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Evaluating Strength Parameters of Simple Clays: Geotechnical Consideration of Residual Soils

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Estimation of Effective Strength Parameters of Well-Behaved Clay

GERALD P. RAYMOND

Many testing techniques and interpretative methods are currently used to determine the strength parameters of saturated clays. The physical circumstances under which clays are likely to be well-behaved are described in this paper. A well-documented procedure for determining the effective stress-strength parameters of such materials is recommended. The direct shear and triaxial compression tests are examined. Other tests or modifications can be conducted, but these are not in common use (e.g., triaxial extension). The point is made that soils often exhibit different properties, depending on the type of testing (e.g., plane strain or radial symmetry).

Three main types of shear tests are used to determine the effective strength parameters of saturated clay. These tests are as follows:

1. Drained direct shear test,
2. Drained triaxial compression test, and
3. Undrained triaxial compression test with pore water pressure measurements.

Other tests or modifications of these tests (i.e., triaxial extension) may be performed but are not in common use and are not covered here. Note that soils often exhibit different properties, depending on the type of testing (e.g., plane strain or radial symmetry). In general, refinements in testing soils are only performed, if at all, on major jobs where savings may be realized despite the additional costs of the testing techniques.

TYPE OF SOIL

The procedures outlined are for soils that do not exhibit large creep effects during testing. The rate of strain should therefore be minimal and the sensitivity and strain softening characteristics should be minor. The soil is assumed to be saturated, to have an unconfined compressive strength of less than 150 kPa (3,000 lb/ft^2), sensitivity less than 8, and an overconsolidation ratio of less than 4.

APPARATUS

Shear Box

The constant rate of strain direct shear box apparatus is shown in Figure 1. The apparatus consists of a square or circular box, generally of brass, approximately one-third its width deep with an open top and bottom. The length depends on the sample size, but it is normally 50-75 mm (2-3 in.). The box is divided horizontally into two halves, a top and a bottom. The two halves can be fitted together by two alignment pins (or screws) in opposite corners that pass vertically through the walls of the upper half and screw into the lower half of the box. In the other two corners are screws that screw into the upper half of the box to rest on the lower half of the box. These latter screws are used once the box has been set up, when the sample in the box is ready for shearing, to raise the upper half slightly and thus reduce friction between the upper and lower parts of the box.

The box is placed in a rectangular brass container designed so that the bottom half of the shear box is held rigidly in position. For the strain-controlled model the container rides on frictionless bearings and is restricted to move in a longitudinal direction only. The sample is sheared, generally at a constant rate of strain, although a constant rate of loading may be used. In the case of a constant rate of strain test, a geared jack driven by an electric motor is normally used. The jack pushes against the container and hence the bottom of the shear box. This movement is transmitted through the sample under test to the upper half of the box, which is connected to a force transducer or proving ring.

Two porous square stones, which fit easily into the shear box, are used above and below the sample. The stones come in various thicknesses, depending on the depth of the box. The total thickness is such that the depth of soil sample is about 20 mm (0.8 in.).

The load to shear of the sample is measured by a proving ring complete with dial gage or by means of a force transducer. Whichever method is used, the force measured is that transmitted to the upper half of the box. The proving ring or force transducer is aligned horizontally with the horizontal shear plane through the sample.

A metal pressure pad that fits into the box on top of the upper porous stone is used to distribute the load from a hanger mechanism that fits over and above the sample normal to the shear plane. The hanger usually rests on the metal pressure pad through a ball bearing located centrally on the pad. Dead weights generally act through a lever system to apply normal loads to the sample. The lever system may be leveled by screw jacks fixed to the underside of the chassis of the apparatus. For light normal pressures the loading may be done directly through the hanger without a lever system.

A square (or circular for circular sample) cutting ring of inside dimensions the same as the inside dimensions of the box and 20-mm (0.8-in.) thick with a 10-degree (or less) sharpened cutting edge is used to obtain the test specimen. A wooden dolly is often used to retrieve the soil specimen from the cutting ring. The wooden dolly will be the same dimensions minus clearances so that it just fits inside the cutting ring.

Figure 1. Schematic of direct shear box.

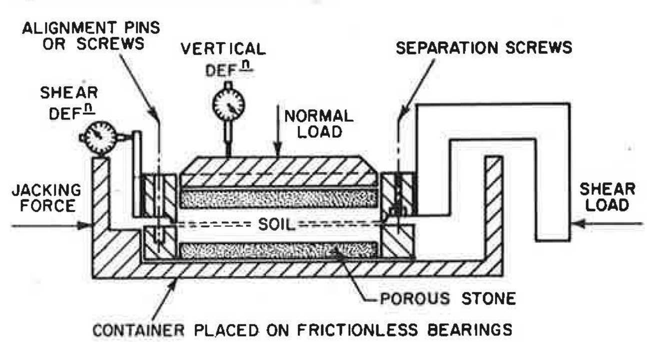


Figure 2. Schematic of triaxial cell.

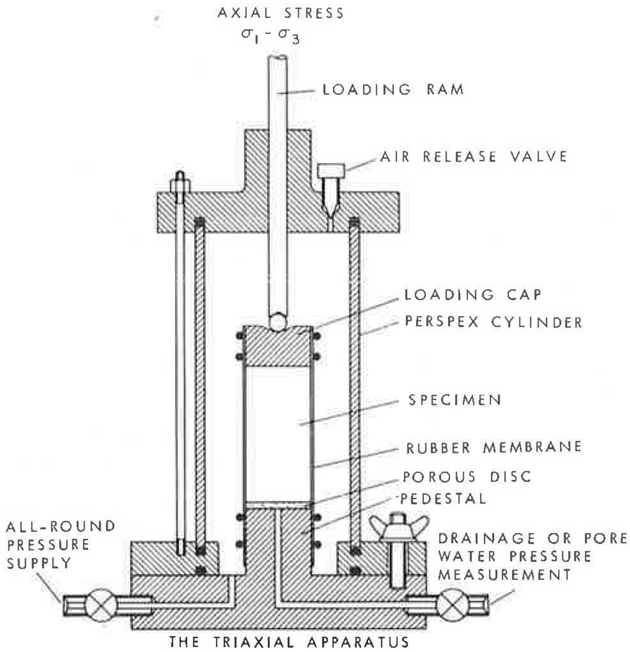
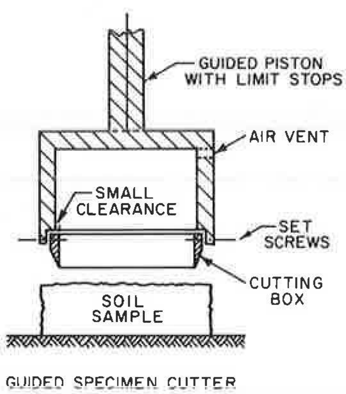


Figure 3. Fine-grained soil specimen cutter for direct shear box.



Triaxial Compression Equipment

A constant rate-of-strain compression machine is normally used to perform the triaxial test. Alternatively, the test may be performed by using incremental loads. The constant rate-of-strain compression machine can be fitted with either a proving ring or force transducer to measure the vertical load. The machine is normally operated at a constant rate through a geared screw jack that is driven by a small electric motor.

A hydraulic or air-operated pressure apparatus is used to apply fluid pressure to the outside of the sample membrane. For short-term tests air is often used acting on water in a water-air reservoir. The cell is nearly always filled with water. In long-term tests, due to the possibility of air permeating through the rubber membrane, some form of mercury constant pressure pot system is often used (1).

Figure 2 shows a typical triaxial cell. In North America a sample 50-mm (2.0-in.) in diameter is commonly used, although other sizes may be used. The length of the sample is generally twice its diameter. It should not be less than one and one-half times the diameter; otherwise end platten friction can become severe and cause major errors in the test results. It should not be more than 3 diameters long to prevent buckling of the specimen. The inner diameter of the cell is generally about 50 mm (2 in.) larger than the specimen size so that the cell can be placed over the specimen without touching and possibly disturbing the set-up. The space between the specimen and the cell is filled with water to allow an all-around hydrostatic test pressure to be applied to the specimen. A piston through the central axis of the top of the cell allows a vertical load to be applied to the specimen through a pressure cap. A rubber membrane is placed around the specimen and O-rings are used at the top and bottom of the rubber membrane to separate the cell fluid from the specimen. This allows the cell pressure to apply a load to the specimen through the rubber membrane and specimen top cap.

A trimming apparatus is usually available for soft soils, along with some form of cradle for trimming the specimen to length. The top cap for the specimen is normally made of plastic or of a light nonporous metal.

General Apparatus

In general, samples will come to the laboratory in field sampling tubes. A universal extruding machine will be required to obtain the undisturbed sample from the sampling tube. Although not common, the ability to apply a suction to the extruding end of the sample will facilitate extrusion of the soil and also minimize disturbance. Other equipment will include stopwatch, moisture content apparatus, balance, and other accessory equipment generally found in most soils laboratories.

PREPARATION OF SAMPLE

Direct Shear Test

The undisturbed soil sample cut from the sample tube should be about 25-mm (1-in.) thick and slightly larger in diameter than the diagonal distance across the shear box cutter. From this sample a specimen is cut with the guided cutting ring apparatus (see Figure 3). In some soils it may be desirable to pretrim the undisturbed sample to a size only slightly greater than the inside of the cutting ring. A wire saw is used to trim the top and bottom of the cutting ring. In some cases final trimming may be performed with a sharp steel straight edge. When the soil specimen has been trimmed to the right thickness, it is pushed out of the cutting ring by using a wooden dolly. In some cases a porous stone is placed next to the soil rather than the wooden dolly. After weighing the sample, saturated porous stones are placed above and below the specimen and the specimen is transferred to a dry shear box. Excess moisture should be dried from the stones before placement next to the soil. A certain amount of skill is needed in cutting the soil, and the use of trial-and-error methods is often required to ensure a good fit of the specimen within the box. Only samples in an undisturbed condition should be tested. Any waste soil may be used to obtain a moisture content of the in situ soil. This may then be used to calculate the bulk dry and wet density of the soil. A check may also be run on the degree of saturation if the specific gravity of the solids is known or assumed.

Triaxial Test

Before a sample is extruded from the sample tube, the triaxial cell should be flushed of all air from the water lines that lead to the specimen pore water. These lines are often known as back pressure lines. If possible the cell should be filled with water and left under pressure when not in use. This will ensure that the cell leads remain de-aired. When the triaxial cell is dismantled for testing, all pore water cocks should be closed.

A sample of undisturbed soil is extruded, generally vertically with the extruding apparatus, from the field sampling tubes. The length of sample extruded from the tubes should be slightly longer than that required for the triaxial specimen. The triaxial specimen is obtained by trimming the extruded soil sample to the correct diameter with the aid of a wire saw. Final trimming of a soft soil can be facilitated with a sharp, steel straight edge. When the required diameter has been achieved, the sample is transferred carefully to a cradle, which permits the sample to be trimmed with the wire saw to the correct length. If the sample is soft and wet, a nonabsorbent piece of paper may be wrapped around the specimen to prevent it from adhering to the cradle or to one's fingers during handling. The trimmed specimen is then weighed. After weighing, porous stones are placed on top and bottom and the specimen is placed on the pedestal of the triaxial machine. The porous stones should have been saturated with de-aired water. Excess water should be removed from the stones. De-airing can be achieved by boiling the stones in water. Either heat or vacuum may be used. Vigorous boiling is not recommended. The stones should be allowed to return to room temperature under water before use.

Filter paper strips along the sides of the specimen may be used to speed testing. The 5-mm (0.2-in.) wide filter paper strips should be saturated and excess moisture removed by squeezing so that they cling to the sample. The strips should cover the soil specimen and stones only. The top cap should now be placed in position.

A suitable rubber sleeve is inspected to detect pin holes. Only sleeves in good condition should be used. The sleeve is placed inside a former, the ends carefully pushed over the former, and the sides straightened so as to remove all wrinkles. By sucking on a rubber tube attached to the side of the former, a slight vacuum is created, which causes the rubber sleeve to cling to the walls of the former. The rubber membrane is now slipped over the specimen and, when in the correct position, the suction on the former is released. The sleeve should fit snugly around the sample without enclosing air bubbles when the vacuum is released. The rubber sleeve should then be slipped over the pedestal and top cap. O-rings, at least two at the bottom and two at the top, are used to seal the rubber sleeve to the metal pedestal and top cap.

Should the material be difficult to trim to a cylinder, a direct shear test should be performed to check the soil properties.

With the pore water cocks closed, the cell should be assembled and filled with water. A cell pressure should be applied and the specimen should be allowed to come to equilibrium, preferably overnight. The drainage cocks should remain closed. If a pore water transducer is connected to the cell base, the cell water loading pressure should be raised to maintain a positive pore water pressure in the specimen during the overnight undrained equilibrium period.

If the specimen pore water pressure rises overnight to equal the cell pressure, leakage through either the membrane or membrane-end cap seal is to be suspected. Alternatively, the sample is set up with excess water on the porous stones or filter paper. This is common when procedures that flush water up the sides of the sample are used. (These procedures are not recommended here.)

The water pressure lines that lead to the pore water of the specimen (often known as back-pressure lines) should be pressure controlled in the same manner as is the cell water.

Depending on the equipment available and its limitations, the cell pressure applied for the overnight equilibrium period should be about one-half that to be used during the shear testing, unless a higher pressure is required to ensure a positive pore water pressure.

CONSOLIDATION OF SPECIMEN

Direct Shear Test

Before water is added to the box, an initial seating load of 2-5 kPa (0.3-0.7 lb/in.^{2}) should be applied to the sample while the dial reading corresponding to the initial height of the sample is quickly obtained. While the dial reading is checked, water should be added. If the sample starts to swell, the load should be increased to prevent swelling and to obtain an initial equilibrium consolidation stress. The standard consolidation testing procedure of doubling the load (or halving the load in the case of swelling) should be conducted on the specimen for each increment until the desired test normal stress is obtained. Readings should be taken to establish 90 percent primary consolidation before the addition of the next increment.

Triaxial Test

Once the specimen has come to equilibrium and before the drainage cock is opened, the back pressure is adjusted to give an effective consolidation pressure equal to that desired for the first load increment. In triaxial testing, unless a void ratio pressure relation is desired, the initial effective consolidation pressure will be half the final effective consolidation test pressure. This allows a consolidation test to be performed, which is free of errors due to trapped water during set-up, by doubling the effective consolidation pressure, which is normal practice in a consolidation test. Time volume change readings are taken until 90 percent or more of the primary consolidation is complete for the drained test and for at least twice the time interval for 90 percent primary consolidation in the undrained test.

In the drained test the drainage cock is momentarily closed, the cell pressure is raised quickly to its test value, and a final consolidation test is started by opening the drainage cock. The value of the coefficient of consolidation obtained from this test is the value to be used for all calculations of testing rate.

In the undrained test the drainage cock and cell pressure cock (if cell leakage is negligible) are closed and the cell pressure is raised quickly to its test value. Pore pressure readings are taken by a method similar to a consolidation test when the cell pressure cock is opened (or the cell pressure is increased if cell leakage is apparent). If the change in pore pressure responds quickly and equals the increase in cell pressure, the drainage cock is opened and a final consolidation test is started. The value of the coefficient of consolidation obtained from this test is to be used for all calculations of testing rate. If the pore pressure response is sluggish or does not equal the increase in cell pressure, either large quantities of air were trapped in the pore water system or the transducer compliance is too large for the soil under test. In both cases undrained testing is not recommended and drained testing is preferred. In special circumstances, where only undrained testing is specified, an attempt to saturate the specimen fully may be undertaken by raising both the cell and back pressure equally. The back pressure should be as high as possible so as to dissolve the trapped air quickly. Note that if the back pressure is later reduced, air may come out of solution, which would defeat the original intention. If the pore pressure responds quickly to a value less than the increase in cell pressure and remains constant, the soil structure compressibility is not large in comparison with the compressibility of water or the soil solids. Discussion of this type of response is beyond the scope of the material being presented here. Such response can be expected from soft or hard rocks.

Where it is desired to measure the axial deformation during consolidation, or to ensure proper seating of the plunger on the top cap, a small axial stress difference equal to no more than 0.1 of the effective cell pressure may be used.

RATE OF TESTING

The usual criterion for the rate of drained testing of soils is based on the time to failure being the same as the time to reach 95 percent average consolidation of an oedometer constant rate-of-loading test. For this criterion to apply, the minimum time to failure is approximately 50 times the time to reach 50 percent average consolidation in a single increment loading test. This ratio is approximately the same for radial drainage; thus the speed of drained testing, whether in the shear box or triaxial cell, is obtained from the last consolidation increment. The result will naturally include any drainage imperfection because it was obtained from the specimen under test.

Similarly the rate of testing for undrained specimens is based on the setting up of a parabolic distribution of pore pressure in a constant rate-of-loading test and the achievement of 95 percent equalization. Under these conditions the minimum time to failure is approximately three times the time to 50 percent consolidation in the last increment test.

If stress-strain-pore pressure and volume change readings before failure are required, the minimum times calculated above are minimum times to the first reliable reading. Larger times to failure are sometimes desirable to facilitate the recording of readings, particularly in more permeable soils.

CRITERIA USED FOR FAILURE

The peak axial stress difference is the most common failure criterion used in soil mechanics. Also used sometimes is the effective principal stress ratio (i.e., ratio of major to minor effective principal stress). In drained tests these two criteria are the same. In undrained tests the former usually occurs first. Although the growth of modern soil mechanics is based on the principle of effective stress, engineering problems are normally load controlled and thus the former criterion is favored. Where a peak deviator stress is pronounced, for both drained and undrained tests, the test should be continued until an approximately constant post-peak stress is obtained.

PLOTTING OF TEST DATA

The consolidation test data, where applicable, should be plotted in accordance with the TRB report, Estimation of Consolidation Settlement (2), and volume reading should be replaced in the triaxial test for dial gage reading. In the triaxial test, when the axial deformation is measured during consolidation, the ratio of axial to lateral strain may be plotted to give an indication of the anisotropy of the soil and its variation with consolidation pressure. The lateral strain is calculated as half the difference of the volumetric and axial strains.

The shear test data should be reduced and the stress-strain data plotted during testing. In the triaxial test the diameter and length of the sample before shear testing should be calculated from the consolidation volume changes (and vertical dial gage if used) and the initial trimmed diameter and height of the specimen. If vertical dial readings are not taken, isotropic consolidation should be assumed.

In the direct shear test the normal and shear stresses are generally calculated by assuming that the shear area remains constant and equal to the area of the shear box. If this is assumed, the shear stress and vertical strain (vertical deformation divided by original height) as ordinates may be simply calculated and plotted against shear strain (lateral deformation divided by original height) as abscissi (see Figure 4). The peak shear stress as ordinates from different tests are then plotted against normal stress as abscissi (see Figure 5). Contours of equal shear strain may be plotted in this graph in the event that shear deformations will control the selection of shear strength.

In the triaxial test, unlike the direct shear test, the instantaneous area of the sample is normally calculated by assuming that it remains a right cylinder. This instantaneous area is used to calculate the axial stress difference, which along with the pore pressure in the undrained test or volume strain in the drained test is plotted as ordinates against axial strain as abscissi. For the drained tests, plots akin to those in Figure 4 are obtained.

The effective stress Mohr circle should be drawn for several different tests from the peak shear stress (see Figure 6).

The plot of shear stress plotted against effective normal stress is used to obtain the effective stress or long-term strength parameters. The slope of three or more tests over the in situ working range is the effective friction angle (ϕ) and the intercept the effective cohesion (c'). As seen in Figure 5, c' and ϕ are not unique and must be selected with care.

Figure 4. Typical plot of direct shear box test data.

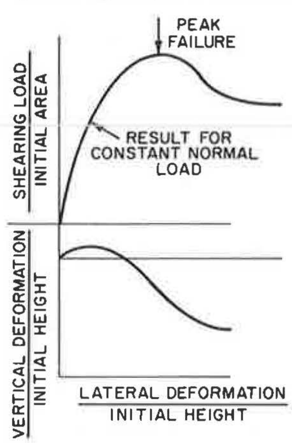


Figure 5. Typical failure criteria plot from direct shear box test results.

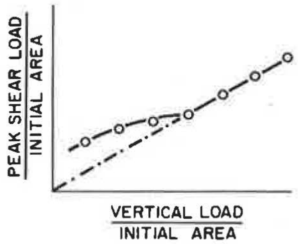
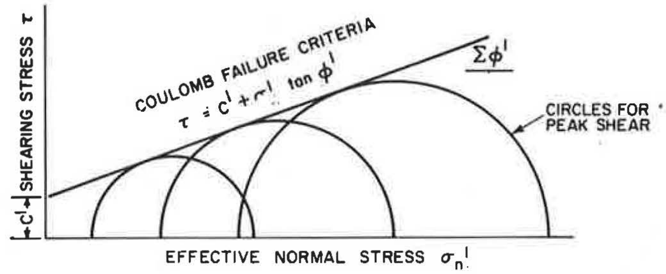


Figure 6. Typical failure criteria from triaxial test results.



