COMMONWEALTH OF KENTUCKY
DEPARTMENT OF HIGHWAYS
FRANKFORT

September 30, 1966

HENRY WARD
COMMISSIONER OF HIGHWAYS

ADDRESS REPLY TO
DEPARTMENT OF HIGHWAYS
DIVISION OF RESEARCH
132 GRAHAM AVENUE
LEXINGTON, KENTUCKY 40506

H-2-16

MEMORANDUM

TO: W. B. Drake, Assistant Projects Management Engineer
Chairman, Kentucky Highway Research Committee

SUBJECT: Research Report, Interim; "Stability Analyses of Earth Masses"; KYHPR-63-16, HPR-1(2), Part II; Development of a Practical Method of Locating and Tracing Seepage Water in Unstable Slopes.

The report submitted herewith appertains precedently to the above-cited study. Eleven case studies are presented, and six more are in progress -- to be included in a subsequent issue. It was felt that a progress report at this time would provide a reference record and helpful guidance for those directly concerned with the repair of slides and with embankment stability problems.

The Division of Materials is presently staffing a force of specialists in soil mechanics and foundations to provide this type of engineering service in the design and construction phases, or as otherwise might be needed. The Research Division has and is co-operating in this plan and in the development of suitable engineering criteria and analytical programs.

For continuity, the six, unreported case studies are listed below:

Lyon County, SP 72-891-1L5, I 24-2(5)45
Carroll County, SP 21-692-15G1, I 71-2(8)45, and
SP 21-692-11G1, I 71-2(10)48

Rowan County, SP 103-502-1L, I 64-6(11)132
Jefferson County, SP 56-898-37L, I 264-1(21)0
Jefferson County, SP 56-273-14L, I 64-2(27)6
Clark County, Mountain Parkway

Situational descriptions -- including plans, cross sections, any existing subsurface data, and a geological reconnaissance -- are essential prerequisites to the orderly solution of stability problems. The need for additional or extensive exploration and borings may be thus minimized.

It appears that prudent use of drainage blankets under the toe-zone of side-hill embankments offers a tactical recourse against roadway slips aggravated by seepage. Natural ground, especially on slopes consisting of weathered shale, is sometimes untrustworthy in seepage zones and should be removed and deployed elsewhere. Fill soils that are likely to dam or prevent free drainage in critical zones might be horizontally but discontinuously and alternately layered between broken rock when available or reserved for the upper reaches of embankments. The use of bench drains in side-hill situations involving shales is advisable.

Embankments founded partly on valley terraces and partly on hill slopes are fraught with treacherous, differential settlement. The depth of foundation soil (natural) and estimates of gross settlement and rate are extremely pertinent inasmuch as differential subsidence may induce fractures in the embankment and thereby trigger a mass rotation in an otherwise stabile situation. Settlement analyses are also pertinent to the subsidence in roadways over culverts and at bridge approaches.

Efforts thus far have not been very successful in the development of a method for locating and tracing seepage waters -- that is, other than by borings and excavation. In some of the cases where excavation was employed in reconstruction of an embankment, only dampness or a mere trickle of water was found at the exposed aquifer strata; in other cases, finger-sized streams were found. In each case, the embankment had great storage capacity and slow dissipative drainage. Infiltration along median strips and upper ditch lines has been suspected as a contributory factor. For these reasons, it is felt that relief drainage at the toe-zone of side-hill embankments will minimize the rise of the free-water surface in the fill and provide a needed buttressing effect.

These studies are continuing and subsequent reports will be forthcoming.

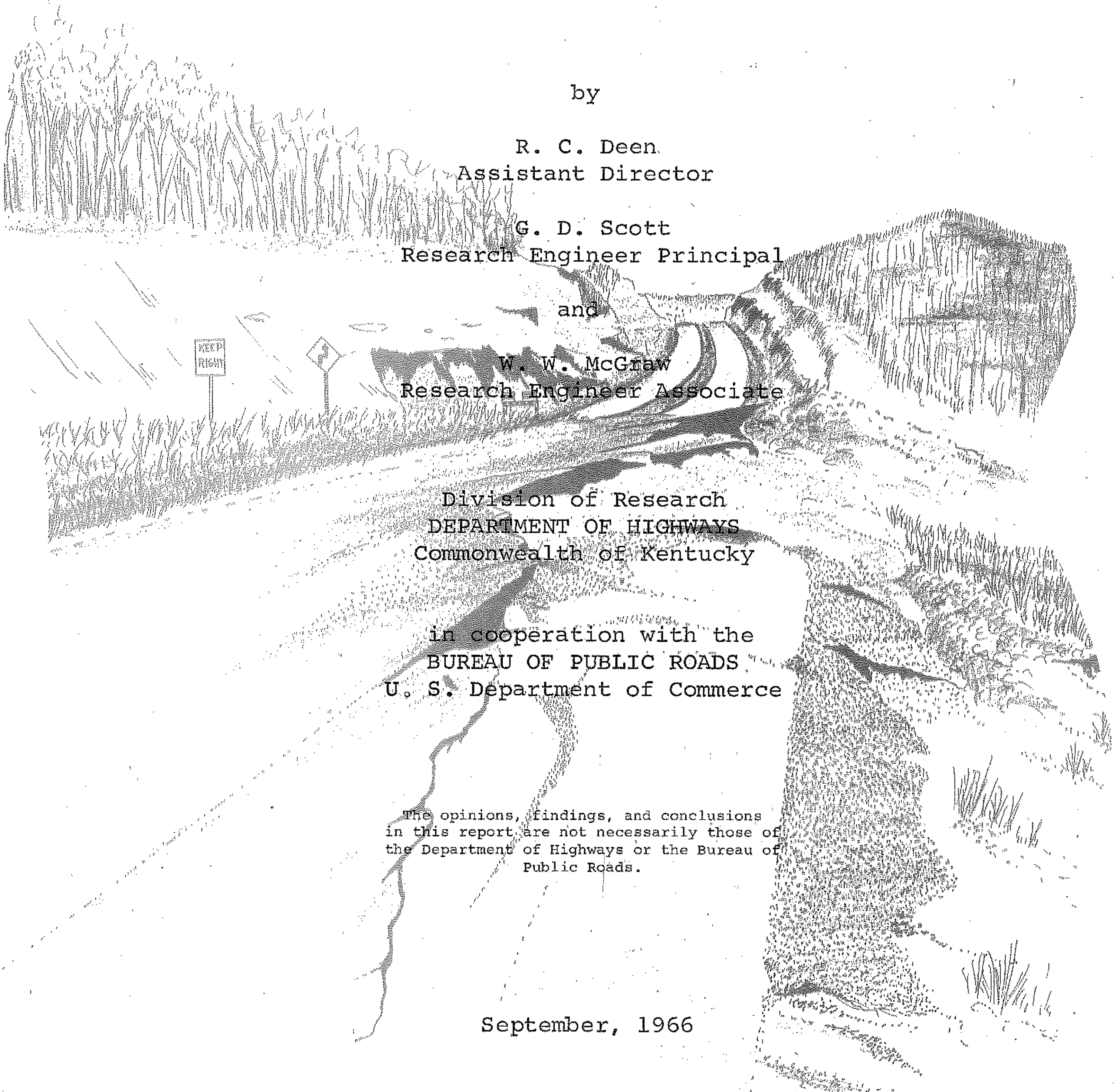
Respectfully submitted,

Jas. H. Havens
Director of Research
Secretary, Research Committee

JHH:mm
Attachment
cc: Research Committee
A. O. Neiser
R. O. Beauchamp
T. J. Hopgood
R. A. Johnson
H. G. Mays

Interim
Research Report

STABILITY ANALYSES OF EARTH MASSES
KYHPR 63-16, HPR-1(2), Part II



by

R. C. Deen
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and

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Division of Research
DEPARTMENT OF HIGHWAYS
Commonwealth of Kentucky

in cooperation with the
BUREAU OF PUBLIC ROADS
U. S. Department of Commerce

September, 1966

INTRODUCTION

During the past few months (January, 1966, to the present) the Division of Research has been called upon to make site investigations and analyses of foundation and slope stability at several locations in the State. These requests have emanated from various offices and divisions of the Department. Some of these investigations involved considerable effort and time to perform the subsurface exploration and to analyze the problem so that recommendations could be made. These investigations were of significant magnitude and are being summarized in this report. These site investigations include:

1) Settlement and Foundation Stability Analysis
Jackson Purchase Parkway
Graves County, SP 42-773-1L1, JPP 111

2) Slope Stability Analysis
Western Kentucky Parkway
Ohio County, Mile Post 75

3) Slope Stability Analysis
Western Kentucky Parkway
Grayson County, Mile Post 83

4) Slope Stability Analysis
US 23
Lawrence County, 1 Mile North of Louisa

5) Settlement and Foundation Stability Analysis
I 65
Barren County, SP 5-682-1L, I 65-2(1)37

6) Slope Stability Analysis
US 119
Bell County, APD 151(18)

7) Retaining Wall Analysis
US 23
Boyd County, SP 10-65-3L2, U 537(15)

8) Slope Stability Analysis
I 64
Boyd County, SP 10-115-25C1

9) Settlement and Foundation Stability Analysis
I 64
Bath County, SP 6-404-5G1, I 64-6(6)117

10) Slope Stability Analysis
Bluegrass Parkway
Nelson County, Mile Post 20

11) Settlement and Foundation Stability Analysis
Ky 55
Adair County, SP 1-10-4L, F 534(3)

Several involved field work, to determine the subsurface conditions, as well as laboratory testing and analysis. In addition to boring logs, Shelby tube specimens of the subsurface materials were obtained and returned to the laboratory for triaxial and (or) consolidation testing. The test results, used in conjunction with the knowledge of the subsurface conditions, were employed in various theoretical analyses to serve as a basis for design and construction recommendations.

