# DESIGN METHODOLOGY:
# MICROPILES FOR SLOPE STABILIZATION AND EARTH RETENTION

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# USE OF MICROPILES FOR SLOPE STABILIZATION

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## ABSTRACT

Geotechnical engineers around the world are faced with the perennial problem of landslides affecting facilities for which they must design safe and economic solutions. Many of these problems can be solved in the conventional ways of reducing driving forces or increasing resisting forces by the displacement of large quantities of earthen materials. Unfortunately, these types of solutions are not always possible due to adverse environmental factors or the high costs of right-of-way acquisition and construction in urbanized areas.

In the past 25 years American engineers have incorporated into their slope stabilization projects some of the innovative solutions developed in Europe. Of particular interest recently is a concept originally developed in Italy for underpinning historic buildings and monuments using *pali radice* or root piles, (now termed, micropiles).

This presentation will discuss the recently developed micropile classification system, CASE 1 and CASE 2 slope stabilization and earth retention methods, construction techniques and materials, recommended design methodology of CASE 1 nonreticulated micropile structures and cost feasibility as presented in the FHWA sponsored "Micropile Design and Construction Guidelines Implementation Manual."

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# CHAPTER 6 - DESIGN METHODOLOGY: MICROPILES FOR SLOPE STABILIZATION AND EARTH RETENTION

## 6.A PURPOSE AND SCOPE

Substantial variance in engineering knowledge and the development of analytical models and design methods pertaining to different applications of micropile systems has a major impact on the current state of practice. For example, the current use of micropiles in the United States for slope stabilization and earth retention has been limited primarily due to the absence of a rigorous design approach, a situation leading to overly conservative, expensive designs and overall lack of confidence in the method.

While substantial research and field testing have been conducted to establish reliable design and construction techniques for axially loaded foundation support piles, the behavior of nonreticulated micropile groups and reticulated micropile networks must be further investigated in order to develop reliable design methods.

In addition, micropiles are being used for structural seismic retrofitting and slope stabilization measures in earthquake zones at an increasing rate. Recent seismic testing programs illustrate that micropile systems can be effectively used in seismic zones. However, the engineering knowledge of the dynamic performance of micropile systems is still very limited and further investigation is needed to develop and evaluate reliable seismic design methods and engineering guidelines. As indicated in Chapter 2, two design concepts for micropile systems have been developed.

## 6.A.1 CASE 1 Non-Reticulated Micropile Groups

CASE 1 non-reticulated systems (Figure 6-1) refer to micropile groups designed to be directly loaded to transfer structural loads through soft or weak ground to more competent strata.

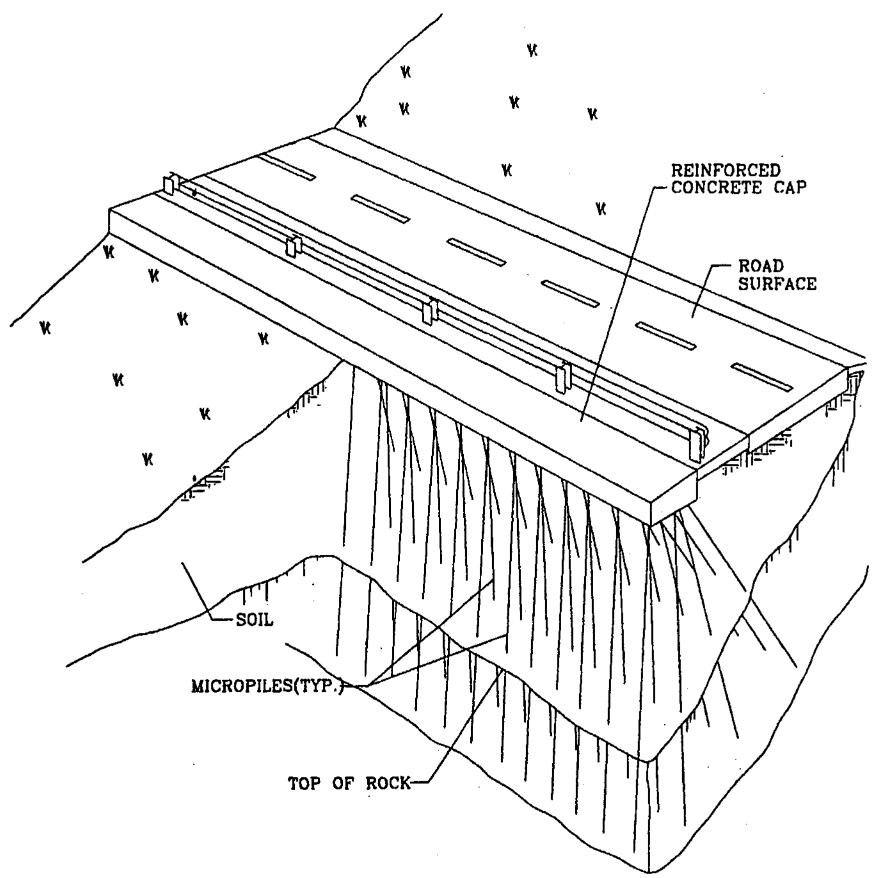



**Figure 6-1.** CASE 1 non-reticulated micropile system

### 6.A.2 CASE 2 Reticulated Micropile Networks

Dr. Lizzi's original "root pile" design concept relies primarily on using a three-dimensional network of reticulated CASE 2 micropiles (Figure 6-2) to create in-situ a coherent, composite, reinforced soil system. According to this design concept, the piles are not designed to individually and directly support the applied load, but rather to circumscribe and internally reinforce the in-situ soil forming a composite gravity structure to support the applied loading with minimal movement. As demonstrated by Lizzi [6], the engineering behavior of the micropile-reinforced soil is highly dependent on the group and network effects that may significantly improve the overall resistance and shear strength of the composite soil-micropile system.

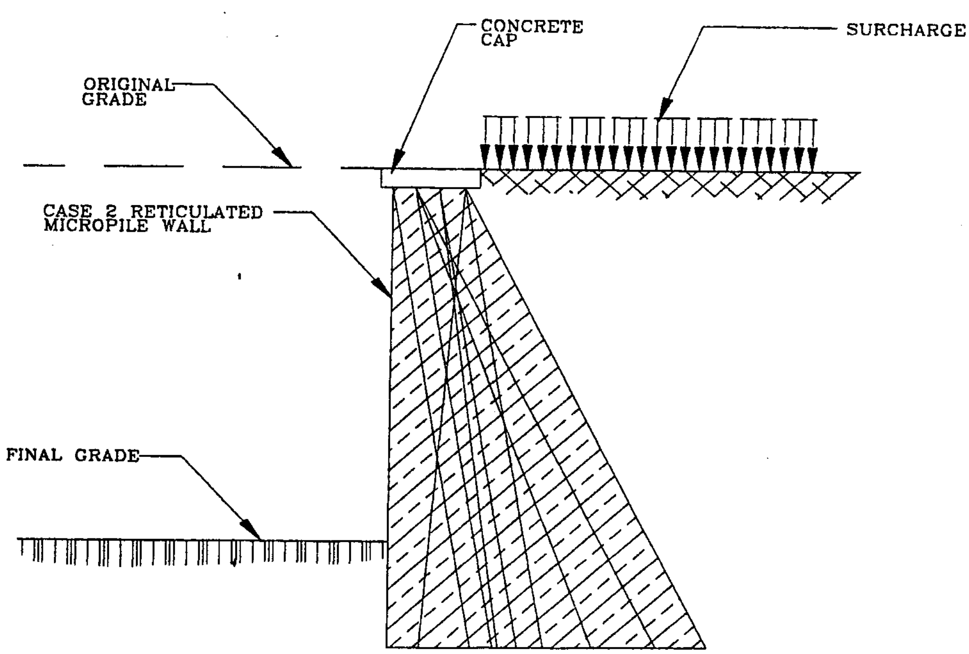



**Figure 6-2.** CASE 2 reticulated micropile network

These two design concepts provide different resisting forces in the micropiles and require significantly different micropile types and installation techniques, especially for slope stabilization and earth retention applications. CASE 1 designs generally involve connection of the micropiles to a reinforced concrete cap as a structural frame that must withstand combined loading and bending moments and, therefore, often demand substantial individual micropile capacities. Hence Type A (gravity grouted, bond in rock), Type B (pressure grouted through the head), and Type D (postgrouted) micropiles with high-strength reinforcements are most commonly used. CASE 2 designs feature a highly redundant, monolithic "gravity" system with low-capacity Type A (gravity grouted, fully bonded in soil) or Type B (low pressure grouting) micropiles. In some cases, specific applications and/or site conditions may require design schemes that combine the two basic design concepts outlined above (Figure 6-3).

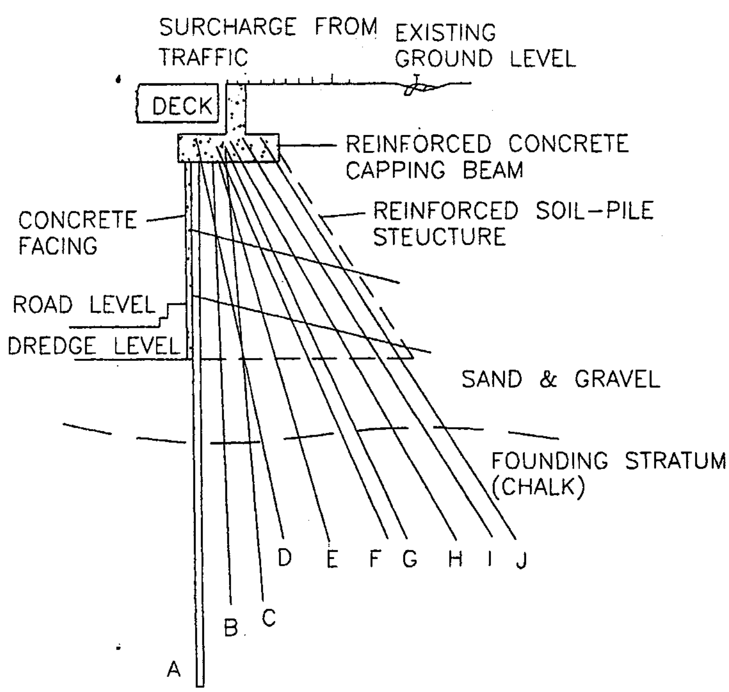



**Figure 6-3. CASE 1/CASE 2 micropile system**

This chapter presents a micropile design method for slope stabilization and earth retention applications that uses only CASE 1 nonreticulated micropile groups.

The impacts of the group and network effects associated with CASE 2 reticulated micropile systems and the overall response of the micropile system to boundary loading have not yet been sufficiently investigated and are not taken into consideration in current design practice of micropile systems. Theories for the design of a single, isolated micropile cannot be expanded to represent the design of reticulated groups. Theories that attempt to model the unique reticulation geometry exist; however, all of these theories are based upon empirical data. Because of these shortcomings, this area of geotechnical and foundation engineering demands a rigorous examination beyond the scope of this manual. However, Section 6.E of this chapter briefly summarizes and illustrates the "reinforced soil" design approach developed by Dr. Lizzi (1957) for slope stabilization applications of reticulated micropile networks.

## 6.B CURRENT MICROPILE SLOPE STABILIZATION TECHNIQUES

The stabilization of slopes by micropiles and/or soil nails consists of placing passive linear inclusions, capable of withstanding tensile forces, shear forces, and bending moments, into an existing or potential sliding surface. The micropiles are generally installed with a uniform density either in a critical zone at the toe of an unstable slope or throughout the sliding or creeping mass, thereby creating a relatively uniform, composite, cohesive mass of reinforced ground.

Micropiles, as well as soil nails, are used to restrain two distinctly different modes of downslope soil movement. In the first case, referred to as *potentially unstable slopes*, little or no movement occurs but safety factors along potential sliding surfaces are unacceptably low thereby allowing potential sliding. The purpose of the reinforcing element is to increase the safety factor to an acceptable level. The second case, referred to as *creeping slopes*, pertains to the situation where movement actually occurs at an unacceptable rate. The upper moving zone is separated from the stable lower zone by either a relatively thin, well-defined failure zone, generally at the interface between two different layers, or a larger zone within which the induced shear stresses are of sufficient magnitude to cause a continuous creep. In this case, the purpose of the reinforcing element is to stop the movement or at least decrease the sliding (or creeping) rate to an acceptable value.

For slope reinforcement, micropiles or soil nails are generally installed with a rather uniform density throughout the unstable zone. In a creeping slope, the reinforcements are generally installed either with a relatively uniform density throughout the creeping zone or in a critical zone at the toe of the slope. The construction process, the choice of the reinforcing element, and the behavior of the slope stabilization system depend on several factors including site conditions, soil type, slope stability and creeping rate, inclination of inclusions with respect to the potential or existing failure surface, spacing of inclusions, and rigidity of the inclusions relative to the soil.

CASE 1 nonreticulated micropiles, as well as soil nails, have been used for in-situ slope reinforcement [1]. The inclusions (e.g., tubes, bars, metallic profiles, etc.) are either installed in boreholes and sealed to the ground by cement grouting, (with or without pressure grouting), or they are simply driven into the ground. The sliding zone is generally uniformly reinforced by the relatively closely spaced inclusions (Figure 6-4a). Alternatively, a CASE 1 nonreticulated micropile group is installed with a rigid cap to form a structural frame (Figure 6-4b). In this mode, each leg experiences a combination of thrust, shear, and bending moment, and its resistance is assumed to be provided by the steel-reinforcing elements. A relatively high density of micropiles can induce a soil-pile interaction, which, although not yet fully understood, is apparently beneficial to overall stability, resulting in a positive group effect.

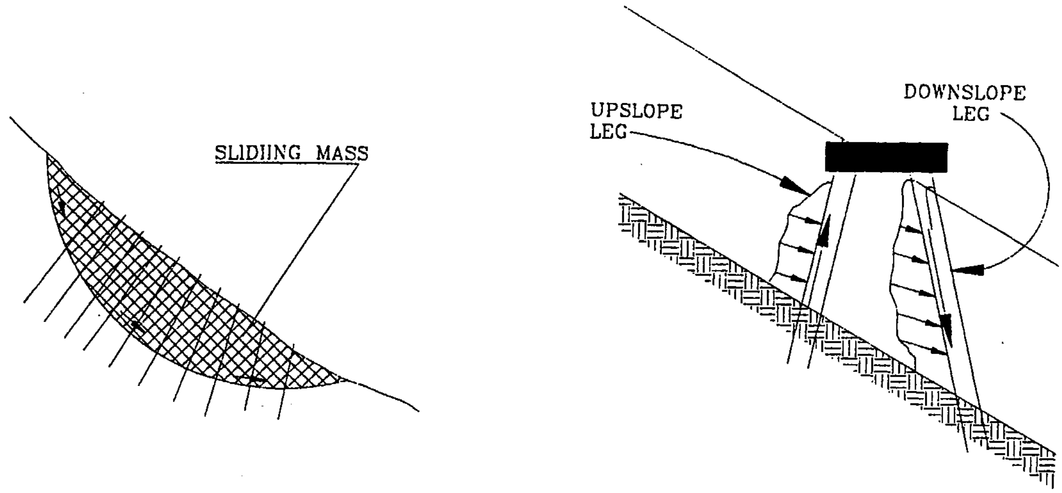
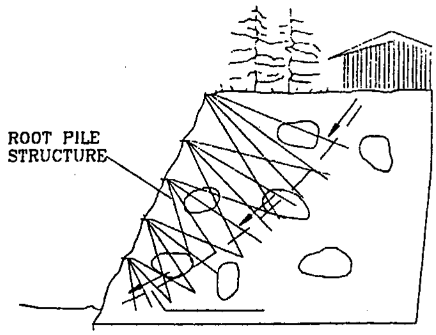
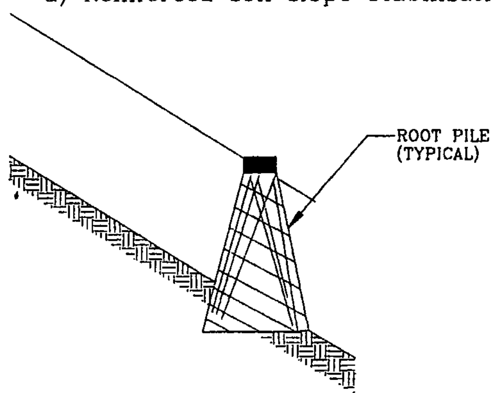



**Figure 6-4. a) Slope reinforcement, and b) slope stabilization**

CASE 2 reticulated micropile networks are designed to create in-situ a coherent, composite, reinforced soil gravity structure significantly different from that of micropile groups or soil nails. This type of structure is highly dependent on the encapsulating network effect that results in an apparent cohesion and an increase of the overall stiffness of the soil-micropile network. As shown in Figure 6-5a, the sliding zone is generally reinforced by the relatively closely spaced, three-dimensional reticulated micropile systems crossing the potential sliding surface. Alternatively, as shown on Figure 6-5b, the micropiles are connected to a reinforced concrete cap creating in-situ a prism of the overburden material and, as postulated by Lizzi, "stitching together the different soil/rock layers, transforming the entire mass into a reinforced soil gravity retaining structure" [6].




a) Reinforced soil slope stabilization




b) Reinforced soil gravity structure

**Figure 6-5. a) Reinforced soil slope stabilization, and b) Reinforced soil gravity structure**

The following briefly presents field experiments Palmerton conducted on two different instrumented micropile systems to investigate the engineering behavior of both nonreticulated (CASE 1) and reticulated (CASE 2) micropile systems used for slope stabilization [21].

A CASE 1 nonreticulated micropile system was used in New York state to stabilize approximately 75 m of roadway [21]. The CASE 1 system was approximately half of the Engineer's estimated cost for a CASE 2 reticulated system. The wall was designed as a structural frame retaining system to resist sliding of the soil above the shear plane. The upslope and downslope micropile systems were assumed to act as a "composite beam," with the upslope piles loaded by earth pressures from the upslope side of the piles and the downslope piles acting as a wall loaded by earth pressures between the upslope and downslope pile clusters. No allowance of long term downhill passive forces were permitted above the slide plane. The reinforced concrete cap was assumed to act as the cross element of the frame. The design anticipated that all of the load would be transferred through the micropiles to the rock in bearing.

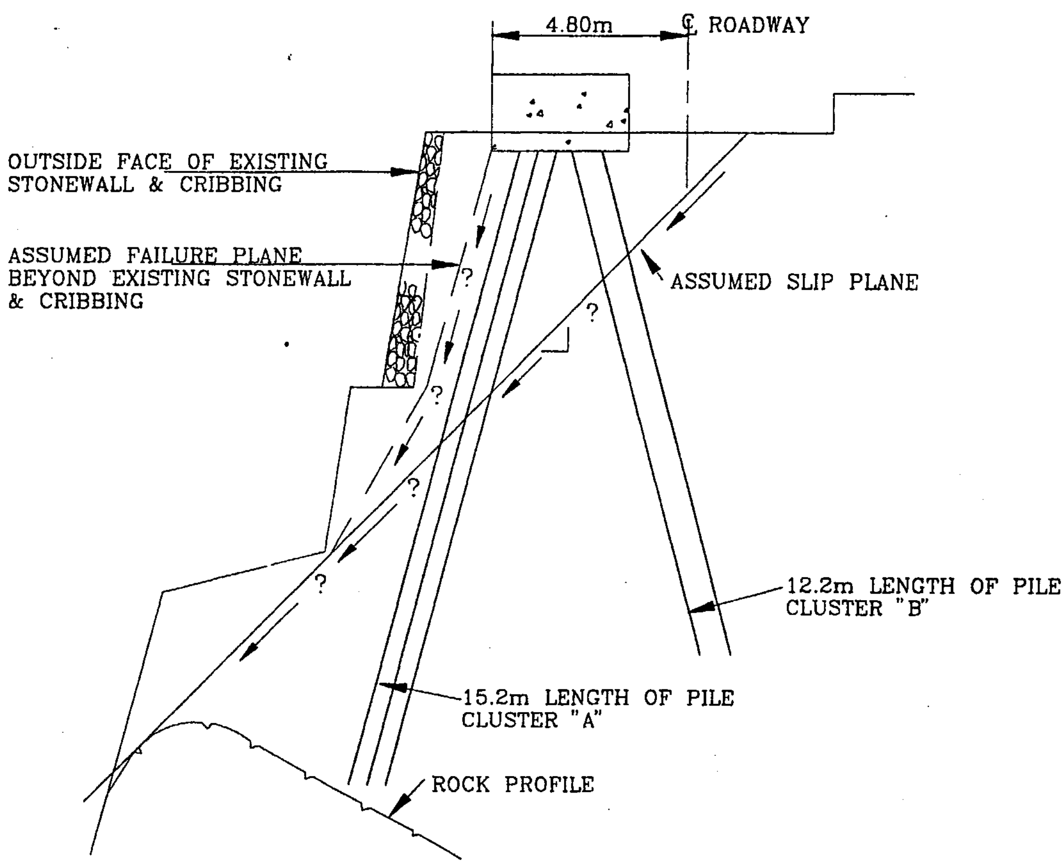
Approximately 700 piles, extending to depths of 12 to 18 m, were used in constructing the wall, as shown in the typical cross section presented in Figure 6-6. The wall is composed of a two-dimensional pattern of 90-mm minimum diameter (with a single 32-mm diameter, steel-reinforcement bar) CASE 1 micropiles inclined upslope and downslope at a maximum angle of 15 degrees from vertical. The center-to-center spacing between adjacent piles ranges between 0.45 and 0.60 m. After micropile installation, a reinforced concrete cap was constructed over the piles. Performance of the micropiles was monitored by strain gauges bonded to the reinforcing steel, and the stabilizing effects of the wall during and after construction on the slope displacements was monitored by tiltmeters and slope inclinometers. The strain-gauge recordings at the New York site did yield noteworthy information.