CONCLUDING REMARKS

Although the discussion has been restricted to the direct shear and triaxial compression test, the reader should understand that other methods of test may be used with equal satisfaction.

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I am grateful for the assistance given to me by many members of the TRB Committee on Soil and Rock Properties. Special recognition is owed to C.C. Ladd, J.A. Tice, and to the past committee chairman, W.F. Brumund.

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Undrained Shear Strength of Saturated Clay

HARVEY E. WAHLS

A commonly used method for determining the undrained shear strength of saturated clays is examined. Some of the advantages and disadvantages of this procedure, which is proposed for use with normally and lightly overconsolidated saturated clays of low to moderate sensitivity, are summarized. The properties of normally consolidated deposits change with time, primarily due to secondary compression effects. Tests of aged, normally consolidated deposits will behave as lightly overconsolidated materials and the measured s_u will be related to the quasi-preconsolidation pressure. This hypothesis serves as the basis for the model described for predicting the in situ undrained shear strength of a saturated clay.

The procedures described in this paper are proposed for use with normally and lightly overconsolidated saturated clays of low to moderate sensitivity. They should be suitable for a saturated clay that has an undrained shear strength less than 1 ton/ft^2 (100 kPa) and an overconsolidation ratio less than 4. Monotonic loading is assumed, and the effects of cyclic or repeated loads are not considered.

CONCEPT OF UNDRAINED STRENGTH

Natural deposits of saturated clay frequently are loaded (or unloaded) rapidly relative to the rate at which consolidation or drainage can occur. For such circumstances an ideal undrained condition may be assumed. The water content and the volume of the clay remain constant during the undrained loading, and excess pore water pressures are generated. The shear strength for such conditions is defined as the undrained shear strength (s_u).

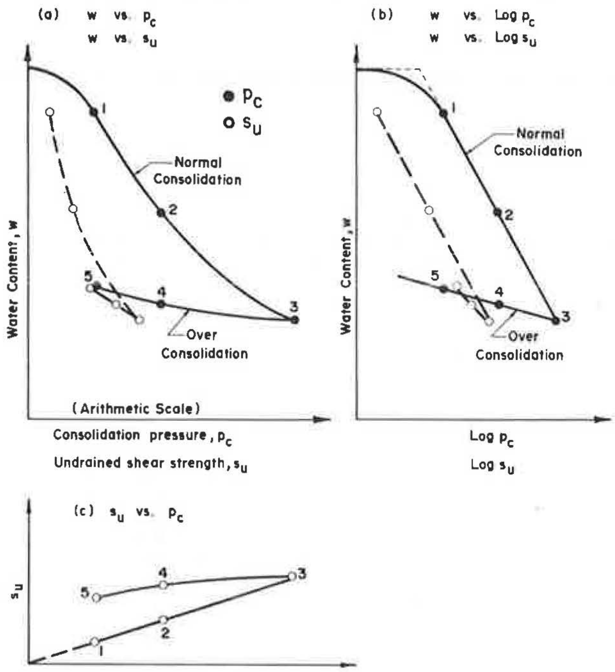
If the undrained behavior of saturated clays is analyzed in terms of total stresses, then the evaluation of pore water pressures is unnecessary. The $\phi = 0$ method of analysis (1) is assumed, and the undrained shear strength (s_u) is assumed equal to the cohesion intercept (c_u) of the Mohr-Coulomb envelope for total stresses. For these assumptions the undrained strength of a saturated clay is not affected by changes in confining stress so long as the water content does not change.

The undrained shear strength of a saturated clay is related to the consolidation history of the deposit. For young, normally consolidated deposits, the water content may be assumed to be uniquely related to the consolidation pressure (p_c), which is equal to the in situ effective overburden pressure (p_o') and thus s_u also is presumed to be a linear function of p_o'. The use of the ratio s_u/p_o' was suggested by Skempton (2). For lightly overconsolidated clays, s_u becomes a function of the current consolidation pressure [or water content (w)] and the maximum past consolidation pressure (p_{cm}). These relations are shown in Figure 1. For normally consolidated conditions, the curve of log s_u versus w is assumed to be approximately parallel to the virgin compression curve.

The properties of normally consolidated deposits change with time, primarily due to secondary compression effects [Bjerrum (3) and Leonards and Ramiah (4)]. Thus, the water content does not remain a unique function of the effective overburden pressure. The undrained shear strength increases and a quasi-preconsolidation pressure develops. As a result, tests of aged, normally consolidated deposits will behave as lightly overconsolidated materials and the measured s_u will be related to the quasi-preconsolidation pressure.

The preceding hypotheses provide a simple model for prediction of the in situ undrained shear strength of a saturated clay. The implication is

Figure 1. Effects of consolidation history on undrained shear strength.



that the in situ s_u can be evaluated by any type of undrained shear test conducted on an undisturbed sample at the in situ water content. These assumptions provide the basis for most undrained analyses of saturated clay in current U.S. practice. The limitations of these assumptions will be discussed in the section on factors that affect undrained test results.

MEASUREMENT OF UNDRAINED STRENGTH

A value of the undrained shear strength of a saturated clay specimen may be obtained by many laboratory and field tests. The common requirement of these tests is that the failure stresses should be developed without drainage or volume change. Also, tests must be conducted on relatively undisturbed soil. The primary tests used in current practice are as follows:

1. Laboratory tests--unconfined compression tests, triaxial compression tests [unconsolidated undrained (UU) and consolidated-undrained (CU)], direct box shear tests (UU and CU), and direct simple shear tests (CU); and
2. In situ tests--vane shear tests, cone penetration tests, and pressuremeter tests.

Unconfined Compression Test

The unconfined compression test (ASTM D2166) is the most widely used laboratory test of undrained strength. The test is performed on an undisturbed cylindrical sample, which is extruded from a thin-walled sampling tube or trimmed from a block sample. The test specimen should be at least 33 mm (1.3 in.) in diameter and have a length (L) to diameter (D) ratio between 2 and 3. In order to minimize effects of sample disturbance, the test specimen should be as large as possible with the available undisturbed soil and should maintain the proper L/D ratio. The length, diameter, and weight of the test specimen should be measured before testing.

The specimen is placed in a strain-controlled axial compression apparatus and loaded at a strain rate of approximately 1 percent/min. The axial load is measured with a calibrated proving ring or force transducer, and the corresponding axial deformation is measured with a dial gage or displacement transducer. Load-displacement data are recorded continuously with a x-y plotter or recorded manually at regular intervals so that a stress-strain curve may be plotted. The test continues until the axial load remains constant (or decreases) or the axial strain reaches an arbitrarily selected limit (e.g., 15 or 20 percent). After completion of the test, the specimen is weighed and its water content is determined.

The unconfined compression strength (q_u) is the peak value of axial load divided by the corrected area, $A_c = A_o/1-\varepsilon$, where A_o is the initial cross-sectional area and ε is the vertical strain. The undrained shear strength (s_u) is assumed equal to one-half of q_u.

The undrained strength as evaluated from the unconfined compression test often underestimates the in situ undrained strength of a saturated clay because of the effects of sample disturbance, discontinuities, and sand partings.

Triaxial Compression Test

The triaxial compression test provides positive control of drainage conditions and the capability for assessing the effect of consolidation pressure on the undrained strength. The apparatus, sample preparation, and test procedures for triaxial compression tests are described by Raymond in a paper in this Record. However, when these tests are used to evaluate only undrained strength, effective stress parameters are not required and pore pressure measurements are not essential.

For UU triaxial tests (ASTM D2850) the drain valves remain closed throughout the test. The chamber pressure is set approximately equal to the in situ effective overburden pressure at the depth from which the undisturbed sample was obtained, and the axial loading may be started immediately.

For CU tests the sample is allowed to consolidate under the chamber pressure before the undrained axial loading is started. When the in situ undrained strength is to be estimated, the consolidation pressure is set equal to the in situ effective overburden pressure.

For UU tests the axial loading is applied at a strain rate of approximately 1 percent/min instead of the slower rates that are suggested by Raymond for tests in which pore pressure measurements are required. The principal stress difference (deviator stress) $(p_1 - p_3)$ is computed as the axial piston load divided by the corrected area of the sample and plotted as a function of the axial strain. The undrained shear strength (s_u) is defined as one-half of the peak value of $(p_1 - p_3)$. For CU tests, the slower strain rates of Raymond should be retained to more closely approximate field rates of loading.

The UU test provides a measure of s_u at the in situ water content of the sample. The value of s_u often underestimates the in situ s_u because of disturbance and stress relief effects associated with sampling and testing. However, a CU test conducted at the in situ effective overburden pressure will usually overestimate the in situ undrained strength of normally and lightly overconsolidated clays because the laboratory specimen will reconsolidate to a water content that is lower than the in situ value. Ladd and Lambe (5) have reported that values of s_u from UU tests of normally consolidated clays may be only 40 to 80 percent of the values obtained from CU tests at the in situ effective overburden pressure.

CU triaxial tests also may be used to evaluate the relation of undrained strength to consolidation pressure. A series of CU tests, which is conducted at several consolidation pressures in excess of the in situ effective overburden pressure of the sample, will estimate the increase of s_u with increasing consolidation pressure and decreasing water content. These tests also are used in conjunction with the relatively new normalized analysis of s_u, which is discussed in the final section of this paper.

Direct Box Shear Test

The direct box shear test is not well suited for undrained strength tests because the drainage conditions are difficult to control. The apparatus, sample preparation, and testing procedures for drained direct box or ring shear tests are described by Raymond. For an undrained test the procedures are similar except that the horizontal shearing force should be applied without any volume change.

Both UU and CU direct shear tests may be attempted. For UU tests the horizontal shear loading is started immediately after application of the vertical normal stress. For CU tests the specimen is allowed to consolidate under the applied vertical normal stress before starting shear.

Often assumed is that undrained shear can be accomplished by performing the test rapidly, but experimental evidence (6) indicates that this assumption is seldom valid. Drainage can only be prevented during shear by varying the vertical normal stress so as to maintain a constant sample thickness and thus a constant volume (7).

The box shear test has several other disadvantages. The rigid boundaries of the apparatus cause extremely nonuniform strains and a progressive failure along the horizontal plane. The state of stress within the sample is indeterminate.

The horizontal shear stress is computed as the horizontal force divided by the horizontal area of the test specimen and is plotted as a function of horizontal displacement. The undrained shear strength (s_u) is defined as the peak value of the horizontal shear stress.

Direct-Simple Shear Test

The direct-simple shear test is conducted on a cylindrical sample encased in a wire-reinforced rubber membrane. The flexible boundary allows the stresses and strains to develop relatively uniformly within the sample. Although the general state of stress within the sample is indeterminate, the average normal and shear stresses acting on ahorizontal plane can be evaluated with sufficient accuracy for practical applications (8,9).

The procedures for conducting the direct-simple test are described by Bjerrum and Landva (10). For undrained shear tests, the vertical normal pressure must be varied to maintain a constant volume. The undrained shear strength (s_u) is defined as the peak value of the horizontal shear stress.

Vane Shear Test

The vane shear test is the most commonly used in situ undrained shear test. The conventional vane has four vertical rectangular blades and a height-to-diameter ratio of two (ASTM D2573). The vane is pushed into undisturbed soil and rotated at a rate of 0.1 degree/sec. The torque required to rotate the vane is measured as a function of angular deformation, and the maximum torque (T_m) is used to compute the undrained shear strength, assuming that the shearing resistance is uniformly mobilized along the surface and ends of a cylinder of the height (H) and diameter (D) of the vane. For this assumption and H/D = 2,

$$s_u = 6T_m/7\pi D^3 \qquad (1)$$

This expression also assumes that s_u is the same on vertical and horizontal planes (i.e., the soil is isotropic). However, because approximately 85 percent of the torque is used to mobilize the shearing resistance around the circumference of the cylinder, the vane test primarily measures s_u along vertical planes.

Cone Penetration Test

The cone penetration test has been used in Europe for many years but rarely in the United States before the mid-1970s. Although many cone penetration tests have been developed, the Dutch cone test has become the most popular. This quasi-static test employs a cone with a 60° point angle and a base diameter of 36 mm (1.4 in.), which provides a projected area of 10 cm^2. The cone is pushed into the ground at the rate of 10 to 20 mm/sec (2 to 4 ft/min), and the penetration resistance of the tip is recorded. Often the cone is fitted with a friction sleeve that can be used to measure local skin friction.

The undrained strength of a saturated clay is computed as

$$s_u = (q_c - p_o)/N_c \qquad (2)$$

where

q_c = cone resistance = R_p/A,
R_p = point resistance of cone,
A = projected area of cone,
p_o = total overburden pressure for the depth at which q_c is measured, and
N_c = cone factor.

The cone factor (N_c) has commonly been assumed equal to 10 for electrical penetrometer tips and equal to 16 for Begemann mechanical tips (11). More recently, Lunne and Eide (12) reported $N_c = 15 \pm 4$ for electrical cones with Scandinavian clays. More research is needed to ascertain the potential variability of N_c and hence the reliability of the cone penetration test for measuring s_u.

Pressuremeter Test

The pressuremeter test measures the pressure required to expand a flexible cylinder against the sides of a bore hole. The original Menard device, which was developed more than 20 years ago, is lowered into a predrilled bore hole. More recent designs in France (13) and England (14) incorporate a small cutting tool, which makes the device self-boring. The undrained shear strength of a saturated clay is evaluated from pressure-radial expansion data for the expandable test cell. Details of the theory and test procedures for pressuremeters are discussed by Baguelin and others (15), Schmertmann (11), and Ladd and others (16).

In recent years use of the Menard pressuremeter has increased in the United States. However, the primary applications appear to have been for cohesionless soils and partly saturated residual soils. For well-behaved saturated clays, the pressuremeter has been used primarily to evaluate the undrained modulus of deformation rather than the undrained strength. Therefore, the pressuremeter test is not discussed further.

FACTORS THAT AFFECT UNDRAINED TEST RESULTS

The evidence in recent literature is ample that different estimates of s_u are obtained by performing different undrained shear tests on identical samples at the in situ water content. These differences generally are attributed to the effects of sample disturbance, anisotropy, strain rate, and creep.

Sample Disturbance

Sample disturbance is present to some degree in every laboratory test specimen. Disturbance results from remolding during the field sampling and laboratory sample preparation and from the stress relief associated with removal of the sample from the ground. The latter effect is not present for in situ shear tests, but some remolding does accompany the insertion of field shear devices into undisturbed soil. However, sample disturbance effects are usually less important for in situ tests than for laboratory tests. Sample disturbance always reduces the undrained shear strength, assuming the water content is unaltered by the sampling and testing procedures.

Anisotropy

Anisotropy is present in most natural clay deposits. Ladd and Foott (17) present an excellent discussion of the effects of anisotropy on various types of undrained shear tests. The s_u measured in each type of shear test depends on the direction of failure plane along which the shearing resistance is mobilized. For example, vane shear, direct shear, and triaxial compression tests measure s_u along vertical, horizontal, and inclined planes, respectively. Each of these tests would be expected to produce a different value of s_u in an anisotropic soil. Ladd and Foott (17) suggest that the s_u measured from a triaxial compression test generally is greater than the value from a direct shear test, which in turn is greater than the value from a triaxial extension test.

Strain Rate and Creep

Strain rate and creep effects are interrelated. The slower the strain rate, the more creep occurs during shear. Undrained creep reduces s_u, and thus a reduction of the strain rate reduces the measured s_u. Ladd and Foott (17) report that each log cycle decrease in strain rate may result in a 10 $\pm$ 5 percent decrease in s_u. They note that the s_u obtained from undrained triaxial tests conducted at an axial strain rate of 1 percent/min may be 20 to 30 percent higher than the value obtained when the strain is reduced to permit meaningful pore pressure measurements in a high plasticity clay. The strain rates used in undrained shear tests are much faster than the strain rates associated with most field design problems. For soils that have significant creep characteristics, the s_u determined from undrained shear tests should be adjusted to provide a more appropriate estimate of s_u for use in undrained analysis and design.

In laboratory tests the above effects tend to produce errors that cancel each other; however, correction factors have been developed to provide improved estimates of s_u from in situ tests. A correction factor for vane shear tests was proposed by Bjerrum (18) on the basis of analyses of embankment failures. Bjerrum's recommended correction curve and the data on which it is based are shown in Figure 2 [Ladd and others (16)]. Additional data subsequently provided by other researchers also are shown in Figure 2. The data vary by $\pm$25 percent from Bjerrum's curve.

ESTIMATING IN SITU STRENGTH

Conventional Method

The current U.S. practice for estimating design values of in situ undrained shear strength from results of undrained shear tests is based on the concepts shown in Figure 1 and described in the section on concepts of undrained strength. The undrained strength is either determined from laboratory or in situ undrained tests performed on undisturbed samples at the in situ water content or from CU laboratory tests consolidated to effective overburden stress. The effects of sample disturbance, anisotropy, and strain rate usually are not considered quantitatively. However, qualitative consideration of these factors undoubtedly plays a role in the selection of design values of s_u. In some instances empirical correlations or correction factors (such as the curve shown in Figure 2) are employed. Field vane, unconfined compression, and unconsolidated undrained triaxial compression tests are the most commonly used tests. Sufficient local experience with a given clay deposit leads to appropriate design estimates of s_u by using one or more of these tests. When local experience is lacking, the potential exists for significant error in the evaluation of s_u.

A degree of empiricism can be found in the application of all of the tests described here. Recognition of this is responsible for the resistance to introduction of new test methods for which less experience and empirical data are available.

Normalized Analysis

Ladd and Foott (17) proposed a new method for evaluating the in situ s_u for design from a normalized analysis of laboratory CU tests. The procedure involves the anisotropic consolidation of undisturbed samples to consolidation stress greater than the in situ maximum past pressure. For some samples the consolidation stress subsequently is reduced to create overconsolidated samples. Undrained shear tests are conducted on these normally consolidated and overconsolidated samples, and the results are plotted in terms of the normalized parameters (s_u/p_{vc} and p_{cm}/p_{vc}), where p_{vc} is the vertical consolidation stress during the shear test and p_{cm} is the maximum past consolidation pressure. A typical diagram is shown in Figure 3 (16). The in situ s_u for design is com-

Figure 2. Correction factor for vane shear data.

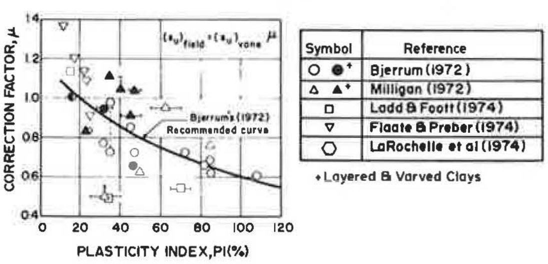
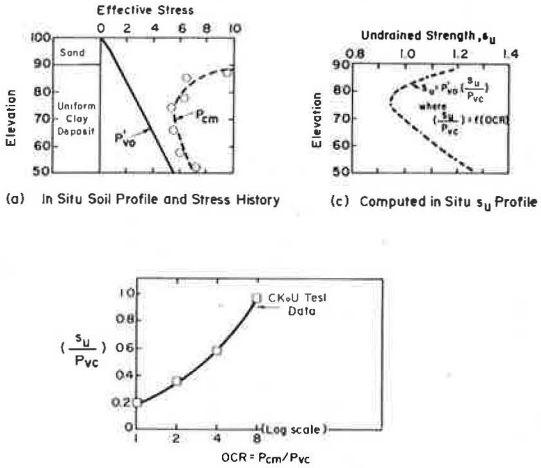


Figure 3. Normalized analysis for in situ s_u.



(a) In Situ Soil Profile and Stress History

(c) Computed in Situ s_u Profile

(b) Normalized Undrained Strength vs. OCR from Laboratory Shear Tests

puted by multiplying the in situ vertical effective overburden pressure by the value of (s_u/p_{vc}) that corresponds to the in situ overconsolidation ratio.

The procedures appear to provide a rational basis for evaluation of the undrained shear strength versus depth for clays of low-to-moderate sensitivity. Effects of sample disturbance, which are inherent in conventional tests at the in situ water content, are minimized. Anisotropy can be considered by selecting the type of shear test that best models the failure mode of the design problem, and strain rate effects are minimized by shearing the CU tests in accordance with the recommendations of Raymond in a paper in this Record. However, the procedure involves much more testing than is required for conventional analyses. Thus, the normalized approach can only be justified for large projects or for many projects on the same widespread clay deposit (16).