Each of the projects cited above are discussed in detail in this report. All data, test results, and analyses pertinent to the respective problems are presented.

FOUNDATION AND SLOPE STABILITY
OF HIGHWAY EMBANKMENTS
AND EXCAVATIONS

In the planning, design, construction, and maintenance of highways, the engineer must always be cognizant of potential problems that might be associated with the stability of man-made and natural slopes. The man-made slopes may be the result of the construction of an embankment or the removal of material to form a cut. Closely associated with the slope stability problem is the consideration of settlements in the foundation material upon which embankments are placed. If excessive settlements occur, there is a possibility that differential strains may be induced within the embankment material and result in a failure. The slope stability problem has always existed in those situations in which embankments and cuts were required. The problem is becoming increasingly evident, however, because of the increased mileage of highways which is being constructed and because of the high standards with regard to grade and curvature resulting in high embankments and deep cuts.

In order to minimize the risks associted with the slope stability problem in highway construction, a knowledge of the general setting is essential in the recognition of potential or actual landslides. The development of, or potential for, landslides--the downward and outward movements of slope-forming materials composed of natural rock, soils, artifical fills or combinations of these materials--is dependent to a large extent upon the character, stratigraphy and structure of the underlying rocks and soils; on the topography, climate and vegetation; and on surface and underground waters. These factors vary widely from place to place and their variations are reflected in the kinds and rates of landslide movements that result from their interaction.

The recognition of potential landslides is extremely difficult with respect to the estimation of the probable severity as well as to the psychological difficulty of predicting troubles for an economy-minded agency or client. For problems associated with the correction of existing slides, the limits and extent of the slide are generally well defined and the seriousness of the problem can be assessed. However, proper correction may be rendered difficult because of the many methods that have been used with varying degrees of success and because the agency or client may not be able to afford an extensive corrective treatment, even though such a treatment is desirable or necessary. Nevertheless, recognition of potential slide areas is extremely important. If such areas are delineated early in the planning stage of a highway facility, steps can be taken in the design of the facility to minimize the occurrence of such problems. This results in a much more efficient construction procedure, and the preventive measures can be taken into account in the funding of the project. If slides can be minimized by proper design and construction techniques, the inconveniences caused by their occurrence during construction or after the facility has been opened to the public can be greatly reduced.

There is no sharp line of demarcation between the prevention and the control or correction of landslides. The basic principles involved in both approaches are the same and many of the general methods of treatment are similar. As might be expected, most of the treatments that might be used to prevent potential slides are also used for correction or control of existing slides. On the other hand, certain corrective measures do not have application in the prevention of potential landslides.

The prevention of landslides is, in many respects, much more difficult than correction from the standpoint of both analysis and design. For an existing slide, the critical boundaries and the nature of the materials can be determined quite satisfactorily by exploration and investigation. In contrast to this, the prevention of an incipient landslide requires the recognition of the hazard, which may or may not be evident from a superficial examination. Secondly, it is necessary that the character and magnitude of the movement be anticipated, and the design of a suitable treatment must be made so that movement will not occur. In addition to these considerations with regard to landslide prevention, it is necessary that a decision be made that the hazard is sufficiently real to justify the expense of preventive treatment.

One type of landslide which is very prevalent and particularly troublesome to the highway engineer is the roadway "slip". This is a landslide which occurs at or below the roadway grade, with a portion or all of the roadbed moving downward and outward. Such slipouts usually occur where the roadbed is partially an embankment and typically do not extend above the roadway grade. Even though this type of landslide may not be quite as spectacular as the large landslides involving slopes above the roadway, the slipout of an embankment is difficult and costly to correct.

Often landslides are spoken of as "individuals", inferring that generalizations are not practical. Even though some landslides do not lend themselves entirely to assumptions normally made in stability analyses, there is the possibility that the classic theories of soil mechanics may provide a quantitative means of evaluating experience and projecting this experience for the purposes of prediction. The principal use of a mathematical, theoretical approach to the stability problem lies in the opportunity to rank relatively the cost and effectiveness of various treatments. The actual quantitative answer thus becomes of lesser importance and provides a means only of ranking the various alternative methods of prevention or correction.

The prevention or correction of all types of landslides may be accomplished by one or more of the following methods: a) avoidance or elimination of the slide, b) reduction of the activating or driving forces, and c) increasing the forces resisting movements.

The most obvious, and often the most economical, method of preventing or correcting a landslide is to avoid the situation entirely. This may be accomplished by a relocation, either major or minor, of the proposed highway or structure in order to avoid the unstable terrain. In certain areas it is also possible to avoid the unstable material by bridging. Another alternative is to eliminate the problem by complete removal of the unstable material.

Reduction of the activating forces can be accomplished by two general methods - removal of material from that portion of the slide which provides the driving force and subdrainage to reduce hydrostatic pressures and (or) to diminish the weight of the material involved in the slide. The subdrainage approach, however, has a more stabilizing effect due to the resulting increase in the shearing resistence rather than by the reduction of the motivating forces.

Many methods of preventing or stabilizing actual slides involve techniques for increasing the forces resisting the sliding movements. Such methods include subsurface drainage in order to increase the shearing resistence of the soil; elimination of weak zones or potential surfaces of rupture by striping, blasting, or benching of smooth sloping surfaces; construction of restraining structures such as piles, walls, cribs, or berms, etc; and the solidification of loose granular material by certain chemical treatments.

Many landslides are caused by man's activities, resulting in an upset of the natural balance of forces. Consideration of the effect of proposed construction cannot be neglected in searching for potential landslides. Any cut or fill will change the local stress conditions within the soil or rock mass. The engineer, therefore, should analyze the possible effects of stress readjustments and estimate the changes which will be experienced by the soil profile, the underlying rock, and the groundwater conditions. Typical situations which may cause unstable conditions and which are the result of man's activities are:

1) restriction of groundwater flow by sidehill fill,
2) overloading of relatively weak underlying soil layers by an embankment,

3) overloading of sloping bedding planes by a heavy sidehill fill,

4) oversteepening of cut slopes in unstable rock or soil,

5) removal by excavation of a thick mantle of pervious soil which is a natural restraining blanket over a softer material,

6) increase in seepage pressure caused by a cut or fill that changes direction or character of the groundwater flow,

7) exposure by excavation of a stiff, fissured clay or a shale that may soften when exposed to surface water and air, and

8) removal of the toe support causing soil or rock above the cut to slide along contact planes with stable soil or bedrock.

Extensive studies by many investigators, particularly Swedish engineers, of slope failures suggests that many slides have developed where the failure surface can be approximated by a curved surface. It has been suggested that this surface of failure can be assumed to have the shape of a logarithmic spiral. This has certain theoretical advantages and is sometimes used for this reason. Taylor, however, has shown that results obtained by assuming the failure surface to be a circular arc, as suggested by Peterson and further developed by Fellenius, are only slightly different from those obtained by procedures based upon the assumption of a logarithmic spiral. The Swedish circle method, as developed in the early part of the century, is extremely simple to use and therefore has had wide application in slope stability analyses.

Figure 1 shows a soil mass, whose stability is to be investigated, bounded by the ground surface and the potential slip surface, abcd. The forces which are acting on the soil mass, abcdef, are the weight of the soil mass, any external forces acting on the surface of the soil mass, and the force acting along the circular arc. Failure occurs when the summation of the moments of the forces tending to cause rotation of the mass around the center of the circular arc is equal to the summation of those moments tending to resist this rotational movement.

In order to more reasonably estimate the distribution of stresses along the circular arc, the analysis of slope stability by the method of slices is an important modification of the circular arc method. It is particularly useful in solving those problems involving nonuniform soil and porewater pressure conditions. In the method of slices, the mass under consideration is divided into a number of segments by vertical lines. The forces acting on each of these segments are evaluated from the equilibrium of the segment. The equilibrium of the entire mass is obtained by summing the forces acting on all of the segments.

As shown in Figure 2, the force system on each individual segment is statically indeterminate. In order to obtain a solution it is necessary to make certain assumptions concerning

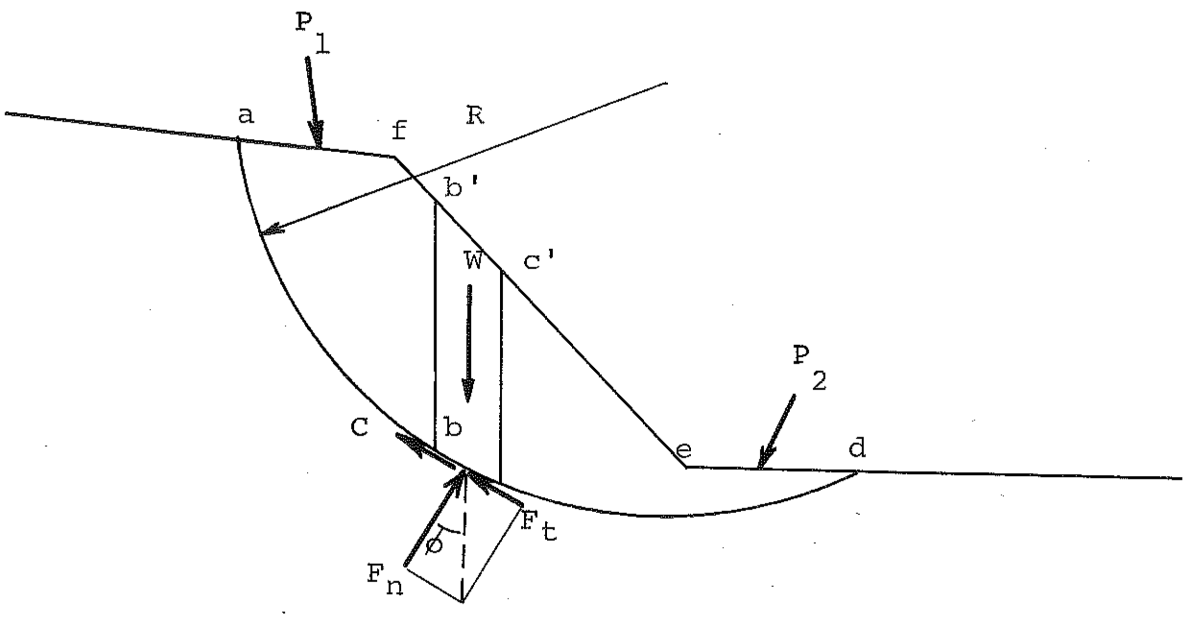


Figure 1. Circular Arc Method of Analysis.

