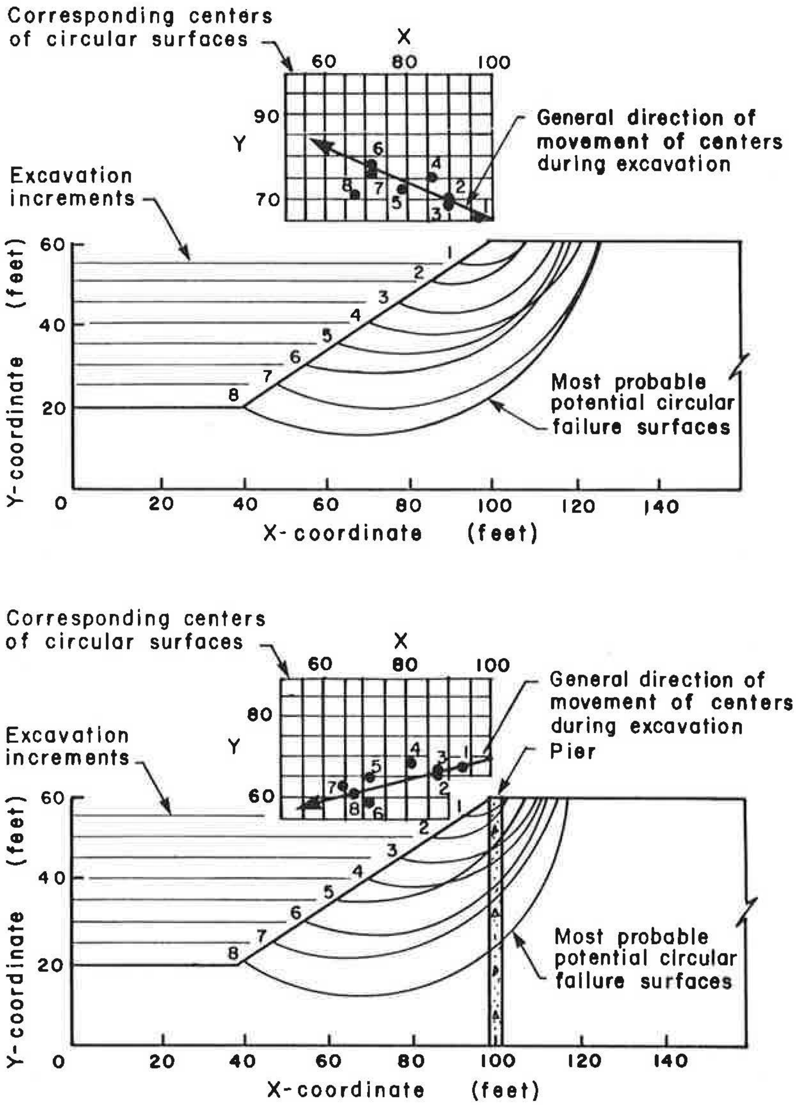

FIGURE 8 Most likely failure surfaces: (*top*) without pier reinforcement; (*bottom*) pier reinforcement at crest.

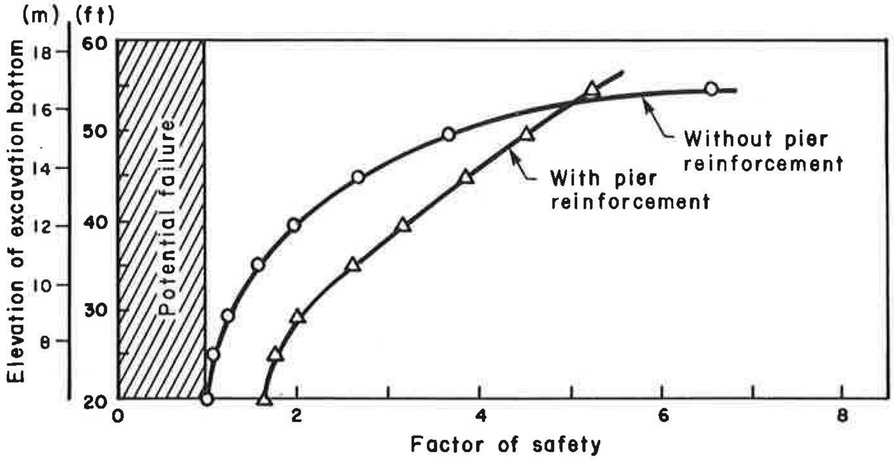

FIGURE 9 Factors of safety during excavation.