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Residual Soils of Piedmont and Blue Ridge

GEORGE F. SOWERS AND THOMAS L. RICHARDSON

The Piedmont and Blue Ridge form a band of crystalline rocks that extend from New Jersey southwest into Alabama. They are deeply and irregularly weathered into residual soils without appreciable transportation. The residual soils retain the mineral segregation, mineral alignment, and structural defects of the parent rocks. These are reflected in nonhomogeneous and anisotropic engineering properties. The soils inherit large residual stresses from the tectonically disturbed rock that are not related to overburden weight. They are stronger than their high void ratios imply. However, they exhibit localized surfaces of weakness that are responsible for excavation cave-ins and landslides despite average strengths that indicate stability. Settlements due to imposed loads can be significant and nonuniform, particularly when the void ratios exceed 1.5. Settlements computed from ordinary consolidation theory usually exceed those observed, probably because of unknown residual stresses and pseudopreconsolidation. Initial settlements are large because the upper strata are partly saturated, hydrodynamic settlement is rapid, and secondary compression is often large because of the large mica content. The deeper horizons of residual soil (partly weathered rock) when loosened by ripping or blasting produce angular gravelly silty sands that are dense, strong, incompressible, and slow to drain when compacted. The more completely weathered residual soils produce sandy silts and silty sands. Their density and incompressibility vary inversely with their mica content. The usual soil classification systems (ASTM-Unified or AASHTO) are poor indexes to residual soil behavior. Instead, void ratio and defect characterization in undisturbed soil and mica content and compacted density in embankment materials are more reliable.

The Piedmont and Blue Ridge provinces of the Southeast compose the largest area in the United States underlain by residual soils derived from igneous and metamorphic rocks. These materials differ in their engineering properties from the commonly encountered transported sands, silts, and clays. Thus, the empirical correlations, design parameters, and mathematical models of conventional soil mechanics are not always valid.

Residual soils are the products of rock weathering that remain above the yet-to-be-weathered parent rock. Residual soils derived from igneous and metamorphic rocks (except marble) retain much of the fabric and many of the structural features of the original rock. The degree of weathering decreases with increasing depth, usually with no well-defined boundary between soil and rock. Although the weathered materials have the texture of soils, they retain enough of the features of rock that their behavior under load can often be modeled better by the methods of rock mechanics than by soil mechanics.

In this paper the residual soils of the Piedmont and Blue Ridge of the Southeast are described and their engineering behavior as contrasted to transported soils is summarized.

PIEDMONT AND BLUE RIDGE

The Piedmont and Blue Ridge are adjacent physiographic provinces that extend from Pennsylvania southwest into Alabama. They lie between the Atlantic Coastal Plain on the east and the Appalachian Ridge and Valley Province on the west (Figure 1). Both are underlain by metamorphic rocks, predominantly gneisses and schists, of early Paleozoic age or older. Younger intrusive bodies of granite and similar silic rocks and occasional smaller bodies of mafic rocks, such as gabbro of late Paleozoic age, are scattered around. Numerous narrow disconnected dikes and occasional quartz-rich pegmatite bodies have been dated from early Mesozoic age. Although the rocks are similar, the last metamorphism of the Blue Ridge was probably more recent than that of the Piedmont. Within the Piedmont are some localized bodies of sandstones and mudstones of Triassic age. They accumulated in basins produced by downwarping and block faulting of the older metamorphic rocks.

The Piedmont is a dissected, tilted plateau. The hilltops (the old plateau) range from elevation 300 m (1,000 ft) on the west to 100 to 150 m (300 to 500 ft) on the east. The relief is typically 30 to 60 m (100 to 200 ft) with broad, sinuous hilltops and narrow zig-zag valleys. The Blue Ridge is higher, which reflects more recent (probably Triassic) uplift with mountain tops at elevation 1000 to 2000 m (3,000 to 6,000 ft) and valleys between elevation 300 to 600 m (1,000 to 2,000 ft). The Blue Ridge-Piedmont boundary is controversial. Some geologists define it by the Brevard Zone, a more-or-less continuous band of sheared rocks more than a mile wide in Georgia and possibly a poorly defined fault in North Carolina and Virginia. Others define the boundary by the topography. The Piedmont hilltops seldom rise above elevation 300 m (1,000 ft) and retain the flatter tops of the old plateau.

Rock Structure

The regions have been subjected to numerous episodes of heat and pressure, which produced varying degrees of metamorphism. The major principal stress direction was probably northwest, which reflects the forces of the crustal plate. The resulting foliation (the segregation of minerals) and orientation of minerals in parallel bands resemble stratification (and is sometimes so described) (Figure 2). However, the orientation of the foliation surfaces is related to the pressure and shear that produced metamorphism and not to any stratification of the unmetamorphosed material. The foliation and banding are often contorted. In both regions the trend of the banding and foliation strikes northeast to southwest, the same as the regional trend of folding. In the Piedmont the trend of the foliation dips is southeast. In the Blue Ridge the dips are far more irregular, which is a reflection of more intense folding and uplift.

Usually several sets of joints result from different episodes of structural deformation. In some areas the joint orientations are uniform and are reflected in zig-zag patterns of second- and third-order streams. In other areas the joints are oriented randomly. Joint spacings are also variable, although in some areas they alternate regularly between close spacing and wide spacing.

Figure 1. Location and idealized section of Piedmont and Blue Ridge.

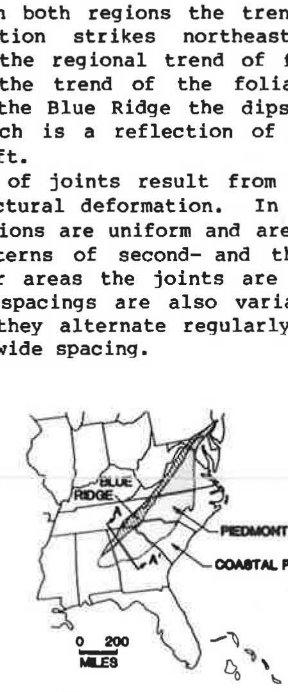


Figure 2. Banded gneiss freshly exposed in presplit highway cut: top to bottom 3 ft.

Both regions are cut by numerous faults. Some are only tens of meters long and have displacements of less than a meter; others are many kilometers long and have undetermined displacements of hundreds of meters. Most of the faults are discrete fractures; however, a few are shear zones with criss-cross fractures. Some faults near sensitive engineering structures have been dated by radioactive isotopes in minerals deposited in the fault surface. The minimum age is about 180 million years, which agrees with the geologic evidence of the most recent major tectonic thrusts.

Residual Tectonic Stresses

The crustal thrusts that produced the metamorphism, folding, and shears of the regions have not all been dissipated by the structural displacements. A few in situ rock stress tests have shown that the horizontal stresses at depths as great as 300 m (1,000 ft) exceed the vertical stresses due to rock weight. The major principal stresses trend northwest to southeast, the same direction as the regional thrusts. These residual stresses are sometimes relieved by erosion, accompanied by opening of the joints or slipping along foliation surfaces in the stress-relieved rock.

RESIDUAL SOIL FORMATION

Residual soils in the region are the products of the chemical decomposition of the various complex aluminum silicate minerals of the original rock. The products are clay minerals, hydrous micas, iron oxides, and semisoluble carbonates and bicarbonates. The temperate-to-warm climate, abundant rain [more than 750 mm (30 in.) per year] and well-established vegetation have been favorable to rapid weathering; the distribution of rain through the year, the vegetation, and the lack of glaciation have been favorable to accumulating the weathering products. Mechanical weathering of the rock is minimal because of the moderate climate and protection of the rock by vegetation on all but the highest, steepest mountains or stream channels.

Volume change is probably associated with weathering. Weathering of the aluminum silicates allows a stressed rigid quartz framework to distort. The geochemical changes of weathering produce volume changes; their effect on the volume of the residual soil compared with the original rock is not known.

Saprolite

When the weathering is accompanied by small volume changes the original fabric (arrangement of minerals) in the rock and most defects are retained. Such residual soils are termed saprolites. If large volume changes occur, such as in the solution of carbonate rocks where only the insoluble impurities remain, little or none of the parent rock's fabric is reflected in the residual soil.

On sloping terrain gravity and the volume changes due to weathering distort and even displace the residual soil. With sufficient movement the saprolite structure is destroyed, and the soil is then a form of colluvium with blocks of saprolite floating in a matrix of sandy silt and silty sands. If the movements are small and creep-like, the saprolite destruction becomes less with increasing depth.

Weathering Profile

Typical weathering profiles are shown in Figure 3, and examples of saprolite weathering are shown in Figures 4-6. The weathering is most advanced at the ground surface and decreases with increasing depth. Four zones can be identified:

1. Upper zone--completely weathered with well-developed pedologic horizons,
2. Intermediate zone--saprolite with soil texture but retains relict structure of the original rock,
3. Partly weathered zone--alternate seams of saprolite and less-weathered rock, and
4. Unaltered or slightly weathered rock.

Figure 3. Weathering profile of crystalline rocks in humid temperate region.

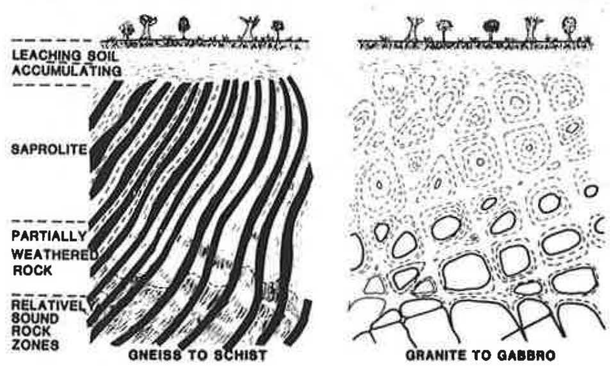


Figure 4. Saprolite from gneiss exposed in footing excavation.

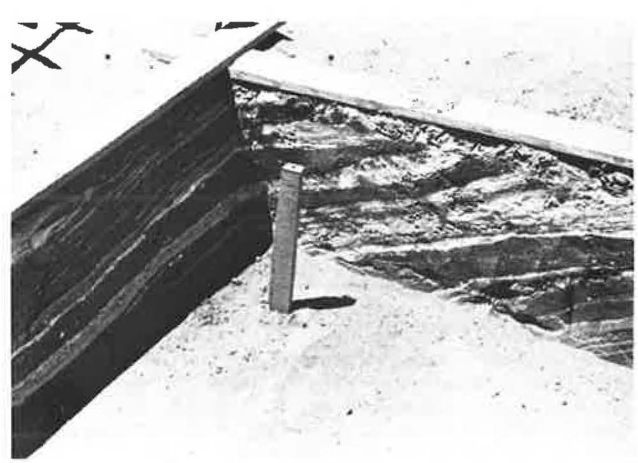
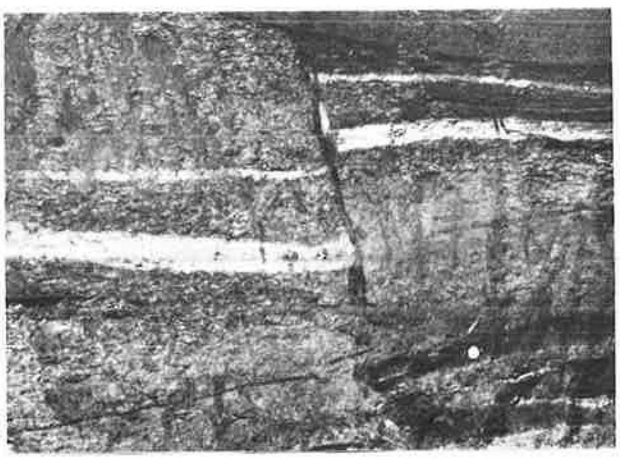


Figure 5. Saprolite and partly weathered zone from granite exposed in highway cut.

Figure 6. Saprolite from gneiss showing fault displacement of parent rock.



The boundaries of the zones are not well-defined; the transition from one to the other is usually gradual. The boundaries are not horizontal. The weathering is deeper and more advanced adjacent to fractures that transmit water and in mineral bands that are more susceptible to decomposition. As a result the soil depth can be irregular, as shown in Figure 3. Variations in soil depth, as reflected by differences in end bearing pile length, of 3 m (10 ft) in a horizontal distance of 3 m are not unusual. Variations in residual soil of 20 m (75 ft) within a city block are not unusual. The variations are greatest in rocks that have steeply dipping foliation.

The upper zone exhibits a humid, temperate region pedologic profile with a leached, somewhat sandy A horizon and a more clayey and oxidized B horizon. The clays are kaolinitic over granites, gneisses, and most schists. The clays are mixed kaolinite-smectite (montmorillonite) over mafic intrusions and dikes.

The saprolite zone exhibits weathering of the feldspar and ferromagnesian minerals into clays and clay-like minerals but retains much of the mica and all the quartz unweathered. In saprolites derived from granite weathering proceeds downward and inward from cracks more or less uniformly. The result is an XX pattern, as shown in Figure 3b and 5, that has boulder-like less weathered rock between the cracks. The boulders have an onion structure and the degree of weathering becomes less toward the boulder center.

In saprolites derived from gneisses and schists the weathering pattern is dominated by the mineral segregation in bands (Figures 4 and 6). The saprolite exhibits brightly colored bands, which reflect the varying amounts and degrees of oxidation of the iron oxides. Depending on scale, the banding represents either anisotropy or nonhomogeneity. On a small scale the foliation produces anisotropy in the orientation of minerals, particularly the micas, parallel to the banding. In the saprolite the unweathered quartz, mica, and the incompletely weathered feldspars retain the same position and orientation as in the original rock, including interlocking and molecular bonding. Volume changes that accompany weathering destroy some but not all of the interparticle bonds. Thus, residual stresses from tectonic loading and previous overburden that has eroded away are partly retained in the saprolite fabric. These residual stresses are modified by the volume changes that accompany mineral decomposition.

The depth of the intermediate zone varies greatly in the Piedmont and Blue Ridge. The greatest depth we have measured is 35 m (120 ft); depths of 60 m (200 ft) have been inferred from caisson construction in Atlanta.

The partly weathered zone reflects the different rates of weathering of the different rock bands. Seams of hard, relatively unweathered rock alternate with seams of more-weathered saprolite. In the weathered rock zones the soils are coarser grained than in the saprolite, which reflects the lesser weathering. Inward weathering from joints in the harder seams produces deep soft seams as well as the boulder-like bodies seen in Figure 5. The partly weathered zone thickness is variable. It is greatest in gneisses with strongly contrasting wide mineral bands and in rocks where the joint spacing is wide: 1 to 3 m (3 to 10 ft).

Slickensides

Many saprolites contain slickensided surfaces that cannot be traced to faults in the rock below. These surfaces are plane to curved, often dipping at angles between 40° and 70°. Their origin is in dispute. We conclude that they develop from stress-relief expansion as the weathering process proceeds (Figure 7). Some are coated with a waxy black mineral. Microtests show the black substance to be an iron-manganese-organic complex. The substance probably leached into the crack from the topsoil above (1).

CLASSIFICATION

The common soil classification systems, Unified-ASTM and AASHTO, have limited application to residual soils. They can reflect the engineering behavior of the upper soil zone where the weathering is virtually complete. However, in the much thicker saprolite and partly weathered zones the relic structure of the rock and the changing physical properties that accompany incomplete weathering are not reflected in the index tests that are the basis for classification.

Texture

The texture of the saprolites and partly weathered zone ranges from boulder to clay sizes. The gradations are often irregular because of variations in

Figure 7. Stress relief expansion of saprolite from gneiss exposed in structural excavation.

Figure 8. Chart of residual soil classification based on weathering.

<table>
<tr><th>SOWERS 1963 (7)</th><th colspan="2">DEERE & PATTON 1971 (3)</th><th>LAW/MARTA RICHARDSON/WHITE 1980 (4)</th><th>BRECKE/BART 1975 (2)</th></tr>
<tr><td rowspan="2">SOIL
N = 5-50</td><td rowspan="3">I
RESIDUAL SOIL</td><td>IA
A-HORIZON</td><td rowspan="2">UPPER HORIZON - NO RESIDUAL STRUCTURE</td><td rowspan="2">RESIDUAL SOIL, RS</td></tr>
<tr><td>IB
B-HORIZON</td></tr>
<tr><td rowspan="2">SAPROLITE
N = 5-50</td><td>IC
C-HORIZON SAPROLITE</td><td rowspan="2">SAPROLITE</td><td rowspan="2">RESIDUAL ZONE 1 RZ-1</td></tr>
<tr><td rowspan="3">II
WEATHERED ROCK</td><td rowspan="2">IIA
TRANSITION SAPROLITE TO WEATHERED ROCK</td></tr>
<tr><td rowspan="2">PARTIALLY WEATHERED ROCK, ALTERNATE HARD & SOFT SEAMS
N > 50</td><td>PARTIALLY WEATHERED ROCK
N > 100
CORE RECOVERY < 50%</td><td rowspan="2">RESIDUAL ZONE 2 RZ-2</td></tr>
<tr><td>IIB
PARTLY WEATHERED ROCK</td><td>ROCK
CORE RECOVERY > 50%
RQD < 50%</td></tr>
<tr><td>ROCK
RQD > 75%</td><td colspan="2">III UNWEATHERED ROCK
RQD > 75%</td><td>SOUND ROCK, CORE RECOVERY > 85%
RQD > 50%</td><td>ROCK: Rx</td></tr>
</table>

weathering, joint spacing, and mineral crystal size. The physical breakdown of partly weathered particles during testing often changes the gradation greatly, so that two tests on identical samples may not produce the same results. Typically the saprolites are micaceous sandy silts and silty sands (ML and SM); the partly weathered zone is similar, with gravel and boulder-sized slabs of less-weathered rock.

Except for those soils derived from mafic rocks, the clay minerals are kaolins and gibbsite and have very low plasticity. However, the soils often feel plastic when squeezed, probably because of mica, vermiculite, and similar minerals. The liquid limit test is often indeterminate, and the soil slides in the cup rather than flows. The thread in the plastic limit test is spongey and breaks at diameters of 5 or 6 mm because of micas. Thus, plasticity indexes are erratic and often are computed as negative.

Void Ratio

The void ratio is related more directly to engineering behavior than to the Atterberg limits and is easier to measure. The range in natural void ratios is greater than for most materials of similar geologic origin. Typical ranges are given in the following table:

Soil	Typical Void Ratio Range
Upper zone, leached	0.6-1
Upper zone, accumulation	0.4-0.8
Saprolite	0.7-3
Partly weathered rock	0.1-0.5
Rock	0.02 or less

The void ratios in the saprolite zone increase with increasing mica content.

Mica Content

The mica content also correlates with engineering properties; however, reliable measurement of the mica content cannot be done cheaply and quickly.

Engineering Classification

The undisturbed rocks have been classified by some investigators by the degree of weathering [Figure 8 (2-5)]. Sowers has used a four-layer system since 1959. A similar system was proposed by Brecke for the Baltimore Region Rapid Transit Authority in about 1975 (3). The Deere-Patton system (4) and the Law/Metropolitan Atlanta Rapid Transit Authority (MARTA) systems (5) are attempts to ascribe differences in weathering primarily to depth. The major problem with any of these is to define the boundaries.

ENGINEERING CHARACTERISTICS

The engineering characteristics, including physical properties, lie between those for soil and rock. Conventional laboratory tests can measure the physical properties required for design in the upper and saprolite zones. The size of samples for testing depends on the banding: Either each band should be tested individually (which would require a large number of small samples) or representative sets of bands should be tested as a whole (which could require large samples).

Permeability

The permeability coefficient (Darcy's coefficient) varies with the degree of weathering, the size of the weathering-resistant particles, and the fracture patterns. Typical permeability values are given in the table below (note that fractures usually govern groundwater movement):

Soil	Value
Soil zone	
A horizon	10^{-3} to 10^{-5} cm/sec, isotropic
B horizon	10^{-5} to 10^{-7} cm/sec, isotropic
Saprolite zone	10^{-4} to 10^{-6} cm/sec, anisotropic
Partly weathered zone	10^{-1} to 10^{-5} cm/sec, anisotropic
Rock	Impervious

Flow in the partly weathered zone is anisotropic; the permeability is typically 10 times greater parallel to foliation than perpendicular to it. The weathered zone is often the most pervious zone. Moreover, the fractures transmit much more water than the intact materials. Therefore, laboratory tests must be accompanied by numerous field bore hole or pumping tests.

Groundwater and Saturation

Groundwater levels are irregular in the Piedmont and even more irregular in the Blue Ridge. The ultimate control levels are the larger streams. Percolation is controlled by the less-pervious B-horizon and the trend of increasing permeability with increasing depth in the saprolite. Sometimes a perched water table is in the leached A-horizon supported by the less pervious B-horizon below.

Typically, a continuous body of groundwater is in the lower part of the saprolite. The phreatic surface level depends largely on topography. It more or less parallels the ground surface; however, its slope can be distorted by anisotropic permeability. It typically fluctuates several meters, which is a reflection of seasonal rainfall.

The groundwater in the partly weathered zone is usually continuous with that of the saprolite. However, impervious unweathered layers and highly pervious cracked zones distort the groundwater seepage patterns and produce localized zones of pressures either higher or lower than the hydrostatic from the saprolite.

Groundwater in the rock is entirely confined to cracks. Depending on the interconnection of cracks, a number of different piezometric levels can be present in a distance of a few meters. Fault and shear zones sometimes exhibit significant artesian pressures, with substantial flows.