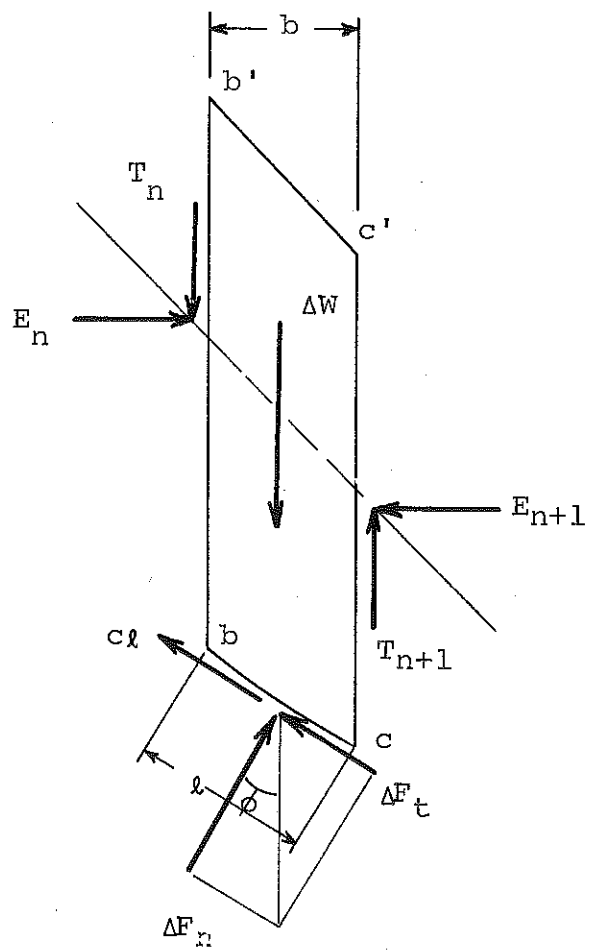


Figure 2. Method of Slices Analysis.

the magnitudes and points of application of the normal and tangential forces acting on the sides of the individual segments. The different assumptions which may be made about the distribution of these forces on the vertical faces of the slices results in different techniques or equations for analyzing the entire mass.

An approximate solution may be obtained by assuming the resultants of the normal and tangential forces are equal and opposite and that their line of action coincides. This assumption greatly simplifies the calculations involved in a slope stability analysis but may result in an error on the order of 15 percent. In order to reduce this error, it is necessary to make assumptions regarding the distribution of the normal and tangential forces acting on the vertical faces of each slice. In making such assumptions, it must be kept in mind that the summation of these forces for the entire mass must be zero.

The vertical as well as areal extent of the various layers of soil and rock material involved in the slope must be known or estimated in order to perform a satisfactory analysis. In addition, the strength characteristics of the materials involved are required in order to predict the shearing forces available to resist the sliding tendency. An estimate must also be made of the changes in the strength parameters as may be effected by variations in the porewater pressures in order to predict what might happen to the slope at various times.

A cursory examination of various slides indicates that a slope may be subjected to several types of loading within its life. In addition, the strength of the soil is not constant but changes with effective stress. It is important, therefore, to consider the strength and loading changes in the stability analysis. The analysis of the relative changes in the applied stresses and the strength is an important part of the total slope stability investigation. Through this type of study, the engineer obtains a clear picture of the changes in the stability of the slope throughout various stages of the project. In this way he is able to determine those stages which are most critical and select these for more detailed investigations. Other less critical times may be disregarded.

Two limiting conditions for soil behavior are well established. The conclusions that can be drawn from these conditions are so informative that the engineer is able, by extrapolation, to arrive at general conclusions for the less simple situations. Illustrated in Figure 3 is a situation in which an embankment is constructed over a deposit of clay. The stresses applied at a point, A, within the foundation material increases as the height of the embankment is increased and reaches a maximum at the end of the construction period. Initially, the pore-water pressure is equal to the hydrostatic pressure. As the

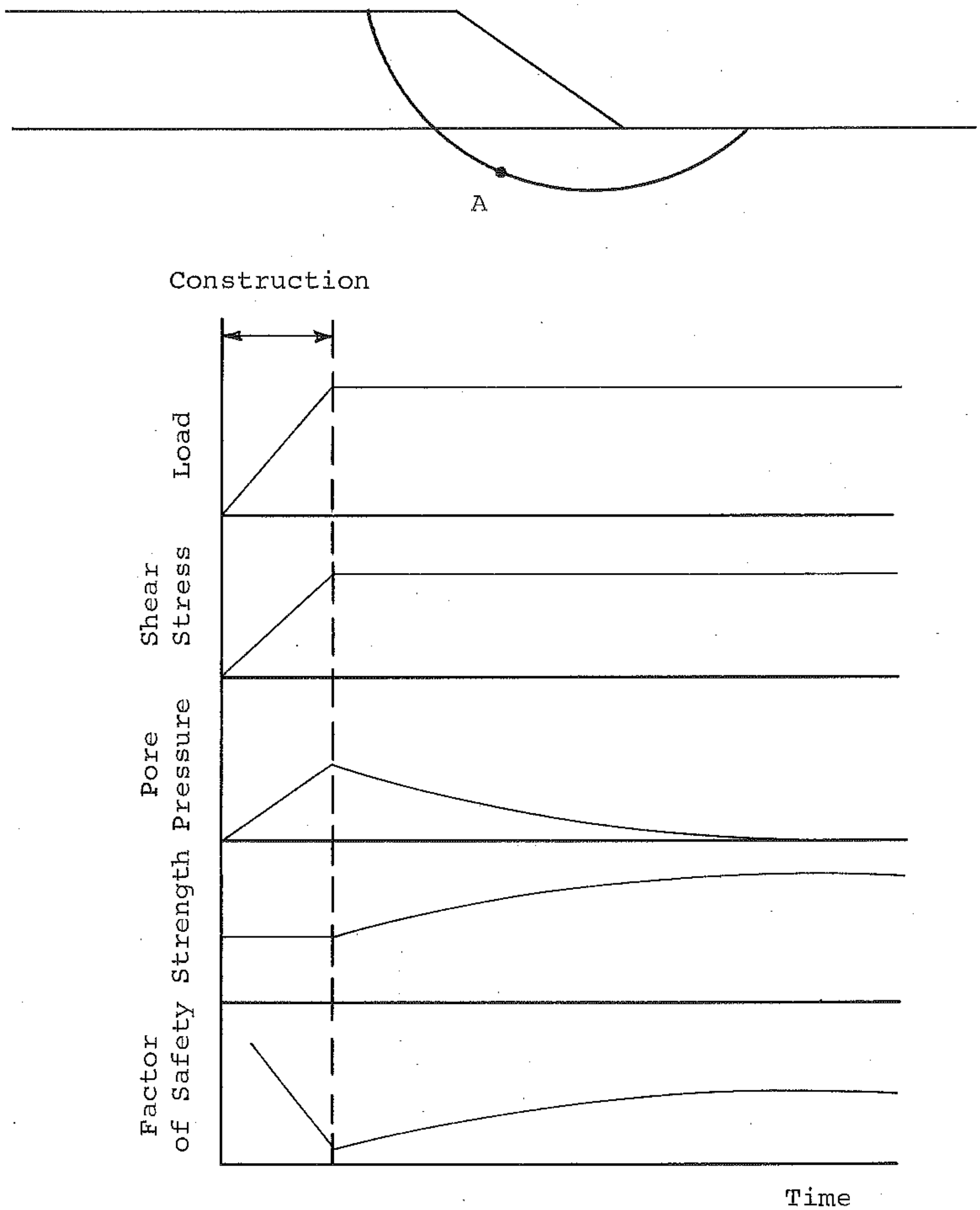


Figure 3. Stability Conditions for an Embankment-Loaded Foundation.

embankment is placed, the pore pressures are increased since it is assumed that there is no drainage, and thus no dissipation of the porewater pressure takes place during the relatively short construction period. As one limiting condition, therefore, it can be assumed that the foundation material is loaded in an undrained condition. After construction, the stresses applied by the embankment loading remain constant. The excess hydrostatic pressures, however, tend to dissipate with time. At some time in the future, the excess pore pressures are reduced to zero. As the pore pressures are dissipated, there is a reduction in the void ratio of the material and a corresponding increase in the effective stress and the shear strength. The second limiting condition is attained at some long time after construction when the excess hydrostatic pressures are zero (that is, the drained state). Since, at this late time in the life of the embankment, the excess hydrostatic pressures are zero, the effective stresses can be calculated from the known loads, the weights of the materials involved, and the hydrostatic pressures. The shear strength then can be determined from the effective stress parameters, c' and ø'.

The two limiting conditions in the example illustrated above involve forces that can be readily calculated. The end-of-construction stage can be studied using a total stress analysis (ø-equal-zero analysis) and the undrained shear strength

The long-term stability of the embankment can be investigated using an effective stress analysis (the excess hydrostatic pressures equal zero) and the effective stress strength parameters. If the distribution of the porewater pressures are known, it is possible, by means of the effective stress analysis, to evaluate the stability of the slope at any other time.