The results of strain gauging the steel reinforcement indicated a generalized pattern of deformation by both "beams," as shown in Figure 6-7. The patterns illustrated in the figure seem to be consistent with downslope movement along the shear plane. The significance of the cap beam on wall performance is clearly demonstrated by the near-immediate reduction in slope movement recorded with the slope inclinometer and the noticeable stabilization of measured tension or compression strain levels in the reinforcement. Pertinent observations associated with recorded movement in the New York site are:

- The repair was installed in an area in which landslide movements were currently active.
- Movements continued at the site during the casting of the piles but before the placement of the pile cap.

- Movement essentially ceased following the placement of the pile cap.

It is of particular interest to note that the various instrumentation (strain gauges and slope indicators) yielded responses that appear to be consistent with the design concept, assuming the micropile group to act as a "composite beam."




**<u>Figure 6-6.</u> Catskill, New York micropile slope stabilization**

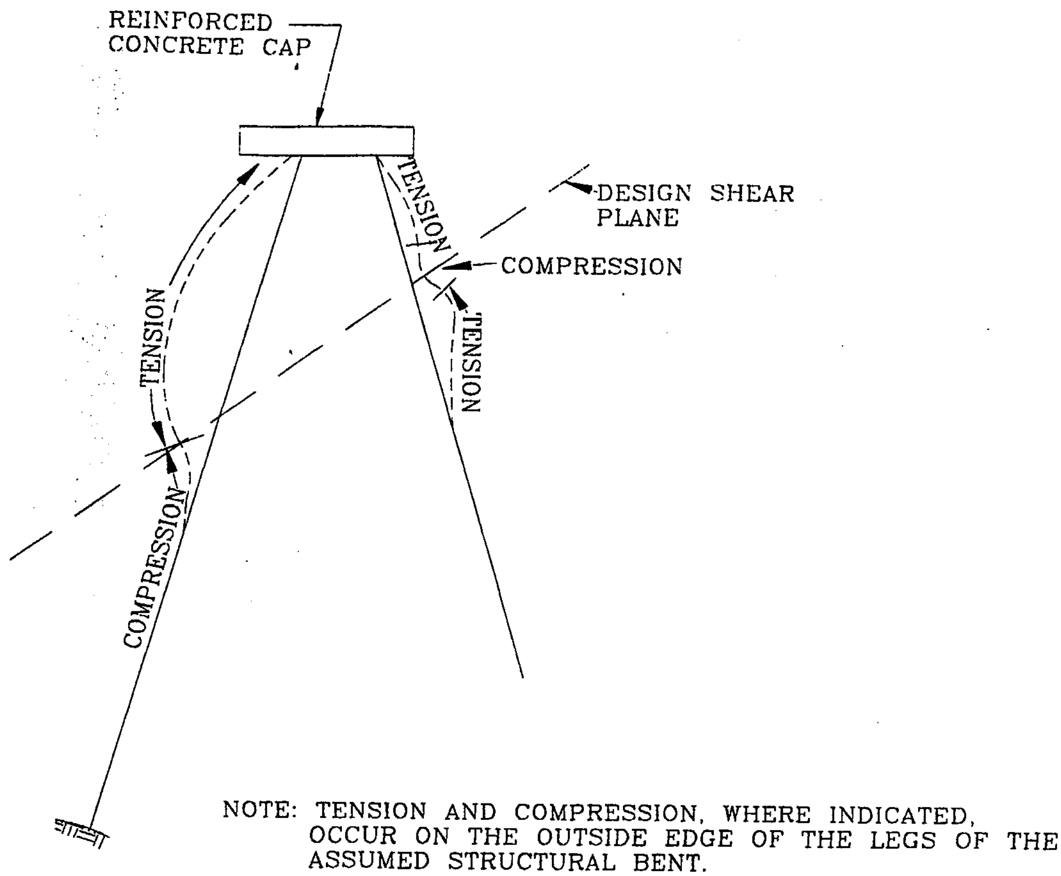



**Figure 6-7. Generalized pile deformation, Catskill, New York**

A CASE 2 reticulated micropile network system was used in Mendicino, California for a highway slope repair [21] in 1983 . A typical section of the pile network, which was connected to a 1.00-m thick reinforced concrete cap beam, is shown on Figure 6-8.

The piles were installed at inclinations ranging from vertical to about 16 degrees from vertical. A total of 730 piles with lengths of 21 m were required to construct the wall. The center-to-center spacing between adjacent piles at the cap beam ranges between 0.45 and 0.9 m. The performance of the pile wall during and after construction was monitored by the U.S. Army Corps of Engineers, Waterways Experiment Station, using strain gauges bonded to reinforcement bars.

The results of strain gauging the steel reinforcement indicated that, with some exception, all steel was loaded in compression with calculated stresses ranging from 5.2 to 52 MPa [21]. Measured tension strains were generally limited to the areas in the vicinity of the cap beam and near the bottom of the piles below the presumed shear surface. Strains in the reinforcing steel developed rapidly during the first and second months following construction, but stabilized thereafter. The recorded postconstruction strains in the rebars were simply too small to establish apparent trends. Evidently, the slope (at least in the area of instrumentation) had stabilized prior to construction.

Subsequent to construction, the downslope portion of the slide mass continued to move, dropping approximately 3 meters vertically away from the downslope piles. However, the roadway and slope behind the reticulated micropile wall have remained stable.

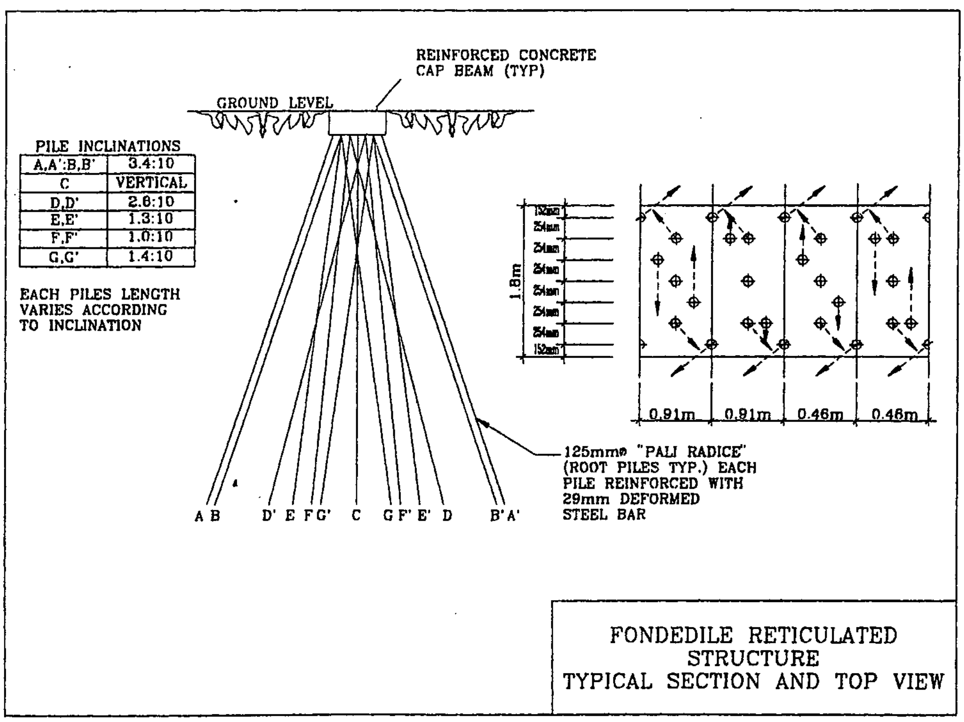



**Figure 6-8. FH-7, Mendicino, California micropile slope stabilization**

## 6.C. DESIGN OF CASE 1 MICROPILE SLOPE STABILIZATION SYSTEMS

This section is devoted to the application and design of CASE 1 nonreticulated micropile systems (Figure 6-9) as in-situ soil reinforcement, used increasingly in a wide range of applications for slope and excavation stability associated with deep foundation, tunneling, and highway construction.

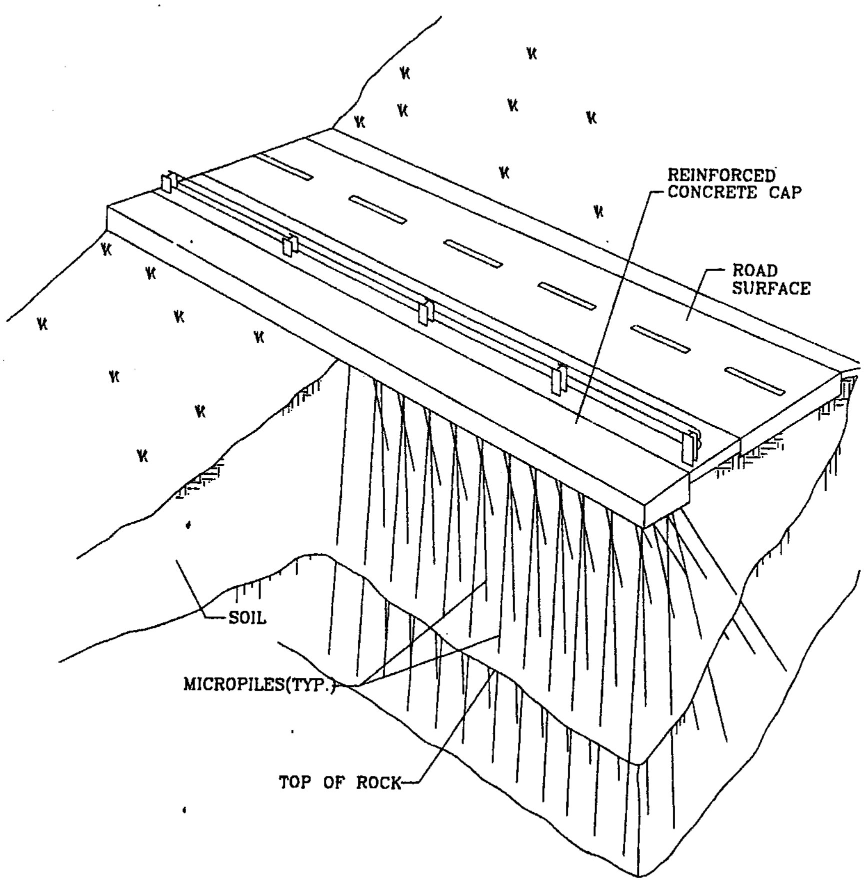



**Figure 6-9.** Typical CASE 1 non-reticulated micropile structure

Within the past few years, intensive research in this area has been conducted by Pearlman et al. Much of this review is based on their research findings [5].

Several of the case histories they analyzed involved projects originally designed assuming that the structure behaved as a gravity retaining wall. However, detailed analysis of wall performance data on these projects indicates that the micropile structures were *not*, in fact, behaving as gravity walls.

Micropile movements seemed to be confined to a relatively thin zone along the slide plane, and additional slope movements were occurring after micropile system construction. Therefore, a new procedure was developed for design of these slope stabilization structures to better model the behavior of this relatively flexible earth retention system. The theoretical basis for the procedure has been verified by comparison with back analyses of the instrumented walls. In general, the new design procedure involves the following:

- Conducting stability analyses to determine the increased resistance required along a potential or existing slip surface to provide an adequate factor of safety.
- Checking the potential for structural failure of the piles due to loading from the moving soil mass.
- Checking the potential for plastic failure, (i.e., flow of soil around the pile) and determine pile spacing.

Typically, movement of marginally stable, noncreeping soil slopes occurs within a relatively thin zone subjected to large shear strains. The large shear strains are not experienced within the materials above and below the zone of failure. Pearlman et al. determined that the function of the micropiles is to connect the moving zone (above the slip surface) to the stable zone (below the slip surface), and thus to increase the sliding resistance along the slip plane, similar to the explanation presented by Palmerton (1984).

Because micropiles are relatively flexible, the maximum bending moments in the piles tend to develop relatively close to the slip plane. Fukuoka devised a theory to evaluate bending moments that develop in the pile-oriented perpendicular to the slip plane, assuming a uniform velocity distribution of the soil above the slip plane [22]. Figure 6-10 is a chart for preliminary design of CASE 1 nonreticulated micropile systems. The chart was developed using the method described in Fukuoka and considers four typical sizes of micropile elements. It should be noted that the ultimate horizontal resistance is either the load that causes yield stresses to develop on the outer edges of the steel pipes or the load that causes crushing of the concrete surrounding the centralized reinforcing bar. The ultimate horizontal load resistance of the piles is a function of the coefficient of subgrade reaction (K) of the soil above and below the slip plane. The K of the soil also has a significant effect on the amount of horizontal movement required before the pile reaches its ultimate horizontal resistance. Typical deflections and bending stresses along a pile are shown in Figure 6-11.

Plastic failure of soil around the piles can be analyzed using a procedure developed by Ito and Matsui [23]. The method is based on the fundamental assumption that soil deformation is restricted to a plane strain condition. Typically, this type of failure occurs if the soil above the slide plane is relatively soft and the piles are stiff and spaced far apart. This is usually not the case with relatively flexible micropiles, but may govern when stiffer pipe elements are employed. The predicted results for various pile spacings and soil conditions, based on the theory proposed by Ito and Matsui, are plotted in Figure 6-12.