The upper portion of the saprolite zone, and frequently the soil zones, are only partly saturated. The degree of saturation varies with height above the phreatic surface and the grain size; it also varies with rainfall and evaporation.

Compressibility

The materials of the saprolite and partly weathered zones become denser with increasing confining stress, as do clays, peats, and even granular materials. A plot of the time rate of consolidation of a partly saturated saprolite exhibits significant initial consolidation (as opposed to immediate or elastic distortion), well-defined primary or hydrodynamic consolidation, and usually significant continuing secondary consolidation (Figure 9). Thus, they resemble partly saturated peats rather than saturated clays (6,7). The hydrodynamic consolidation is usually much more rapid than for clays because of the higher permeability and the anisotropic effect of more pervious seams and mineral orientations favorable to drainage. Structures often settle one-fourth to one-half of their ultimate amount during construction followed by a year or two of remaining hydrodynamic consolidation that develops at an ever-decreasing rate. Significant secondary compression continues for years.

The typical stress-void ratio curves, plotted for initial plus hydrodynamic consolidation, resemble those for undisturbed preconsolidated clays. All exhibit an apparent preconsolidation load that can be estimated by Casagrande's empirical method. There appears to be no relation between depth and preconsolidation stresses. They vary erratically, typically between 50 and 250 kN/m^2 (1 and 5 kips/ft^2), although values as high as 500 kN/m^2 (10 kips/ft^2) have been observed. The preconsolidation load probably reflects residual mineral bonds between unweathered or partly weathered grains, augmented by residual tectonic stresses that are only partly relieved by weathering and erosion.

Sowers has published an empirical correlation between void ratio and compression index for the straight-line portion of the logarithmic stress-void ratio curve (2,8):

C_c = 0.75 (e-0.55), average;
C_c = 0.75 (e-0.3), 90 percent of data points below; and
C_c = 0.75 (e-0.85), 10 percent of data points below.

The higher range contains the most mica; the lower range contains the least. Similar relations exist for waste fills, peats, and even rockfills (6).

A more useful relation for preliminary design is the constrained compression modulus for the stress range, 50-250 kN/m^2 (1 kip/ft^2-5 kips/ft^2). This is shown in Figure 10. It is similar to the previously quoted relationship. The wide range of data is due in part to sampling disturbance and test procedures.

Shear Strength

The shear strengths of residual soils are still the subject of controversy. Sowers pointed out in 1963 (2) that (a) typical Mohr envelopes are somewhat curved, (b) saturated saprolites in drained shear exhibit shear strength at zero confinement (true cohesion), and (c) the shear strengths of the partly saturated saprolites are greater than when saturated. More recent data (shown in Figure 11a) indicate that the average trend of saturated residual soil does not exhibit true cohesion. However, as may be seen, the data above the trend line show many samples to have true cohesion.

These observations have been confirmed by us in numerous triaxial tests for MARTA that encompass a range of different gneiss and schist parent rocks, different depths, different void ratios, and different degrees of saturation. These strengths are expressed in Figures 11 and 12 for effective and total strength, respectively, in plots of maximum shear strength as a function of the mean of the major and minor principal stresses (p-q plots). Strengths are shown separately for both the partly saturated and saturated conditions. The partly saturated condition shows a higher apparent cohesion, probably due to capillary tension. The diagonal banding in the test data is the result of tests run at a similar confining pressure.

Figure 9. Time-rate of settlement: partly saturated saprolite.

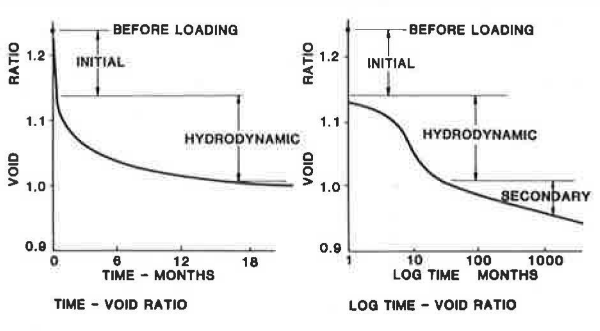


Figure 10. Constrained modulus from consolidation test versus void ratio.

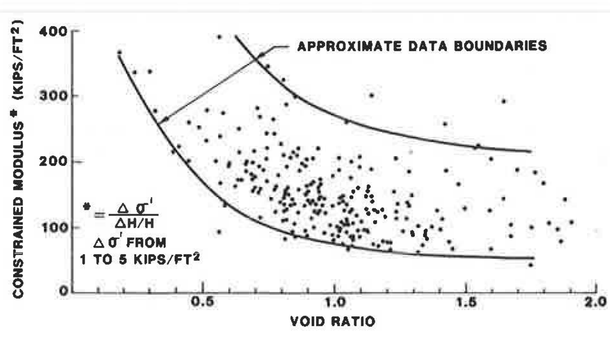


Figure 11. Effective shear strength.

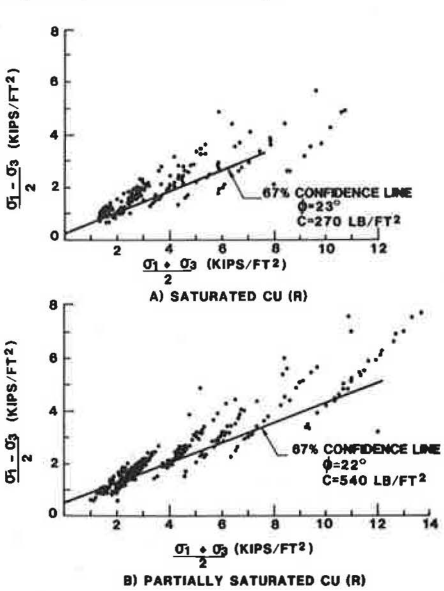
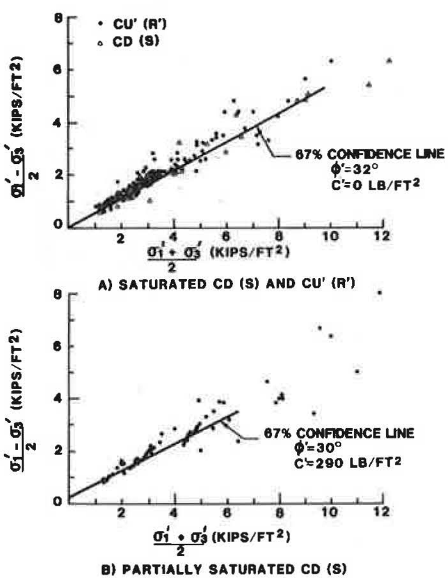


Figure 12. Total shear strength.



The shear strength varies greatly from point to point. In addition, it is anisotropic. Tests with the flakey minerals oriented parallel to the potential shear plane exhibit about two-thirds to three-fourths the strength perpendicular to it. The black slickenside surfaces previously described also exhibit one-half to two-thirds the shear strengths of the intact material.

Stress Relief

Local areas retain residual lateral stresses that sometimes exceed the vertical. These are sometimes evident in the saprolite zone, where new excavations sometimes allow expansion that generates shear displacements and local slickensides (Figure 7). These slickensides, often dipping steeply, are the cause of slope failures and excavation bracing collapse in otherwise stable excavations (1). Stress relief in the partly weathered and sound rock opens up joints and produces displacements along foliation weaknesses during blasting. Closer-spaced presplit and trim holes in blasting can reduce these adverse effects. The joints in partly weathered rock and sound rock in steep dam abutments are frequently 3- to 5-cm (1- to 2-in.) wide. The stress relief openings sometimes extend back into the abutments about as far as the eroded valley is deep. Excavation of the open-jointed rock in a core trench is not always effective because deeper excavation generates more stress relief strains and previously closed joints become open. Similar conditions are sometimes present in the rock stream bed below the dam.

Construction Drainage

The micaceous sandy silt and silty sand saprolites are slick and slippery when wet. They churn up easily and become a soft slurry when exposed in construction below the water table. Moreover, a significant part of their strength at low confinement is derived from capillary tension. Drainage is imperative, therefore, for excavations more than a few feet below the groundwater level. Because of medium to low permeabilities, and anisotropic permeability, drainage parallel to foliation by well points, wells, or even deep sumps is successful. Because the permeabilities are not high (except for the occasional fractures) the amount of water pumped is seldom great.

Engineered Fill

The A-horizon is the only reliable source of silty sands in the region. It is typically only 0.3 to 1 m (1 to 3 ft) below the thin organic layer. Therefore, many roads have been built with topsoil or A-horizon bases. The B-horizon is the most reliable source of low-permeability clay. The only problems are that it frequently is desiccated and hard and only 0.3- to 1-m (1- to 3-ft) thick.

The saprolites as excavated have the texture of sandy silts and silty sands. As such, they usually make good embankment materials. However, unlike the sedimentary soils of similar texture, the mica reduces their maximum compacted densities and increases their compressibilities. Sowers (2) reported a median maximum dry density of 1.6 g/cm^3 (100 lb/ft^3) for the standard Proctor (standard AASHTO) maximum and a range from 1.25 to 2 g/cm^3 (78 to 125 lb/ft^3). However, those soils that have low maximum densities make strong but somewhat rubbery embankments at densities between 95 and 100 percent of the standard maximum density.

CONSTRUCTION EXCAVATION

Ordinary earth-moving equipment is suitable for excavation of the soil horizon. The A-horizon usually includes up to 20 cm (8 in.) of soil mixed with humus that could be termed topsoil that is often stockpiled for landscaping. Local drainage is sometimes necessary if the impervious B-horizon produces a perched water table.

The saprolites are usually easily excavated, although loosening with rippers is sometimes necessary. Drainage is required below the water table.

The partly weathered zone usually presents excavation problems (and subsequently claims and litigation). The softer soil-like seams can be excavated easily. The hard seams can be loosened by ripping if thin; otherwise, blasting is required. Therefore, contracts that do not describe the mixed condition adequately or that define rock excavation as if there were a sharp, smooth line between rock and soil materials will produce problems. The defining of excavation loosening by seismic velocities has been reasonably effective in this zone. An occasional pinnacle of resonably sound rock can be expected even in the saprolite and soil zones; an occasional weathered zone can also be expected deep in the sound rock.

SUMMARY

The residual materials of the Piedmont and Blue Ridge exhibit engineering behaviors typical of both soils and rocks. In-place they retain the relict structure of the rock as well as the nonhomogeneity, anisotropy, and surfaces of weakness. When excavated and recompacted as fill they have the texture of sandy silts and silty sands. The mica in most of the residual soils contributes to high void ratios, high compressibilities, and low compacted densities. The strengths are higher than might be expected for soils of such void ratios. The usual index tests do not reflect the engineering potential of Piedmont and Blue Ridge materials (except the soil horizon). Instead, the degree of weathering as exhibited in the total profile, the void ratio, and mica content are more useful indexes.

ACKNOWLEDGMENT

The data on the properties of the soils were obtained from the files of Law Engineering Testing Company. These data reflect thousands of projects and tens of thousands of samples throughout the Piedmont and Blue Ridge. The greatest concentration of data was from MARTA, for which Law Engineering was the Geotechnical Consultant (acting through the general engineering consultant, Parsons Brinckerhoff/Tudor).

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Thickness of Weathered Mantle in Piedmont: Variability and Survey Techniques

ANTONIO V. SEGOVIA

The thickness of overburden measured at construction sites is, in many instances, different from that predicted by regional geomorphologic models. Variations in the thickness of residual soils and weathered rock, on the order of 10 m, have been found to occur over distances as short as a few meters. Problems, ranging from tilting and fracturing of structures to spots that require chronic maintenance on roads, can result from differential settling induced by such variability of the weathered mantle. Geologic and soil maps help to predict problem areas where the variations result from changes in underlying rock facies, such as soluble to insoluble rock. However, sudden variations are common in areas where little or no change in either lithologies or soil types is apparent in such maps. In these areas thickening of the weathered layer is commonly associated with the presence of fracture zones (or zonesof closely spaced jointing), which provide preferential loci for water percolation and weathering of the underlying rocks. Such fracture zones can vary in width from one meter to tens of meters. Methods effective in locating fracture zones in the Piedmont include: (a) analysis of aerial photographs, topographic maps, and satellite images to map possible fracture traces and (b) field check carried out by one or more of the following techniques: shallow seismic, magnetic or electric surveys, and trenching or augering. Expeditious field evaluations, based on the examination of samples obtained with a hand auger to determine variations in pedologic characteristics associated with varying rates of water infiltration, can provide a useful guide to the more expensive and time-consuming geophysical surveys.

Estimations of the thickness of the weathered mantle are necessary to determine depth of excavation to bedrock, expected settling of structures, attenuation capacity of the mantle for potential groundwater contaminants from waste disposal, cost of landscaping, and the impact of soil erosion on future land productivity. Incomplete evaluation of the variations in the nature and thickness of the weathered mantle can result in problems. These may include the tilting and fracturing of structures, cost overruns in excavation contracts, high maintenance cost of highways, and unexpected contamination of water supplies.

The standard county soil surveys and county maps, which depict the thickness of overburden, are not adequate for site-specific design purposes. Soil maps rarely provide information beyond 1-1.5 m below the surface, whereas maps of thickness of overburden are usually generalized and, in most cases, they neglect the effect of fracture zones. The maps for Montgomery County, Maryland, among the most recent

Figure 1. Variations in thickness of the weathered mantle associated with changes in lithology of unfractured substrata.

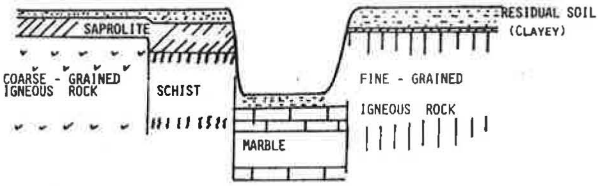


Figure 2. Conversion of rocks to soil in the Piedmont.

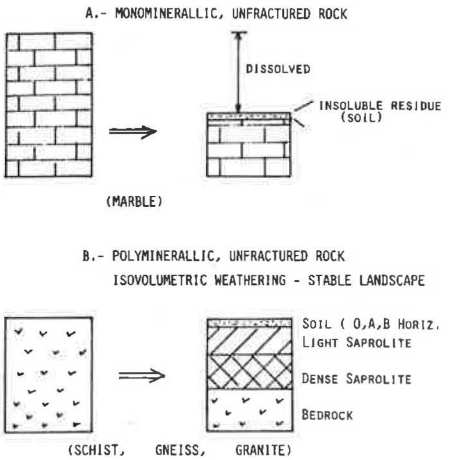
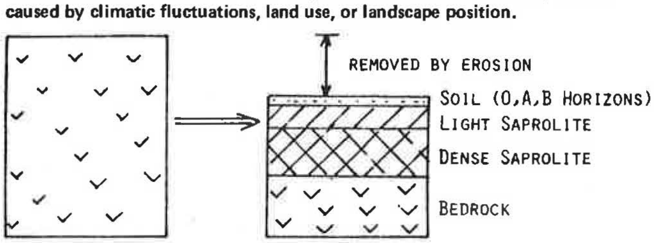


Figure 3. Conversion of polymineralic rocks to soil in the Piedmont under conditions of isovolumetric weathering and unstable landscape; i.e., erosion caused by climatic fluctuations, land use, or landscape position.



in the Piedmont, are an example of competent geologic and pedologic work that, for reasons of scale and purpose, do not satisfy the requirements of site-specific interpretation (1-3).

Conceptual models for the variations in thickness of residual soils and saprolite have been advanced (4-6), but they should be used with caution because the depth to bedrock has been found to depart from the models in many localities. The depth can vary suddenly from less than 1 m to about 10 m over a distance of a few meters (7-9). The terms overburden and weathered mantle in this paper include all materials above bedrock. Saprolite is included by pedologists as the C-horizon of residual soils (10). However, the engineering properties of saprolite are often distinctive enough that, for convenience, it is separated from the overlying (O, A, B) horizons. This differentiation is maintained in this paper, where it is helpful in clarifying some concepts.

GENERAL MODEL OF CONVERSION OF ROCKS TO SOILS IN THE PIEDMONT

The conceptual models of thickness of the weathered mantle commonly used are based on (a) an interpretation of the evolution of the landscape; (b) effective time of action of the processes of weathering, or intensity and rates of weathering; (c) assumptions about the volumetric relations between residual soils (including saprolite) and parent rock (4); and (d) relative resistance of the various rock types to chemical decomposition (11).

Figure 1 represents schematically a model based on differential resistance to weathering for the most common rock types found in the Piedmont. The basic assumptions in this model are as follows:

1. Each rock type is homogeneous and isotropic to the action of weathering agents,
2. Each agent of weathering acts uniformly over the area considered,
3. Transformation operated by weathering is essentially an isovolumetric phenomenon, and
4. Degradation of the landscape is either negligible or fairly uniform, except for areas of soluble rocks such as marble, where the ratio of volume of residual soil to volume of parent material is very small compared with the ratio for other rocks (Figure 2).

A model that combines isovolumetric weathering and landscape degradation for areas underlain by polymineralic rocks is shown schematically in Figure 3. Overall erosion of the land could have been episodically accelerated by climatic fluctuations and land use (12,13). Locally, strong erosion could be caused by a combination of moderate-to-steep slopes with runoff and mass wasting.

LOCAL DEPARTURES FROM THE GENERAL MODEL

Models of weathering based on the factors and assumptions outlined in the previous paragraphs have often turned out to be inadequate to explain local variations in depth to bedrock. Additional factors appear to be responsible for variations in the thickness of the weathered column. Some of these additional factors are discussed in the following paragraphs.

Inhomogeneities are found within mappable geologic units because small bodies of diverse lithologic composition, resistance to weathering, and engineering properties may be grouped within a given geologic unit for mapping purposes.

Variations in thickness of the weathered mantle can also be caused by folding and faulting, which followed or accompanied the regional metamorphism that affected the Piedmont rocks. The folding and faulting that accompany metamorphism would have resulted from the action of regional compactional stresses acting on a complex, anisotropic mass. Even though each lithologic component of the mass may have behaved uniformly under stress, the geometry of some of the rock bodies probably varied independently from that of the others. In such a case, even though the region was subjected to a fairly uniform stress, differential compaction of irregularly shaped bodies of varying mechanical properties would have required the development of tight folds and faults with horizontal, oblique, and even vertical components of movement. Such faults resulted in the juxtaposition of materials of different resistance to weathering. Given geologic time, this resulted in the current variations in the thickness of residual soils. The bewildering discontinuities of topographic ridges in metamorphic terrains reflect the great numbers of disruptions and dislocations caused by such adjustments. The left side of Figure 4 illustrates variations of this type.

The presence of thin, long intrusive bodies, usu-

Figure 4. Soil-landscape associations in the Piedmont and their relations to bedrock type and structure.

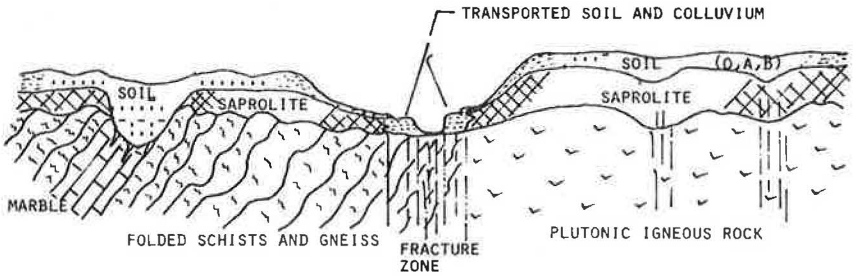
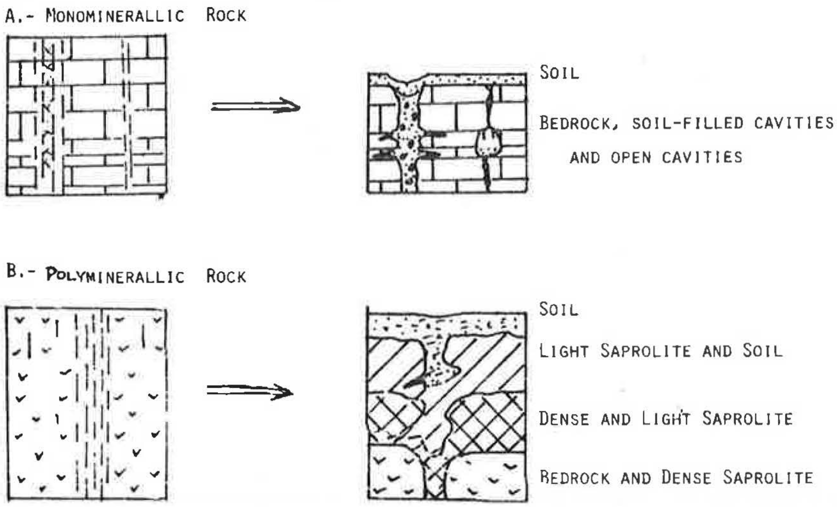


Figure 5. Irregular weathering caused by presence of fracture zones in rocks of Piedmont.



ally vertical or nearly vertical in attitude also creates variations in thickness of the weathered column. Some of these dikes are granitic, but the most common ones in the Piedmont are basaltic in composition.