A second example illustrating the application of limiting conditions is illustrated in Figure 4. Here a cut is made in a clay. As the excavation of the soil progresses, the average overburden pressure at some given point, A, is reduced and results in a decrease inthe porewater pressures. The applied shearing stresses at point A increases to a maximum at the end of construction. As in the previous example, it can be considered that the limiting condition of no drainage during construction still applies. Therefore, at the end of construction the shear strength remains equal to the undrained strength. With time, the pore pressures increase, accompanied by a swelling of the clay and a reduction in the shear strength of the material. As before, the second limiting condition is reached after a long time--when the excess hydrostatic pore pressures are equal to zero. Again the strength of the materials can be represented by the effective stress parameters.

In many instances the placement of a sidehill embankment causes changes in the natural flow of underground seepage waters.

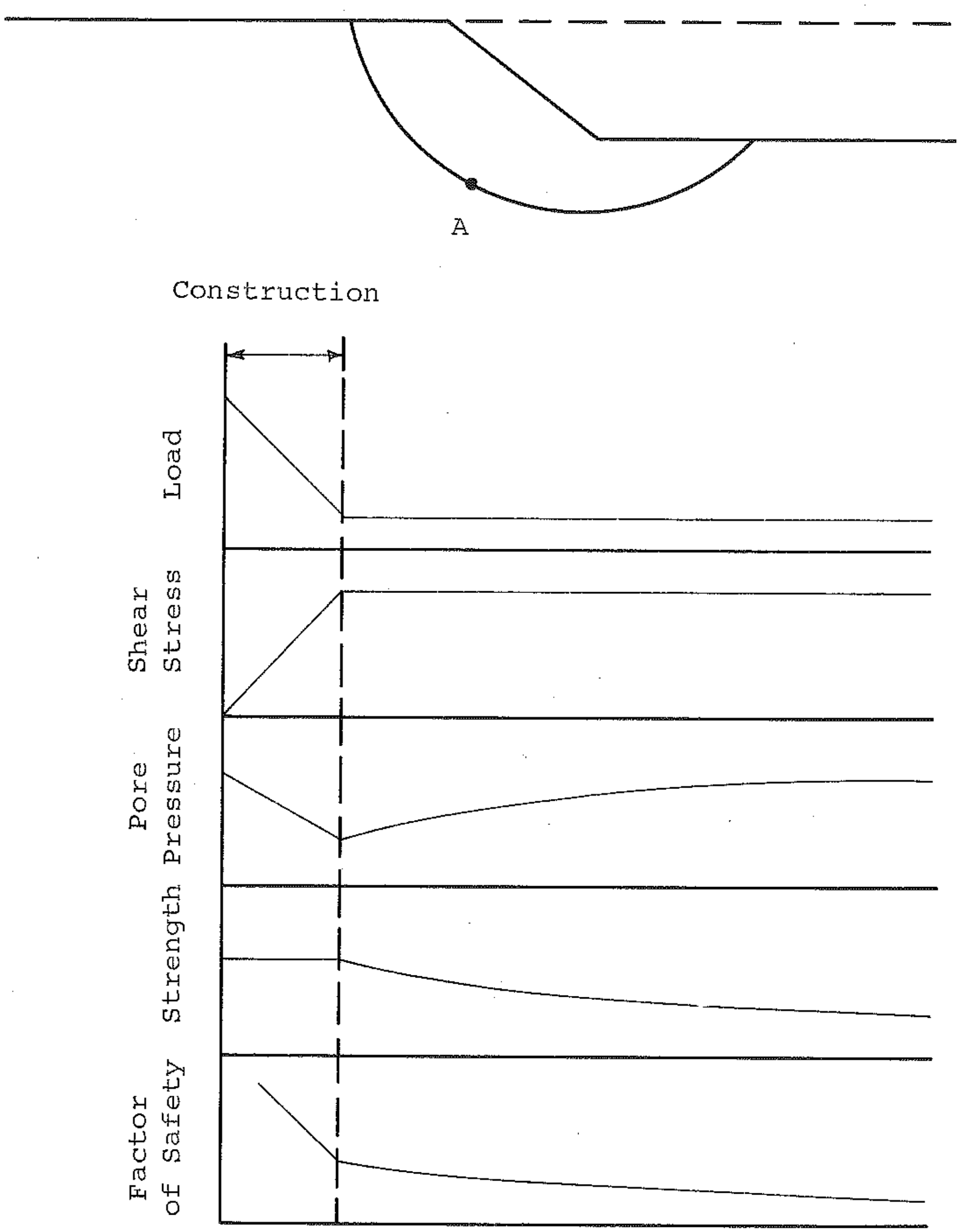


Figure 4. Stability Conditions of an Excavated Slope.

The toe of the embankment may actually serve to dam seepage waters which would normally have outcropped on the slope and not have caused any particular trouble. The entrapped water may, after a time, cause the embankment to become saturated. The water table will rise, resulting in excess hydrostatic pressures which may reduce the shear strength of the embankment material to a critical value and therefore cause a slide. This type of situation illustrates the importance of providing adequate drainage systems at the toe of such embankments so that these excess hydrostatic pressures are not permitted to develop.

A study of various situations in the qualitative manner illustrated above demonstrates the importance of the distribution of excess hydrostatic porewater pressures. In order to make reasonable estimates of slope performance, it is necessary to have some knowledge of expected changes in porewater pressure distributions within the soil mass.

In the design of a cut or embankment slope, there is no known apriori location for the critical arc. The analysis of a slope stability problem, therefore, involves a trial-and-error method of locating the potential failure surface that is most critical. In such a trial-and-error approach, numerous repetitive calculations must be made. This lends itself to the use of electronic computers. Accordingly, a program has been developed, based upon the method of slices and the Swedish circular arc technique, to perform the tedious calculations involved in such an analysis. The use of such a program, however, must be tempered with engineering judgement in selecting those conditions and results of analysis which are applicable to the situation. The details of the computer program are contained in an appendix to this report.

SETTLEMENT AND FOUNDATION STABILITY ANALYSIS

Jackson Purchase Parkway

Graves County, SP 42-773-1L1, JPP 111

SETTLEMENT AND FOUNDATION STABILITY ANALYSIS
Jackson Purchase Parkway
Graves County, SP 42-773-1L1, JPP 111

To the north of Water Valley in Graves County, the route of the Jackson Purchase Parkway crosses the Bayou du Chien, a significant stream draining from east to west towards the Mississippi River. The stream valley is rather wide and flat, and the materials deposited in this area are alluvium in nature and consist predominately of sands, silts, and clays. According to the boring logs, the upper portion of the alluvium has been classified as a silt. At depths of about 24 feet the deposit becomes more sandy in texture. A more detailed discussion of the geology of the Jackson Purchase Parkway area is contained in a previous report entitled "A General Survey of Highway Construction Materials, Jackson Purchase Region" by R. C. Deen and J. H. Havens, dated March, 1966.

A preliminary review of test results from routine subsurface exploration and soil testing indicated to personnel of the Division of Materials the possibility of large amounts of settlement in the foundation material beneath embankments across the Bayou du Chien. In order to investigate this possibility in more detail, ten Shelby tube samples were obtained from three drill holes, at

Stations 465+25, 469+75 and 476+00, by the project consulting engineers, Adam K. Grafe, Associates. The samples were submitted to the Division of Research where they were immediately extruded from the tubes. Material at either end of the tube samples which could have been disturbed during the sampling procedure was discarded. The remainder of the sample was then cut into four parts, each approximately four inches long, which were then dipped in melted wax for protection during storage until testing.

Consolidation tests were performed on some of the specimens in order to determine the compression parameters which might be used in predicting the amount and time-rate of settlement under loading. Triaxial tests were also performed in order to obtain strength parameters so that a bearing-capacity analysis could be made.

The void ratio-log pressure curves obtained from the consolidation tests are shown in Figure 1-1. An attempt was made to find the preconsolidation pressures by Casagrande's graphical procedure. The values obtained, which average 1.7 tons per square foot, were much higher than the effective overburden pressure and thus indicate a preconsolidated condition. However, the preconsolidation pressures were somewhat difficult to determine because the