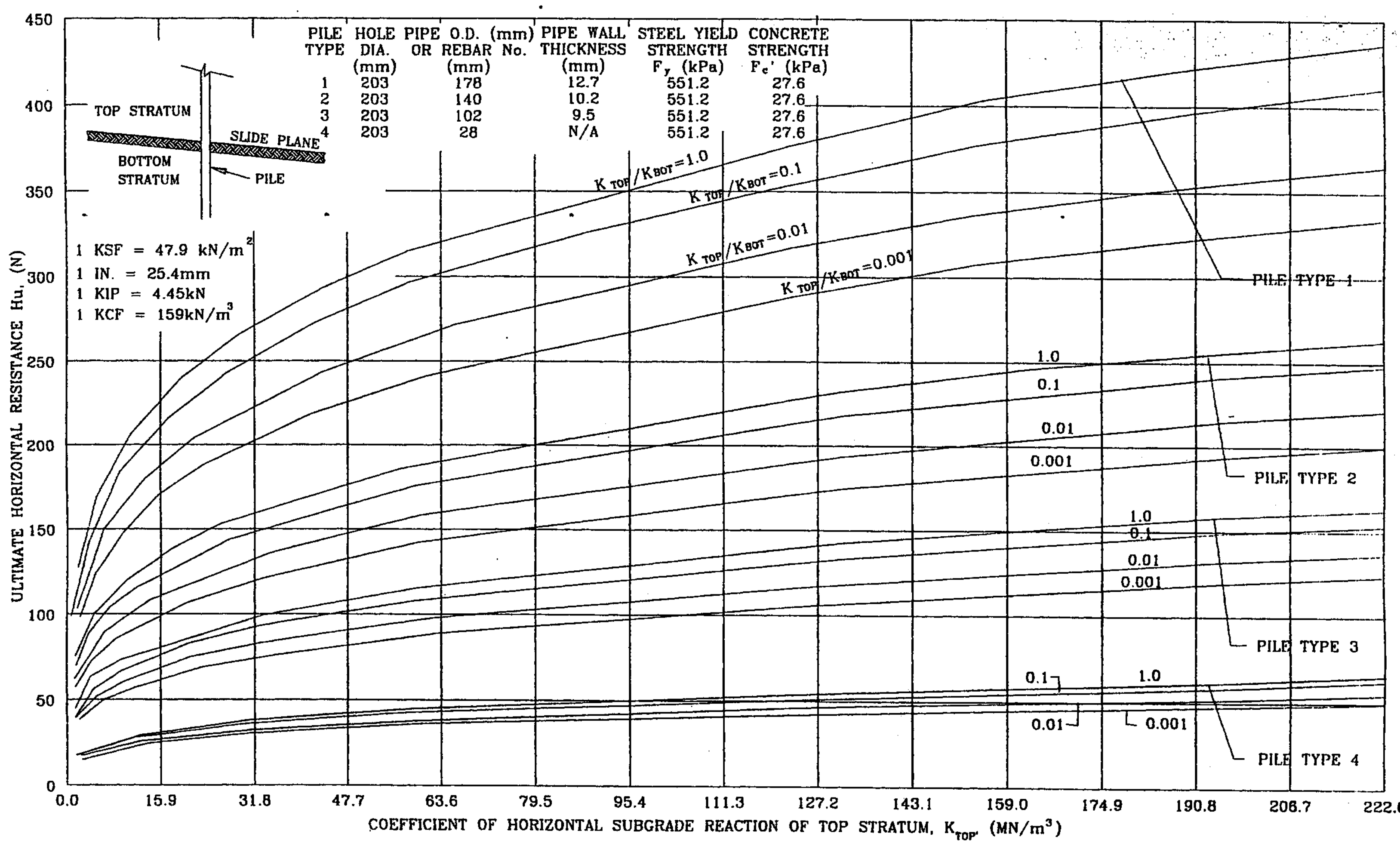



**Figure 6-10.** **Preliminary design chart for pile ultimate horizontal resistance**

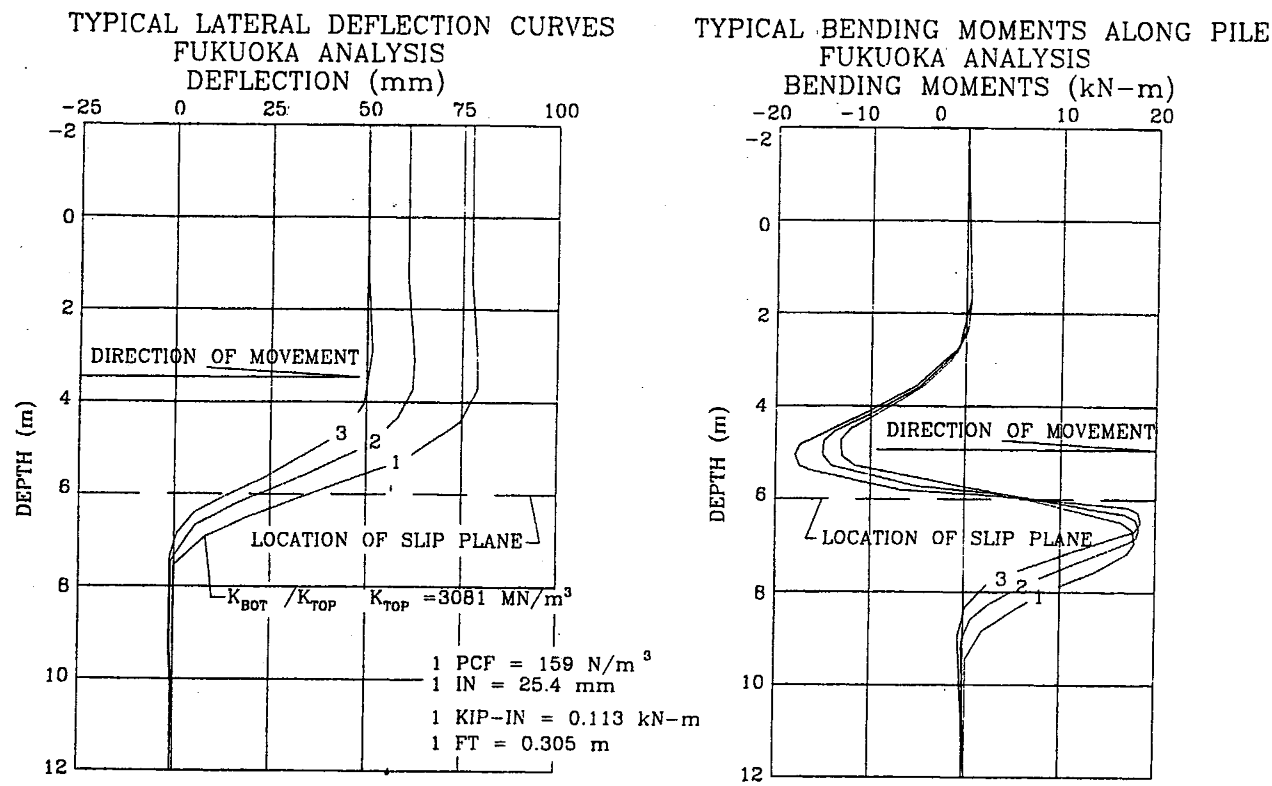
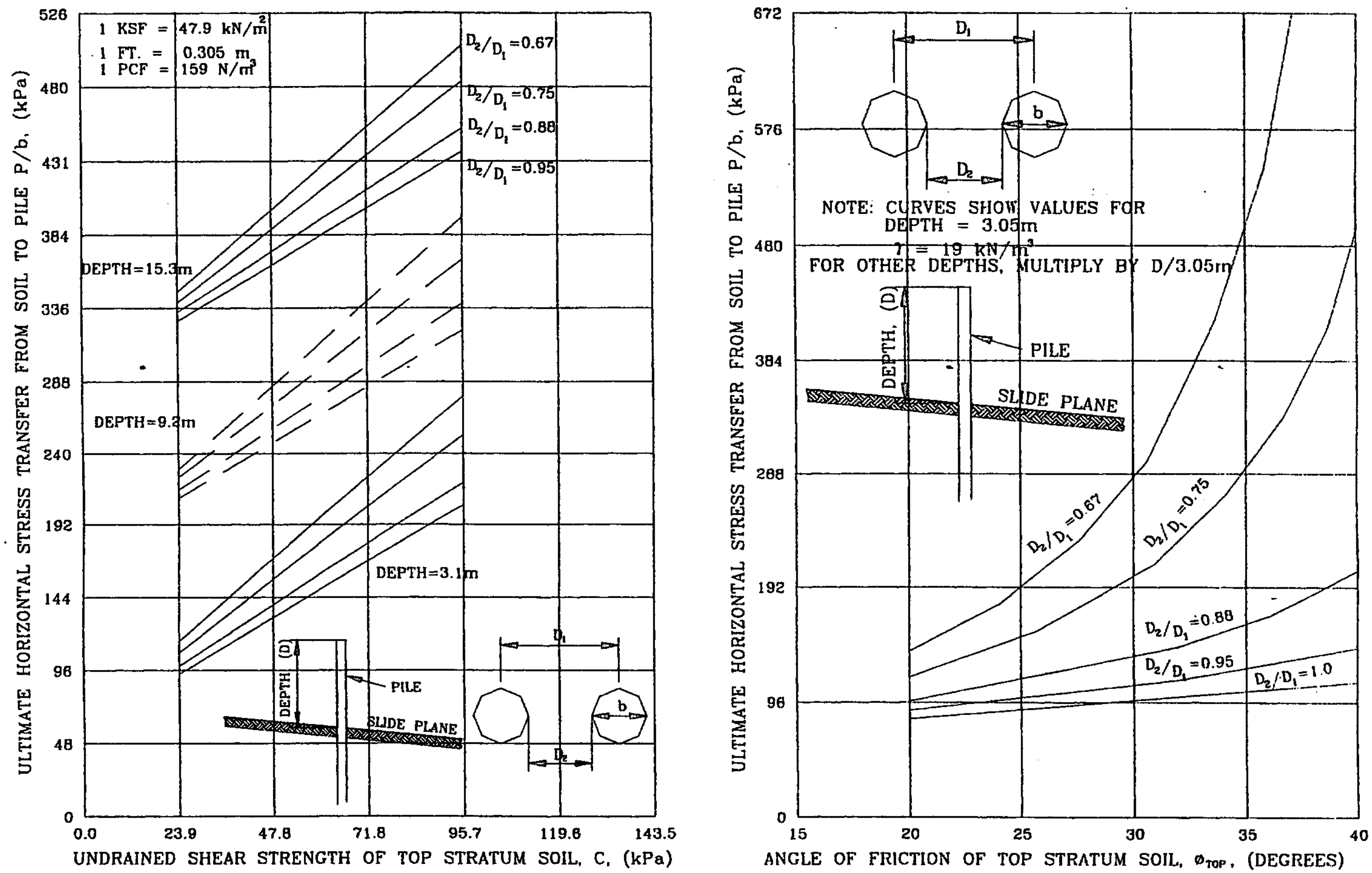



**Figure 6-11.** Typical pile deflection and bending stresses




(A) COHESIVE SOIL

(B) COHESIONLESS SOIL

**Figure 6-12.** **Ultimate stress transfer from soil to pile vs shear strength of soil (Note: For mixed soils that have both cohesive and frictional shear strength, use whichever gives the higher ultimate P/b)**

Ito and Matsui suggested two analytical approaches, considering respectively: (1) plastically deforming soil around the piles (limit analysis approach), and (2) visco-plastic flow of the soil around the piles.

The limit analysis approach, considered appropriate only for overly consolidated soils, assumes that the soil just up-slope of the pile is in a plastic limit state, and that this soil is a perfectly plastic solid that follows the Mohr-Coulomb yield criterion. The static equilibrium conditions of the plastic solid yields a solution for lateral force acting on a unit length of the pile as a function of the pile diameter, the spacings, and the effective strength characteristics of the soil. Typical analytical results based on this method of analysis do not take into account soil arching, and no consideration is given to the creep behavior of the sliding ground. Therefore, it cannot be considered valid for normally consolidated, saturated, soft clayey soils.

The visco-plastic flow approach assumes that the soil just around the piles behaves as a visco-plastic solid in quasi-steady state of a visco-plastic flow. The sum of the quasi-static lateral earth pressure and the viscous shear force due to the soil-pile interaction yields the solution for the lateral force acting on a unit length of the pile as a function of the pile diameter, the spacings, the soil visco-plastic properties, and the sliding velocity. The second approach incorporates the viscous flow conditions of the creeping soil, but it raises difficulties concerning the boundary conditions at the soil-pile interfaces, the appropriate determination of the viscosity properties of the soil, and the reasonably accurate evaluation of the flow velocity.

The design charts are useful in providing a preliminary estimate of the pile density and type of piles that are feasible for a particular application. The procedure is conservative for cross sections that use inclined piles. Inclining the micropiles with respect to the slide plane, and/or direction of slope movement, tends to mobilize the axial resistance of the micropiles. Since the piles are typically small in diameter, their surface area-to-cross- sectional area ratio is relatively large. Hence, they are very efficient at mobilizing skin friction, and typically have much higher axial capacity than lateral capacity.

It should then be noted that these charts are for preliminary design only, to establish general requirements for pile size and spacing and budget cost analysis. Final design of CASE 1 nonreticulated micropile slope stabilization systems requires consideration of various factors, including pile inclination relative to the orientation of slope movement, the depth of the slide plane relative to the stiffness of the piles, and the additional capacity of the reinforcing bars after concrete crushing occurs.

For final design of CASE 1 nonreticulated micropile slope stabilization systems following the Pearlman et al. method, the following design procedure is suggested:

1.) Conduct slope stability analysis on existing slope condition. If currently an active slide exists, assume a F.S. =1.0.

2.) Determine the increased resistance required along an existing failure surface to provide a minimum owner-specified factor of safety;

3.) Determine the potential for structural failure of the micropiles due to loading from the moving soil mass, including the following;

- Determine pile structural properties and capacities;
- Perform lateral pile analysis to evaluate ultimate resistance of pile located perpendicular to slide plane;
- Evaluate micropile geotechnical capacity in lower stable zone;
- Evaluate added pile resistance due to inclination [25].

4.) Determine final micropile spacing. Evaluate spacing required to prevent soil flow around piles.

In developing a design, it is necessary that certain information be available regarding the location of the slide plane and the engineering properties of the ground mass above and below the slide plane. The location of the slide plane is an important consideration in the design to ensure that each pile extends a sufficient length across either side of the zone of movement. Information from subsurface explorations, in-situ measurements (e.g., inclinometers), sheared buried utilities, or the locations of displacement at the ground surface can be used to measure or infer the location of the slide plane. The determination of soil and rock properties should be based on the results of subsurface explorations, and in-situ and laboratory testing. In particular, the type, consistency, strength of soil and rock materials and groundwater table location are necessary if reliable and rational designs are to be developed.

The unconfined compressive strength or internal angle of friction of material above and below the slide plane can be estimated by in-situ testing (e.g., pressuremeter or cone penetrometer), laboratory testing of samples from borings, and/or empirical correlations with standard penetration test blow counts. Back analysis of shear strength parameters can also be performed on existing active slides. Use of the design procedure should be based on input provided by a geotechnical engineer experienced in interpreting such test results and in performing slope stability analysis, to develop material parameters needed for design.

An example design problem for slope stabilization utilizing this method of analysis for a CASE 1 nonreticulated micropile system is presented in the following section.

## 6.D SAMPLE PROBLEM NO. 2 - CASE 1 NONRETICULATED MICROPILE SLOPE STABILIZATION

*Given*: Roadway along the flank of a hill (valley floor downslope, river is present). The road is built across unstable ground (sometimes side cast fill, of early origin and often uncompacted). Subsurface conditions, site topography, and design parameters for this example are shown on the attached sketches. The existing road is marginally stable, failing during winter due to high groundwater.

Project constraints include working in an environmentally sensitive area. No work is to be performed outside of the roadway prism. One-way traffic must be maintained, with temporary road closures for periods up to 30 minutes allowed. Micropiles are determined to be the best solution to meet the project constraints.

Geotechnical site characterization defining soil and rock units with material properties and groundwater levels is provided herein. Stability analyses have been performed as shown for the unstable condition. For this example, the landslide failure surface location has been determined via the subsurface exploration program and installation of slope inclinometers.

*Solution:* Provide CASE 1 nonreticulated micropile slope stabilization as follows:

1. Analyze existing stability condition of the slope using conventional stability analyses (e.g., use XSTABL, STABL4, or hand solution etc.). If currently an active slide, assume existing slope stability F.S. = 1.0.
2. Determine additional resisting shear force needed to maintain the required factors of safety.

   $F.S._{static} \geq 1.3$

   $F.S._{seismic} \geq 1.1$
3. Design CASE 1 nonreticulated micropile structure in lower roadway shoulder to stabilize roadway.

4. Provide internal and external stability design calculations for required micropile structure including final micropile spacing.