2.4-m (8-ft) center-to-center spacing, is implemented to control movements and provide stability. The piers are installed below the existing ground surface, 3.7 m (12 ft) from the toe. The shafts are drilled as usual; however, the piers are constructed only up to the future ground surface. The remainder of the shafts are backfilled with soil. Excavation of the slope then proceeds unhindered to the level of the top of the pier.

A soil with a frictional component of strength and a variable Poisson's ratio (i.e., extended Duncan-Chang model) is used. For the basic example, the soil has a cohesion of 200 psf (980 kg/m^2) and a friction angle of 36 degrees. The other parameters are given in Table 2. The initial stresses are established for the original 1:2 slope through the gravity turn-on procedure. These stresses, once established, are transferred to corresponding elements in the final 1:1 geometry, to which stresses created by surface loads representing the excavation operation are added. The surface nodal loads that simulate the excavation process are determined from the average elemental initial stresses and are applied in increments.

For excavations made in soils with a frictional component of strength, in addition to the piers reducing the shear stresses imposed by the steeper geometry, improvements to stability are also dependent on retention of the confinement initially present in the slope. The degree to which retention of confinement influences the final stability is a function of the degree to which the stability without piers is dependent on the frictional capacity of the soil. This mechanism can be demonstrated in this example by monitoring the elements with reduced modulus (i.e., failed elements in accordance with the Mohr-Coulomb criteria) for a variety of cohesion and friction values.

TABLE 2 SOIL PARAMETERS FOR CUT SLOPE PROBLEM

Type	Parameter	Value
Duncan-Chang for variable modulus of elasticity	Constant, K	47.2
	Constant, n	0.5
	Constant, R_f	0.8
Duncan-Chang for variable Poisson's ratio	Constant, G	0.33
	Constant, F	0.06
	Constant, d	4.0
Soil properties	Unit weight, γm	2.1 g/cm^2 (130 pcf)
	Cohesion, C	980 kg/m^2 (200 psf)
	Friction angle, ϕ	36°

For the original soil model (strength parameters given in Table 2), no element failed under the final, excavated, loading condition. If, however, the same excavation was made in a significantly weaker soil (values of friction angle reduced to 18 and 9 degrees), a bulb of failed elements did form as shown in Figure 10 for both the unreinforced slope and the slope with a row of piers at 3.7 m (12 ft) from the toe. As the friction angle was decreased to 18 degrees, failure initiated from the toe and at the foundation level below the toe. (These were the locations where the principal stress differences were highest.) As the friction angle of the soil was further reduced to 9 degrees and the stability became more dependent on the cohesive strength, additional failure spread from the foundation level, near the toe, where the deviator stresses were high before excavation, into areas where confinement prevented failure in soils with higher frictional capacity.

In this example, a significant amount of shear stress was present in the slope before excavation. Much of the additional shear stress was due to unloading and resulting increases in principal stress differences, which was not effectively absorbed by the piers. For this reason, as confinement became less important to stability (i.e., lower frictional strength with regard to cohesive strength), the failure bulbs for both the unreinforced slope and pier-supported slope became similar. Hence, in this example, the beneficial effects of the piers were fewer because the soil strength became dominated by its cohesive component.