Fracture zones usually have very sharp boundaries and may vary in width from less than a meter to a hundred or more meters. They are perhaps the most troublesome features encountered in attempting to predict depth to bedrock. The fracture zones control water infiltration into the ground (14). In turn, water circulation controls weathering and the rate of formation of residual soils. Thickness of soils may vary from a few tens of centimeters outside the fracture zone to ten meters within the fracture zone, over a horizontal distance of a few meters, as shown in Figures 4 and 5.

The following characteristics of fracture zones have been observed in the Piedmont:

1. Orientation: Even though there appear to be two conjugate, dominant sets, with orientations that probably rotate gradually with the regional tectonic grain or latitude, local sets are also superposed on the regional ones (15). From the engineer's point of view the local sets, in some instances, may be more important than the regional ones. The local sets are apparently related to deformation caused by the intrusion of igneous masses, such as the Boyds laccolith, whereas others may be associated with changes in compositional and mechanical properties of the rocks below (15).
2. Age: Observations of soils developed on fracture zones in the Piedmont suggest that some fracture zones are probably very old, affecting weathering for more than a hundred thousand years, whereas others appear to be young, perhaps no more than a few thousand years old.

Grouped collectively under geomorphologic factors, and more restricted in occurrence than fracture zones are (a) filled, paleostream channels, (b) fluvial terraces and floodplain alluvium in stream valleys, (c) veins filled with crushed quartz that promote fast water percolation, (d) erosion associated with local land use, (e) eolian erosion and preferential deposition in certain loci (wind-transported material is slowly incorporated into the soil profile and becomes difficult to differentiate from some residual soils), and (f) stream and gully erosion.

DETERMINATION OF LOCAL VARIATIONS IN DEPTH TO BEDROCK

Emphasis is placed on techniques for mapping variations in thickness of the weathered mantle caused by the presence of fracture zones. However, the same field procedures listed below can be used to locate similar features of different origin.

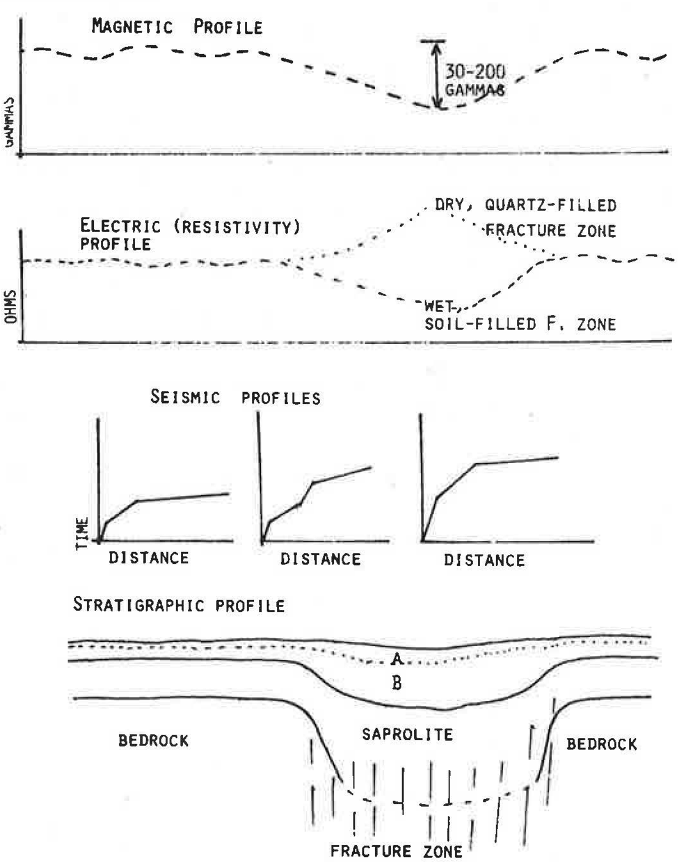
Lattman (16) described the techniques still in common use to map photogeologic fracture traces and lineaments. Unfortunately, the features thus mapped have come to mean fracture traces, fracture zones, or simply faults and fractures to many later workers. Many of the linear features so mapped are not associated with fracture zones. For this reason, such features must be evaluated in the field in every case. For the same reason, the term regmite from the Greek regma (fracture) and mimos (imitator, mimic), was introduced to differentiate mere linear features traced on aerial photographs from those verified in the field (17). The terms linear, lineament, and fracture traces commonly mean regmites in the literature and are rarely used to mean verified fracture zones.

Survey techniques to map fracture zones that may affect the thickness of the weathered mantle include the following steps:

1. Aerial photographs should be analyzed, preferably in stereoscopic models, to identify changes in soil tone and linear features manifested as vegetation alignments, stream deflections, or rectilinear reaches, topographic breaks in slope, and scarps (16), complemented by analysis of topographic maps and satellite images. The preferred scale of the aerial photographs should be 1:12,000 or larger.

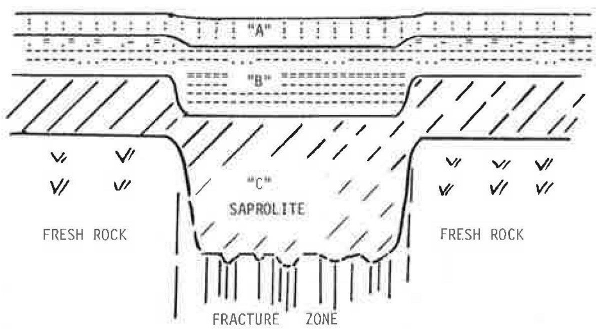
2. The importance of field evaluation cannot be overemphasized. This includes field observation of vegetation, soil variations, and geomorphologic features. These observations are accompanied by examination of soil samples obtained with a hand auger. A bucket auger can easily reach down to about 1.5-2 m in weathered material. This depth is usually sufficient to provide reliable clues as to differential rates of water percolation and soil development. The colors are characteristically redder and more vivid in the fracture zone than in the surrounding areas. The field check is completed by using one or more portable geophysical tools, such as a seismic device, magnetometer, and resistivity meter. The magnetometer is an excellent tool for a fast check of suspected fracture zones in most areas of the Piedmont, whereas the seismic devices, though slightly more cumbersome to use, can provide reliable data on depth to bedrock.

Figure 6. Expression in geophysical surveys of variations in thickness of weathered mantle.



Figures 6 and 7 illustrate the variations in geophysical parameters and soils parameters usually found along traverses as one crosses a fracture zone. Figure 8 shows the expression of two fracture zones in the magnetometric survey of a site in Olney, Maryland. Figure 9 shows a seismic traverse across a fracture zone in the vicinity of Clarksburg, Montgomery County, Maryland. Bedrock is a biotitic schist within the Ijamsville Formation (1).

Figure 7. Hand-auger traverse across a fracture zone in the Piedmont.



Note: All soil horizons are typically thicker in the fracture zone. Colors are also redder and more vivid than outside the zone.

Figure 8. Expression of fracture zones in magnetometric survey site in Olney, Md.

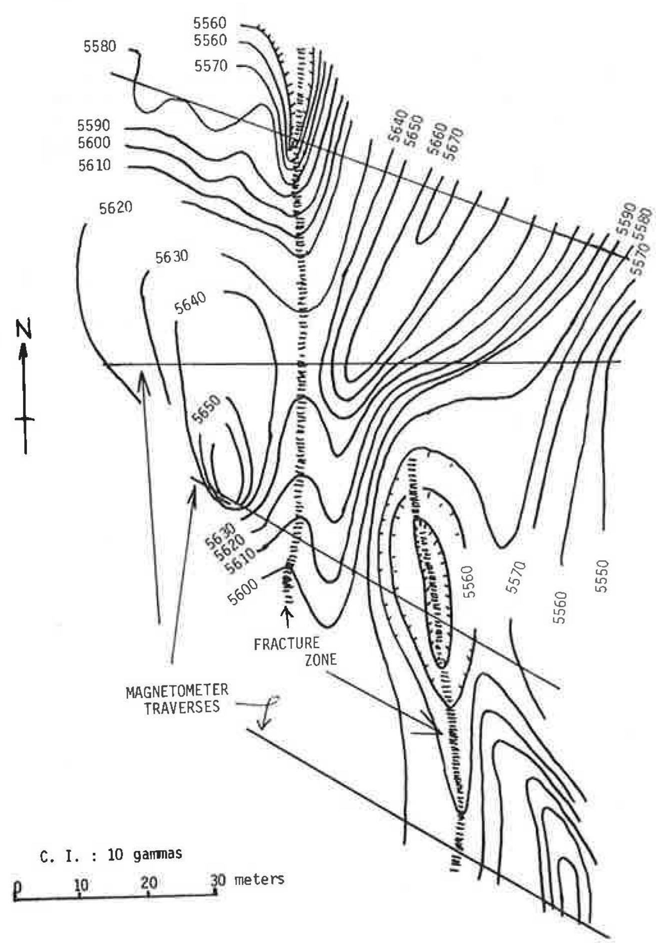
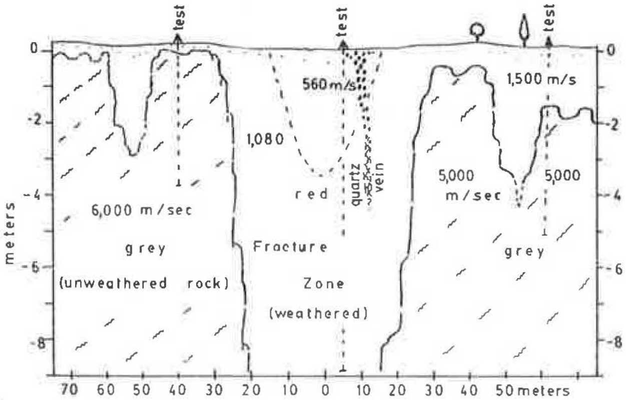


Figure 9. Interpretation of a seismic traverse across a fracture zone in the vicinity of Clarksburg, Montgomery County, Md.



Note: Geophone spacing: 3 m.

SUMMARY

Variations in thickness of the weathered mantle at specific sites are difficult to predict on the basis of regional models of weathering for the Piedmont. Therefore, design and site selection of engineering works that can be affected adversely by differential settling and tilting, or for which there is a required amount of soil thickness between base and bedrock, should be preceded by detailed terrain evaluation. The emphasis should be on the detection of fracture zones. Such zones can cause sharp variations in thickness of overburden over short distances because they tend to localize water percolation and increase depth of weathering. Detailed geomorphologic analysis by means of interpretation of aerial photographs, followed by field evaluation using hand augers and portable geophysical equipment such as magnetometers, resistivity meters, and seismic devices have proved to be an effective method for locating fracture zones and variations in depth to bedrock in the Piedmont.

ACKNOWLEDGMENT

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Cautions of Reinforced Earth with Residual Soils

VICTOR ELIAS AND PAUL SWANSON

For many years Reinforced Earth has been an alternative to the conventional retaining system. Experience has been gained in many soil types across the United States. The result has been the development of reinforced earth backfill specifications that provide for granular backfill properties in most soils. The performance and problems of a project that uses fine-grained residual soils as Reinforced Earth backfill are documented in this paper. Fines and moisture content caused strength reductions, which resulted in wall deformations. A thorough investigation was conducted, including extensive laboratory testing. Test results demonstrate the reduction in strength with increased fines and water content. The design strip pull-out capacities in fine-grained residual soils and coarse-grained highly micaceous soils are sensitive to variations in the amount of fines and water content. Strict quality control is necessary during construction of any earth structure in these soils.

Construction of a Reinforced Earth wall began during the winter of 1978-1979 in Virginia. The wall consisted of nine sections, which totaled more than 35,000 ft^2 of exposed surface area. The walls varied in height; the maximum section was approximately 23 ft. A simplified location plan is shown in Figure 1. Many similar walls have been designed and constructed in Virginia by the Reinforced Earth Company; however, the problems that evolved on this job provide a valuable lesson for future construction in residual soils.

The project site is located in the Piedmont geologic province. On-site soils are residual fine sandy micaceous low-plasticity silts. These soils, although often suitable for use as structural fill, were not considered suitable Reinforced Earth backfill. The job specifications required that the Reinforced Earth backfill be nonplastic and less than 15 percent pass the No. 200 sieve. The gradation requirements were as follows:

Sieve Size	Percentage Passing
No. 200	0-15
3 in.	75-100
6 in.	100

This was a standard requirement for Reinforced Earth walls constructed at that time. Research has shown that nonplastic soils that have less than 15 percent fines derive their internal strength from intergranular friction (1).

A compaction moisture content within ±2 percent of optimum with a minimum density greater than 95 percent of standard Proctor maximum dry density was also specified. For a truly granular fill this criteria are not as critical as others. Compaction moisture content significantly affects undrained strength of clayey or silty soils. However, the internal friction of a granular soil is only moderately reduced with increased compaction, moisture content, or saturation (2,3).

DESIGN

The wall facings were precast reinforced concrete. Typically, four Reinforced Earth strips were attached to each panel on approximately 2.5-ft spacings. The strips were ribbed galvanized steel 1.6x 0.2 in. in dimension. The strips varied in length relative to their position in the wall from 14 ft to approximately 20 ft.

The subsurface investigation performed for this project revealed no permanent groundwater table within the project grading limits. The Reinforced Earth granular backfills were expected to provide sufficient drainage of transient groundwater behind the wall. Therefore, no subsurface drainage system was installed.

During the design phase the Reinforced Earth Company had conducted laboratory strip pull-out capacity tests and strength tests on coarse-grained soils from local borrow sources. This information, in combination with much field testing in a variety of soil types, provided the basis for designing the wall. The backfill selected by the contractor was a residual gravelly, silty sand to be trucked in from off-site. Initial sampling of the borrow source indicated that it would meet the criteria specified for grain size.

The reinforcing strips were designed for strip pull-out capacity based on data obtained from samples that met the gradation specifications. The design pull-out capacity is dependent on length, position in the wall, and overburden pressure. The internal wall design allowed for a safety factor of at least two against strip pullout. The ultimate strip pull-out capacity allowed for a small safety margin in the event that hydrostatic pressures were applied to the wall from a transient groundwater condition.

PROJECT HISTORY

The sequence of events related to the displacements and reconstruction of the wall are summarized in Figure 2. Construction of the wall began in October 1978. The walls were nearly completed by December of that year. Some construction continued during the winter; however, earthwork was halted due to adverse weather conditions. In February 1979 an early thaw was preceded by several days of above normal precipitation (Figure 2). This amounted to approximately 22 in. of accumulated snowfall that melted over a 7-day period. During this time movements were noted along some sections of the Reinforced Earth walls. Typical movements consisted of tilting 10 to 12 in. out of plumb at its maximum section, which caused the wall facing to apply a lateral force on some adjacent piers. An example of the magnitude of movement is shown in Figure 3.

A detailed investigation was subsequently conducted into the probable cause of the movements. Recommendations were given concerning remedial construction of walls after the investigation was completed. The damaged panels were removed along with the Reinforced Earth backfill. New backfill was placed and the walls were reconstructed by December 1979. The reconstructed wall has experienced no problems in 3 years of service.

INVESTIGATIVE FINDINGS

The investigation consisted of 10 test borings and four hand-dug excavations along the wall (see Figure 1). All test holes were located 8 ft behind the face of the Reinforced Earth walls.

Three types of sampling procedures were used during the investigation to determine whether the method of sampling altered the grain size distribution of the samples obtained. Sampling at test holes TH-1-10 was performed with a 3-in. diameter hollow stem auger that had 1-7/8-in. wide spiral flights. All samples obtained were auger cuttings except for three undisturbed 3-in.-diameter Shelby

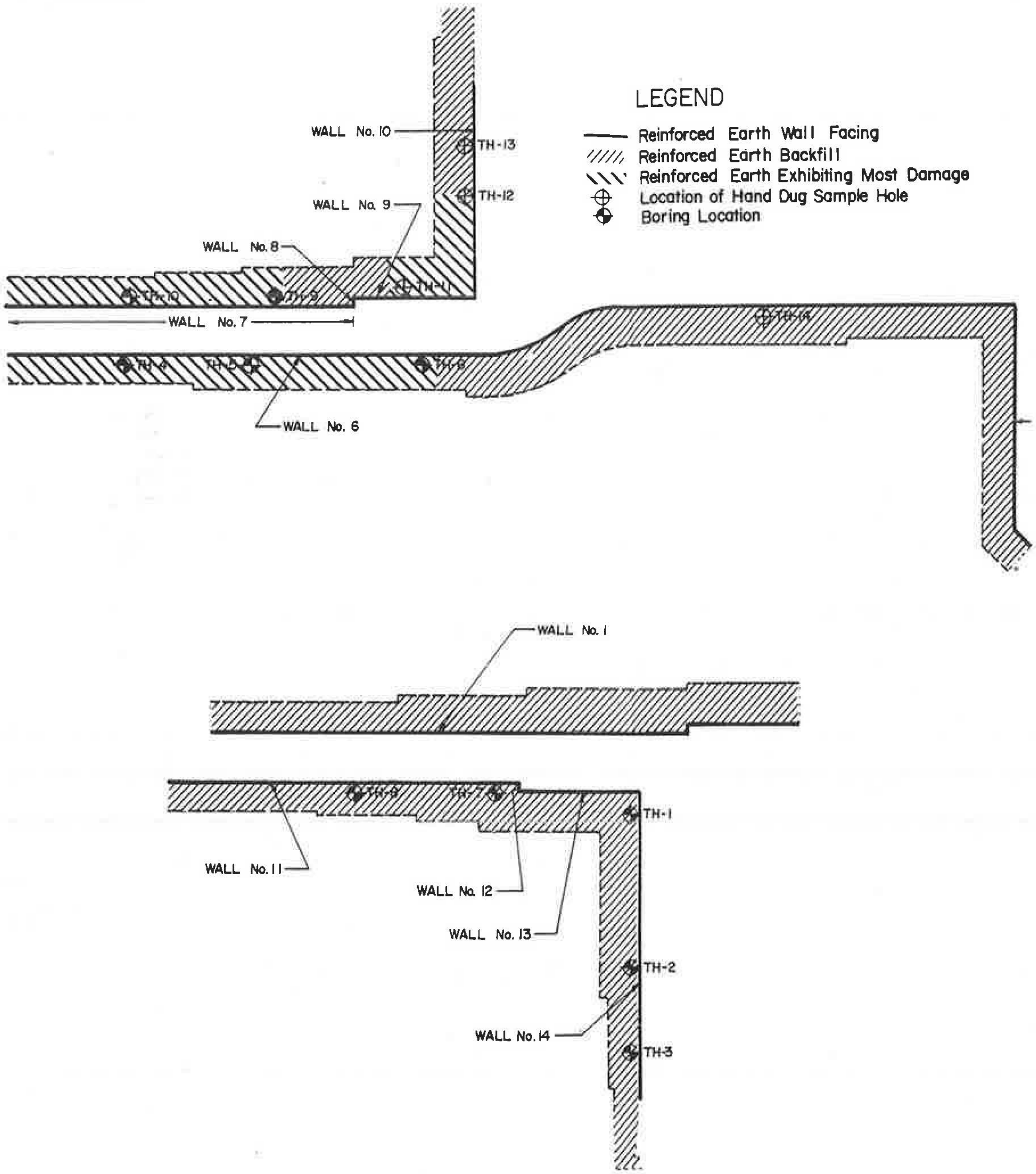


Figure 1. Site plan.

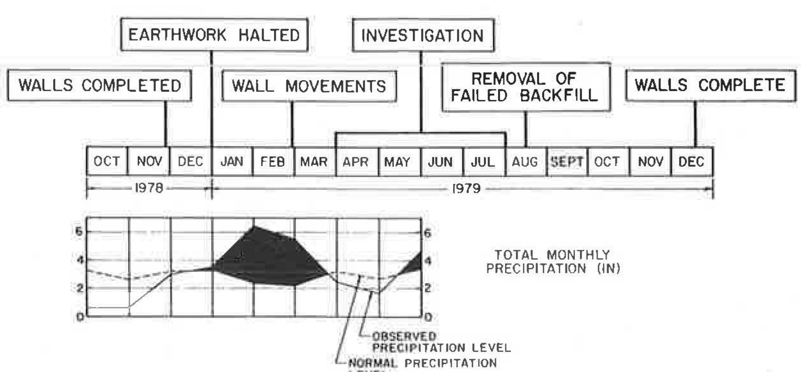


Figure 2. Sequence of events.

tube samples taken at test holes TH-6, TH-9, and TH-10. Hand-excavated bag samples were obtained at test holes 11-14, where test locations were inaccessible to truck-mounted drilling equipment.

Composite grain size distribution curves were plotted for each test hole location. Comparison of these curves indicated that similar grain size distribution curves were obtained by auger cuttings, undisturbed samples, and hand excavations. Also, the majority of the hand-excavated samples showed that less than 7 percent of the sample by weight was larger than the 1-1/2-in. sieve size. Therefore, grain size distribution curves for all of the samples should be considered representative of the in situ soils at that location.