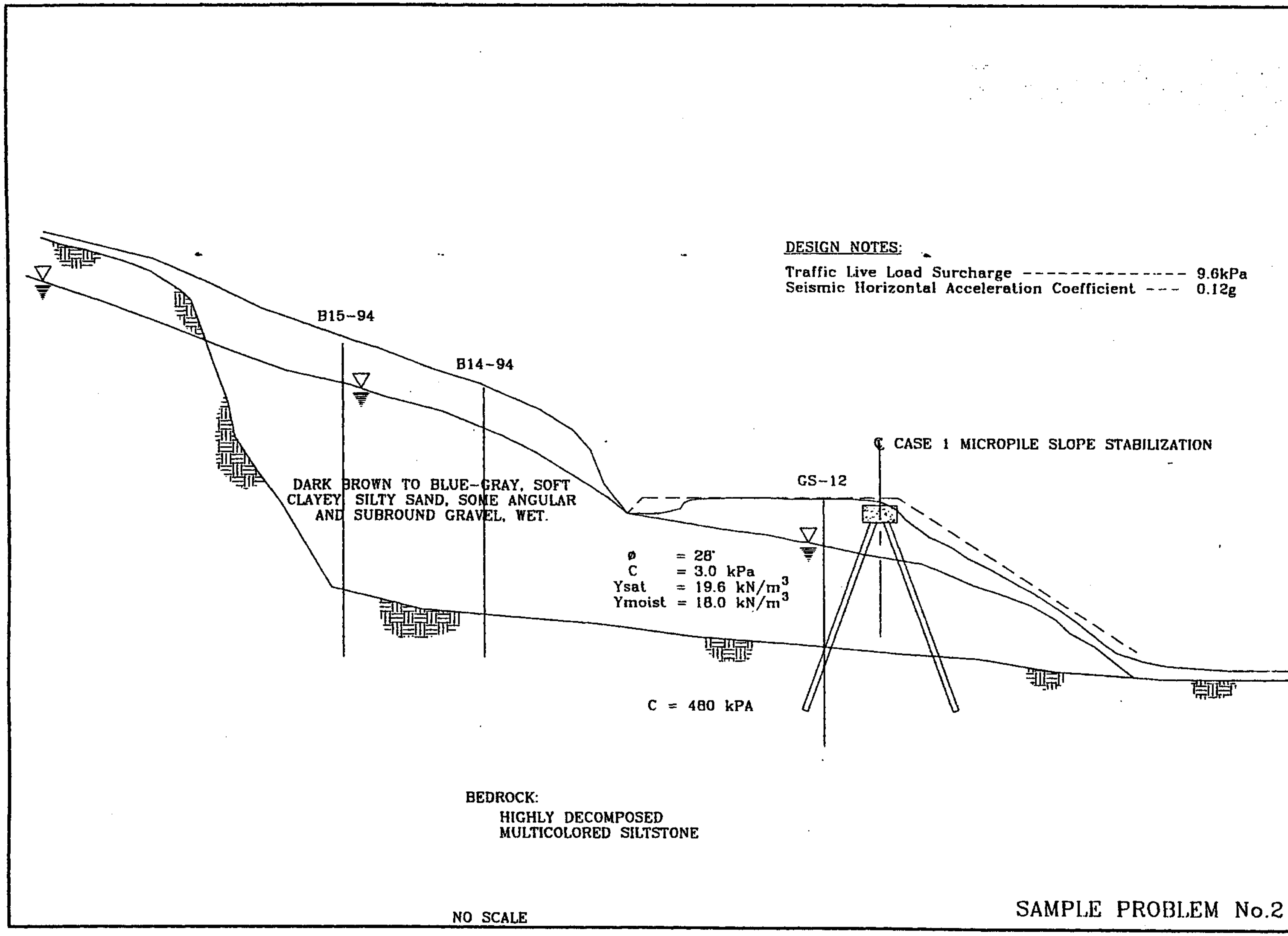



**Figure 6-13.** **Sample Problem No. 2, Case 1 non reticulated micropile slope stabilization**

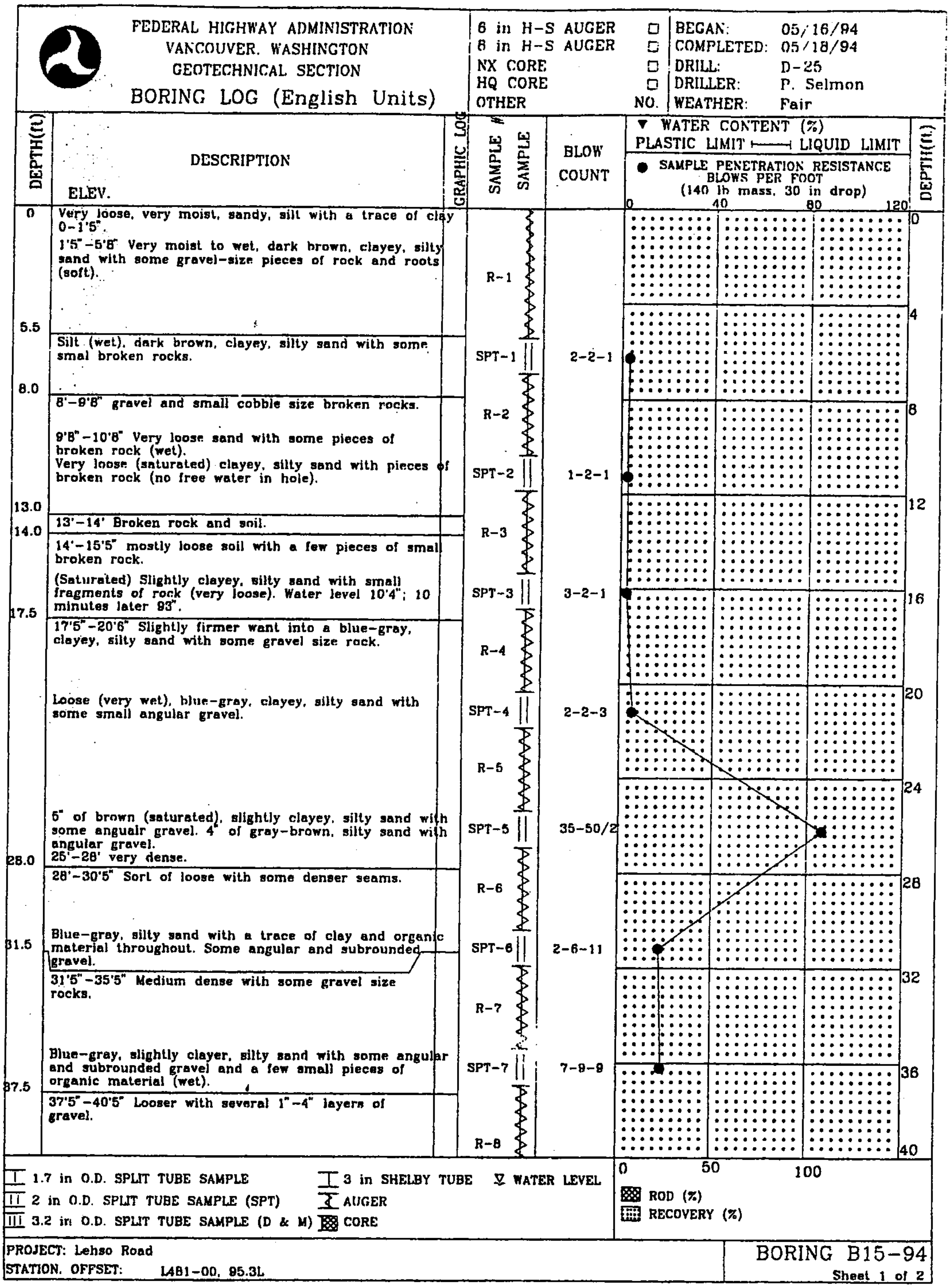



**Figure 6-14. Boring log B15-94 information (station L481+00)**

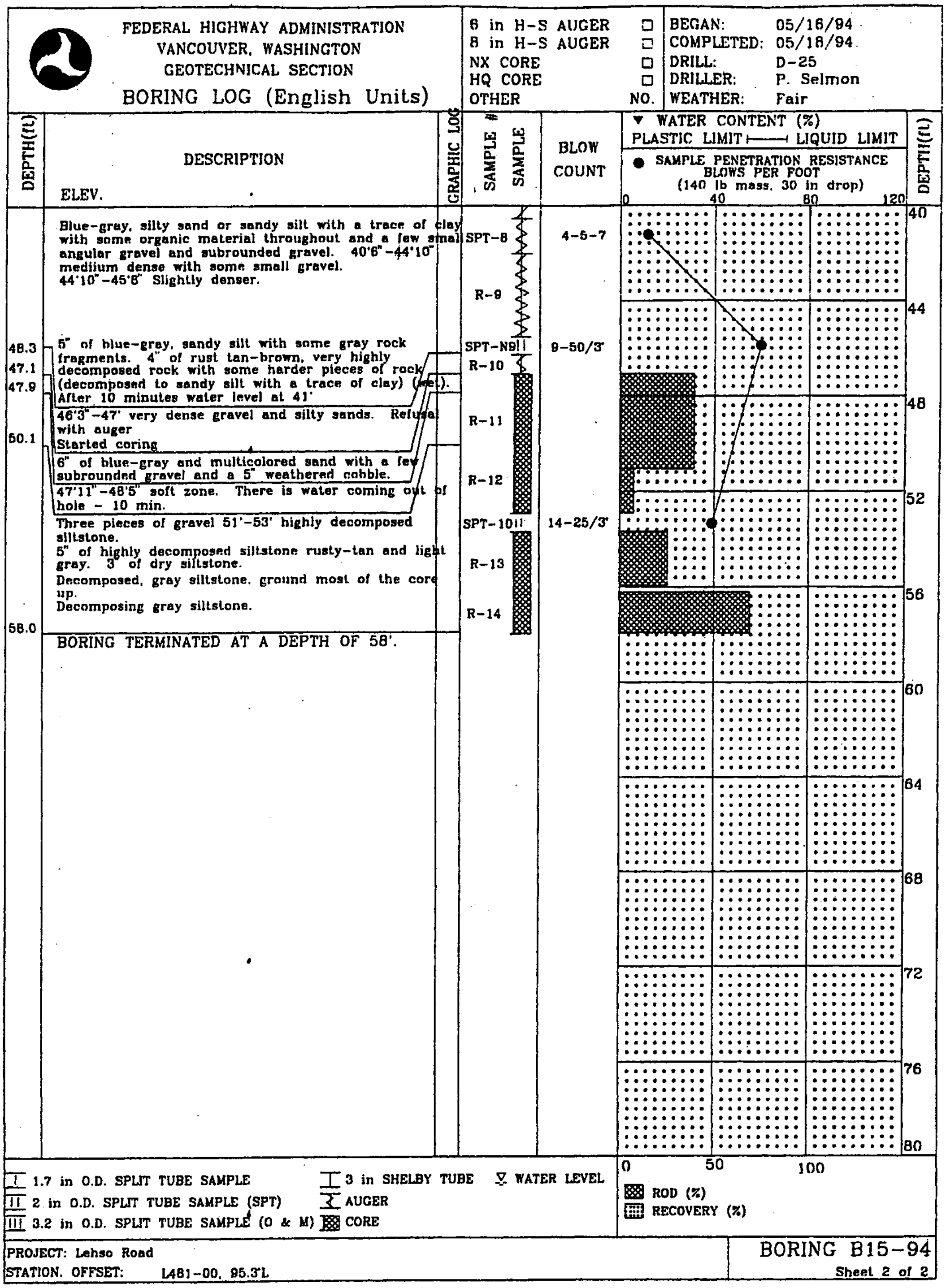



**Figure 6-14 (continued).** Boring log B15-94 information (station L481+00)

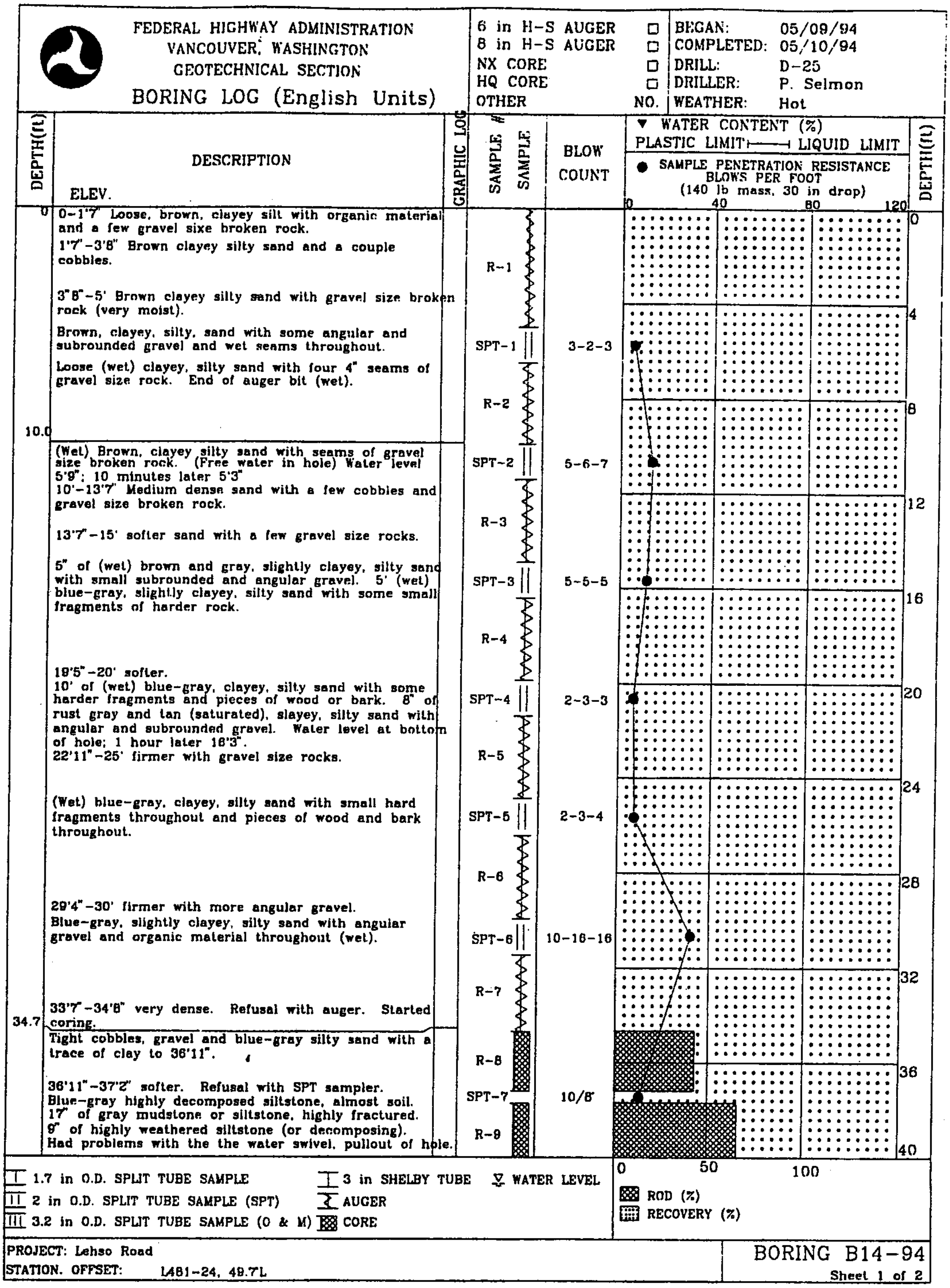



**Figure 6-15. Boring log B14-94 information (station L481+24)**

| FEDERAL HIGHWAY ADMINISTRATION<br>VANCOUVER, WASHINGTON<br>GEOTECHNICAL SECTION<br>BORING LOG (English Units) | 6 in H-S AUGER ☐<br>8 in H-S AUGER ☐<br>NX CORE ☐<br>HQ CORE ☐<br>OTHER NO. | BEGAN: 05/09/94<br>COMPLETED: 05/10/94<br>DRILL: D-25<br>DRILLER: P. Selmon<br>WEATHER: Hot |
|---|---|---|

| DEPTH(ft) | DESCRIPTION | GRAPHIC LOG | SAMPLE # | SAMPLE | BLOW COUNT | ▼ WATER CONTENT (%) PLASTIC LIMIT ├─┤ LIQUID LIMIT<br>● SAMPLE PENETRATION RESISTANCE BLOWS PER FOOT (140 lb mass, 30 in drop)<br>0 40 80 120 | DEPTH(ft) |
|---|---|---|---|---|---|---|---|
| 40.4 | Water level was at 3' 8:00 am 5-11-94 the water was running out top of the hole when we pulled core barrel at 37'2"; we had water run out of drill still.<br>BORING TERMINATED AT A DEPTH OF 40'5" | | | | | | 40 |
| | | | | | | | 44 |
| | | | | | | | 48 |
| | | | | | | | 52 |
| | | | | | | | 56 |
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| | | |
|---|---|---|
| ⌶ 1.7 in O.D. SPLIT TUBE SAMPLE<br>⌶ 2 in O.D. SPLIT TUBE SAMPLE (SPT)<br>⌶ 3.2 in O.D. SPLIT TUBE SAMPLE (D & M) | ⌶ 3 in SHELBY TUBE<br>⌶ AUGER<br>▩ CORE | ▽ WATER LEVEL |
| 0 50 100<br>▩ ROD (%)<br>▤ RECOVERY (%) | | |
| PROJECT: Lehso Road<br>STATION, OFFSET: L481-24, 49.7L | | BORING B14-94<br>Sheet 2 of 2 |

**Figure 6-15 (continued). Boring log B14-94 information (station L481+24)**

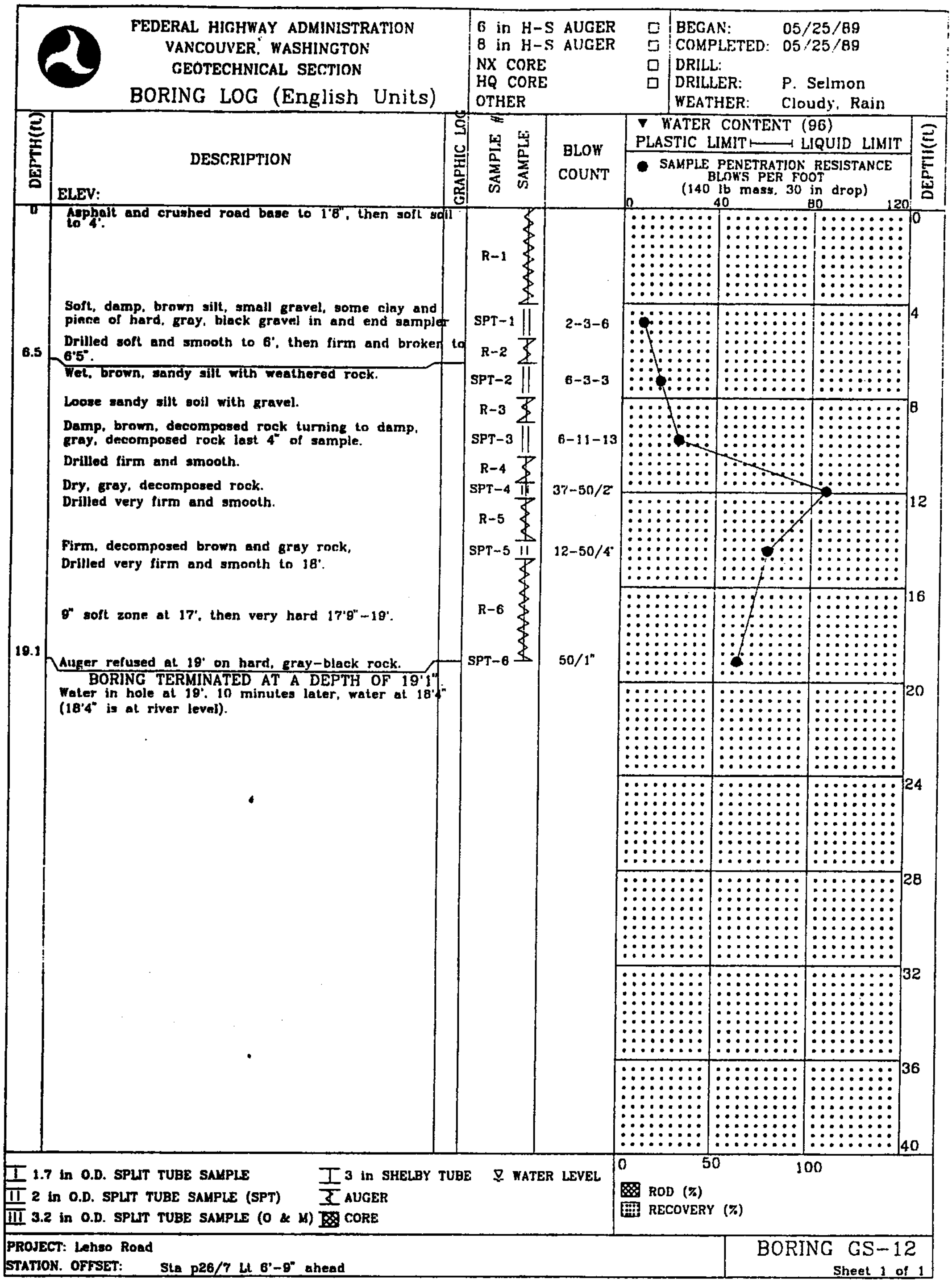


**Figure 6-16.** **Boring log GS-12 information (station P26/7 Lt 6'-9")**

*Design:* For ease of understanding, results of XSTABL slope stability and COM624P pile stiffness, bending capacity, and lateral pile analyses, have been summarized. All computer runs are not included.

**Step 1: Perform slope stability analyses to determine the increased resistance required to stabilize roadway slope and meet the specified slope stability factor of safety.**

To do this, the XSTABL computer program was utilized to perform the analyses. Run No.1 was performed to analyze the unstabilized slope. The following factors of safety were verified for the current unstabilized slope condition: (computer output for static condition shown on pages 189-193.)

$$F.S._{static} = 1.0$$
$$F.S_{.seismic} = 0.81$$

**Step 2: A.) Perform slope stability analyses to determine the required wall resistance necessary to provide the owner desired factor of safety ($F.S._{static} = 1.3$ and $F.S._{seismic} \doteq 1.1$).**

In the XSTABL analysis (Run No. 2), a thin slice of strong cohesive material was included in the design to model the shear resistance provided by the micropiles. The cohesive strength was varied by trial and error until the specified minimum static and seismic factors of safety were satisfied. Run No.2 indicates the required ultimate additional shear resistance needed to meet the required minimum factors of safety is 63 kN/m for static conditions and 90 kN/m for seismic conditions.

Therefore, the seismic condition governs in this case. Use an ultimate additional shear resistance value of 90 kN/m of wall length for design purposes, (computer output shown on pages 194-198).

**Step 2: B.) Perform slope stability analyses on slope in front of micropile structure to verify stability of unreinforced slope.**

Performance of this additional stability analysis will determine the following:

1.) Stability of existing unstabilized slope in front of micropile structure, and

2.) The need to reduce any passive resistance provided by the existing downslope soil. *(Commentary: If the required factors of safety are not satisfied for the existing lower slope, the soil may continue to creep downslope away from the micropile structure. The designer must then neglect a portion or all of the passive resistance provided by the existing lower slope. This can be performed by remodeling the geometry of the lower slope to obtain the required factors of safety).*

Run No.3 was performed to analyze the existing unstabilized lower slope. For this example problem, the analysis verified a $F.S._{static} = 1.46$ and $F.S._{seismic} = 1.21$ for the existing downslope conditions. Therefore, full passive resistance is provided and can be utilized for the micropile structures analysis. (Computer output shown on pages 199-209).

XSTABL
Slope Stability Analysis
using the
Method of Slices

Interactive Software Designs, Inc.
Moscow, ID 83843, U.S.A.



Ver. 5.001 94 Ä 1231

Problem Description: EXAMPLE 2 (EXISTING-STATIC)

SEGMENT BOUNDARY COORDINATES

9 surface boundary segments

| Segment No. | x-left (m) | y-left (m) | x-right (m) | y-right (m) | Soil Unit Below Segment |
|---|---|---|---|---|---|
| 1 | .0 | 6.1 | 6.1 | 6.4 | 1 |
| 2 | 6.1 | 6.4 | 9.1 | 7.0 | 1 |
| 3 | 9.1 | 7.0 | 12.5 | 9.1 | 1 |
| 4 | 12.5 | 9.1 | 15.9 | 12.2 | 1 |
| 5 | 15.9 | 12.2 | 23.8 | 12.8 | 1 |
| 6 | 23.8 | 12.8 | 24.7 | 12.2 | 1 |
| 7 | 24.7 | 12.2 | 25.9 | 12.2 | 1 |
| 8 | 25.9 | 12.2 | 28.0 | 15.9 | 1 |
| 9 | 28.0 | 15.9 | 58.2 | 26.5 | 1 |

ISOTROPIC SOIL PARAMETERS

1 soil unit(s) specified

| Soil Unit No. | Unit Moist (kN/m3) | Weight Sat. (kN/m3) | Cohesion Intercept (kPa) | Friction Angle (deg) | Pore Parameter Ru | Pressure Constant (kPa) | Water Surface No. |
|---|---|---|---|---|---|---|---|

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 1 | | | | | | | |
| | 18.0 | 19.6 | 3.0 | 28.00 | .000 | .0 | 1 |

1 Water surface(s) have been specified

Unit weight of water = 9.81 (kN/m3)

Water Surface No. 1 specified by 7 coordinate points

PHREATIC SURFACE

| Point No. | x-water (m) | y-water (m) |
|---|---|---|
| 1 | .00 | 5.00 |
| 2 | 6.10 | 5.50 |
| 3 | 9.10 | 6.00 |
| 4 | 16.80 | 11.90 |
| 5 | 25.90 | 12.00 |
| 6 | 29.60 | 15.00 |
| 7 | 58.20 | 24.00 |

BOUNDARY LOADS

1 load(s) specified

| Load No. | x-left (m) | x-right (m) | Intensity (kPa) | Direction (deg) |
|---|---|---|---|---|
| 1 | 16.2 | 23.5 | 9.6 | .0 |

NOTE- Intensity is specified as a uniformly distributed force acting on a horizontally projected surface.

BOUNDARIES THAT LIMIT SURFACE GENERATION HAVE BEEN SPECIFIED

LOWER limiting boundary of 6 segments:

| Segment No. | x-left (m) | y-left (m) | x-right (m) | y-right (m) |
|---|---|---|---|---|
| 1 | .0 | 6.1 | 6.1 | 6.4 |
| 3 | 6.1 | 6.4 | 18.3 | 7.0 |
| 4 | 18.3 | 7.0 | 41.8 | 9.8 |
| 5 | 41.8 | 9.8 | 47.3 | 11.4 |
| 6 | 47.3 | 11.4 | 53.3 | 24.4 |
| 7 | 53.3 | 24.4 | 58.2 | 26.5 |

A SINGLE FAILURE SURFACE HAS BEEN SPECIFIED FOR ANALYSIS

Trial failure surface specified by the following 5 coordinate points:

| Point No. | x-surf (m) | y-surf (m) |
|---|---|---|
| 1 | 6.10 | 6.40 |
| 2 | 12.20 | 7.01 |
| 3 | 16.80 | 8.54 |
| 4 | 19.80 | 10.67 |
| 5 | 21.34 | 12.61 |

SELECTED METHOD OF ANALYSIS: Simplified Janbu

SUMMARY OF INDIVIDUAL SLICE INFORMATION

| Slice | x-base (m) | y-base (m) | height (m) | width (m) | alpha | beta | weight (kN) |
|---|---|---|---|---|---|---|---|

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 1 | 7.60 | 6.55 | .15 | 3.00 | 5.71 | 11.31 | 8.1 |
| 2 | 9.63 | 6.75 | .57 | 1.05 | 5.71 | 31.70 | 10.8 |
| 3 | 11.18 | 6.91 | 1.37 | 2.05 | 5.71 | 31.70 | 52.9 |
| 4 | 12.35 | 7.06 | 1.95 | .30 | 18.40 | 31.70 | 11.2 |
| 5 | 14.20 | 7.68 | 2.97 | 3.40 | 18.40 | 42.36 | 194.2 |
| 6 | 16.05 | 8.29 | 3.92 | .30 | 18.40 | 4.34 | 22.6 |
| 7 | 16.50 | 8.44 | 3.81 | .60 | 18.40 | 4.34 | 44.2 |
| 8 | 18.30 | 9.60 | 2.78 | 3.00 | 35.37 | 4.34 | 161.1 |
| 9 | 20.30 | 11.31 | 1.23 | 1.01 | 51.60 | 4.34 | 23.3 |
| 10 | 21.07 | 12.28 | .31 | .53 | 51.60 | 4.34 | 3.0 |

SLICE INFORMATION .....continued:

| Slice | Sigma (kPa) | c'-value (kPa) | phi | U-base (kN) | U-top (kN) | Q-top (kN) | Delta |
|---|---|---|---|---|---|---|---|
| 1 | 2.3 | 3.0 | 28.00 | .0 | .0 | .0 | .00 |
| 2 | 9.5 | 3.0 | 28.00 | .0 | .0 | .0 | .00 |
| 3 | 20.2 | 3.0 | 28.00 | 8.7 | .0 | .0 | .00 |
| 4 | 23.3 | 3.0 | 28.00 | 2.8 | .0 | .0 | .00 |
| 5 | 35.9 | 3.0 | 28.00 | 49.4 | .0 | .0 | .00 |
| 6 | 47.3 | 3.0 | 28.00 | 5.9 | .0 | .0 | .00 |
| 7 | 52.9 | 3.0 | 28.00 | 12.6 | .0 | 5.8 | .00 |
| 8 | 27.9 | 3.0 | 28.00 | 83.4 | .0 | 28.8 | .00 |
| 9 | 13.6 | 3.0 | 28.00 | 10.1 | .0 | 9.7 | .00 |
| 10 | 6.8 | 3.0 | 28.00 | .0 | .0 | 5.1 | .00 |

For the single specified surface, corrected JANBU factor of safety = 1.046 (Fo factor = 1.052)

Resisting Shear Strength = 250.03E+00 kN

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Slope Stability Analysis
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Problem Description: EXAMPLE 2 (STABILIZED-SEISMIC)

SEGMENT BOUNDARY COORDINATES

9 surface boundary segments,

| Segment No. | x-left (m) | y-left (m) | x-right (m) | y-right (m) | Soil Unit Below Segment |
|---|---|---|---|---|---|
| 1 | .0 | 6.1 | 6.1 | 6.4 | 1 |
| 2 | 6.1 | 6.4 | 9.1 | 7.0 | 1 |
| 3 | 9.1 | 7.0 | 12.5 | 9.1 | 1 |
| 4 | 2.5 | 9.1 | 15.9 | 12.2 | 1 |
| 5 | 5.9 | 12.2 | 23.8 | 12.8 | 1 |
| 6 | 23.8 | 12.8 | 24.7 | 12.2 | 1 |
| 7 | 24.7 | 12.2 | 25.9 | 12.2 | 1 |
| 8 | 25.9 | 12.2 | 28.0 | 15.9 | 1 |
| 9 | 28.0 | 15.9 | 58.2 | 26.5 | 1 |

1 SUBSURFACE boundary segments

| Segment No. | x-left (m) | y-left (m) | x-right (m) | y-right (m) | Soil Unit Below Segment |
|---|---|---|---|---|---|
| 1 | 15.0 | 1.9 | 16.0 | 9.0 | 2 |

ISOTROPIC Soil Parameters

2 soil unit(s) specified

| Soil Unit No. | Unit Moist (kN/m3) | Weight Sat. (KN/m3) | Cohesion Intercept (kPa) | Friction Angle (deg) | Pore Parameter Ru | Pressure Constant (kPa) | Water Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 18.0 | 19.6 | 3.0 | 28.00 | .000 | .0 | 1 |
| 2 | 18.0 | 19.6 | 90.0 | 28.00 | .000 | .0 | 1 |

1 Water surface(s) have been specified

Unit weight of water = 9.81 (kN/m3)

Water Surface No. 1 specified by 7 coordinate points

PHREATIC SURFACE

| Point No. | x-water (m) | y-water (m) |
|---|---|---|
| 1 | .00 | 5.00 |
| 2 | 6.10 | 5.50 |
| 3 | 9.10 | 6.00 |
| 4 | 16.80 | 11.90 |
| 5 | 25.90 | 12.00 |
| 6 | 29.60 | 15.00 |
| 7 | 58.20 | 24.00 |

A horizontal earthquake loading coefficient of .120 has been assigned.