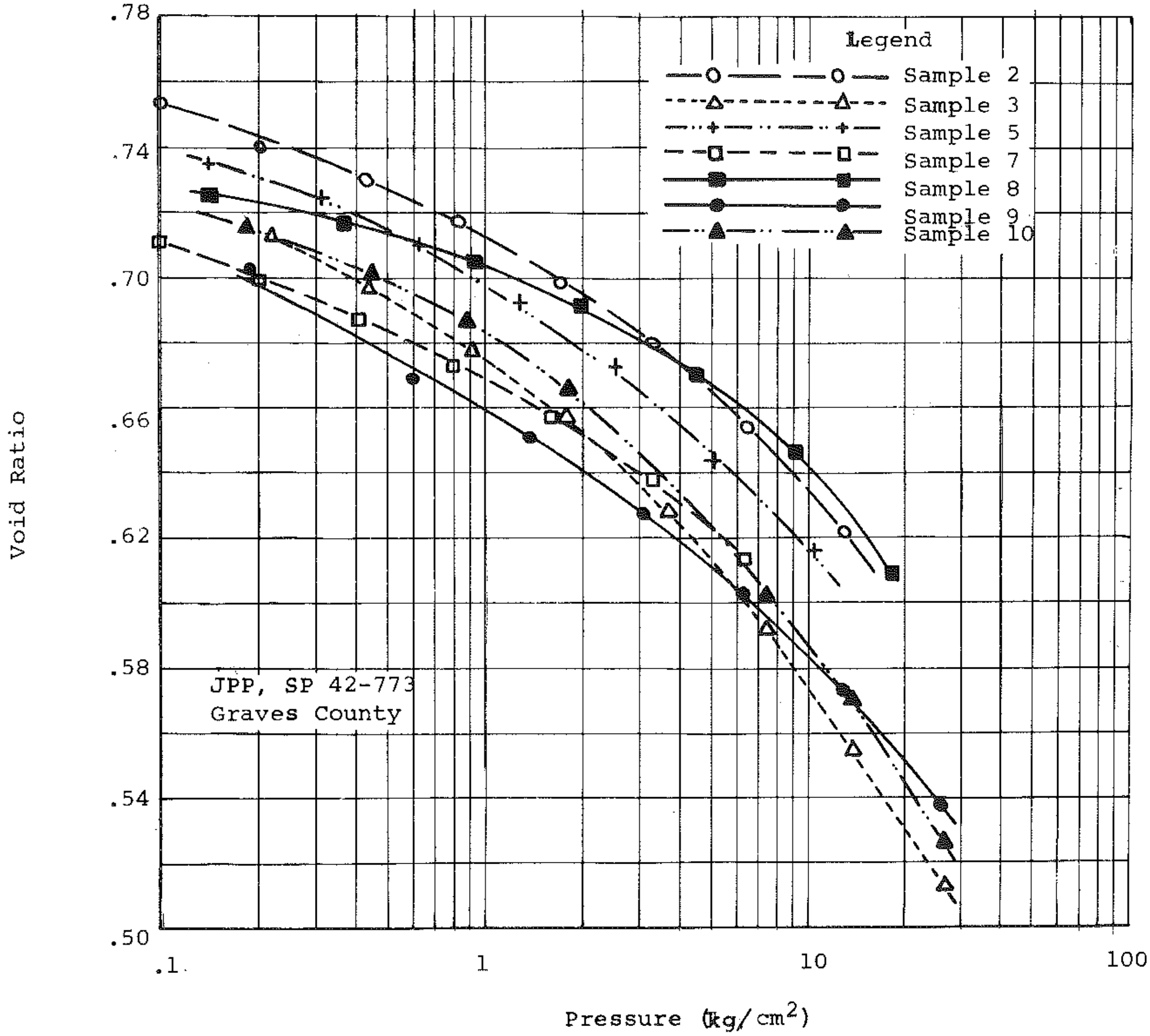


Figure 1-1. Void Ratio - Log Pressure Curves.

void ratio-log pressure curves are relatively flat in the testing range. But since the curves are flat, the expected ultimate settlement in much smaller than would be anticipated if the loading range were in excess of the preconsolidation pressure.

The initial overburden pressure on the foundation material was assumed to be 0.36 tons per square foot, and the final pressure after loading with the embankment was taken to 1.33 tons per square foot. The value of the compression index (C_c), the slope of the void ratio-log pressure curve, used in the settlement calculations was taken to be 0.057. This value was obtained by averaging the slopes of all of the curves at the midpoint of the pressure range due to the loading of the embankment. The maximum settlement was calculated to be approximately five inches.

Using an average value of 0.738 feet^2 per day for the coefficient of consolidation (C_v), it is anticipated that 90 percent of the ultimate settlement will be reached within about 180 days, and 50 percent will be obtained in about 55 days from the date that construction of the embankment is started. For the rate of settlement calculations, it was assumed that the construction period would be 30 days long (see Figure 1-2). Calculations to

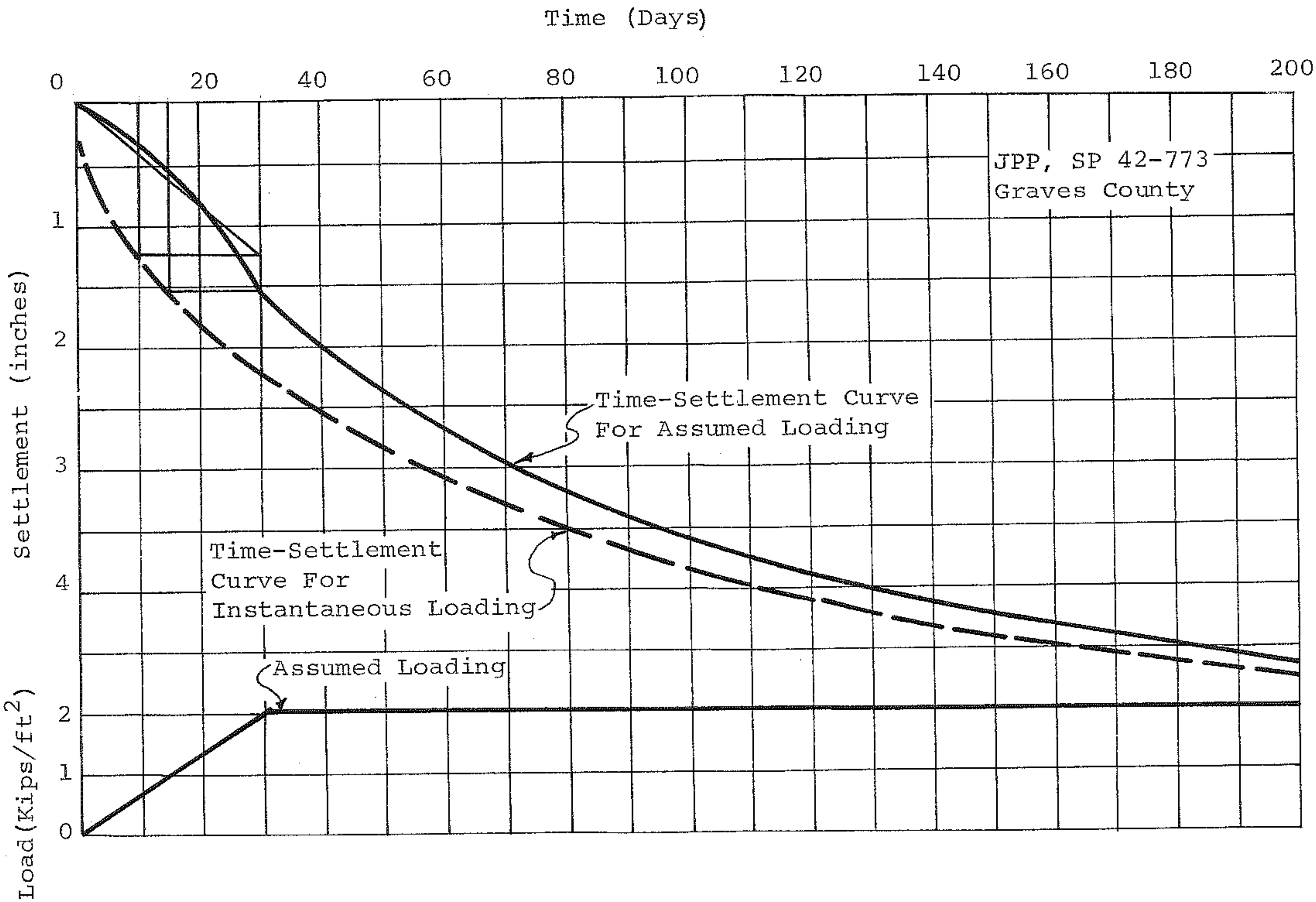


Figure 1-2. Rate of Settlement Curves

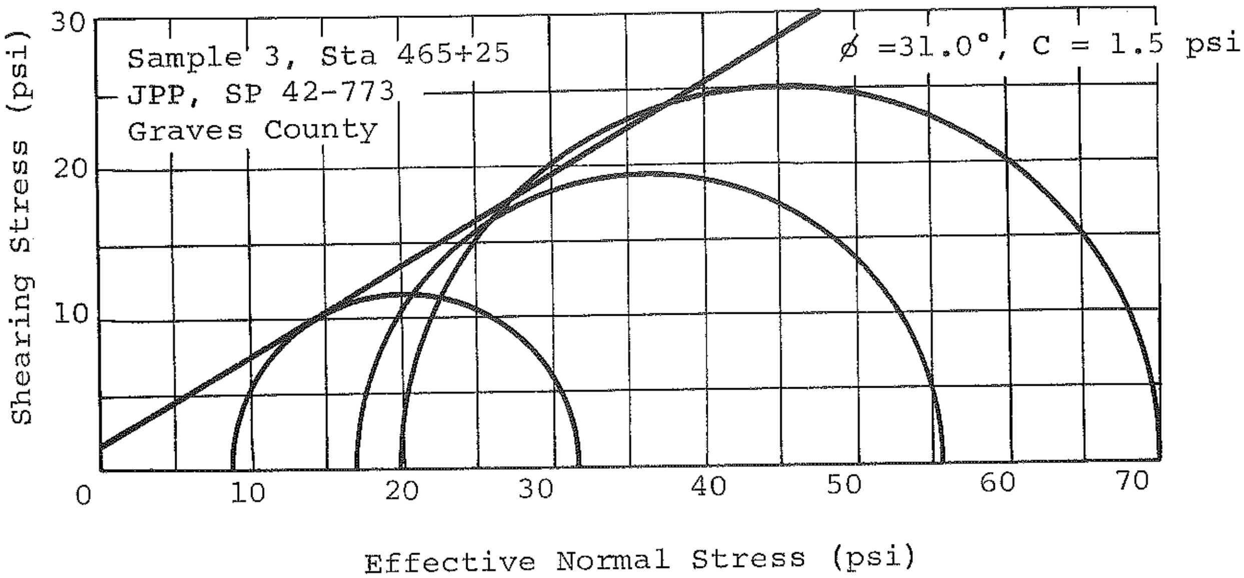
estimate the rate of settlement were based upon the assumption that the field boundary conditions will permit double drainage, which is likely since the compressible material is underlaid by a sandy silt or sand. The drill logs, however, indicate that there are small artesian pressures in the sand layer. Water rose to the surface in two holes and flowed out at the surface in one hole. Thus it may be desirable to provide relief of the artesian pressures in the form of wells outside the embankment area or in the form of sand drains along the center line at perhaps 100-foot spacing. Also, to insure drainage at the surface, the first layer of the embankment should be constructed of porous granular material (a sand blanket).