Conversely, as the stability of the slope became more dependent on the frictional strength (i.e., the cohesion was reduced), the failure bulb grew along the slope surface and crest, where the confining pressures were small. The deeper soils at the foundation level, which failed with decreasing frictional capacity, did not fail under decreasing cohesion because the remaining overburden provided enough confinement not to require a substantial mobilization of cohesion. The effect of a decreasing cohesion with a constant friction angle is shown in Figure 11. As described above, the piers were ineffective in absorbing the shear stresses created in this

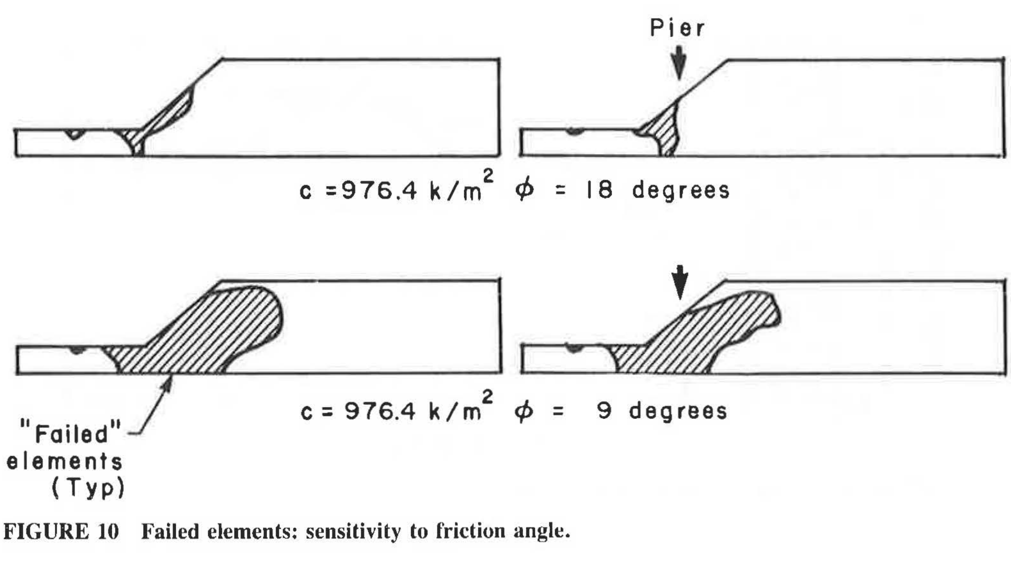

FIGURE 10 Failed elements: sensitivity to friction angle.