A vertical earthquake loading coefficient of .000 has been assigned.

BOUNDARY LOADS

1 load(s) specified

| Load No. | x-left (m) | x-right (m) | Intensity (kPa) | Direction (deg) |
|---|---|---|---|---|
| 1 | 16.2 | 23.5 | 9.6 | .0 |

NOTE - Intensity is specified as a uniformly distributed force acting on a HORIZONTALLY projected surface.

BOUNDARIES THAT LIMIT SURFACE GENERATION HAVE BEEN SPECIFIED

LOWER limiting boundary of 6 segments:

| Segment No. | x-left (m) | y-left (m) | x-right (m) | y-right (m) |
|---|---|---|---|---|
| 1 | .0 | 6.1 | 6.1 | 6.4 |
| 2 | 6.1 | 6.4 | 18.3 | 7.0 |
| 3 | 18.3 | 7.0 | 41.8 | 9.8 |
| 4 | 41.8 | 9.8 | 47.3 | 11.4 |
| 5 | 47.3 | 11.4 | 53.3 | 24.4 |
| 6 | 53.3 | 24.4 | 58.2 | 26.5 |

A SINGLE FAILURE SURFACE HAS BEEN SPECIFIED FOR ANALYSIS

Trial failure surface specified by the following 5 coordinate points:

| Point No. | x-surf (m) | y-surf (m) |
|---|---|---|
| 1 | 6.10 | 6.40 |
| 2 | 12.20 | 7.01 |
| 3 | 16.80 | 8.54 |
| 4 | 19.80 | 10.67 |
| 5 | 21.34 | 12.61 |

SELECTED METHOD OF ANALYSIS: Simplified Janbu

SUMMARY OF INDIVIDUAL SLICE INFORMATION

| Slice | x-base (m) | y-base (m) | height (m) | width (m) | alpha | beta | weight (kN) |
|---|---|---|---|---|---|---|---|
| 1 | 7.60 | 6.55 | .15 | 3.00 | 5.71 | 11.31 | 8.1 |
| 2 | 9.63 | 6.75 | .57 | 1.05 | 5.71 | 31.70 | 10.8 |
| 3 | 11.18 | 6.91 | 1.37 | 2.05 | 5.71 | 31.70 | 52.9 |
| 4 | 12.35 | 7.06 | 1.95 | .30 | 18.40 | 31.70 | 11.2 |
| 5 | 13.75 | 7.53 | 2.71 | 2.50 | 18.40 | 42.36 | 130.3 |
| 6 | 15.45 | 8.09 | 3.70 | .90 | 18.40 | 42.36 | 63.9 |
| 7 | 15.95 | 8.26 | 3.95 | .10 | 18.40 | 4.34 | 7.6 |
| 8 | 16.10 | 8.31 | 3.91 | .20 | 18.40 | 4.34 | 15.0 |
| 9 | 16.50 | 8.44 | 3.81 | .60 | 18.40 | 4.34 | 44.2 |
| 10 | 18.30 | 9.60 | 2.78 | 3.00 | 35.37 | 4.34 | 161.1 |
| 11 | 20.30 | 11.31 | 1.23 | 1.01 | 51.60 | 4.34 | 23.3 |
| 12 | 21.07 | 12.28 | .31 | .53 | 51.60 | 4.34 | 3.0 |

SLICE INFORMATION .....continued:

| Slice | Sigma (kPa) | c-value (kPa) | phi | U-base (kN) | U-top (kN) | Q-top (kN) | Delta |
|---|---|---|---|---|---|---|---|
| 1 | 2.3 | 3.0 | 28.00 | .0 | .0 | .0 | .00 |
| 2 | 9.5 | 3.0 | 28.00 | .0 | .0 | .0 | .00 |
| 3 | 20.3 | 3.0 | 28.00 | 8.7 | .0 | .0 | .00 |
| 4 | 23.6 | 3.0 | 28.00 | 2.8 | .0 | .0 | .00 |
| 5 | 33.1 | 3.0 | 28.00 | 33.2 | .0 | .0 | .00 |
| 6 | 22.0 | 90.0 | 28.00 | 16.3 | .0 | .0 | .00 |
| 7 | 24.9 | 90.0 | 28.00 | 1.9 | .0 | .0 | .00 |
| 8 | 47.5 | 3.0 | 28.00 | 4.0 | .0 | .0 | .00 |
| 9 | 53.4 | 3.0 | 28.00 | 12.6 | .0 | 5.8 | .00 |
| 10 | 28.5 | 3.0 | 28.00 | 83.4 | .0 | 28.8 | .00 |
| 11 | 14.0 | 3.0 | 28.00 | 10.1 | .0 | 9.7 | .00 |
| 12 | 7.2 | 3.0 | 28.00 | .0 | .0 | 5.1 | .00 |

For the single specified surface, corrected JANBU factor of safety =1.115 (Fo factor =1.052)

Resisting Shear Strength = 331.52E+00 kN

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Problem Description: EXAMPLE 2 (FRONT SLOPE)

SEGMENT BOUNDARY COORDINATES

9 surface boundary segments

| Segment No. | x-left (m) | y-left (m) | x-right (m) | y-right (m) | Soil Unit Below Segment |
|---|---|---|---|---|---|
| 1 | .0 | 6.1 | 6.1 | 6.4 | 1 |
| 2 | 6.1 | 6.4 | 9.1 | 7.0 | 1 |
| 3 | 9.1 | 7.0 | 12.5 | 9.1 | 1 |
| 4 | 12.5 | 9.1 | 15.9 | 12.2 | 1 |
| 5 | 15.9 | 12.2 | 23.8 | 12.8 | 1 |
| 6 | 23.8 | 12.8 | 24.7 | 12.2 | 1 |
| 7 | 24.7 | 12.2 | 25.9 | 12.2 | 1 |
| 8 | 25.9 | 12.2 | 28.0 | 15.9 | 1 |
| 9 | 28.0 | 15.9 | 58.2 | 26.5 | 1 |

1 SUBSURFACE boundary segments

| Segment No. | x-left (m) | y-left (m) | x-right (m) | y-right (m) | Soil Unit Below Segment |
|---|---|---|---|---|---|

| | | | | | |
|---|---|---|---|---|---|
| 1 | 15.0 | 9.0 | 16.0 | 9.0 | 2 |

ISOTROPIC Soil Parameters

2 soil unit(s) specified

| Soil Unit No. | Unit Moist (kN/m3) | Weight Sat. (kN/m3) | Cohesion Intercept (kPa) | Friction Angle (deg) | Pore Parameter Ru | Pressure Constant (kPa) | Water Surface No. |
|---|---|---|---|---|---|---|---|
| 1 | 18.0 | 19.6 | 3.0 | 28.00 | .000 | .0 | 1 |
| 2 | 18.0 | 19.6 | 100.0 | 28.00 | .000 | .0 | 1 |

1 Water surface(s) have been specified

Unit weight of water = 9.81 (kN/m3)

Water Surface No. 1 specified by 7 coordinate points

PHREATIC SURFACE,

| Point No. | x-water (m) | y-water (m) |
|---|---|---|
| 1 | .00 | 5.00 |
| 2 | 6.10 | 5.50 |
| 3 | 9.10 | 6.00 |
| 4 | 16.80 | 11.90 |
| 5 | 25.90 | 12.00 |
| 6 | 29.60 | 15.00 |
| 7 | 58.20 | 24.00 |

A horizontal earthquake loading coefficient of .120 has been assigned.

A vertical earthquake loading coefficient of .000 has been assigned.

BOUNDARY LOADS

1 load(s) specified

| Load No. | x-left (m) | x-right (m) | Intensity (kPa) | Direction (deg) |
|---|---|---|---|---|
| 1 | 16.2 | 23.5 | 9.6 | .0 |

NOTE - Intensity is specified as a uniformly distributed force acting on a HORIZONTALLY projected surface.

BOUNDARIES THAT LIMIT SURFACE GENERATION HAVE BEEN SPECIFIED

LOWER limiting boundary of 6 segments:

| Segment No. | x-left (m) | y-left (m) | x-right (m) | y-right (m) |
|---|---|---|---|---|
| 1 | .0 | 6.1 | 6.1 | 6.4 |
| 2 | 6.1 | 6.4 | 18.3 | 7.0 |
| 3 | 18.3 | 7.0 | 41.8 | 9.8 |
| 4. | 41.8 | 9.8 | 47.3 | 11.4 |
| 5 | 47.3 | 11.4 | 53.3 | 24.4 |
| 6 | 53.3 | 24.4 | 58.2 | 26.5 |

A critical failure surface searching method, using a random technique for generating CIRCULAR surfaces has been specified.

25 trial surfaces will be generated and analyzed.

5 Subsurfaces initiate from each of 5 points equally spaced along the ground surface between x = 9.1 m and x =12.5 m

Each surface terminates between x = 13.0 m
and x =15.0 m

Unless further limitations were imposed, the minimum elevation at which a surface extends is y = 7.0 m

.5 m line segments define each trial failure surface.

ANGULAR RESTRICTIONS

The first segment of each failure surface will be inclined within the angular range defined by:

Lower angular limit: = -45.0 degrees
Upper angular limit: = 45.0 degrees

Factors of safety have been calculated by the:

SIMPLIFIED JANBU METHOD

The 10 most critical of all the failure surfaces examined are displayed below - the most critical first.

Failure surface No. 1 specified by 13 coordinate points

| Point No. | x-surf (m) | y-surf (m) |
|---|---|---|
| 1 | 10.80 | 8.05 |
| 2 | 11.30 | 8.10 |
| 3 | 11.79 | 8.20 |
| 4 | 12.26 | 8.35 |
| 5 | 12.72 | 8.56 |
| 6 | 13.15 | 8.82 |
| 7 | 13.55 | 9.12 |

| | | |
|---|---|---|
| 8 | 13.91 | 9.46 |
| 9 | 14.23 | 9.85 |
| 10 | 14.51 | 10.26 |
| 11 | 14.74 | 10.70 |
| 12 | 14.92 | 11.17 |
| 13 | 14.97 | 11.35 |

Corrected JANBU FOS= 1.145 (Fo factor = 1.063)

Failure surface No. 2 specified by 11 coordinate points

| Point No. | x-surf (m) | y-surf (m) |
|---|---|---|
| 1 | 10.80 | 8.05 |
| 2 | 11.30 | 8.06 |
| 3 | 11.79 | 8.14 |
| 4 | 12.27 | 8.30 |
| 5 | 12.71 | 8.52 |
| 6 | 13.12 | 8.81 |
| 7 | 13.48 | 9.16 |
| 8 | 13.79 | 9.55 |
| 9 | 14.03 | 9.99 |
| 10 | 14.20 | 10.46 |
| 11 | 14.25 | 10.70 |

Corrected JANBU FOS = 1.273 (Fo factor = 1.069)

Failure surface No. 3 specified by 16 coordinate ponts

| Point No. | x-surf (m) | y-surf (m) |
|---|---|---|
| 1 | 9.10 | 7.00 |
| 2 | 9.59 | 7.10 |
| 3 | 10.07 | 7.24 |
| 4 | 10.54 | 7.40 |
| 5 | 11.00 | 7.59 |
| 6 | 11.45 | 7.82 |
| 7 | 11.88 | 8.07 |
| 8 | 12.30 | 8.34 |
| 9 | 12.70 | 8.65 |
| 10 | 13.08 | 8.97 |
| 11 | 13.43 | 9.33 |
| 12 | 13.77 | 9.70 |
| 13 | 14.08 | 10.09 |
| 14 | 14.36 | 10.50 |
| 15 | 14.62 | 10.93 |
| 16 | 14.71 | 11.12 |

Corrected JANBU FOS = 1.328 (Fo factor = 1.049)

Failure surface No. 4 specified by 13 coordinate points

| Point No. | x-surf (m) | y-surf (m) |
|---|---|---|
| 1 | 9.10 | 7.00 |
| 2 | 9.60 | 7.04 |
| 3 | 10.09 | 7.13 |

| | | |
|---|---|---|
| 4 | 10.57 | 7.26 |
| 5 | 11.04 | 7.44 |
| 6 | 11.48 | 7.67 |
| 7 | 11.91 | 7.93 |
| 8 | 12.30 | 8.24 |
| 9 | 12.66 | 8.59 |
| 10 | 12.99 | 8.97 |
| 11 | 13.28 | 9.38 |
| 12 | 13.52 | 9.81 |
| 13 | 13.69 | 10.18 |

Corrected JANBU FOS = 1.334 (Fo factor = 1.059)

Failure surface No. 5 specified by 8 coordinate points

| Point No. | x-surf (m) | y-surf (m) |
|---|---|---|
| 1 | 12.50 | 9.10 |
| 2 | 12.98 | 9.25 |
| 3 | 13.43 | 9.46 |
| 4 | 13.84 | 9.74 |
| 5 | 14.21 | 10.08 |
| 6 | 14.52 | 10.47 |
| 7 | 14.77 | 10.91 |
| 8 | 14.93 | 11.32 |

Corrected JANBU FOS = 1.435 (Fo factor = 1.053)

Failure surface No. 6 specified by 8 coordinate points

| Point No. | x-surf (m) | y-surf (m) |
|---|---|---|
| 1 | 12.50 | 9.10 |
| 2 | 12.99 | 9.22 |
| 3 | 13.44 | 9.43 |
| 4 | 13.85 | 9.71 |
| 5 | 14.20 | 10.07 |
| 6 | 14.48 | 10.48 |
| 7 | 14.68 | 10.94 |
| 8 | 14.73 | 11.13 |

Corrected JANBU FOS = 1.450 (Fo factor = 1.060)

Failure surface No. 7 specified by 9 coordinate points

| Point No. | x-surf (m) | y-surf (m) |
|---|---|---|
| 1 | 10.80 | 8.05 |
| 2 | 11.30 | 8.04 |
| 3 | 11.79 | 8.13 |
| 4 | 12.26 | 8.31 |
| 5 | 12.68 | 8.58 |
| 6 | 13.05 | 8.92 |
| 7 | 13.35 | 9.32 |
| 8 | 13.56 | 9.77 |
| 9 | 13.66 | 10.16 |

Corrected JANBU FOS = 1.504 (Fo factor = 1.071)

Failure curface No. 8 specified by 7 coordinate points

| Point No. | x-surf (m) | y-surf (m) |
|---|---|---|
| 1 | 12.50 | 9.10 |
| 2 | 12.99 | 9.20 |
| 3 | 13.44 | 9.41 |
| 4 | 13.84 | 9.72 |
| 5 | 14.15 | 10.11 |
| 6 | 14.37 | 10.56 |
| 7 | 14.43 | 10.86 |

Corrected JANBU FOS = 1.552 (Fo factor = 1.065)

Failure surface No. 9 specified by 11 coordinate points

| Point No. | x-surf (m) | y-surf (m) |
|---|---|---|
| 1 | 11.65 | 8.58 |
| 2 | 12.12 | 8.73 |
| 3 | 12.58 | 8.93 |
| 4 | 13.02 | 9.18 |
| 5 | 13.43 | 9.46 |
| 6 | 13.81 | 9.78 |
| 7 | 14.17 | 10.14 |
| 8 | 14.48 | 10.52 |
| 9 | 14.76 | 10.94 |
| 10 | 15.00 | 11.38 |
| 11 | 15.00 | 11.38 |

Corrected JANBU FOS = 1.557 (Fo factor = 1.045)

Failure surface No. 10 specified by 7 coordinate points

| Point No. | x-surf (m) | y-surf (m) |
|---|---|---|
| 1 | 11.65 | 8.58 |
| 2 | 12.15 | 8.59 |
| 3 | 12.63 | 8.75 |
| 4 | 13.04 | 9.03 |
| 5 | 13.37 | 9.41 |
| 6 | 13.57 | 9.86 |
| 7 | 13.61 | 10.12 |

Corrected JANBU FOS = 1.810 (Fo factor = 1.072)

The following is a summary of the TEN most critical surfaces

Problem Description: EXAMPLE 2 (FRONT SLOPE)

| | Modified JANBU FOS | Correction Factor | Initial x-coord (m) | Terminal x-coord (m) | Available Strength (kN) |
|---|---|---|---|---|---|
| 1. | 1.145 | 1.063 | 10.80 | 14.97 | 3.381E+01 |
| 2. | 1.273 | 1.069 | 10.80 | 14.25 | 2.792E+01 |
| 3. | 1.328 | 1.049 | 9.10 | 14.71 | 4.082E+01 |
| 4. | 1.334 | 1.059 | 9.10 | 13.69 | 3.627E+01 |
| 5. | 1.435 | 1.053 | 12.50 | 14.93 | 1.668E+01 |
| 6. | 1.450 | 1.060 | 12.50 | 14.73 | 1.584E+01 |
| 7. | 1.504 | 1.071 | 10.80 | 13.66 | 2.188E+01 |
| 8. | 1.552 | 1.065 | 12.50 | 14.43 | 1.372E+01 |

| | | | | | |
|---|---|---|---|---|---|
| 9. | 1.557 | 1.045 | 11.65 | 15.00 | 2.057E+01 |
| 10. | 1.810 | 1.072 | 11.65 | 13.61 | 1.354E+01 |