A summary of the appropriate settlement calculations is given in Table 1-1.

Mohr's circles and failure envelopes obtained from the triaxial testing program are shown in Figure 1-3. The circles are plotted on an effective stress basis. Although the two series of triaxial tests were performed on materials of widely different consistencies, the shear strengths were almost identical. The effective angle of friction (ϕ') was found to be 30°, and the cohesion (c') intercept was 288 pounds per square foot. Ultimate bearing capacity based on the ϕ' and c' values was calculated to be 80 tons per

TABLE 1-1. SUMMARY OF SETTLEMENT COMPUTATIONS

ITEM	FORMULA AND DEFINITION	COMPUTED VALUE
Initial Effective Pressure, P_o (At Midpoint of Compressible Layer - Depth of 12 Feet)	$P_o = \gamma_f Z$ Where γ_f = Unit Weight of Foundation Material $= [(G-1)/(1+e_o)] \cdot \gamma_w$ G = Specific Gravity of Foundation Material = 2.64 e_o = Initial Void Ratio of Foundation Material = 0.709 γ_w = Unit Weight of Water = 62.4 Lbs/Ft^3 Z = Depth = 12 Ft	P_o = 720 Lbs/Ft^2 = 0.36 T/Ft^2
Pressure Increase, ΔP (Due to Weight of Embankment — At Depth of 12 Feet on Center Line of Embankment)	$\Delta P = \gamma_e DI$ Where γ_e = Unit Weight of Embankment = 130 Lbs/Ft^3 D = Height of Embankment = 15 Ft I = Loading Influence Value (From Influence Chart by J.O. Osterberg)	ΔP = 1950 Lbs/Ft^2 = 0.97 T/Ft^2
Final Effective Pressure, P_f (At Depth of 12 Feet on Center Line of Embankment)	$P_f = P_o + \Delta P$	P_f = 2670 Lbs/Ft^2 = 1.33 T/Ft^2
Maximum Ultimate Settlement, S_u (on Center Line of Embankment)	$S_u = 2HC_c \log [(P_o + \Delta P)/P_o]/(1 + e_o)$ Where C_c = Compression Index = 0.057 $2H$ = Thickness of Compressible Layer= 24Ft	S_u = 5 in.
Time-Rate of Settlement	$t = TH^2/c_v$ Where t = Time for Settlement to Occur c_v = Coefficient of Consolidation = 0.738 Ft^2/Day T = Theoretical Time Factor	t (At 50% Consolidation) = 38 Days T = 0.196 t (At 90% Consolidation) = 165 Days T = 0.848



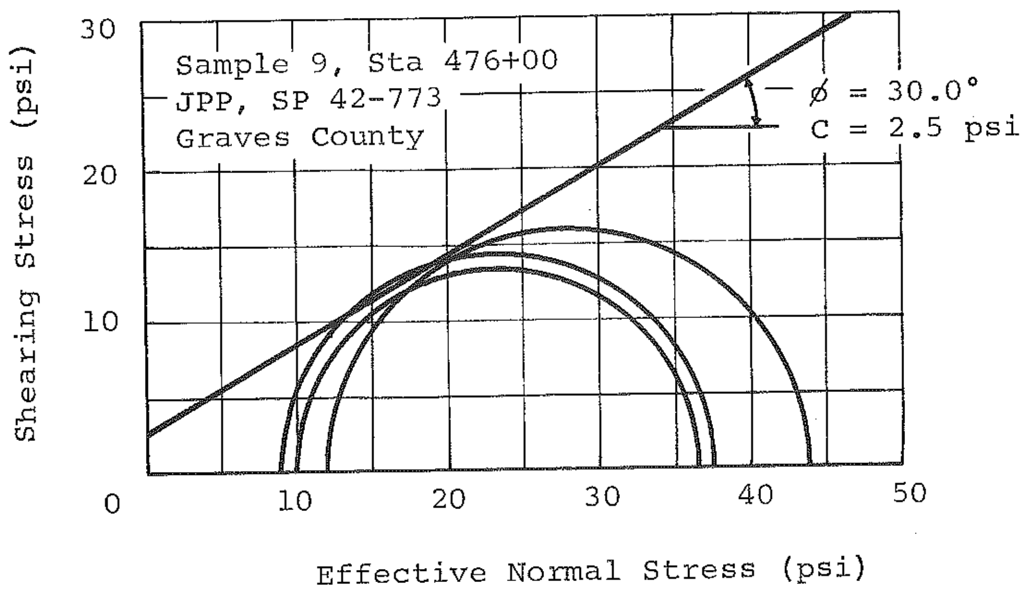


Figure 1-3. Mohr's Circles and Failure Envelopes.

square foot. The bearing capacity based on a ϕ'-equal-zero analysis is only 4.5 tons per square foot. The expected maximum pressure increment due to the placement of the fill is 1.0 tons per square foot. The value for the unconfined compression (q_u) used in the ϕ'-equal-zero analysis was actually obtained from a triaxial test in which a confining pressure of ten pounds per square inch was employed. The ϕ'-c' analysis reflects long-term stability after some strength gain due to consolidation whereas the ϕ'-equal-zero analysis more nearly reflects the "after construction" or short-term condition.

There is a fairly large factor of safety, 80, against a bearing failure on a long-term basis. Although the factor of safety immediately after construction is adequate, it is much smaller, being 4.5. The ϕ'-equal-zero analysis is probably conservative due to the high drained shear strength of the material and the high rate at which consolidation is expected to occur--that is, the soil may gain considerable strength during construction due to drainage under the initial construction loads. The bearing capacity calculations are summarized in Table 1-2.

Because the width of the embankment or loaded area is very large compared with the thickness of the compressible layer, it was throught that the bearing capacity

TABLE 1-2. SUMMARY OF BEARING CAPACITY COMPUTATIONS

ITEM	TYPE OF ANALYSIS AND PARAMETERS	FORMULA AND DEFINITION	'N' VALUES (From Terzaghi)	COMPUTED BEARING CAPACITY (Tons/Foot^2)
Ultimate Bearing Capacity, Q_u	$\phi' - c'$ $\phi' = 30°$ $c' = 288$ Lbs/Ft^2	$Q_u = c'N_c + \gamma D_f N_q + 1/2 \gamma BN_\gamma$ Where γ = Unit Weight = 123 Lbs/Ft^3 D_f = Depth of Footing = 0 B = Width of Footing = 132 Ft (Average Width of Embankment)	$N_c = 37$ $N_q = 22$ $N_\gamma = 20$	80.4
	$\phi' = 0$ $q_u = 1580$ Lbs/Ft^2	$Q_u = q_u N_c$	$N_c = 5.7$	4.5
Allowable Bearing Capacity, Q_a (Factor of Safety= 3)	$\phi' - c'$ $\phi' = 30°$ $c' = 288$ Lbs/Ft^2	$Q_a = Q_u/3$	$N_c = 37$ $N_q = 22$ $N_\gamma = 20$	26.8
	$\phi' = 0$ $q_u = 1580$ Lbs/Ft^2	$Q_a = Q_u/3$	$N_c = 5.7$	1.5
Actual Pressure, Q	Osterberg's Chart	$Q = \gamma_e DI$ Where γ_e = Unit Weight of Embankment= 130 Lbs/Ft^3 D = Height of Embankment = 15 Ft I = Loading Influence Value		1.0

factors of safety as obtained from bearing capacity equations would not necessarily be appropriate to the situation. In order to further check the stability of the foundation under the embankment loading, an analysis of the stability of the embankment was made using the Swedish circle technique. With the water table taken to be at the ground surface and assuming the embankment material to offer resistance to shearing to the extent of one-half of that offered by the foundation material, the minimum factor of safety against failure of the embankment-foundation system was found to be 1.3. If the embankment was assumed to have the same strength parameters as the foundation material, the minimum factor of safety against failure was found to be 2.4 with the water table at the ground surface and 2.7 with the water table at great depth. This analysis again suggests that the embankment would be safe when placed upon the foundation material.

A summary of the pertinent test data is presented in Table 1-3.

Table 1-3. Summary of Test Data.

Location	Sample No.	Description	Depth (Feet)	Moisture Content (Shelby Tube Sample) (Percent)	Specific Gravity	Consolidation Parameters *		Triaxial Test Data					
						C_v (Ft^2/Day)	C_c	Dry Unit Weight (Lbs/CuFt)	Moisture Content (Percent)		Effective Confining Pressure (Psi)	Coehesion (Psi)	Friction Angle (Degrees)
									Before Test	After Test			
STA 465+25	1	Tan Sandy Silt	1-3	32.9									
	2	Tan and Gray Silt	11-13	28.5	2.64	.400	.053						
	3	Tan and Gray Silt	16-18	22.7	2.66	.440	.065	102.5	25.1	24.3	8.6		
								102.8	19.2	17.0	17.0	1.5	31.0
								98.1	27.4	25.0	19.8		
	4	Tan Sandy Silt	21-23	17.4									
STA 469+75	5	Tan and Gray Silt	6-8	26.5		.540	.061						
	6	Tan and Gray Silt	11-13	28.9	2.65								
STA 476+00	7	Tan and Gray Silt	4-6	22.1	2.65	.800	.051						
	8	Tan and Gray Silt	9-11	25.3	2.64	.630	.038						
	9	Dark Blue Silt	14-16	22.7	2.67	2,000	.074	97.8	28.0	28.0	9.2		
								92.0	29.5	29.5	9.9	2.5	30.0
								93.9	24.8	26.0	11.9		
	10	Blue Sandy Silt	19-21	22.7	2.61	.360	.057						

*Range of Loading—P_o=0.35 kg/cm^2, P_f=1.31 kg/cm^2

SLOPE STABILITY ANALYSIS

Western Kentucky Parkway

Ohio County, Mile Post 75

SLOPE STABILITY ANALYSIS
Western Kentucky Parkway
Ohio County, Mile Post 75

The Division of Research has been observing a slide at Mile Post 75 (Station 5800+00) on the Western Kentucky Parkway in Ohio County as a part of its research study dealing with tracing seepage water in unstable slopes. Movement of the shoulder and guardrail where observed in January, 1966, in the westbound lanes at this location. At that time there was no observable movement at the toe of the slope but tension cracks were noted on the fill slope. Movement increased appreciably from January to February, 1966. In an attempt to stabilize the embankment, piles were placed in the westbound shoulder during the spring of 1966. A stability analysis was made in order to estimate the number of piles that might be required to stabilize the moving mass.