A detailed set of testing was conducted, including field sampling, moisture contents, Atterberg limits, unit weights, Proctor compaction tests, and grain size analyses. The results of these tests are summarized in Table 1. The cause of the problem is best shown in Figure 4. The Reinforced Earth walls that exhibited the most severe damage were composed of excessively wet fill soils with a high fines content.

The investigation revealed that a significant portion of the Reinforced Earth backfill was not within the project gradation specifications. In the areas of severe wall distress, the backfill contained well over 30 percent and up to 50 percent fines. Hydrometer analyses of these soils also indicated that the majority of the backfill contained more than 15 percent finer than the 15 μm size, which has been shown by others to be the point where fines control the frictional nature of the soil (1).

Plasticity limits of the backfill were also outside of project specifications. Liquid limits ranged from 23 to 40 for samples with 15 percent or

Figure 3. Wall damage.

Table 1. Classification properties for Reinforced Earth backfill.

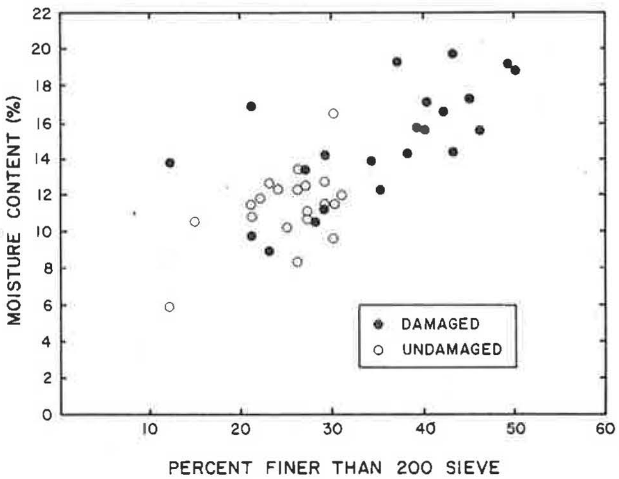
			Percentage Finer Than				Limits			
Test	Sample	Depth (ft)	No. 4	No. 200	15 μm	Natural Moisture (%)	Liquid Limit	Plasticity Index	Optimum Moisture (%)	Maximum Density[a] (lb/ft³)
TH-1	1	0-5	37	12		5.8				
TH-1	2	5-10	86	30		11.4				
TH-1	3	10-15	62	21		11.4				
TH-1	4	15-20	59	23		12.6				
TH-1	5	20-25	64	29		12.7				
TH-2	1	0-5	80	25		10.2				
TH-2	2	5-10	77	30	20	11.6				
TH-2	3	10-15	74	29		11.5	25	8		
TH-2	4	15-20	65	26		13.4				
TH-3	1	0-1	54	26		8.2				
TH-3	2	1-5	76	27		12.5				
TH-3	3	5-10	78	27		11.2				
TH-3	4	10-15	83	29		11.3				
TH-4	1	0-3	26	12		13.9				
TH-4	2	3-6	92	46	37	15.7	40	20		
TH-4	3	6-10	82	40	31	15.4	34	13		
TH-5	1	0-3	62	29		11.1				
TH-5	2	3-6	89	38		14.1				
TH-5	3	6-9	67	29	22	14.1	33	15		
TH-5	4	9-13	83	43		19.7				
TH-6	1	0-3	63	21		16.8				
TH-6	2	3-6	93	50	39	18.9	39	16		
TH-6	3	6-9	90	49		19.1			13.4	117.9
TH-6	4	9-11	71	34		13.9	33	12		
TH-6	5	11-15	77	39		15.7			13.8	119.5
TH-7	1	0-2	46	15	8	10.5	23	6		
TH-7	2	2-7	65	22	15	11.9	30	13		
TH-7	3	7-12	76	27		11.7				
TH-7	4	12-17	69	26		12.3				
TH-8	1	0-5	61	21		10.7				
TH-8	2	5-10	84	30	20	9.6	24	8		
TH-8	3	10-15	79	31		12.0				
TH-9	1	0-5	78	28	18	10.6				
TH-9	2	5-9	76	27		13.3	28	11	11.0	123.9
TH-9	3	9-11		45		17.3	36	NP		
TH-9	4	11-15	90	43		14.6	35	15	13.7	118.3
TH-9	5	15-20	84	42	30	16.5				
TH-10	1	0-3	66	21		9.7				
TH-10	2	3-6	63	23	17	9.0	26	6	10.1	124.5
TH-10	3	6-8	58	21		9.9	38	13		
TH-10	4	8-11	86	35		12.3			11.9	122.1
TH-11	1	0-2	78	40	22	17.1	32	7		
TH-12	1	0-2	71	37		19.2				
TH-13	1	0-2	68	30		16.5				
TH-14	1	0-2	55	24		12.1				

[a]All Proctors performed in accordance with ASTM D698.

greater passing the No. 200 sieve. Plasticity indices ranged from 6 to 20.

The shear strength of the backfill was estimated from conventional laboratory tests performed on samples that had different fines content. The tests consisted of classification testing as well as triaxial shear tests and direct shear tests. Test results are summarized in Table 2.

Figure 4. Moisture content and percentage fines for damaged and undamaged walls.



In addition, strip pull-out capacity was determined for each soil type at various confining pressures. The pull-out tests were performed in a large direct shear box. A diagram of the test apparatus is shown in Figure 5. The shear box accommodates a 3-ft-square, 19-in.-thick sample. The lower half of the box is stationary and is attached to the reaction frame. The upper half of the box consists of 9-in. steel channels and is separated from the rest of the apparatus by 1-in.-diameter steel ball bearings. The normal pressure is applied by nine calibrated springs restrained between two rigid plates. The uppermost plate is bolted to six threaded rods around the perimeter of the box, which are anchored to the base of the apparatus.

Slots for the reinforcing strips were cut in both the upper and lower halves of the box approximately 3 in. from the mid-height of the sample. The reinforcing strips were stressed by jacking against a reaction frame placed across the sides of the box at a deformation rate of approximately 0.10 in./min. The shear load was measured by a load cell positioned between the jack and the reaction frame. Dial gages attached to a stationary reference beam were used to measure strip movement.

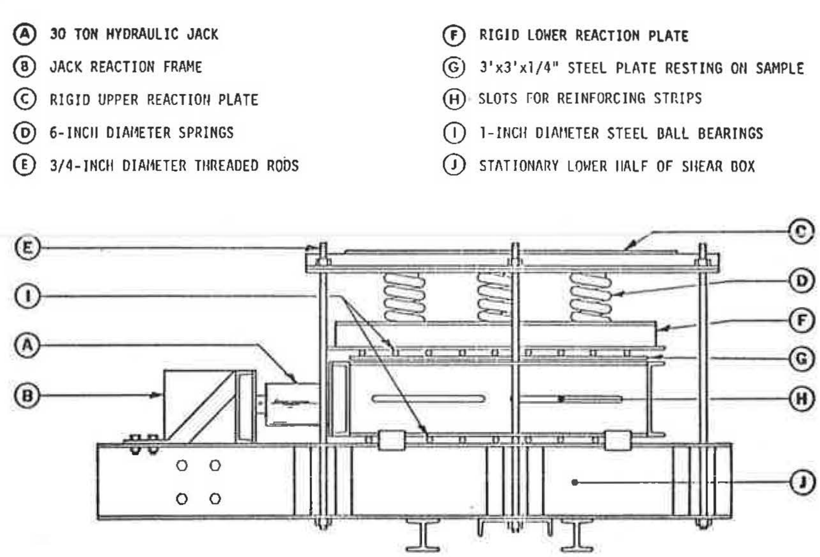
Test results are summarized in Figure 5. Compaction levels and moisture contents used to prepare the samples for testing were similar to those levels

Table 2. Residual soil strength property summary.

		Classification			Compaction		Direct Shear		Triaxial Shear			
Sample Source	Sample No.	Liquid Limit	Plasticity Index	Percent Fines	Optimum Moisture (%)	Max.[a] Density (lb/ft³)	C (lb/ft²)	Ø' Degrees	C (lb/ft³)	Ø' Degrees	Test Density (lb/ft²)	Test Moisture (%)
Project backfill	1	33	NP	23	10.1	124.5	600	33	175	32.9	115.3	13.1
	2	26	NP	26	11.0	124.0	280	34	100	31.4	118.3	11.8
	3	34	30	41	15.7	114.0	0	39	0	36.9	109.0	17.8
Related testing of residual silts	4	44	35	66	18.0	106.5	0	35			98.3	21.9
	5	63	NP	73	30.0	86.0	500	34			86.8	28.4
Related testing of residual sands	6	20	18	11	9.9	128.4	400	47			122.3	9.7
	7	20	18	11	9.9	128.4	500	39.5			124.0	11.8

[a]ASTM D698

Figure 5. Strip pull-out testing device.



observed from field testing of the backfill. The strip pull-out capacity has been reported in terms of the maximum shear stress applied to the strip divided by the strip normal pressures. This value is termed the apparent friction coefficient (f*).

All samples tested exhibited a significant decrease in f* with increased fines content. The apparent friction coefficient of Reinforced Earth strips in soils with 40 percent fines was observed to be a factor of 2 less than that for soils with 25 percent fines.

An evaluation of the as-built condition of the Reinforced Earth wall was based on these test results. The reduced pull-out capacity of the backfill soils and the possibility that hydrostatic pressures may have developed within the backfill due to poor drainage is sufficient to produce a near-failure condition. Wall movement was not noted in areas where the Reinforced Earth backfill soils were determined to contain less than 25 percent fines. An acceptable safety factor against strip pull-out was back-calculated for these wall sections based on the higher values of apparent friction coefficient available.

Figure 6. Strip pull-out capacity tests with increasing fines content.

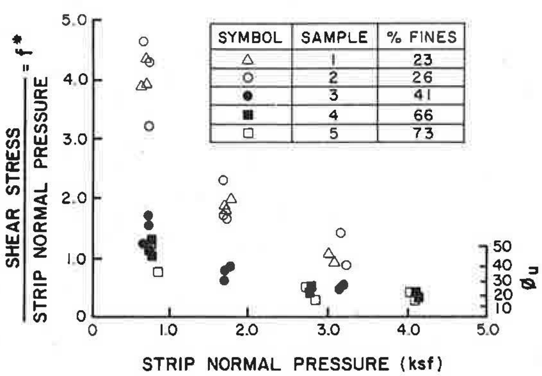
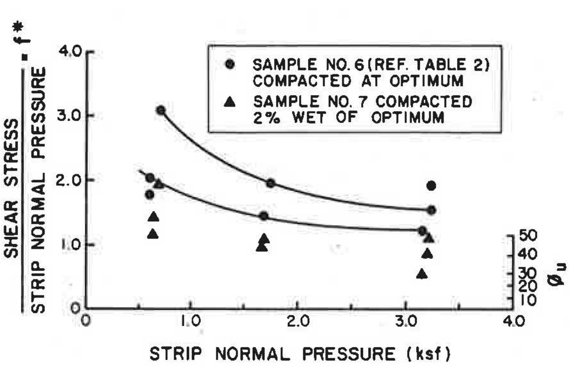


Figure 7. Strip pull-out capacity with increasing moisture content.



Based on the investigation, the areas of Reinforced Earth backfill with more than 25 percent fines were identified, excavated, and replaced with select backfill. The fines content of the new backfill was limited to 25 percent. Most of the concrete panels were reused; however, many of the reinforcing strips were damaged during excavation.

RELATED TESTING

This project provided an opportunity for test results to be compared with previous testing of residual soils. In the past the Reinforced Earth Company has conducted research into the pull-out capacity of ribbed reinforcing strips subjected to variations in fines content and moisture conditions for various residual soils. A summary of the tests previously conducted on residual soils is also shown in Figures 6 and 7.

The apparent friction coefficient (f*) varies considerably with the normal pressure applied to the strip. This trend is observed for all compacted soils. High f* values at low confining pressures are the result of dilatancy in soils that are granular and to some extent to locked-in compactive stresses that cause the soil to be heavily overconsolidated. At higher normal pressures the f* approaches a limiting value. At all pressures the effect of fines content is pronounced. Between 30 and 40 percent fines there is a drastic reduction in the value of f*. Within this range the fine-grained portion of the soil becomes dominant and controls the strength characteristics. It is also evident that the same trend of decreasing f* occurs with increasing fines content from 40 to 70 percent; however, relatively, the effect is much less.

Table 3 is a summary of laboratory direct shear tests (quick tests) performed on French soils to determine short-term internal friction of sand with increasing percentages of fines. Quick tests were performed to compare with results of pull-out tests on reinforcing strips. All samples were compacted to optimum conditions. Some samples were tested at optimum conditions whereas others were saturated before shear.

Figure 8 compares these test results (dashed lines) with those of similar tests performed on a clay-sand mixture without any silt component (solid line). The ratio of ϕ/ϕ_{ou} is shown as a function of the proportion of fines. The value ϕ_{ou} is the angle of internal friction of the saturated soil containing only sand. The value of ϕ_u is that of the saturated soil consisting of sand and fines. Fines has been interpreted in one case as the amount of clay and silt in the mixture. A second interpretation of the same test results define fines as the amount of clay grouping the silt with the sand skeleton.

The soils exhibit a marked reduction in strength; more than 20 percent are finer than the 15 μm size. The curves also indicate that little or no reduction in strength occurs for soils with less than 10 percent passing the 15 μm size. Only

Table 3. Property and strength summary of laboratory soil mixtures.

Sample	Percentage Passing			Liquid Limit, W_L	Plasticity Index, IP	Shear Test Under Optimum Conditions		Shear Test on Soil Compacted to Optimum and then Saturated	
	<80 μm	<15 μm	<2 μm			ϕ	C(kPa)	ϕ	C(kPa)
M0	14	6	3			36	12	33	0
M5	19	10	7			36	8	35	3
M15	26	19	15	19	6	36	3	27	34
M25	34	27	23	23	10	30	37	21	40
M35	41	34	31	29	15	17	51	17	44
M100	97	86	82	93	58	16	131	0	95

Figure 8. Strength reduction with fines.

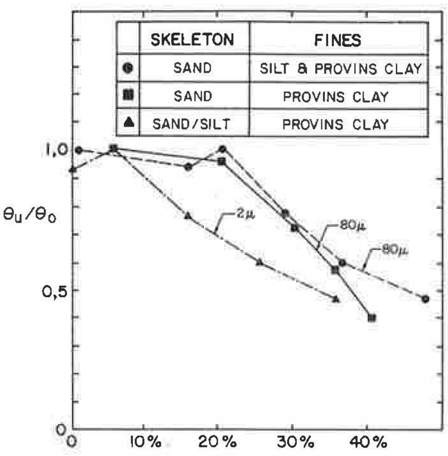
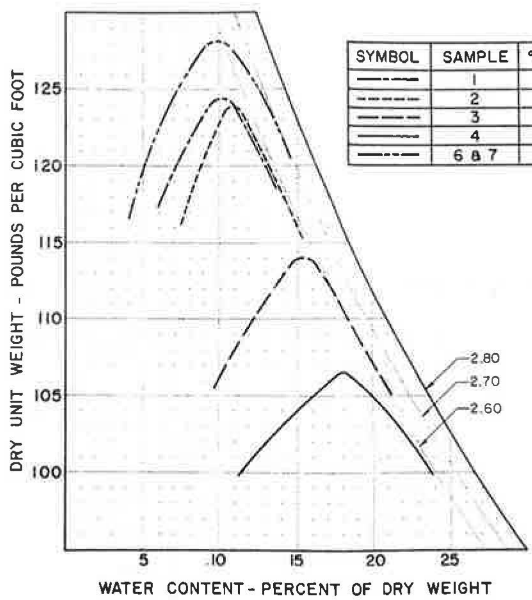


Figure 9. Moisture density curves for residual soils tested.



minor strength loss was observed for soils with 15 percent finer than the 15 μm size. The departure of the silt-sand skeleton and clay fines mixture from the other soils confirms that silt size particles should be grouped with the clay fraction rather than the sand fraction.

A series of tests conducted on a 6-m-high test wall completed in 1981 in a Reinforced Earth test facility confirmed a reduction in pull-out capacity when the soils were saturated before testing. With increasing fines the reduction due to saturation was greater. Within the Reinforced Earth backfill specification guidelines the reduction ranged up to approximately 30 percent. Beyond the specification limits (more than 15 percent passing the 15 μm size) the reduction ranged up to 70 percent.

The effect of saturation can also be seen from these test results. The percentage of strength reduction with saturation is increased with increasing fines. The point at which the frictional strength component is controlled by the fine-grained materials appears to occur between 10 and 20 percent passing the 15 μm size. This trend is apparently valid for both partly saturated and fully saturated soils.

A practical application of these trends can be related to the compaction moisture content. Figure 7 is a summary of f* for two samples of a micaceous residual sand from Virginia compacted at slightly different moisture contents. The effect is a significant decrease in f* with a compaction moisture content only 2 percentage points above optimum. Optimum moisture is within 3 percentage points of saturation. Consequently, this soil is sensitive to moisture variations. One explanation may be the effect of mica in the soil matrix. These soils were derived from in situ weathering of the Wissahickon schist. The test samples appeared highly micaceous based on touch and texture. Soils such as these can be difficult to compact even at optimum moisture content (4). The moisture density relations shown in Figure 9 are typical of many residual soils. The optimum moisture content is relatively high, near the zero air voids curve. Soils compacted at or slightly wet of optimum moisture are nearly saturated and, therefore, are sensitive to moisture variations. Soils such as these will exhibit a marked reduction in strip pull-out capacity if compacted wet of optimum.

CONCLUSIONS

The study shows that fines content and moisture content are important factors when constructing Reinforced Earth in residual soils. Between 10 and 20 percent passing the 15 μm size, the fine-grained portion of the fill controls the desired strength properties. The use of Reinforced Earth backfill with a higher percentage of fines can result in a significant reduction in strip pull-out capacity and decrease the internal stability of the wall. The reduction in frictional strength with increasing percentage of fines is even more pronounced in saturated soils.

Caution should be exercised when using residual soils as Reinforced Earth backfill. The fines content of residual soils is subject to large variations over small distances. Also, coarse-grained highly-micaceous sands common to the residual soils of the Piedmont region exhibit a marked sensitivity to moisture and significant strength reduction with increased compaction moisture content. The combination of moisture sensitivity and highly variable fines content requires strict quality control during construction to avert problems in these soil types.

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Landslide Analysis Concepts for Management of Forest Lands on Residual and Colluvial Soils

RODNEY W. PRELLWITZ, TERRY R. HOWARD, AND W. DALE WILSON

A forest land management analysis scheme is discussed for dealing with landslides that occur in residual and colluvial soils. No one geotechnical or statistical model can be expected to apply to all levels of land management where an assessment of the potential for landslide is vital to a rational decision-making process. The U.S. Department of Agriculture Forest Service in cooperation with the University of Idaho is developing a scheme for evaluating soil-mantle landslide potential to provide information at three levels of land management activities: (a) resource planning; i.e., relative landslide hazard evaluation for resource allocation; (b) project planning; i.e., evaluation of management impacts for comparing alternate transportation routes and timber harvest techniques; and (c) road design and landslide stabilization; i.e., evaluation of alternate road stabilization techniques at a specific critical site. Both geotechnical and statistical analysis techniques are advocated so that the information can be in geotechnical form (factor of safety against failure or critical height of slope) or in statistical form (probability of landslide occurrence) with landslide inventories used as a link between the two. A hypothetical example of the three-level analysis is given.

Many forest lands in the West, particularly those on residual and colluvial soils, are classified as unstable and have a high potential for mass failure. Timber-harvesting operations, road construction, and other resource-management activities in these areas can accelerate mass erosion and cause significant degradation of water quality unless carefully planned and executed. Successful management of these lands requires development of a specialized body of knowledge to quantify and integrate those site factors that influence slope stability. Site factors that require special attention are slope, soil depth, soil shear strength, seasonal ground water levels, and the strength derived from vegetation (effective root strength). Geotechnical characterization of these site factors can then be the basis for a landslide hazard analysis tailored to a specific management decision level.

MANAGEMENT COMPLEXITY

The management of lands that have a high potential for landslide is inherently complex, not only because of the nature of the interacting natural processes and management activities but also because of the number of persons of varied disciplines who must possess a degree of understanding of the slope failure processes and be able to contribute to the total stabilization effort. Considerable overlap and interaction between members of key disciplines must be coordinated.

Members of different disciplines must deal with problems of slope stability at several levels of intensity. For example, the resource planner must recognize high-hazard areas, but only on a general scale. The road locator needs to recognize potentially unstable areas along proposed routes and to avoid the problem through adjustment in alignment. The engineer must be able to use soil mechanics in the stability analysis of remedial measures before, during, and after construction to prevent or correct specific road cut or fill slope failures.