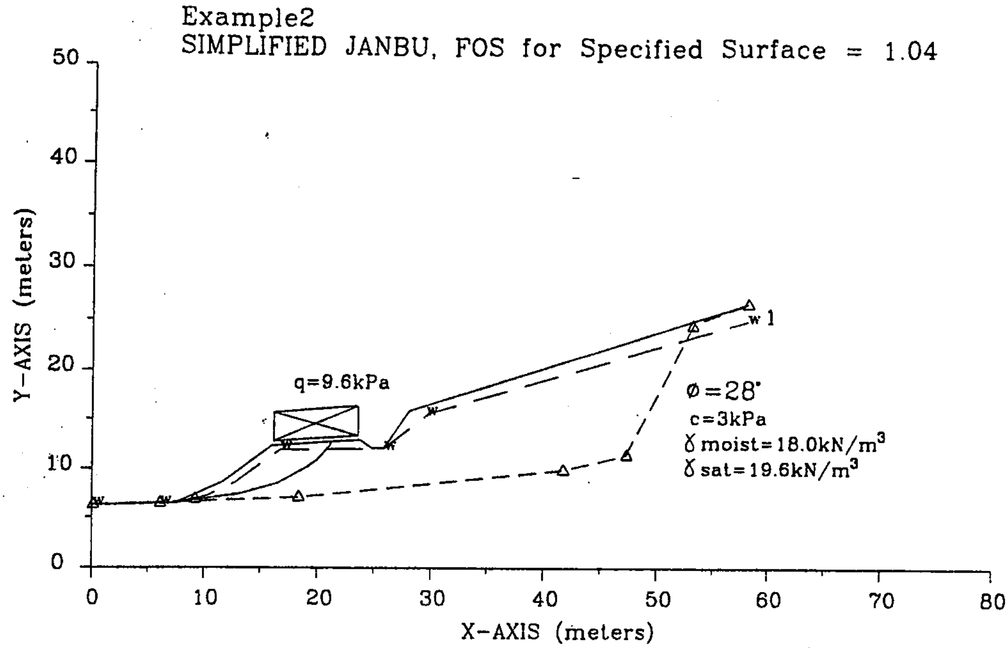



**Figure 6-17.** **Sample Problem No. 2, slope stability results (static-existing condition)**

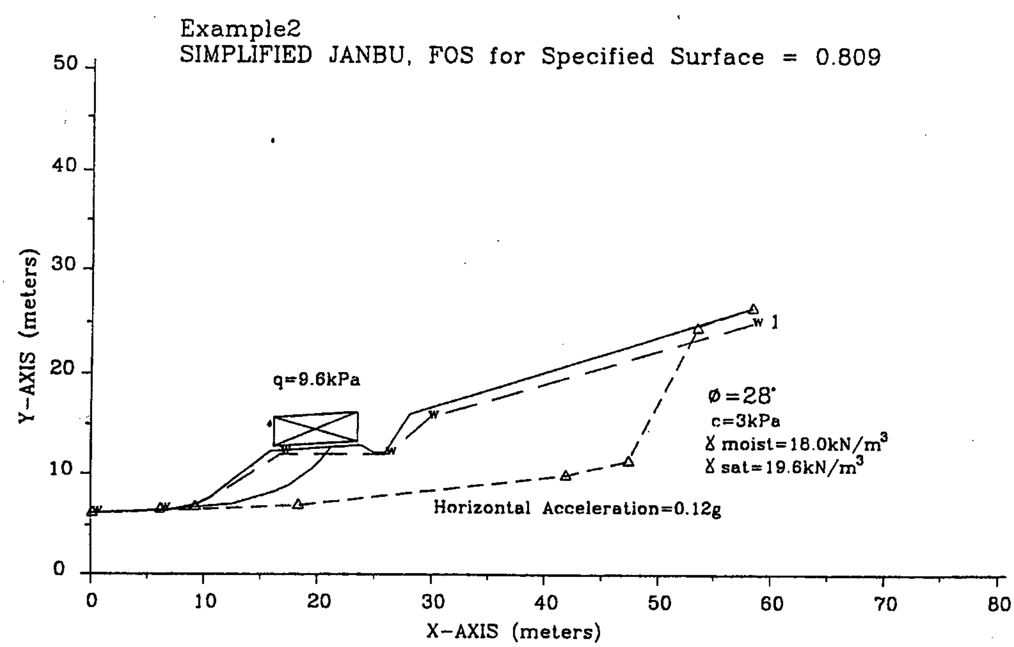



**Figure 6-18.** **Sample Problem No. 2, slope stability results (seismic-existing condition)**

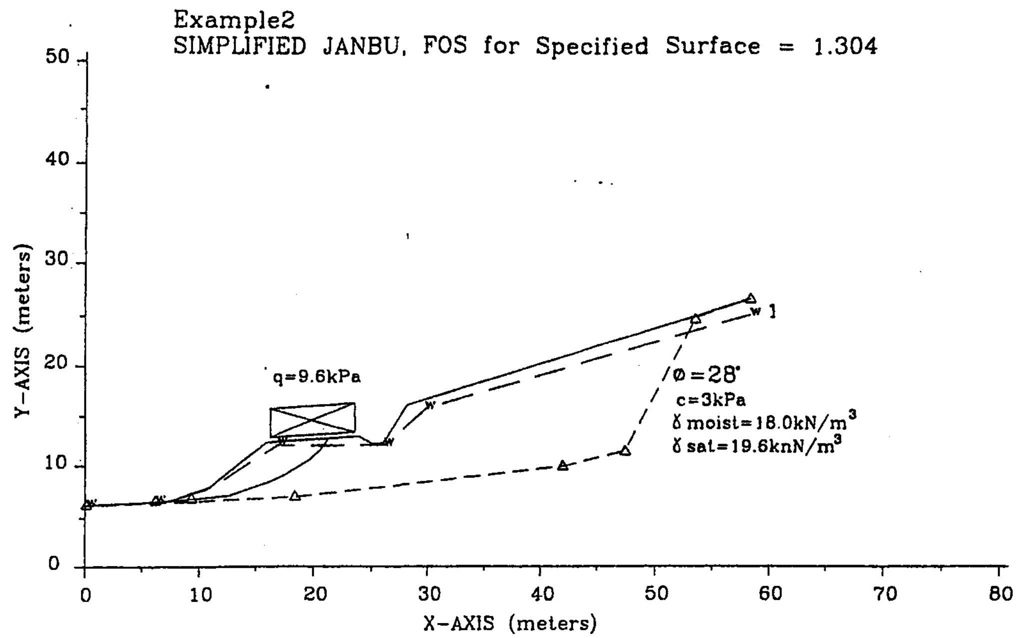



**Figure 6-19.** **Sample Problem No. 2, slope stability results (static-stabilized condition)**

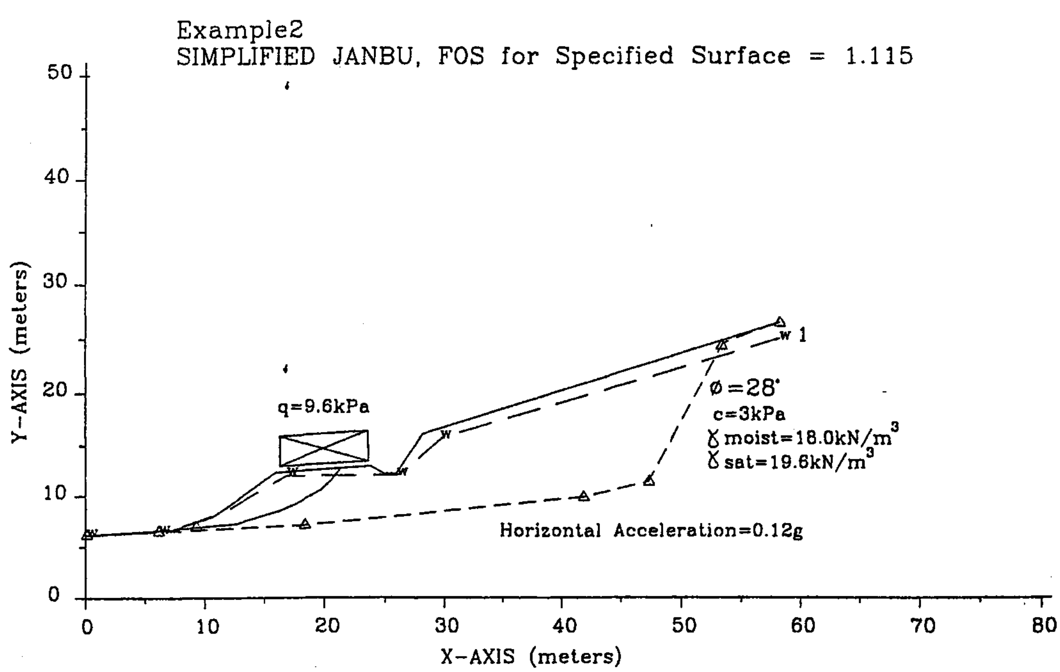



**Figure 6-20.** **Sample Problem No. 2, slope stability results (seismic-stabilized condition)**

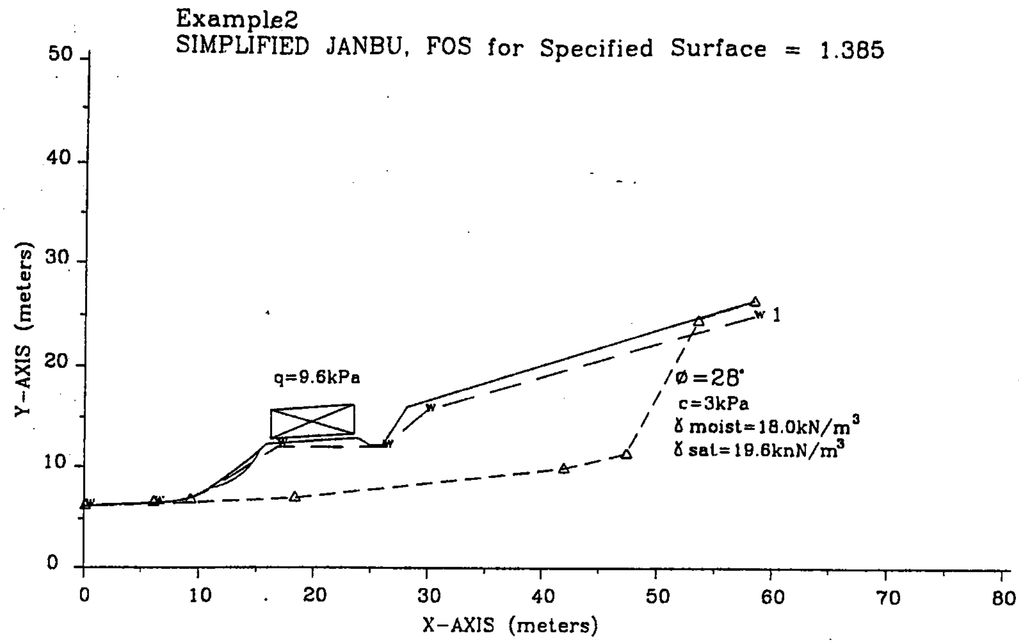



**Figure 6-21.** **Sample Problem No. 2, slope stability results (static-front slope condition)**

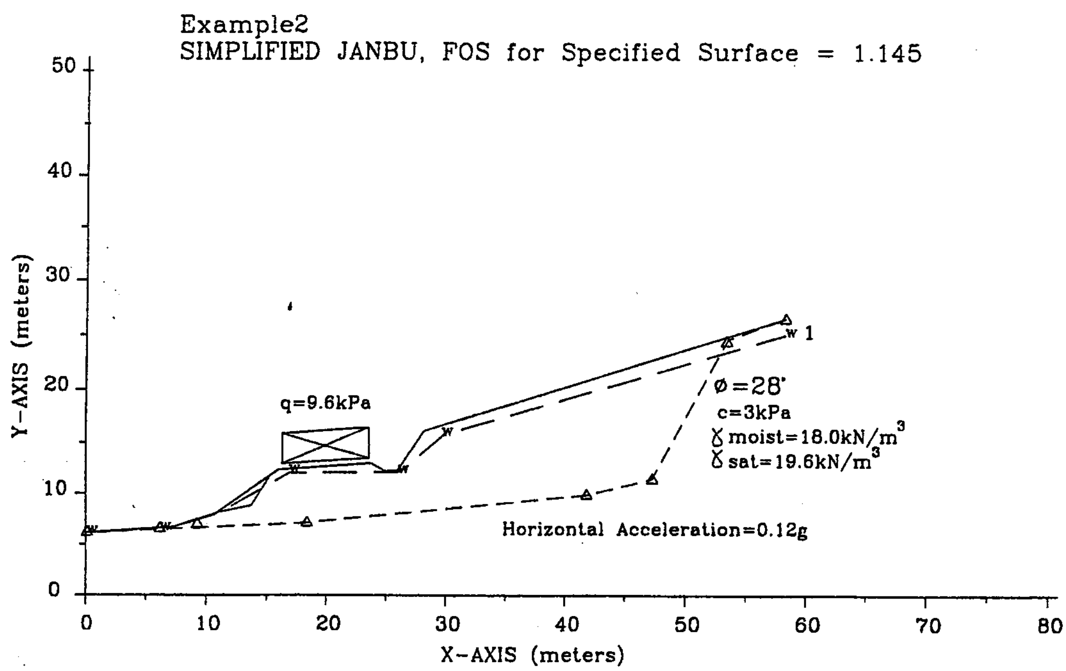



**Figure 6-22.** **Sample Problem No. 2, slope stability results (seismic-front slope condition)**

**Step 3: A. Evaluate the ultimate bending capacity and flexural rigidity (EI) of the proposed micropile using the COM624P Ultimate Bending Analysis computer program.**

Assume 102 mm diameter, 9.5 mm wall thickness API N-80 pipe casing in 178 mm diameter drilled hole.

**Pile Dimensions**

Casing outside diameter - $od_{casing} = 102 \cdot mm$

Casing wall thickness - $t_{wall} = 9.5 \cdot mm$

Casing inside diameter - $id_{casing} = od_{casing} - 2 \cdot t_{wall} = 83 \ mm$

Thickness for sacrificial steel- (corrosion loss) $t_{sacrifice} = 1.5 \cdot mm$

Steel area - $Area_{steel} = \frac{\pi}{4} \cdot \left[ (od_{casing} - 2 \cdot t_{sacrifice})^2 - id_{casing}^2 \right]$

$Area_{steel} = 2.286 \cdot 10^3 \cdot mm^2$

Grout area- $Area_{grout} = \frac{\pi}{4} \cdot id_{casing}^2$ $\quad Area_{grout} = 5.41 \cdot 10^3 \cdot mm^2$

Total section area - $Area_{pile} = \frac{\pi}{4} \cdot (od_{casing} - 2 \cdot t_{sacrifice})^2$ $\quad Area_{pile} = 7.694 \cdot 10^3 \ mm^2$

**Casing Section Properties**

Casing steel yield strength - $F_y casing = 552000 \cdot \frac{kN}{m^2}$

Casing section modulus - $S_{casing} = 0.098175 \cdot \left[ \frac{(od_{casing} - 2 \cdot t_{sacrifice})^4 - id_{casing}^4}{od_{casing} - t_{sacrifice}} \right]$

$S_{casing} = 4.82 \cdot 10^4 \cdot mm^3$

Casing plastic modulus - $Z_{casing} = \frac{(od_{casing} - 2 \cdot t_{sacrifice})^3}{6} - \frac{id_{casing}^3}{6}$

$Z_{casing} = 6.64 \cdot 10^4 \cdot mm^3$

Casing yield moment - $M_{yield} = F_{y_{casing}} \cdot S_{casing}$ $M_{yield} = 26.6 \cdot kN \cdot m$

Casing plastic moment - $M_{plastic} = F_{y_{casing}} \cdot Z_{casing}$ $M_{plastic} = 36.7 \cdot kN \cdot m$

**Composite Section Properties**

Material Modulus of elasticity - $E_{grout} = 2.48 \cdot 10^7 \cdot \frac{kN}{m^2}$ $E_{steel} = 2.00 \cdot 10^8 \cdot \frac{kN}{m^2}$

Grout Compressive Strength - $f'_{c\ grout} = 27{,}600\ kN/m^2$

The flexural behavior of the composite micropile subjected to bending is dependent upon its flexural rigidity, EI, where E is the modulus of elasticity of the composite micropile material and I is the moment of inertia of the cross section about the axis of bending. In general, the flexural rigidity of a micropile, consisting of a cement grout filled pipe section, varies nonlinearly with the applied bending moment. The EI of the composite micropile section will experience a significant change when cracking of the cement grout occurs. COM624P uses a rigorous approach to control the use of cracked EI values during computations. Micropile bending stiffness is provided as a function of applied moment. The COM624P analysis allows the selection of a more representative cracked EI value providing a more accurate determination of the ultimate lateral load capacity for a given micropile cross-section.

From COM624P Ultimate Bending Analysis (shown on pages 216-219), for $M = M_{yield}$, determine the flexural rigidity (EI) of the pile section:

$$EI = 597\ kN \cdot m^2$$

<u>For input into COM624P lateral analysis =</u>

$$E_{pile} = \frac{Area_{steel} \cdot E_{steel} + Area_{grout} \cdot E_{grout}}{Area_{pile}}$$

$$E_{pile} = 7.69 \cdot 10^7 \cdot \frac{kN}{m^2}$$

<u>Solve for I =</u>

$$I_{pile} = \frac{EI}{E_{pile}} = 7.77 \cdot 10^{-6} \cdot m^4$$

Sample Problem No. 2 - Case 1 Non-Reticulated Micropile Slope Stabilization

*************************************************************

ULTIMATE BENDING RESISTANCE AND FLEXURIAL RIGIDITY

*************************************************************

DIAMETER = .10 M (INPUT VALUE = 0.099 m)

STEEL SHELL THICKNESS = .01 M (INPUT VALUE=0.008 M)

CONCRETE COMPRESSIVE STRENGTH = 27600.00 KN/ M**2

REBAR YIELD STRENGTH = 414000.00 KN/ M**2

STEEL SHELL OR CORE YIELD STRENGTH = 552000.00 KN/ M**2

MODULUS OF ELASTICITY OF STEEL = 200000000.00 KN/ M**2

COVER THICKNESS = .042 M

NUMBER OF REINFORCING BARS = 1

NUMBER OF ROWS OF REINFORCING BARS = 1

SQUASH LOAD CAPACITY = 1678.67 KN

| ROW NUMBER | AREA OF REINFORCEMENT M**2 | DISTANCE TO CENTROIDAL AXIS M |
|---|---|---|
| 1 | .000000 | .0000 (No Reinforcing Bar) |

OUTPUT RESULTS FOR AN AXIAL LOAD = .00 KN

*******************************************************

| MOMENT M-KN | EI 'KN-M**2 | PHI I/M | MAX STR M/M | N AXIS M |
|---|---|---|---|---|
| .001 | 630.93 | .000001 | .00000 | .050 |
| .032 | 630.92 | .000051 | .00000 | .050 |
| .663 | 630.55 | .001051 | .00004 | .050 |
| 1.292 | 630.18 | .002051 | .00008 | .050 |
| 1.841 | 603.47 | .003051 | .00012 | .048 |
| 2.444 | 603.31 | .004051 | .00015 | .048 |
| 3.046 | 603.15 | .005051 | .00019 | .048 |
| 3.649 | 602.98 | .006051 | .00023 | .048 |
| 4.251 | 602.82 | .007051 | .00027 | .048 |
| 4.852 | 602.66 | .008051 | .00031 | .048 |
| 5.453 | 602.50 | .009051 | .00035 | .048 |
| 6.054 | 602.34 | .010051 | .00038 | .048 |
| 6.655 | 602.18 | .011051 | .00042 | .048 |
| 7.255 | 602.02 | .012051 | .00046 | .048 |
| 7.855 | 601.86 | .013051 | .00050 | .048 |
| 8.454 | 601.70 | .014051 | .00054 | .048 |
| 9.054 | 601.53 | .015051 | .00058 | .048 |
| 9.653 | 601.37 | .016051 | .00061 | .048 |
| 10.251 | 601.21 | .017051 | .00065 | .048 |
| 10.850 | 601.05 | .018051 | .00069 | .048 |
| 11.448 | 600.89 | .019051 | .00073 | .048 |
| 12.045 | 600.73 | .020051 | .00077 | .048 |
| 12.643 | 600.57 | .021051 | .00081 | .048 |
| 13.240 | 600.41 | .022051 | .00084 | .048 |
| 13.836 | 600.24 | .023051 | .00088 | .048 |
| 14.433 | 600.08 | .024051 | .00092 | .048 |
| 15.029 | 599.92 | .025051 | .00096 | .048 |
| 15.624 | 599.76 | .026051 | .00100 | .048 |
| 16.220 | 599.60 | .027051 | .00103 | .048 |
| 16.815 | 599.44 | .028051 | .00107 | .048 |
| 17.410 | 599.28 | .029051 | .00111 | .048 |
| 18.004 | 599.12 | .030051 | .00115 | .048 |
| 18.598 | 598.96 | .031051 | .00119 | .048 |
| 19.192 | 598.79 | .032051 | .00123 | .048 |
| 19.785 | 598.63 | .033051 | .00126 | .048 |
| 20.379 | 598.47 | .034051 | .00130 | .048 |
| 20.971 | 598.31 | .035051 | .00134 | .048 |
| 21.564 | 598.15 | .036051 | .00138 | .048 |
| 22.156 | 597.99 | .037051 | .00142 | .048 |
| 22.748 | 597.83 | .038051 | .00146 | .048 |

| MOMENT M-KN | EI KN-M**2 | PHI I/M | MAX STR M/M | N AXIS M |
|---|---|---|---|---|
| 23.339 | 597.67 | .039051 | .00149 | .048 |
| 23.931 | 597.51 | .040051 | .00153 | .048 |
| 24.522 | 597.34 | .041051 | .00157 | .048 |
| 25.112 | 597.18 | .042051 | .00161 | .048 |
| 25.702 | 597.02 | .043051 | .00165 | .048 |
| 26.292 | 596.86 | .044051 | .00168 | .048 |
| **For Pile $M_{yield}$-26.6 kN.m** | **EI= 597 kN.m$^2$** | | | |
| 26.882 | 596.70 | .045051 | .00172 | .048 |
| 27.471 | 596.54 | .046051 | .00176 | .048 |
| 28.060 | 596.38 | .047051 | .00180 | .048 |
| 28.649 | 596.22 | .048051 | .00184 | .048 |
| 29.237 | 596.06 | .049051 | .00188 | .048 |
| 29.825 | 595.89 | .050051 | .00191 | .048 |
| 30.413 | 595.73 | .051051 | .00195 | .048 |
| 31.000 | 595.57 | .052051 | .00199 | .048 |
| 31.587 | 595.41 | .053051 | .00203 | .048 |
| 32.174 | 595.25 | .054051 | .00207 | .048 |
| 32.748 | 594.86 | .055051 | .00211 | .048 |
| 33.318 | 594.42 | .056051 | .00214 | .048 |
| 33.865 | 593.59 | .057051 | .00218 | .048 |
| 34.410 | 592.75 | .058051 | .00222 | .048 |
| 34.915 | 591.27 | .059051 | .00226 | .048 |
| 35.416 | 589.77 | .060051 | .00230 | .048 |
| 35.874 | 587.61 | .061051 | .00233 | .048 |
| 36.320 | 585.33 | .062051 | .00237 | .048 |
| 36.736 | 582.64 | .063051 | .00241 | .048 |
| 37.122 | 579.56 | .064051 | .00245 | .048 |
| 37.503 | 576.52 | .065051 | .00249 | .048 |
| 37.827 | 572.69 | .066051 | .00253 | .048 |
| 38.149 | 568.96 | .067051 | .00256 | .048 |
| 38.451 | 565.03 | .068051 | .00260 | .048 |
| 38.718 | 560.72 | .069051 | .00264 | .048 |
| 38.986 | 556.53 | .070051 | .00268 | .048 |
| 39.229 | 552.13 | .071051 | .00272 | .048 |
| 39.447 | 547.49 | .072051 | .00276 | .048 |
| 39.665 | 542.97 | .073051 | .00279 | .048 |
| 39.869 | 538.40 | .074051 | .00283 | .048 |
| 40.049 | 533.62 | .075051 | .00287 | .048 |
| 40.229 | 528.97 | .076051 | .00291 | .048 |
| 40.408 | 524.43 | .077051 | .00295 | .048 |

| MOMENT M-KN | EI KN-M**2 | PHI I/M | MAX STR M/M | N AXIS M |
|---|---|---|---|---|
| 40.560 | 519.67 | .078051 | .00298 | .048 |
| 40.710 | 514.98 | .079051 | .00302 | .048 |
| 40.589 | 510.42 | .080051 | .00306 | .048 |
| 41.005 | 505.92 | .081051 | .00310 | .048 |
| 41.129 | 501.27 | .082051 | .00314 | .048 |
| 41.254 | 496.73 | .083051 | .00318 | .048 |
| 41.378 | 492.30 | .084051 | .00321 | .048 |
| 41.501 | 487.96 | .085051 | .00325 | .048 |
| 41.606 | 483.50 | .086051 | .00329 | .048 |
| 41.709 | 479.14 | .087051 | .00333 | .048 |
| 41.812 | 474.87 | .088051 | .00337 | .048 |
| 41.916 | 470.69 | .089051 | .00341 | .048 |
| 42.012 | 466.53 | .090051 | .00344 | .048 |
| 42.097 | 462.35 | .091051 | .00348 | .048 |
| 42.182 | 458.25 | .092051 | .00352 | .048 |
| 42.268 | 454.24 | .093051 | .00356 | .048 |
| 42.353 | 450.32 | .094051 | .00360 | .048 |
| 42.434 | 446.43 | .095051 | .00363 | .048 |
| 42.504 | 442.52 | .096051 | .00367 | .048 |
| 42.574 | 438.68 | .097051 | .00371 | .048 |
| 42.644 | 434.92 | .098051 | .00375 | .048 |
| 42.714 | 431.23 | .099051 | .00379 | .048 |
| 42.784 | 427.62 | .100051 | .00383 | .048 |