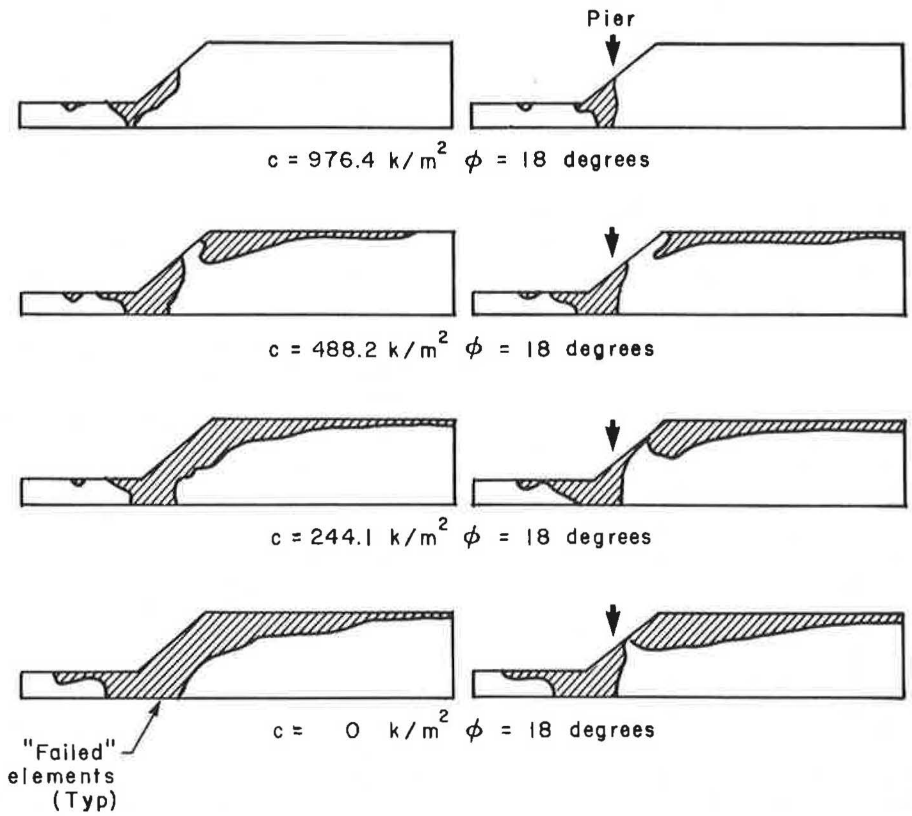

FIGURE 11 Failed elements: sensitivity to cohesion.

unloading situation. However, the piers did retard the loss of confinement through their skin friction. Thus, as reliance on the frictional component of the soil became more important with decreasing cohesion, the difference in the size of the failure bulb between the cases of the unreinforced slope and slope with piers became greater.

CONCLUSIONS

Three applications of the use of drilled piers for slope stabilization were investigated: surcharge loading (*19*), excavation from a horizontal ground surface, and cut slope stabilization. Several conclusions regarding slope-pier interaction can be drawn.

Of prime importance is that the piers must be positioned at a point where relatively large displacements are expected to occur; the magnitude of these displacements determines how much stress will be mobilized against the pier (i.e., how much stress the pier will absorb).

The direction of the displacement, and thus the direction of the stresses on the pier, should also be considered. Maximum benefit of the piers can be gained in axial compression, thus taking advantage of their greatest structural strength in that direction. For this reason, the case of surcharge loading lends itself well to this type of reinforcement.

The movements are primarily lateral for the case of excavation from a horizontal ground surface. The piers can still effectively absorb the stresses and transmit them to the foundation layers; however, the lateral load on the piers must be limited (unless tiebacks are used). The piers in our example were placed at the crest to take full advantage of the maximum vertical load. However, the limited mobilization of horizontal load due to limited movement of the soil at this point limited their ability to improve the stability. Better stability can be achieved by placing the piers lower in the slope, where horizontal movements are greater; however, analysis of these cases (*21*) showed that this results in high shearing stresses in the piers.

The relative mobilization of cohesive and frictional strengths is important to the performance of the stabilizing piers. Where upward movement is an important component, the support offered by the piers becomes indirect, increasing the confining pressure on the potential failure surface. The effectiveness of the piers in a soil with a frictional component and vertical downward component of movement is more complicated. Reducing the vertical stresses in the slope may have a negative effect because confining stresses are also being relieved. Thus, a major component of the piers' supporting action is of no benefit. The stability of the piers can be improved by retaining the vertical component of stress in areas of uplift, as shown in the example of cut slope stabilization. In general, however, unless the lateral reinforcement properties of the piers can be used, drilled piers may not always be an optimal solution to stability problems in frictionless soils.

In summary, when evaluating drilled piers for slope stabilization, the entire soil-structure interaction that occurs between the piers and soil mass must be considered, not simply the added shearing resistance provided by the piers. On the basis of analyses performed to date, the best applications of the piers seem to be in purely cohesive materials under loading conditions that can use the vertical resistance of the piers. When used to their best advantage, drilled piers have been shown to be a versatile slope-stabilization alternative.

ACKNOWLEDGMENTS

Support for this research was provided by the Indiana Department of Highways and the Federal Highway Administration, U.S. Department of Transportation. The research was administered through the Joint Highway Research Project, Purdue University, West Lafayette, Indiana.

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Publication of this paper sponsored by Committee on Transportation Earthworks.