FAILURE MODE

Consistent with Varnes (1), landslides may be grouped into two broad categories, depending on the type of slide mass material--either soil (debris or earth) or bedrock. This grouping enables orderly selection of stability analysis techniques and the data required. The concept should apply to soil or bedrock landslides with the proper selection of slope analysis techniques and required data. However, this discussion is directed at landslides where the failure is confined to a soil mantle primarily of colluvial or residual origin.

The usual setting for this type of failure is a relatively loose, cohesionless soil mantle that overlies a less permeable bedrock or denser soil mass. An exception to this is an extremely altered bedrock or residual soil near the surface that overlies a less altered bedrock at some depth. Each of these conditions can result in similar failures and can be analyzed in the same manner. The contact with the underlying, less permeable, material forms a drainage barrier for the normal downward migration of ground water that originates from rainfall, snowmelt, or both. Ground water is concentrated at the drainage barrier and, if sufficient quantities are available, the soil mantle develops within it a perched water table with seepage moving along the barrier. The drainage barrier, phreatic surface (water table), and ground surface are often parallel or nearly so. Seepage of this form is usually considered to be of the infinite slope form because of this parallelism.

Failure of the entire soil mantle can occur naturally due to higher-than-normal ground water concentrations that result from unusually high rainfall or snowmelt. Failure also may result from wildfire, which destroys vegetation and thus the beneficial effects of evapotranspiration and root strength. Failure more often occurs through land management activities such as timber harvest and road construction, which in some manner increase ground water concentration, destroy root strength, or affect the natural parallelism of the ground surface or phreatic surface in relation to the drainage barrier.

Failures are often confined to the soil mantle because the underlying material usually has a higher strength and the critical failure surface is usually at the maximum depth of the soil and water table (tangent to the contact with the drainage barrier). The failure surface may be circular arc or translational in shape, depending on local conditions. Translational failures may begin as a small circular arc and progress into a translational shape or a series of circular arc failures as more of the soil mantle is mobilized.

IDEALIZED LANDSLIDE EVALUATION SYSTEM

A complete system of landslide hazard evaluation is needed that begins early in the resource planning phase, follows through into project development, and provides information back to the planning phase to improve future hazard analyses. The system should be structured on a common scheme but branch early into either soil-mantle landslide analyses or bedrock landslide analyses and use the respective analysis techniques and data. In either case, the complete system should be structured on a common basic analysis form that is simplistic in the resource planning phase and requires primarily available resource inventory data and becomes more complex and

Table 1. Idealized analysis system.

Item	Level 1, Resource Allocation	Level 2, Project Planning	Level 3, Critical Site
Base map	Landslide hazard map on resource inventory scale; 1:24,000; 1 in. = 2,000 ft	Project map of larger scale; 1 in. = 500 ft	Critical site map on even larger scale; 1 in. = 20 ft to 1 in. = 100 ft
Stability analysis	Infinite slope equation requires values for geotechnical variables and their inherent variance	Combination of infinite slope analysis from level 1 but used to model effects of tree removal and critical height analysis of anticipated road cut and fill slopes	Critical failure path analysis by computer program with search routine for circular arc, translation failures, or both; anticipated drained phreatic surfaces generated through computer analysis to predict effects of road with and without various stabilization techniques on infinite-slope-recharged phreatic surface (2,3)
Data display	Resource inventory map overlay of factor of safety against failure or probability of landslide occurrence	Same as level 1 but for more localized project area that has potentially unstable locations of road cut and fill slopes shown on proposed route	Cross-sections of critical site conditions with proposed road and alternate stabilization techniques superimposed
Required data	Available forest resource inventory data, values for geotechnical variables and variance through broad characterization of forest land forms, variables and analysis model tested and refined through association with landslide inventory and subsequent evaluation in levels 2 and 3	Level 1 data, data from timber and route reconnaissance to delineate local areas within project where failures are most likely	Surface and subsurface critical site data; subsurface data from geophysical methods and drilling if severity warrants; soils and ground water hydrologic data from soil sampling and testing and ground water monitoring
Prime use	To delineate areas susceptible to landslides on broad scale to alert land manager to land units where hazard intensity is greatest; through statistical correlation to landslide inventory, to predict number and magnitude of landslides as a result of resource development	To assess severity of instability more accurately as local islands of instability are predicted through reconnaissance; to make decisions to limit development or to continue to level 3 analysis based on improved assessment of probable failure magnitude and intensity; to better evaluate transportation planning, timber harvest techniques, and route locations for project so critical sites can be isolated along selected routes where level 3 analysis will have most benefit	To select and design road stabilization measures through relative stability-probability of failure cost analysis of feasible alternatives

requires more exact data only as the intended use demands greater accuracy.

For soil-mantle landslide analysis the ideal system should be structured to

1. Provide landslide hazard evaluation to guide management decisions on unstable lands at three crucial phases: resource allocation, project planning, and road design;
2. Include soil, vegetation, slope, and ground water hydrologic variables together with their inherent natural variance in a geotechnical analysis (factor of safety against failure or critical height of slope), a statistical analysis (probability of landslide occurrence), or both;
3. Begin with a simplified analysis that requires primarily available resource inventory data and progresses into more complex analyses that require more exact data (the selection of technique should be commensurate with the level of management decision; thus, the user at any level is faced only with the complexity and need for data required at that level); and
4. Facilitate the inventory of new landslides as they occur and slope failures as they are corrected and feed back the data gathered into earlier processes to improve the planning of subsequent projects.

Three levels of analysis complexity and data are visualized for the idealized system in Table 1.

RESTRICTIONS ON USE

Existing Stability Analyses

Current restrictions on the use of an idealized evaluation system for soil-mantle landslides are not due to the lack of slope stability analysis technology. The program recently developed by Simons, Li, and Ward (4) for mapping potential landslides is based on an infinite slope analysis and includes both factor of safety against failure and probability of landslide occurrence options for a level 1 analysis. Stability number charts that have seepage correction factors are being developed for infinite slope seepage conditions (5) and converted to computer programs for the critical height analysis of typical road cut and fill slopes in a level 2 analysis. Numerous programs are available and in use by geotechnical specialists in stability analysis for the correction of existing landslides that have either circular arc and translational failure surfaces. The most widely used methods of slices (primarily the Fellenius, the simplified Bishop, and the Janbu methods) can be integrated into one program to cover a variety of failure surface analyses for level 3. Statistical counterparts for the probability of landslide occurrence option used in level 1 are planned for levels 2 and 3 based on methods currently used in geotechnical engineering (4,6).

Existing Data Base

One current restriction on using the system is the small existing data base for most forests. Many forest managers have (or are in the process of developing) resource inventory maps for soils, bedrock, topography, timber type, and other features. These maps could provide the start of a level 1 data base through proper characterization of geotechnical variables for the inventoried conditions. Statistical analyses used by Simons and Ward (4) and DeGraff (7) will prove invaluable for linking inventoried physical factors such as bedrock, aspect, and slope to inventoried landslides. The accuracy of the values assigned for geotechnical variables, analysis models, and the probability of landslide occurrence can be tested through association with corresponding physical factors. Currently, only a few forests have landslide inventory data. Geomorphic landtype maps (8), where available, should be the most useful tool for geotechnical variable characterization because the landtype classification includes the major physical factors on which to assign values for the variables.

Existing Variable Definition Methodology

The main restriction to implementing the system is the current state of the art in defining certain geotechnical variables. Techniques for defining slope, soil depth, and soil shear strength have progressed to a state where the values and their variance can be used with some degree of confidence. This is not true for the two most dynamic variables--ground water concentration and tree root strength.

The part of the soil mantle that can be expected to be below the phreatic surface at any point in time is perhaps the most dynamic of the variables. It can fluctuate constantly in response to precipitation. Practical and inexpensive methods are needed to develop local correlations between rainfall and snowmelt and the resulting rise in ground water. Although general knowledge of the time-related effects of tree root strength on forest slope stability has been advanced through research, currently no cost-effective quantitative methods are available for determining the effective tree root strength to use in analysis. To use the system now, it may be necessary to back-calculate to determine values for these two important variables until the state of the art progresses.

ILLUSTRATIVE PROBLEM

The following hypothetical problem illustrates the concept of the three-level analysis system. Where available, actual analysis results are used to demonstrate current progress. All studies within the project should be completed by mid-1985.

Level 1 Analysis for Developed Area

Step 1

Figures 1-3 show drawings of three inventory map overlays for part of the Clearwater National Forest in northern Idaho--the transportation map, landslide

Figure 1. Transportation inventory map of developed part of Clearwater National Forest, Idaho.

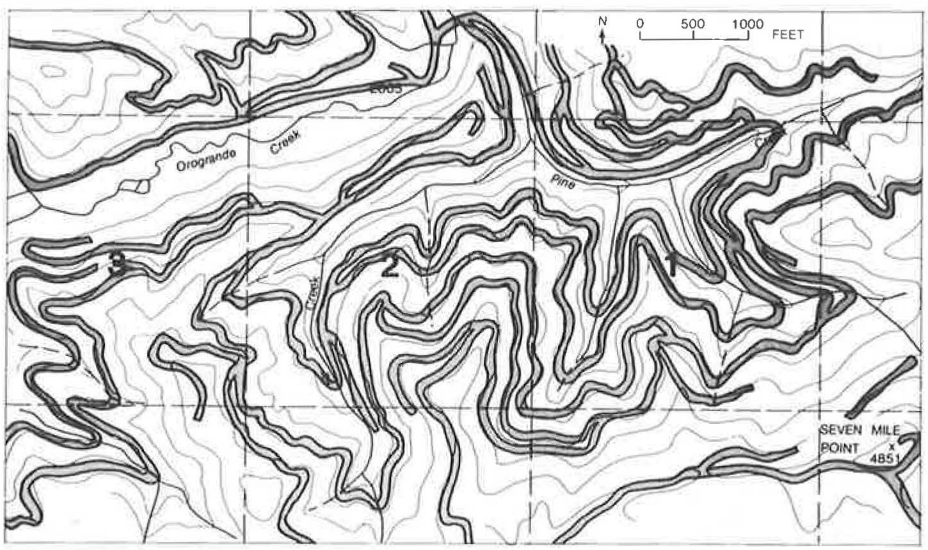


Figure 2. Landslide inventory map of area in Figure 1.

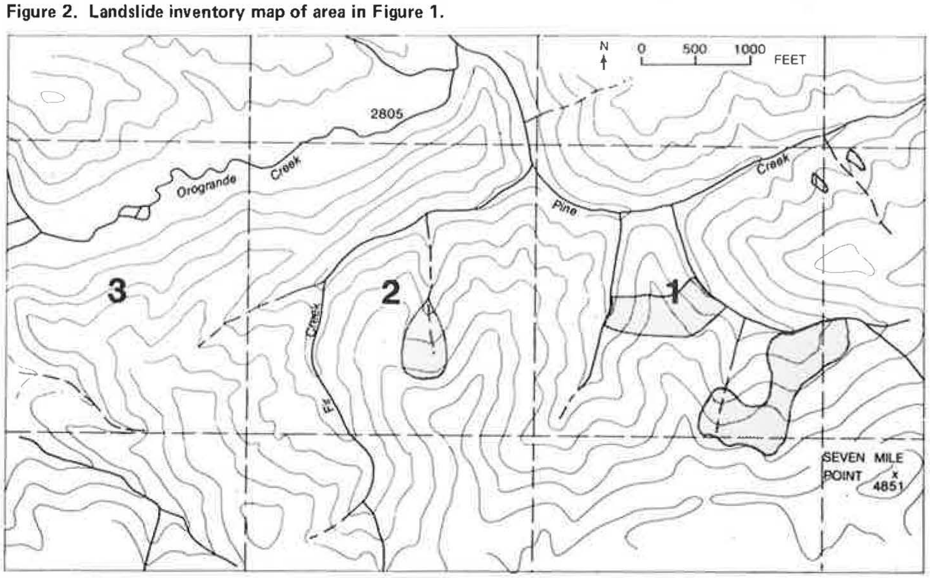


Figure 3. Geomorphic landtype inventory map of area in Figure 1.

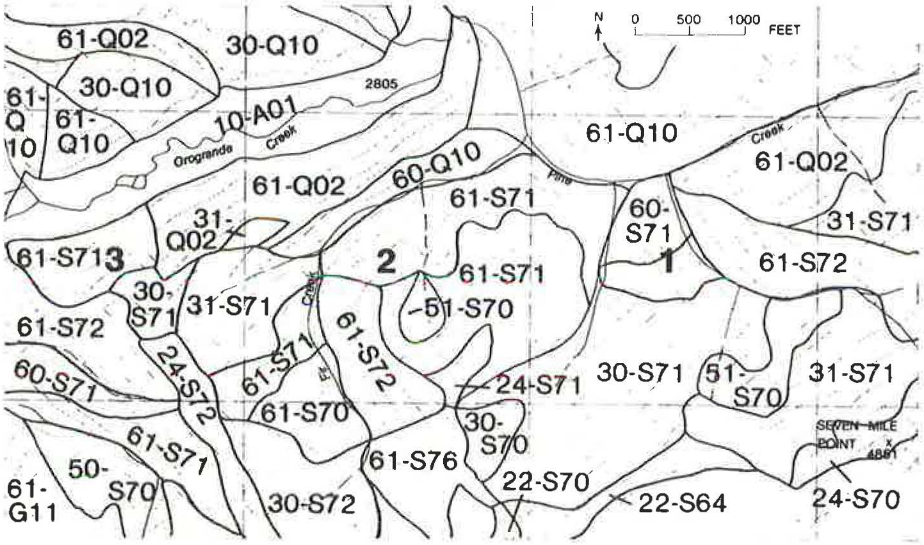
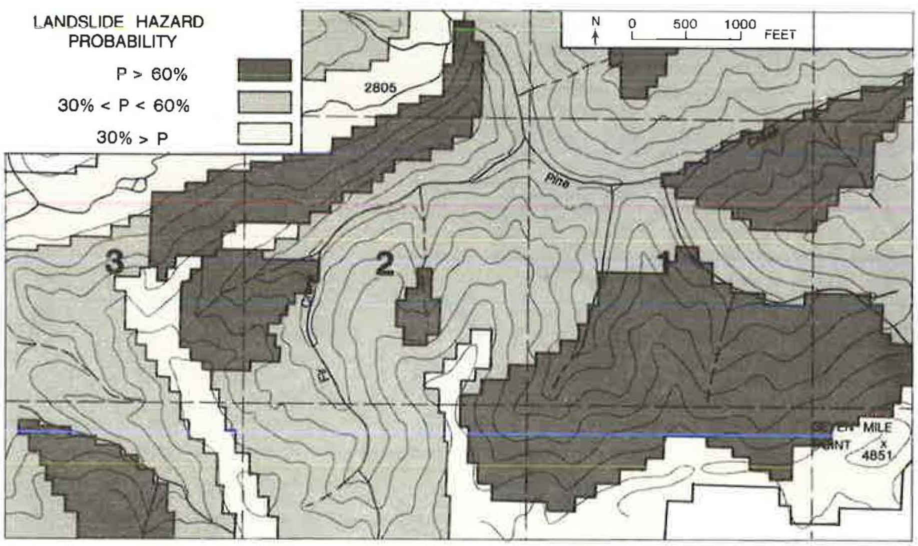


Figure 4. Results of level 1 analysis for Figure 1 area by using probability of landslide occurrence option.



inventory map, and geomorphic landtype map, respectively. The association of these inventories is the most useful for this forest as an initial level 1 data base. The transportation and landslide maps show the location of all existing roads and landslides. The landtype map shows the boundaries of distinctive geomorphic landforms. The coded landtype classification includes the major geomorphic physical factors that make this mapping unit distinctive (8). Physical factors such as parent rock type, aspect, slope, and timber type are used in the classification and corresponding values for geotechnical variables such as soil shear strength, soil depth, slope, and ground water concentration, and are characterized and stored into an initial level 1 data base. This data base will be updated by using new data from levels 2 and 3.

Step 2

The level 1 (infinite slope) analysis is completed. The results will be either in ranges of factor of safety against failure or probability of landslide occurrence. Figure 4 shows the printout of the Simons, Li, and Ward program by using the option of ranges of probability of landslide occurrence for the data from step 1. This analysis for developed areas can be repeated as necessary to test and refine the model and variable values until the planner is satisfied that the results correlate realistically to the landslide inventory. In this procedure, the planner must remember that the values are to be used in conjunction with the infinite slope equation. This model is selected for this level of analysis because of its simplicity but not necessarily because of its accuracy. Accuracy depends largely on how well the model fits the ground water concentration mechanism and whether translational failures develop; even then the model will probably be applicable only to parts of any landtype (where the worst conditions exist).

Level 1 Analysis for Undeveloped Area

Step 3

Step 3 is similar to steps 1 and 2 for adjacent undeveloped areas with similar landtypes. Figure 5 shows the transportation map of the undeveloped area. Figure 6 is the level 1 analysis printout of landslide hazard probability. By beginning the analysis in this manner, the planner can calibrate the analysis by using the developed areas for predictions about the undeveloped areas to aid the land manager in resource planning decisions on whether or not to develop, how intensely to develop, and the landslide risk involved as a result of development. In addition, the following advantages are available through a level 1 analysis:

1. The land manager can be given a comparison of landslide magnitude and consequences by relating to experiences in the developed areas.
2. The accuracy of at least some of the level 1 data base can be improved through the feedback loop from levels 2 and 3, which follows.
3. The intensity and location of the level 2 analysis can be planned commensurate with the anticipated landslide hazard.

Level 2 Analysis

Step 4

Figures 5 and 7 show the area selected for level 2 analysis on levels 1 and 2 scales. In this case, the level 2 analysis is used to evaluate two possible routes to a proposed log landing site. Reconnaissance data are gathered at selected cross-sections along each route for better assessment of the extent of the anticipated problem areas and estimation of values for geotechnical variables.

Figure 5. Transportation resource inventory map of undeveloped part of Clearwater National Forest, Idaho, showing existing road terminal.

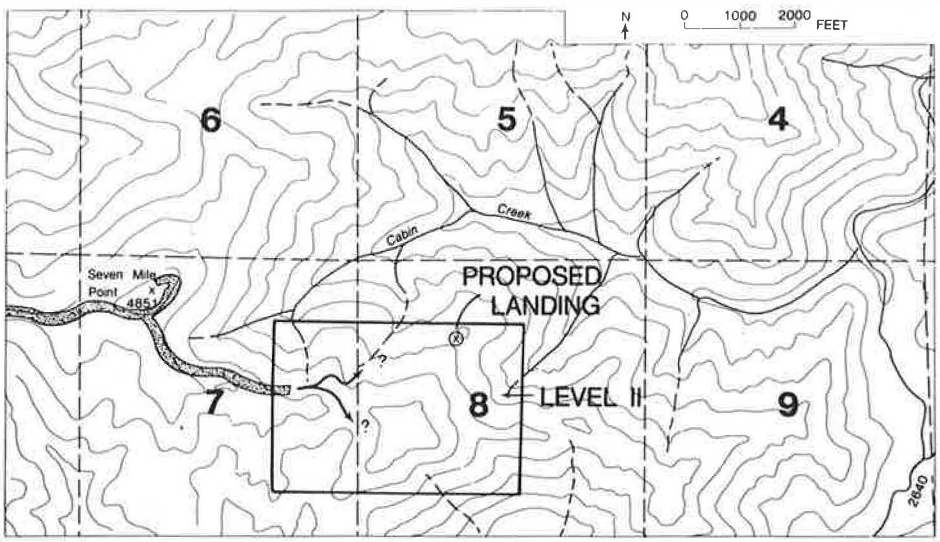
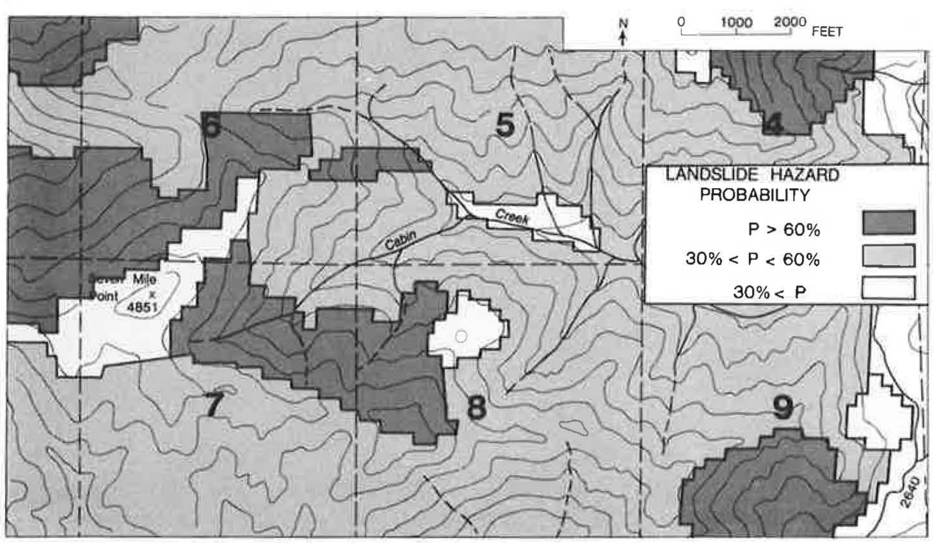


Figure 6. Results of level 1 analysis for undeveloped area in Figure 5.