The ultimate bending moment at a concrete strain of 0.003 is: .406E+02 M-KN

**Step 3: B. Perform lateral pile capacity analysis:**

Utilize COM624P computer program to model the overburden material above the slide plane as a loose-to-medium-dense silty sand all below the water table. Depth from top of proposed piles to siltstone bedrock is 5.3 m. Worst-case conditions include a depth to slide plane of approximately 5 m. Use COM624P computer program internal p-y curves and evaluate the ultimate lateral resistance a micropile can provide if it is located perpendicular to the slide plane. The siltstone bedrock has been modeled assuming a stiff clay above the water table. This provides the most reasonable p-y curve modeling without specific information. *(Commentary: To achieve the most accurate p-y curves, the designer could recommend additional pressuremeter testing to develop site specific "rock" p-y curves).* In the COM624P analysis, the pile is assumed to act as a free headed pile at the slide surface. For this example, we have assumed the slide plane to occur slightly above the bedrock. Therefore, soil materials immediately above and below the slide plane have similar stiffness and free headed pile conditions exist. For this example, the slide loading (90 kN/m) is modeled as a distributed load, resisted by the pile in a triangular pressure distribution (Figure 6-23). The pile must be designed to provide the required resistance for slope stability and structurally withstand the ultimate load. Pile spacing is assumed to be 1.0 m. Full passive resistance is used in front of micropile structure.

Input parameters for the COM624P lateral pile capacity analysis are developed as follows:

<u>Pile Length</u>---measured from top of pile to bottom of pile. Pile extended 2 m into siltstone bedrock, (7.3 m).

<u>Modulus of Elasticity of Pile ($E_{pile}$)</u>---use value developed from COM624P pile ultimate bending analysis ,($7.69 \times 10^7$ kN/m$^2$).

<u>Pile Diameter-</u>--neglect outer grout material and use diameter of pile after corrosion allowance, (0.099 m).

<u>Moment of Inertia of Pile ($I_{pile}$)</u>---use value developed from COM624P pile ultimate bending analysis, ($7.77 \times 10^{-6}$m$^4$).

<u>Area of Pile ($A_{pile}$)</u>---use value calculated previously ($7.69 \times 10^3$mm$^2$).

Variation of Soil Modulus (k)---use representative value from COM624P Users Manual ($7.69 \times 10^3$ kN/m$^3$).

Results of the COM624P input and analysis output data follow (Computer output shown on pages 222-226 ):

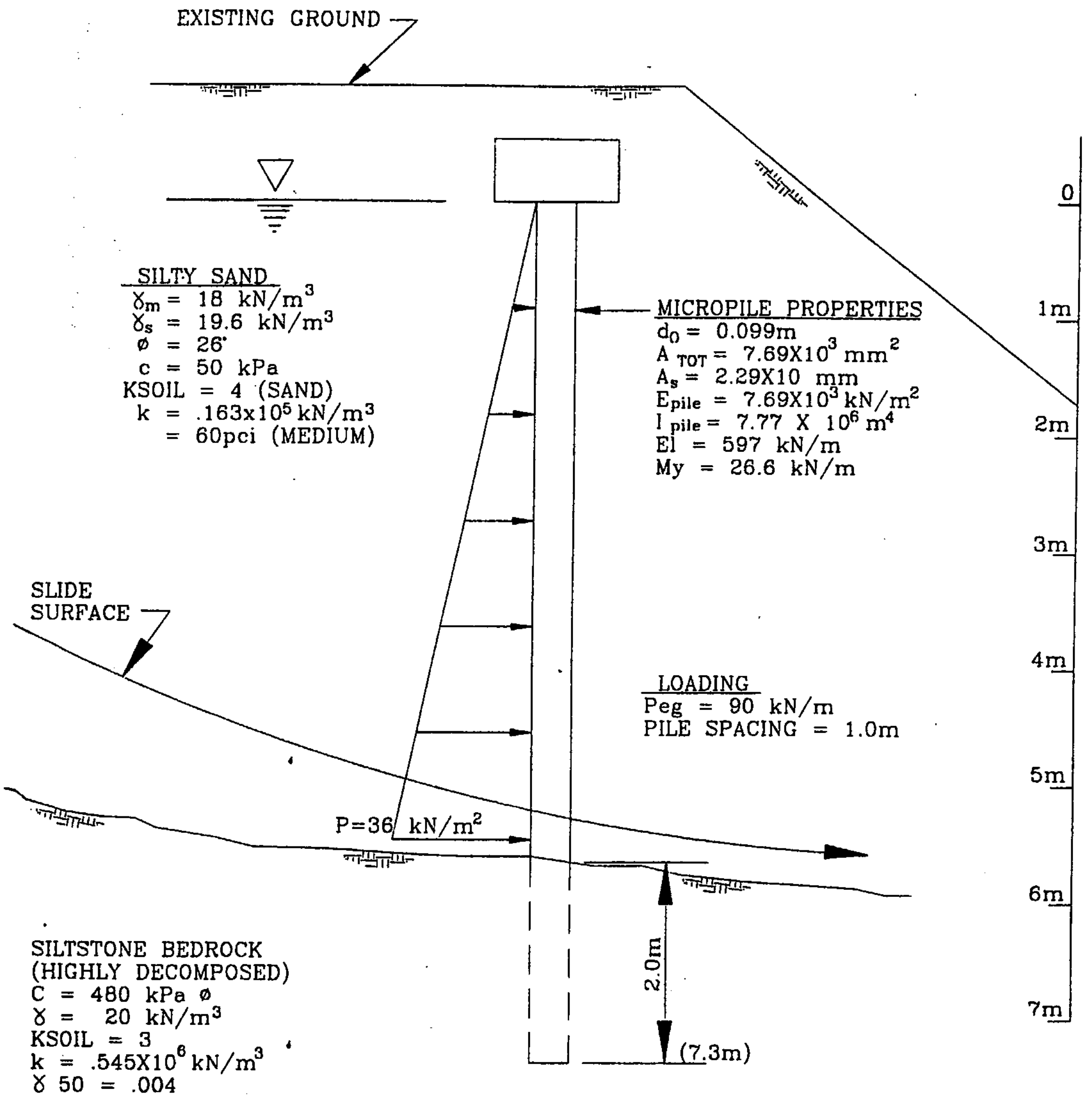



**Figure 6-23** Sample Problem No. 2, lateral pile capacity input values.

SAMPLE PROBLEM NO. 2 CASE 1 NON-RETICULATED MICROPILE SLOPE STABILIZATION

UNITS-METR

*******************************************************************
PILE DEFLECTION, BENDING MOMENT, SHEAR & SOIL RESISTANCE
*******************************************************************

INPUT INFORMATION
*********************

DISTRIBUTED LOAD CURVE 2 POINTS

| X,M | LOAD, kN/M |
|---|---|
| .00 | .000E+00 |
| 5.00 | .360E+02 |

THE LOADING IS STATIC

PILE GEOMETRY AND PROPERTIES

PILE LENGTH = 7.30 M
MODULUS OF ELASTICITY OF PILE = .769E+02 KN/M**2
1 SECTION(S)

| X | DIAMETER | MOMENT OF INERTIA | AREA |
|---|---|---|---|
| M | M | M**4 | M**2 |
| .00 | | | |
| | .099 | .780E-02 | .770E-02 |
| 7.30 | | | |

SOILS INFORMATION

X-COODINATE AT THE GROUND SURFACE = .00 M

SLOPE ANGLE AT THE GROUND SURFACE = .00 DEG.

2 LAYER(S) OF SOIL

LAYER 1
THE LAYER IS A SAND
X ATHT THE TOP OF THE LAYER = .00 M
X AT THE BOTTOM OF THE LAYER = 5.30 M
VARIATION OF SOIL MODULUS,k = .163E+06 KN/M**3

LAYER 2
THE LAYER IS A STIFF CLAY ABOVE THE WATER TABLE
X AT THE TOP OF THE LAYER = 5.30 M
X AT THE BOTTOM OF THE LAYER = 10.00 M
VARIATION OF SOIL MODULUS,k = .543E+06 KN/M**3

DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
4 POINTS

| X.M | WEIGHT, kN/M*3 |
|---|---|
| .00 | .20E+02 |
| 5.00 | .20E+02 |
| 5.30 | .20E+02 |
| 10.00 | .20E+02 |

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
5 POINTS

| X,M | C,kN/M**2 | PHI,DEGREES | E50 |
|---|---|---|---|
| .00 | .300E+01 | 28.000 | --- |
| 5.00 | .300E+01 | 28.000 | --- |
| 5.30 | .300E+01 | 28.000 | --- |
| 5.03 | .480E+03 | 28.000 | --- |

| | | | |
|---|---|---|---|
| 10.00 | .480E+-3 | .000 | .400E-02 |

FINITE DIFFERENCE PARAMETERS
NUMBER OF PILE INCREMENTS = 50
TOLERANCE ON DETERMINATION OF DEFLECTIONS = .100E-03 M
MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS=100
MAXIMUM ALLOWABLE DEFLECTION = .10E+03 M

INPUT CODES
OUTPT = 1
KCYCL = 1
KBC = 1
KPYOP = 0
INC = 1

SAMPLE PROBLEM NO. 2 CASE 1 NON-RETICULATED MICROPILE SLOPE STABILIZATION

UNITS-METR

***********************
OUTPUT INFORMATION
***********************

PILE LOADING CONDITION
LATERAL LOAD AT PILE HEAD = .000E+00 KN
APPLIED MOMENT AT PILE HEAD = .000E+00 M-KN
AXIAL LOAD AT PILE HEAD = .000E+00 KN

DISTRIBUTED LOAD CURVE 2 POINTS

| X,M | LOAD,kN/M |
|---|---|
| .00 | .000E+00 |
| 5.00 | .360E+02 |

| X | DEFLECTION | MOMENT | STRESS | TOTAL RESIST | SHEAR RIDIDITY | SOIL | FLEXURAL |
|---|---|---|---|---|---|---|---|
| M | M | M-KN | kN/M**2 | KN | kN/M | KN-M**2 | |
| .00 | .606E-03 | .000E+00 | .000E+00 | .000E+00 | .000E+00 | .600E+03 | |
| .15 | .588E-03 | .000E+00 | .000E+00 | -.214E-01 | .126E+01 | .600E+03 | |
| .29 | .569E-03 | -.626E-02 | .397E-01 | -.387E-01 | .193E+01 | .600E+03 | |
| .44 | .550E-03 | -.113E-01 | .717E-01 | .278E-01 | .219E+01 | .600E+03 | |
| .58 | .531E-03 | .185E-02 | .117E-01 | .142E+00 | .335E+01 | .600E+03 | |
| .73 | .512E-03 | .302E-01 | .191E+00 | .205E+00 | .494E+01 | .600E+03 | |
| .88 | .494E-03 | .617E-01 | .392E+00 | .168E+00 | .679E+01 | .600E+03 | |
| 1.02 | .478E-03 | .792E-01 | .503E-+00 | .758E-01 | .782E+01 | .600E+03 | |
| 1.17 | .465E-03 | .839E-01 | .532E+00 | -.514E-03 | .875E+01 | .600E+03 | |
| 1.31 | .455E-03 | .791E-01 | .502E+00 | -.532E-01 | .968E+01 | .600E+03 | |
| 1.46 | .447E-03 | .683E-01 | .434E+00 | -.836E-01 | .106E+02 | .600E+03 | |
| 1.61 | .443E-03 | .547E-01 | .347E+00 | -.954E-01 | .116E+02 | .600E+03 | |
| 1.75 | .440E-03 | .405E-01 | .257E+00 | -.930E-01 | .126E+02 | .600E+03 | |
| 1.90 | .438E-03 | .275E-01 | .175E-01 | -.809E-01 | .136E+02 | .600E+03 | |
| 2.04 | .438E-03 | .168E-01 | .107E+00 | -.633E-01 | .146E+02 | .600E+03 | |
| 2.19 | .438E-03 | .902E-02 | .572E-01 | -.434E-01 | .157E+02 | .600E+03 | |
| 2.34 | .438E-03 | .417E-02 | .264E-01 | -.235E-01 | .167E+02 | .600E+03 | |
| 48 | .439E-03 | .215E-02 | .137E-01 | -.511E-02 | .178E+02 | .600E+03 | |
| 2.63 | .439E-03 | .267E-02 | .170E-01 | .107E-01 | .188E+02 | .600E+03 | |
| 2.77 | .440E-03 | .529E-02 | .336E-01 | .231E-01 | .199E+02 | .600E+03 | |
| 2.92 | .441E-03 | .942E-02 | .598E-01 | .305E-01 | .210E+02 | .600E+03 | |
| 3.07 | .442E-03 | .142E-01 | .902E-01 | .306E-01 | .221E+02 | .600E+03 | |
| 3.21 | .444E-03 | .183E-01 | .116E+00 | .193E-01 | .232E+02 | .600E+03 | |
| 3.36 | .446E-03 | .199E-01 | .126E+00 | -.880E-02 | .244E+02 | .600E+03 | |
| 3.50 | .450E-03 | .158E-01 | .100E+00 | -.608E-01 | .257E+02 | .600E+03 | |
| 3.65 | .453E-03 | .210E-02 | .133E-01 | -.144E+00 | .270E+02 | .600E+03 | |
| 3.80 | .457E-03 | -.263E-01 | .167E+00 | -.263E+00 | .283E+02 | .600E+03 | |
| 3.94 | .460E-03 | -.748E-01 | .475E+00 | -.418E+00 | .295E+02 | .600E+03 | |
| 4.09 | .460E-03 | -.148E+00 | .941E+00 | -.592E+00 | .306E+02 | .600E+03 | |
| 4.23 | .455E-03 | -.248E+00 | .157E+01 | -.747E+00 | .314E+02 | .600E+03 | |
| 4.38 | .441E-03 | -.366E+00 | .233E+01 | -.810E+00 | .315E+02 | .600E+03 | |
| 4.53 | .414E-03 | -.484E+00 | .307E+01 | -.658E+00 | .306E+02 | .600E+03 | |
| 4.67 | .370E-03 | -.558E+00 | .354E+01 | -.110E+00 | .282E+02 | .600E+03 | |
| 4.82 | .306E-03 | -.516E+00 | .328E+01 | .107E+01 | .241E+02 | .600E+03 | |
| 4.96 | .224E-03 | -.247E+00 | .157E+01 | .313E+01 | .182E+02 | .600E+03 | |
| 5.11 | .133E-03 | .399E+00 | .253E+01 | .362E+01 | .112E+02 | .600E+03 | |
| 5.26 | .556E-04 | .809E+00 | .514E+01 | .246E+01 | .495E+01 | .600E+03 | |
| 5.40 | .747E-05 | .112E+00 | .710E+01 | -.190E+01 | .660E+02 | .600E+03 | |

| X | DEFLECTION | MOMENT | STRESS | TOTAL RESIST | SHEAR RIDIDITY | SOIL | FLEXURAL |
|---|---|---|---|---|---|---|---|
| M | M | M-KN | kN/M**2 | KN | kN/M | KN-M**2 | |
| 5.55 | -.941E-06 | .255E+00 | .162E+01 | -.386E+01 | -.414E+02 | .600E+03 | |
| 5.69 | -.299E-06 | -.970E-02 | .616E-01 | -.901E+00 | -.370E+02 | .600E+03 | |
| 5.84 | -.127E-08 | -.833E-02 | .529E-01 | .331E-01 | -.202E+02 | .600E+03 | |
| 5.99 | .232E-09 | -.489E-04 | .310E-03 | .286E-01 | .108E+02 | .600E+03 | |
| 6.13 | .307E-12 | .650E-05 | .413E-04 | .167E-03 | .607E+01 | .600E+03 | |
| 6.28 | -.256E-14 | .878E-08 | .557E-07 | -.223E-04 | -.261E+01 | .600E+03 | |
| 6.42 | -.562E-18 | -.721E-10 | .457E-09 | -.301E-07 | -.142E+01 | .600E+03 | |
| 6.57 | .886E-22 | -.158E-13 | .100E-12 | .247E-09 | .250E+00 | .600E+03 | |
| 6.72 | .195E-25 | .249E-17 | .158E-16 | .542E-13 | .337E-01 | .600E+03 | |
| 6.86 | -.307E-29 | .548E-21 | .348E-20 | -.854E-17 | -.372E-04 | .600E+03 | |
| 7.01 | -.675E-33 | -.863E-25 | .547E-24 | -.188E-20 | -.440E-05 | .600E+03 | |
| 7.15 | .106E-36 | .000E+00 | .000E+00 | .000E+00 | .276E-09 | .600E+03 | |
| 7.30 | .468E-40 | .000E+00 | .000E+00 | .000E+00 | .616E-09 | .600E+03 | |

COMPUTED LATERAL FORCE AT PILE HEAD = .00000E+00 KN
COMPUTED MOMENT AT PILE HEAD = .00000E+00 M-KN
COMPUTED SLOPE AT PILE HEAD = -.12743E-03

THE OVERALL MOMENT IMBALANCE = .798E-16 M-KN
THE OVERALL LATERAL FORCE IMBALANCE= -.854E-14 KN

OUTPUT SUMMARY

PILE HEAD DEFLECTION = .606E-03 M
MAXIMUM BENDING MOMENT = .112E+01 M-KN
MAXIMUM TOTAL STRESS = .710E+01 kN/M**2

NO. OF ITERATIONS = 3
MAXIMUM DEFLECTION ERROR = .408E-04

******************

SUMMARY TABLE

******************

| LATERAL LOAD (KN) | BOUNDARY CONDITION BC2 | AXIAL LOAD (KN) | YT (M) | ST (M/M) | MAX. MOMENT (M-KN) | MAX. STRESS (KN/M**2) |
|---|---|---|---|---|---|---|
| .000E+00 | .000E+00 | .000E+00 | .606E-03 | -.127E-03 | .112E+01 | .710E+01 |

**Step 4: Determine final micropile spacing and layout.**

From the COM624P computer run, the assumed pile sizing and spacing produced the following results:

Input

Pile spacing = 1.0 m

Minimum required ultimate shear resistance = 90 kN/m

Pile dimensions = 102 mm x 9.5 mm wall thickness

Drill hole diameter = 178 mm

Results

$M_{max} = 1.1$ kN.m $<$ $M_y = 26.6$ kN.m

Pile head deflection = $6.1 \times 10^{-4}$m

**Step 4: A. Evaluate added pile resistance due to pile inclination with respect to the slide mass direction of moment.**

Placing piles on an inclination (from vertical) with respect to the slide mass direction of movement provides an added pile resistance due to the addition of the pile's axial load carrying component.