The materials which outcrop in the southeastern portion of Ohio County are predominately of Pennsylvanian age. The Caseyville Sandstone has been observed at the lower elevations with the Tradewater Formation above it. At the highest elevations in this portion of the county, materials classified as the Carbondale Formation have been observed.

The Caseyville Sandstone is a crossbedded, conglomeratic, medium- to very coarse-grained sandstone intertonguing with shales. The middle portion contains several thin coal beds and thin layers of limestone. The Tradewater Formation is a series of interbedded sandstones and shales with occassional thin layers of limestone and thin coal beds. The sandstones may be relatively thick and well-cemented locally. The sandstones range from very thin laminations to very thick, crossbedded units in places. The shale, locally silty, is gray or black when unweathered but changes to slightly lighter colors when affected by water and air. The No. 1A coal is located near the base of the Tradewater Formation and the No. 7 coal is located near the top. Underclays approximately a foot thick have been noted to the associated with coal beds throughout the formation. Layers of gray clay up to two feet thick have been noted in drill holes in the area. The Carbondale Formation is predominately a shale or sandy shale with thin coal beds. The Sebree Sandstone, a crossbedded friable to well-cemented, shaly quartz sandstone, is located at the base. The Carbondale Formation in this area is capped by the No. 9 coal bed. Figure 2-1 illustrates a generalized columnar section for this portion of Ohio County.

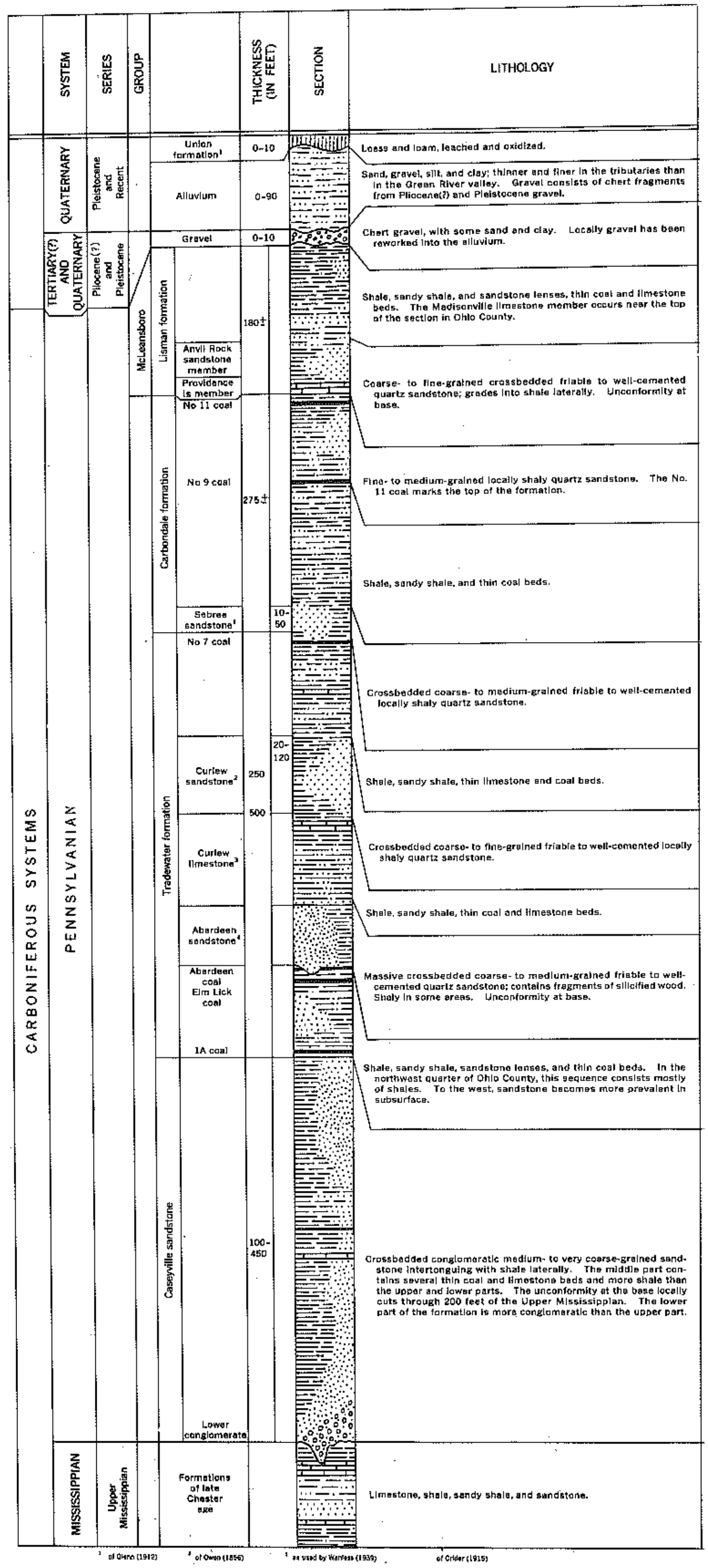


Figure 2-1. Generalized Columnar Section for Ohio County, Kentucky (from "Availability of Ground Water in Butler and Ohio Counties, Kentucky," B.W. Maxwell and R.W. Devaul, U.S. Geological Survey, 1962).

It is thought that the material involved in the slide at Mile Post 75 consists of portions of the Tradewater Formation. The site is a side-hill cut-fill section. Outcrops of thinly bedded shales have been covered with the embankment portion of the section. These shales are known to be paths along which subsurface waters seep. It is felt that subsurface seepage water has been dammed by the embankment--the embankment becoming saturated and losing strength and therefore slipping down the original slope. Shelby tube samples were obtained from the slide zone, and a series of triaxial tests were performed on two-inch diameter undisturbed samples. The Mohr's circles and rupture envelopes for the test series are shown in Figure 2-2. A summary of the test results is given in Table 2-1.

On the basis of the analysis, it was suggested that three rows of the nominally six-inch diameter piles, which were placed at the site, would be adequate to stabilize the soil above the zone in which the piles are placed. The piles in these three rows would be placed at a spacing of two feet, center-to-center. This recommendation appears to have been verified in the field. In those zones of the slide in which the equivalent of three rows of piles, two feet, center-to-center, were placed, the slide appears to have stabilized (see Figure 2-3). In other zones where

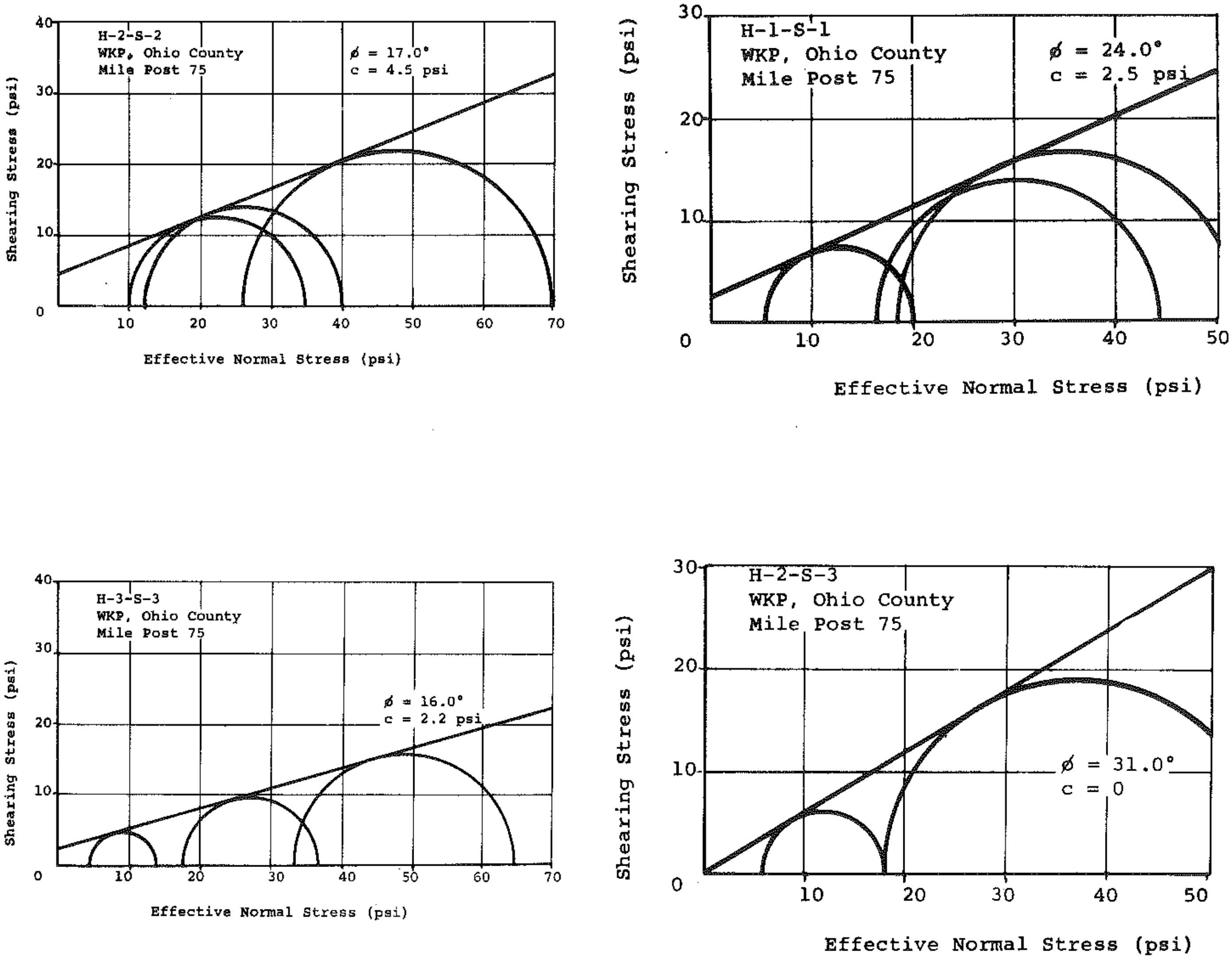


Figure 2-2. Mohr's Circles and Failure Envelopes.