Step 5

Typical road template sections are superimposed on the selected cross-sections and cut slope height, fill slope height, and the relation of cut and fill to the ground water level, root zone, and drainage barrier contact are determined by computer analysis. Figure 8 shows a self-balance road template commonly used on forest roads (cut volume balances fill volume with appropriate compaction factor). The critical heights of the cut and fill slopes are then determined and compared with the anticipated slope heights. Figure 9 shows the prototype program printout from a programmable calculator for a combined levels 1 and 2 analysis of the cross-section of Figure 8. The compaction factor can also be evaluated by this analysis. A full-bench road template may also be used on steep slopes where a fill slope will not catch or would be too high.

Step 6

A program similar to that used for Figure 9 will be developed as a subroutine for a computer analysis that represents the results as either S for stable or U for unstable on a project map. In addition, a statistical subroutine will be developed similar to that in level 1 for an optional output in terms of probability of slope failure. Figure 10 is a hypothetical drawing of the anticipated display.

Step 7

To assess the impact of timber harvest (tree removal) on the stability of the natural slopes, the level 1 analysis will be repeated at level 2 with changes made in tree-root strength, tree surcharge, and ground water concentration to reflect the impact

Figure 7. Level 2 analysis area showing location of alternate routes to proposed landing and selected cross-section locations on each route.

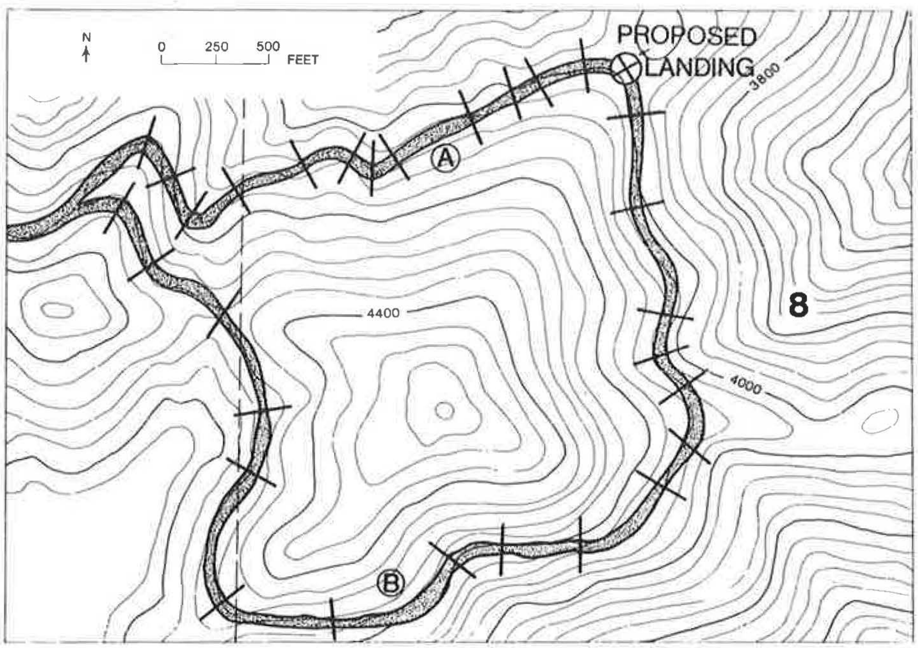


Figure 8. Self-balancing road template cross-section from level 2 analysis summarized on Figure 9.

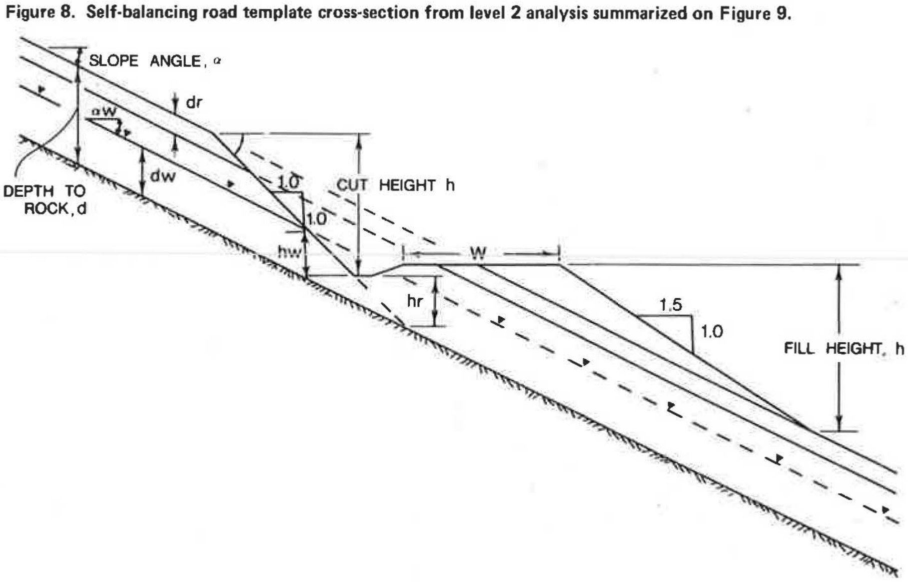
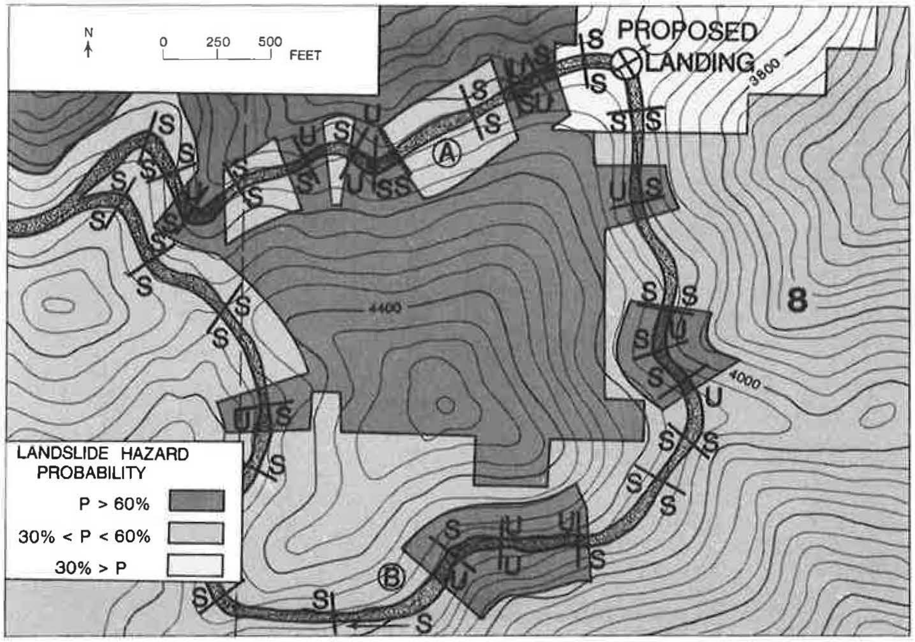


Figure 9. Printout of level 2 analysis of Figure 8 cross-section data.

```
STA.134+50

SOIL DATA
 DEN.1 DEN.2  PHI  COH.
 120.0 130.0 32.0 40.0

ROOT DATA
 5 YRS. AFTER HARVEST
ROOT COH.= 20.0PSF/FT.

SITE DATA
ALPHA= 50.0%= 26.6DEG.
AL.W= 50.0%= 26.6DEG.
  d   dw   dr
 10.0 5.0 2.0
   INF. SLOPE F.S.= 1.02
          STABLE

ROAD DATA
       CUT SLOPE= 1.00:1
DITCH
 SLOPE  DEPTH  BOT.W
 3.0:1   1.0   2.0
     ROAD WIDTH= 16.0FT.
      FILL SLOPE= 1.50:1
       COMP. FACT.= 25.%
FOR 50.% LOSS IN dr,
        COMP. FACT.=22.%

SELF-BALANCE SECTION

CUT
 h    hw    hr
 14.4  4.4 -5.6
 N     S    Hc
 42.8 0.52 14.7
          STABLE

FILL
 DEN.  PHI  COH
 130.0 34.0 40.0
 h    hw    hr
 17.3 -1.4 -17.3
 N     S    Hc
 151.8 0.73 34.0
          STABLE
```

Figure 10. Hypothetical drawing of the probability of landslide occurrence for level 2 analysis.



(9). The uses of the level 2 analysis are then as follows:

1. To facilitate management decisions on development through evaluation of alternate transportation routes and alternate timber harvest techniques and
2. To locate the critical sites where level 3 analyses are necessary on the selected routes.

Level 3 Analysis

Step 8

Figures 11 and 12 show one critical site selected for level 3 analysis on levels 2 and 3 scales. A critical site investigation (both surface and subsurface) is made for each site selected. The extent of this investigation and the subsequent analysis are planned by the geotechnical specialist in the same manner as a landslide correction project is planned.

Step 9

The antipipated road section is superimposed on cross-sections of the critical site and the stability of the anticipated cut and fill slopes are analyzed for circular arc, translational failure, or both. This step differs from step 5 in that the mode of failure is analyzed to determine the failure surface that has the least factor of safety and the anticipated extent of the slide mass. Many stability analysis programs are in use that would serve as a level 3 analysis for either shape of failure surface. Plans are to formulate the most functional of these as subroutines for one master program. Figure 13 shows possible translational and circular arc failure surfaces for the cut slope on the cross-section of the critical site. Figure 14 shows a programmable calculator printout for a program that combines the Fellenius (ordinary method of slices), simplified Bishop, and Janbu methods of slices solution for failure along these surfaces. The master computer program will combine analyses such as these, which can be preselected by the designer in conjunction with failure surface predicting, slice generating, and optional search for minimum factor of safety subroutines. Subroutines for predicting the steady-state drained phreatic surface to be expected from an infinite slope seepage source will also be programmed to evaluate the various drainage conditions in steps 9 and 10.

Step 10

The analysis of the unstabilized case in step 9 serves as a standard of comparison for the relative stabilization technique analysis that begins with step 10. In step 10 all feasible stabilization alternatives are analyzed to determine the relative increase in factor of safety over the unstabilized case.

Step 11

Decision analysis components (6) are determined for each alternative:

1. Probability of failure,
2. Construction and maintenance costs,
3. Consequences of failure (cost of failure).

Level 3 analysis provides the design engineer a decision analysis through which to select the optimum stabilization alternative for the current constraints.

Feedback to Level 1

Step 12

The data gathered for levels 2 and 3 are fed back into the level 1 data base to improve future analy-

Figure 11. Level 2 base map showing one area selected for level 3 analysis.

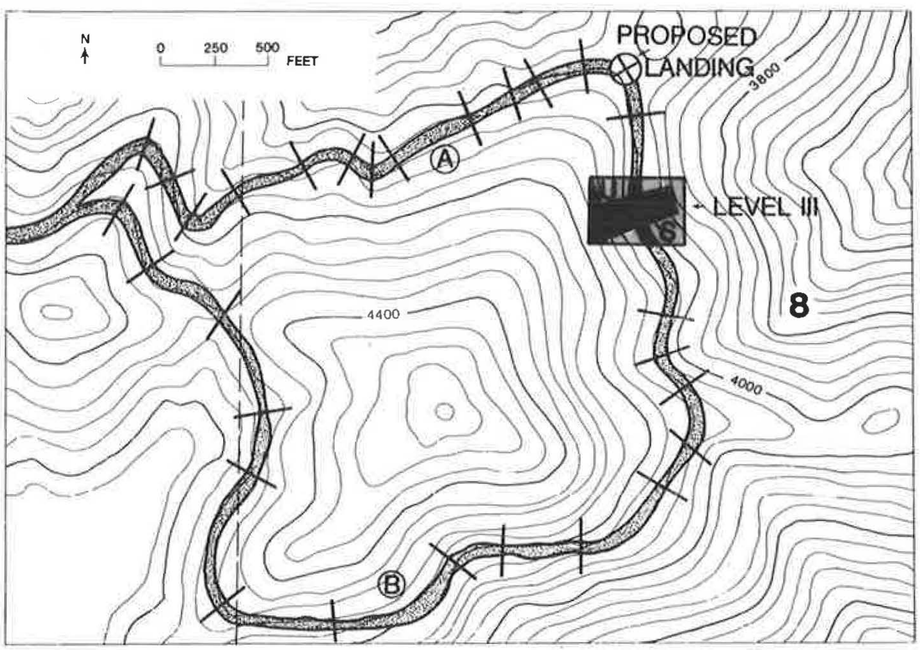


Figure 12. Level 3 analysis area showing proposed road.

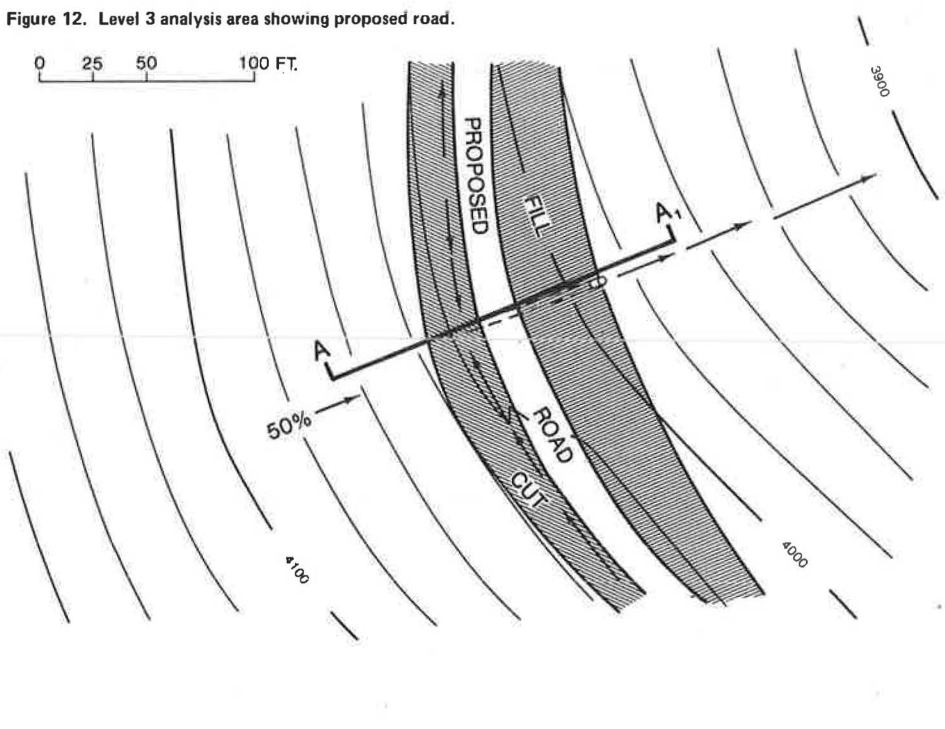


Figure 13. Cut slope portion of cross-section A-A' from Figure 12 showing possible circular arc and translational failures analyzed in Figure 14.

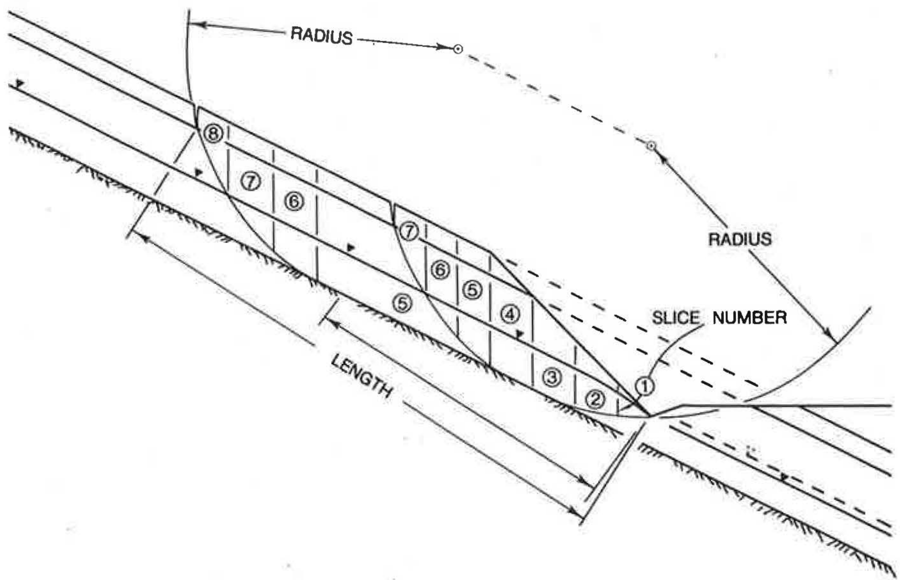


Figure 14. Printout of the level 3 analysis of Figure 13 cross-section data.

```
I.D.: CIRCULAR ARC

OMS BISHOP JANBU
           chord d= 4.9
           chord l= 29.5
TENSION CRACK
                Zw= 2.0
                 a= 6.6
                 R= 25.0
MIN FS= 0.70
MAX FS= 0.90

SLICE 1
NEW SOIL
 DEN.1 DEN.2 PHI  COH.
 110.0 125.0 32.0 0.0
THETA  dl  dW  X  AL.W
 4.0 0.5 1.0 3.0 33.5

SLICE 2
THETA  dl  dW  X  AL.W
 11.5 1.7 2.8 4.0 28.0

SLICE 3
THETA  dl  dW  X  AL.W
 20.9 3.6 3.8 4.0 26.6

SLICE 4
NEW SOIL
 DEN.1 DEN.2 PHI  COH.
 106.0 125.0 32.0 0.0
THETA  dl  dW  X  AL.W
 31.0 5.5 3.6 4.0 26.6

SLICE 5
NEW SOIL
 DEN.1 DEN.2 PHI  COH.
 103.0 125.0 32.0 0.0
THETA  dl  dW  X  AL.W
 41.5 6.8 2.1 3.0 26.6

SLICE 6
THETA  dl  dW  X  AL.W
 51.5 6.8 1.1 3.0 26.6

SLICE 7
NEW SOIL
 DEN.1 DEN.2 PHI  COH.
 101.0 125.0 32.0 60.0
THETA  dl  dW  X  AL.W
 65.0 4.6 0.0 3.0 0.0

 OMS FS=0.78

 BMS FS=0.87

 JMS FS=0.87

I.D.: TRANSLATIONAL

OMS BISHOP JANBU
           chord d= 6.1
           chord l= 49.5
TENSION CRACK
                Zw= 2.0
                 a= 6.6
                 R= 25.0
MIN FS= 0.80
MAX FS= 1.00

SLICE 1
NEW SOIL
 DEN.1 DEN.2 PHI  COH.
 110.0 125.0 32.0 0.0
THETA  dl' dW  X  AL.W
 4.0 0.5 1.0 3.0 33.5

SLICE 2
THETA  dl  dW  X  AL.W
 11.5 1.7 2.8 4.0 28.0

SLICE 3
THETA  dl  dW  X  AL.W
 20.9 3.6 3.8 4.0 26.6

SLICE 4
NEW SOIL
 DEN.1 DEN.2 PHI  COH.
 106.0 125.0 32.0 0.0
THETA  dl  dW  X  AL.W
 26.6 5.5 4.0 4.0 26.6

SLICE 5
NEW SOIL
 DEN.1 DEN.2 PHI  COH.
 103.0 125.0 32.0 0.0
THETA  dl  dW  X  AL.W
 26.6 6.8 4.0 16.0 26.6

SLICE 6
THETA  dl  dW  X  AL.W
 36.2 6.8 3.5 4.0 26.6

SLICE 7
THETA  dl  dW  X  AL.W
 49.5 6.8 1.7 4.0 26.6

SLICE 8
NEW SOIL
 DEN.1 DEN.2 PHI  COH.
 101.0 125.0 32.0 60.0
THETA  dl  dW  X  AL.W
 65.0 4.6 0.0 3.0 0.0

 OMS FS=0.88

 BMS FS=0.92

 JMS FS=0.94
```

ses. Techniques for data storage and analysis that upgrade the values for geotechnical variables for each landtype as the sample size is expanded (10) will be used.

SUMMARY AND CONCLUSIONS

The concept for a three-level landslide analysis system has been outlined. Important points regarding the system are as follows:

1. Each level of analysis is designed to require its own data base and to provide guidance for land management decisions at that level only. The level of analysis complexity, data required, and accuracy must be commensurate with the type of management decision they are intended to support.

2. A loop that channels levels 2 and 3 data back into the level 1 data base will upgrade the accuracy for future analyses.

3. Although the system described is for soil-mantle failures common in residual and colluvial soils, the concept is a series of building blocks that may be made applicable to rock slope failures by the proper substitutions.

4. Current restrictions on use of this system are not in the analysis techniques that are either in existence or at least feasible for development. The current restrictions are (a) the general lack of a dynamic and easily upgraded storage system and (b) the present state of the art for determining the values for certain geotechnical variables such as ground water concentration and effective tree root strength.

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