An elastic method presented by Poulos and Davis [25] is utilized to evaluate the inclined pile conditions.

For Sample Problem No. 2, the maximum pile bending moment and deflection values are very low and are not close to yield. Therefore, evaluation of added pile resistance due to pile inclination is not necessary. Pile spacing, for this example, will be governed by the potential flow of soil around the piles.

**Step 4: B. Evaluate potential for flow of soil around piles.**

From Figure 6-24,

$$\text{For } \frac{D_2}{D_1} = \frac{(1.0\ \text{m} - 0.099\ \text{m})}{1.0\ \text{m}} = 0.90$$

$\text{Ø} = 28^0$

$$\frac{P}{b} = (120\ \text{kN/m}^2)\ \frac{(5.3\ \text{m})}{3.05\ \text{m}} = 209\ \text{kN/m}^2$$

$$P_{avg} = (209\ \text{kN/m}^2)(0.099\ \text{m}) = 21\ \text{kN/m}$$

$$\text{Hult} = (21\ \text{kN/m})\ (5.3\ \text{m}) = 111.3\ \text{kN/pile}$$

Therefore, increase pile spacing to:

$$\frac{111\ \text{kN/pile}}{90\ \text{kN/m}} = 1.23\ \text{m} \qquad \text{say } 1.25\ \text{m (maximum pile spacing)}$$

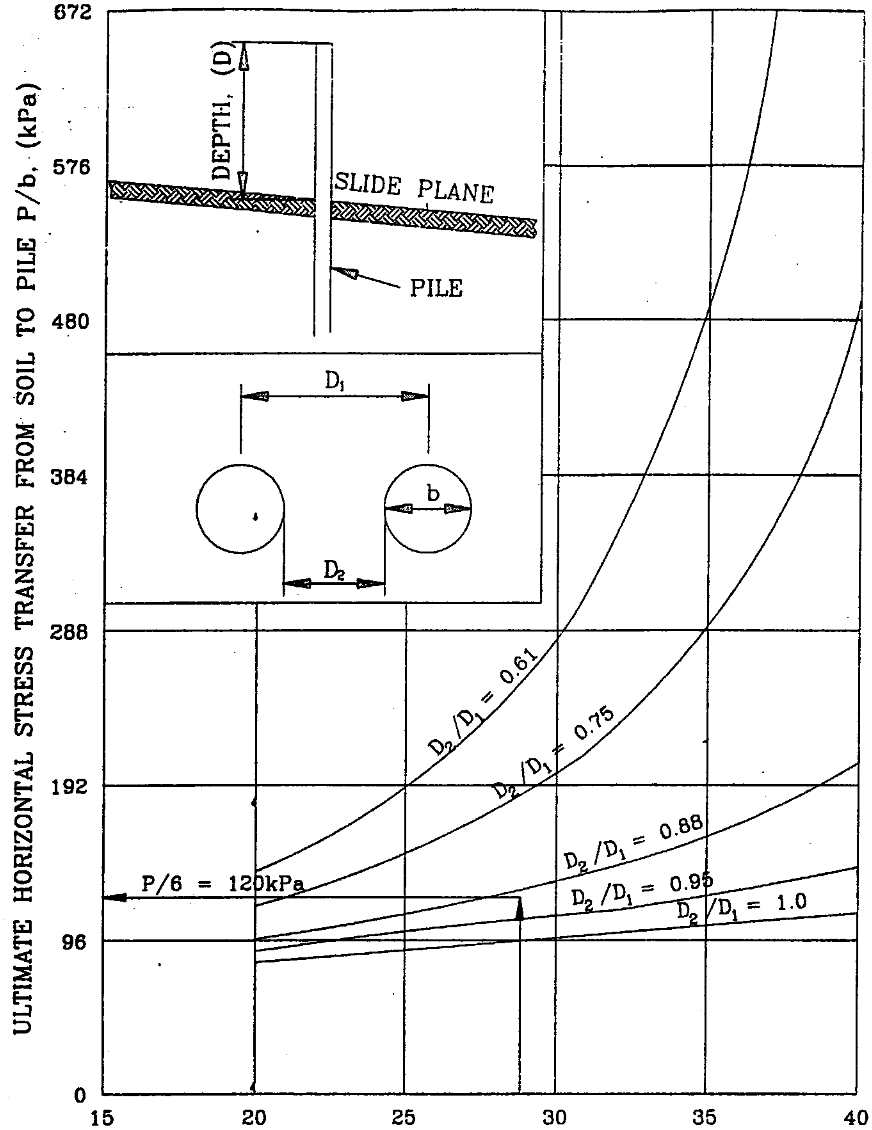
**Step 4: C. Reevaluate lateral pile capacity at increased spacing of 1.25 m.**

From the COM624P computer run, the revised pile spacing produces the following results:

$M_{max} = 1.4$ kN.m $< M_y = 26.2$ kN.m
Pile head deflection = $9.3 \times 10^{-4}$m

**Design Summary**

Place two rows of inclined micropiles on 1.25 m centers. Inclination of micropiles provides added resistance and stability to the overall slope stabilization structure. Incline piles $20^0$ uphill and $20^0$ downhill. Install micropiles 2 m minimum into siltstone bedrock. Furnish and install 1.0 m wide x 0.75 m thick reinforced concrete cap beam. Design reinforced concrete cap beam in accordance with AASHTO requirements. Figure 6-25 illustrates the final design configuration of the CASE 1 non reticulated micropile slope stabilization structure for Sample Problem No. 2. *(Commentary: The above referenced method of design analysis has been recently developed but not yet proven by long term instrumented field performance. To date, many CASE 1 non reticulated micropile slope stabilization structures have been designed and constructed using this method and are performing as required. It is recommended that owners considering the use of these types of slope stabilization structures implement a long term instrumentation program to fully understand the behavior and performance of the structure.)*




(B) COHESIONLESS SOIL

Note: Curves show values for depth = 3.05m
$\gamma$ = 19 kN/m$^3$, For other depths, multiply by D/3.05m

**Figure 6-24. Ultimate stress transfer from soil to pile vs. shear strength of soil (Figure 6-12B)**

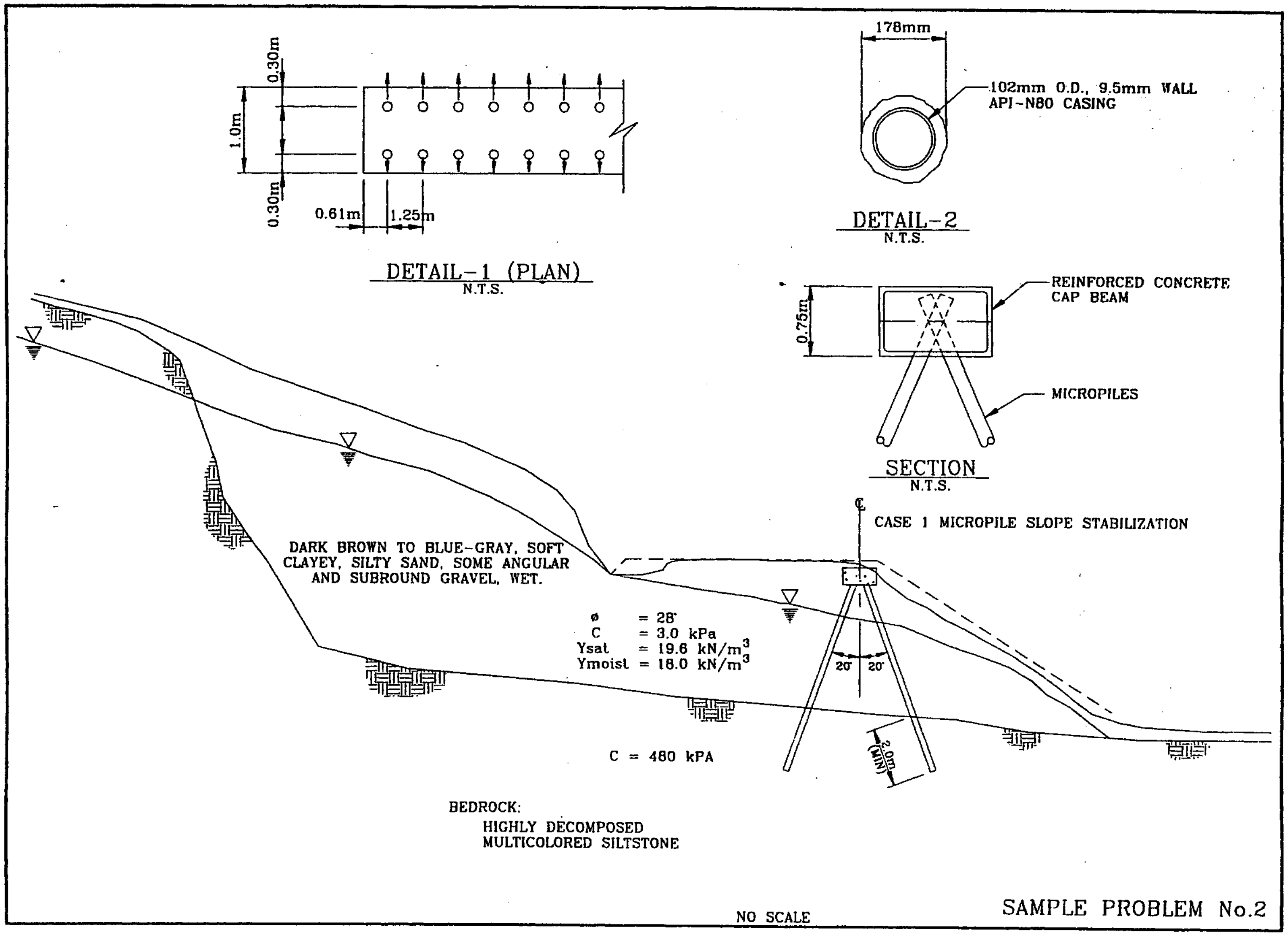
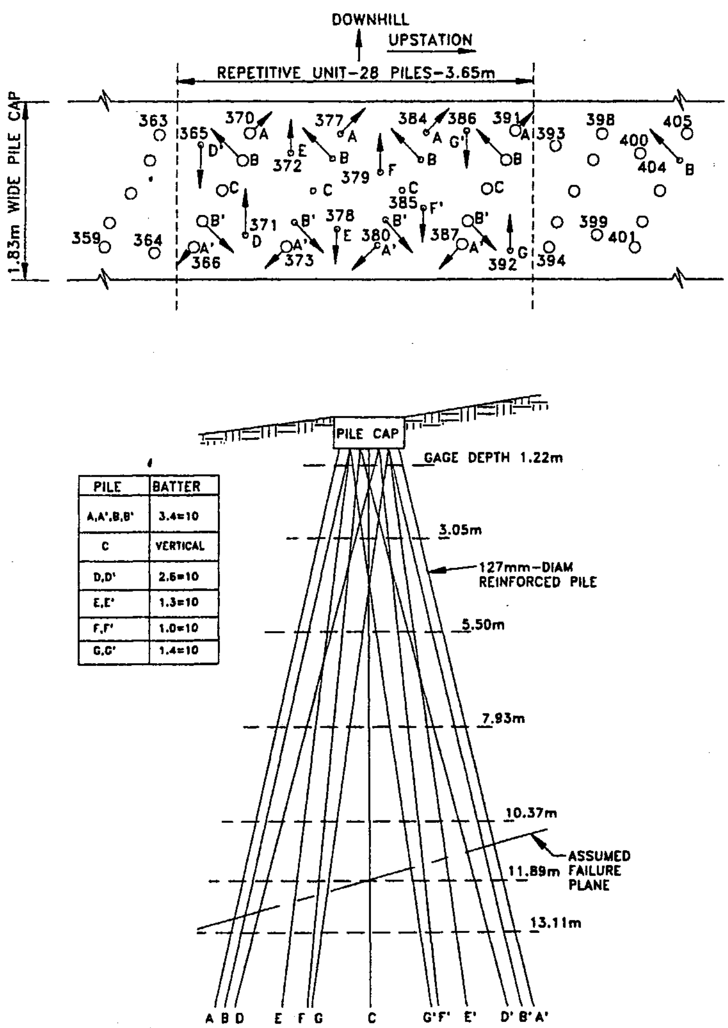



**Figure 6-25.** **Sample Problem No. 2, Final configuration of Case 1 non reticulated micropile slope stabilization**

## 6.E DESIGN CONCEPT OF CASE 2 RETICULATED MICROPILE NETWORKS

The CASE 2 reticulated micropile network design concept developed by Dr. Lizzi and illustrated in Figure 6-29 consists of "a three-dimensional lattice structure built into the soil according to a pre-established scheme depending on the purpose that the structure has to carry out," [24]. Apart from several earlier tests on models and full-scale structures performed to establish the design concept, the first official test on a full-size structure was performed in 1957 for the Milan, Italy subway system.




| PILE | BATTER |
|---|---|
| A,A',B,B' | 3.4=10 |
| C | VERTICAL |
| D,D' | 2.6=10 |
| E,E' | 1.3=10 |
| F,F' | 1.0=10 |
| G,G' | 1.4=10 |

**Figure 6-26.** **CASE 2 reticulated micropile networks**

Introducing the basics of his design approach, Lizzi stated that the design is "not an easy task. It is a very complex soil-pile interaction, there are many factors whose influence on the final behavior of the structure cannot be conveniently assessed." He cited the potential variations in the soil, in the piles, and the "practically unknown" relationship between the two parameters. He suggested that designs be based on "some simple assumptions" using the concept of reinforced soil, and those used for reinforced concrete. The soil supplies the weight in a manner more or less as supplied in a monolithic gravity wall, whereas the piles, introducing reinforcing elements into the soil, supply the lines of force, allowing the composite mass to support compression, tension, and shear stresses. The system is based on the pile-soil interaction, which results in a network (or "knot") effect provided the piles are not too far apart. The purpose of the micropiles is twofold:

1.) The pile must retain the soil and prevent its spoiling by any "flow" through the network formed by the piles, which would cause a reduction of the continuity and the unity of the composite gravity structure.

2.) The piles must supply a nailing of the various soil layers by providing additional shear resistance along the possible critical sliding surfaces.

Listed herein is a brief summary of the CASE 2 reticulated micropile network design approach developed by Dr. Lizzi for slope stabilization and earth retention applications, [24].

Figure 6-30 illustrates the design principles of a CASE 2 reticulated micropile network earth retention structure. This design anticipates a highly redundant system in which no tension is applied to any of the piles. This system is therefore subjected to compression and shear, and the micropile network provides confinement to the in-situ soil, thereby improving its deformation modulus and increasing its shear resistance. The design presents an analogy to that of reinforced concrete design considering an homogenized transformed section of a "composite beam."

The transformed section area, $A_{trans}$ is given by:

$$A_{trans} = A_{conc} \frac{E_{conc}}{E_{soil}} + A_{stl} \frac{E_{stl}}{E_{soil}}$$

where

$A_{conc}$ = area of the concrete;

$A_{stl}$ = area of the steel;

$E_{conc}$ = Young's modulus of the concrete;

$E_{stl}$ = Young's modulus of the reinforcing steel,

$E_{soil}$ = Young's modulus of the soil.

The moment of inertia of the base of the structure, $I_{trans}$, is computed by assigning equivalent areas of soil to the concrete and steel based upon the ratios of Young's moduli. Extreme fiber stresses are computed as:

$$\sigma = \frac{P}{A_{trans}} \pm \frac{P.e}{I_{trans}} (B/2)$$

where

P = Vertical component of the resultant force acting on the structure;

e = eccentricity of the force P;

$A_{trans}$ = area of the transformed section;

$I_{trans}$ = moment inertia of the base of the structure; and

B = width of the transformed section.

The extreme fiber stresses are kept under compression in the heel of the "wall" by the proper choice of design parameters. In order to resist overturning moments and maintain compression stresses, the design should verify that the resultant of the earth pressure and dead-load forces acts within the middle-third of the foundation (shown in Figure 6-30). The horizontal component, H, of the resultant force acting on the base of the "structure" is resisted by the combined shear resistance of the soil plus the shear resistance provided by the piles acting as dowels. It is recommended that the piles be extended into the rock if possible, and always be extended below the zone in which failure is suspected.

Stability of the structure is generally analyzed with respect to the following failure mechanisms:

A. External Stability
   1. Safety factor against overturning
   2. Safety factor against sliding

B. Internal Stability
   1. Plastic deformation of the soil between adjacent micropiles
   2. Structural failure of the pile cross section

Conditions A1 and A2 establish overall structure geometry and spacing between pile rows. Condition B1 allows the determination of the transverse spacing of the micropiles, and condition B2 establishes the total number of micropiles and verifies the spacing between the pile rows.

For final design of CASE 2 reticulated micropile slope stabilization systems, the following design procedure is suggested:

1) Conduct slope stability analysis to determine the increased resistance required along an existing failure surface to provide an owner-specified factor of safety;
2) Develop geometry of CASE 2 reticulated micropile structure including preliminary micropile

layout, spacing and lengths. Determine preliminary pile structural properties and capacities;

3) Evaluate external stability for given gravity structure geometry;

   a) Safety factor against overturning about toe of structure.

$$S.F._{OT} = \frac{\Sigma M_{resisting}}{\Sigma M_{overturning}} \geq 2.0$$

   b) Safety factor against sliding about base of structure.

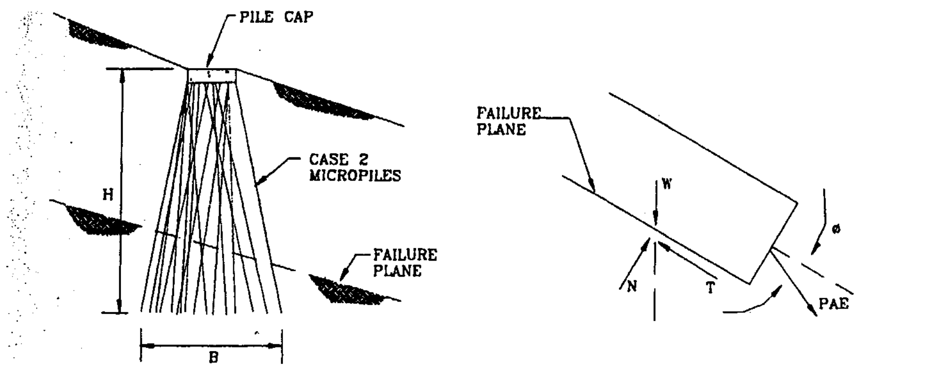
$$S.F._{sliding} = \frac{(\Sigma \text{Vertical Loads})(\text{Shear strength of base soil}) + (\text{Shear capacity of piles})}{\Sigma \text{Horizontal Loads}} \geq 1.5$$

4) Evaluate internal stability of gravity structure for given micropile layout and spacing;

   a) Final spacing of micropiles: Use Ito and Matsui theory (Figure 6-12) to analyze plastic deformation of soil around micropiles.

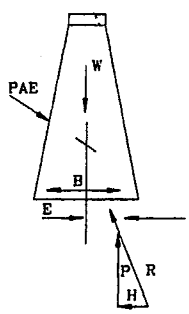
   b) Structural capacity of micropiles: Analyze structural capacity of given micropile cross section.

*(Commentary: The above referenced design procedure has been recently developed but not yet proven by long term instrumented field performance. It is recommended that owners considering the use of these types of slope stabilization structures implement a long term instrumentation program to fully understand the behavior and performance of the structure.)*




A) CASE 2 RETICULATED PILE STRUCTURE

B) CALCULATION OF ACTIVE EARTH PRESSURE




C) EXTERNAL STABILITY OF CASE 2 "GRAVITY" WALL

STEP:

1) $\sigma = \frac{P}{A_{Trans}} \pm \frac{P.\,E}{I_{Trans}}(B/2);\ @\ \sigma \geq 0$

2) $S.F. = \frac{\left[\left(\text{shear strength of soil} \times B\right) + \left(\text{shear capacity of piles}\right)\right]}{H}$

3) Pile Capacity $> (B/6 \times \sigma_{max}\ A_p)$

D) DESIGN CRITERIA

**Figure 6-27. Lizzi design principle of CASE 2 reticulated micropile networks, [24]**