Table 2-1. Summary of Test Results.

Location	Sample No.	Description	Depth (Feet)	Moisture Content (Shelby Tube Sample) (Percent)	Liquid Limit (Percent)	Plasticity Index (Percent)	Specific Gravity	Classification		Triaxial Test Data					
								AASHO	Unified	Dry Unit Weight (Lbs/CuFt)	Moisture Content (Percent) Before Test	Moisture Content (Percent) After Test	Effective Confining Pressure (Psi)	Cohesion (Psi)	Friction Angle (Degrees)
STA 5800+00 16'RT of Pave-Ment Edge	H-1-S-1	Very Moist, Gray Clay	10-12	20.5	33	11	2.67	A-6	CL	105.1	22.5	20.6	5.4		
										112.5	9.8	10.8	16.4	2.5	24.0
										102.7	22.5	20.6	18.4		
	H-1-S-2	Moist Clay, Firm Tan Shale	15-17	18.0	40	14	2.59	A-6	CL						
STA 5800+50 2'RT of Pave-ment Edge	H-2-S-1	Very Moist Tan Clay	10-12	19.0	NP	NP	2.65	A-2-4	SM						
	H-2-S-2	Very Moist Tan Clay, Firm Sandstone near Bottom	15-17	21.0	38	17	2.67	A-6	CL	108.4	19.0	21.0	10.0	4.5	17.0
										110.6		20.7	12.0		
										106.6	17.8	18.6	25.8		
	H-2-S-3	Moist Tan Clay and Shale	20-22	19.0	37	12	2.69	A-6	CL	115.6	16.3	18.1	5.8	0	31.0
										125.1	12.5	12.9	17.7		
STA 5800+30 16'RT of Pave-ment edge	H-3-S-3	Moist Brown Clay with Shale Fragments	15-17	21.0						92.7	30.3		4.6		
										113.0		21.8	17.6	2.2	16.0
										113.8	19.3	19.7	33.6		

Figure 2-3. Slide at Mile Post 75 Western Kentucky Parkway, in May, 1966.

Figure 2-4. Site in Preparation for Driving Rail Piles, July, 1966.

fewer piles were installed, the piles have very definitely been bent, and the soil mass appears to still be moving. It should be pointed out, however, that the soil located downslope from the piles was not stabilized by the placement of the piles. The soil above the piles has tended to "flow" through the piles, particularly during the wet season.

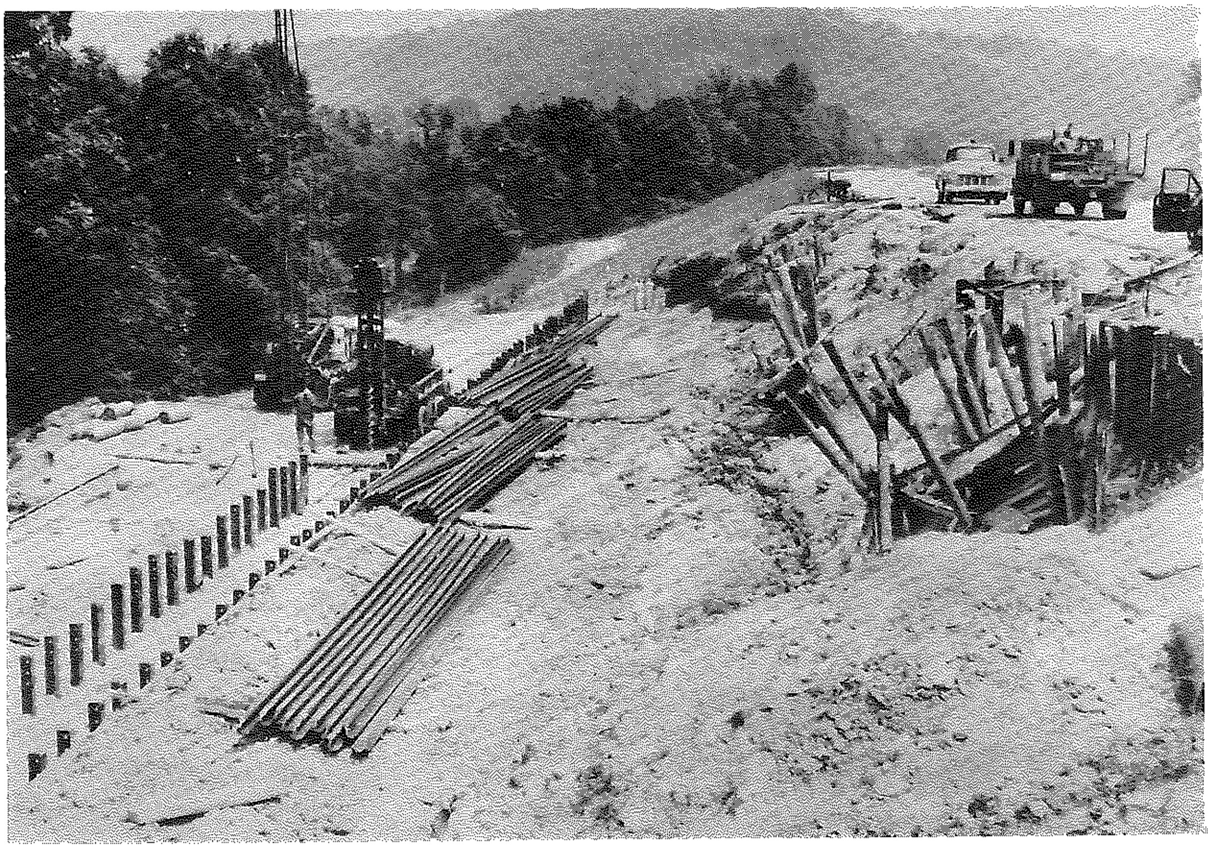


Figure 2-5. Installation of Rail Piles, August, 1966.

SLOPE STABILITY ANALYSIS

Western Kentucky Parkway

Grayson County, Mile Post 83

SLOPE STABILITY ANALYSIS
Western Kentucky Parkway
Grayson County, Mile Post 83

In February, 1966, undisturbed Shelby tube samples were obtained from drill holes along the left shoulder between Stations 6192+70 and 6194+00, near Mile Post 83, on the Western Kentucky Parkway in Grayson County. The fill in the westbound lanes was just beginning to show signs of distress at that time (see Figure 3-1 and 3-2). No movement at the toe was noticeable, but tension cracks were forming in the fill slope. In April, after a heavy rain, the slide moved an appreciable amount (Figure 3-3). Early in May, the slide progressed eastward and the westbound pavement cracked and settled on the order of two to ten feet (Figures 3-4 through 3-6). At this time, the Division of Research was requested by the Office of Engineering Management to investigate the failure and to suggest a method or methods to correct the slide.

The materials which outcrop in the southwestern portion of Grayson County are predominately of the Pennsylvanian age. The Caseyville Sandstone has been observed at the lower elevations with the Tradewater Formation above it. At the highest elevations in this portion of the county, materials classified as the Carbondale Formation have been observed.

Figure 3-1. Slide at Mile Post 83, Western Kentucky Parkway, in Feburary, 1966.

Figure 3-2. Slide at Mile Post 83, Western Kentucky Parkway, in February, 1966.

Figure 3-3. Slide at Mile Post 83, Western Kentucky Parkway, in April, 1966.

Figure 3-4. Slide at Mile Post 83, Western Kentucky Parkway, in May, 1966.

Figure 3-5. Slide at Mile Post 83, Western Kentucky Parkway, in May, 1966.

Figure 3-6. Slide at Mile Post 83, Western Kentucky Parkway, in May, 1966, Showing Roll at the Toe of Slide.

The Caseyville Sandstone is a crossbedded, conglomeratic, medium- to very coarse-grained sandstone intertonguing with shales. The middle portion contains several thin coal beds and thin layers of limestone. The Tradewater Formation is a series of interbedded sandstones and shales with occassional thin layers of limestone and thin coal beds. The sandstones may be relatively thick and well-cemented locally. The sandstones range from very thin laminations to very thick, crossbedded units in places. The shale, locally silty, is gray or black when unweathered but changes to slightly lighter colors when affected by water and air. The No. 1A coal is located near the base of the Tradewater Formation, and the No. 7 coal is located near the top. Underclays approximately a foot thick have been noted to be associated with coal beds throughout the formation. Layers of gray clay up to two feet thick have also been noted in drill holes in the area. The Carbondale Formation is predominately a shale or sandy shale with thin coal beds. The Sebree Sandstone, a crossbedded friable to well-cemented shaly, quartz sandstone, is located at the base. The Carbondale Formation in this area is capped by the No. 9 coal bed. Figure 3-7 illustrates a generalized columnar section for this portion of Ohio County.

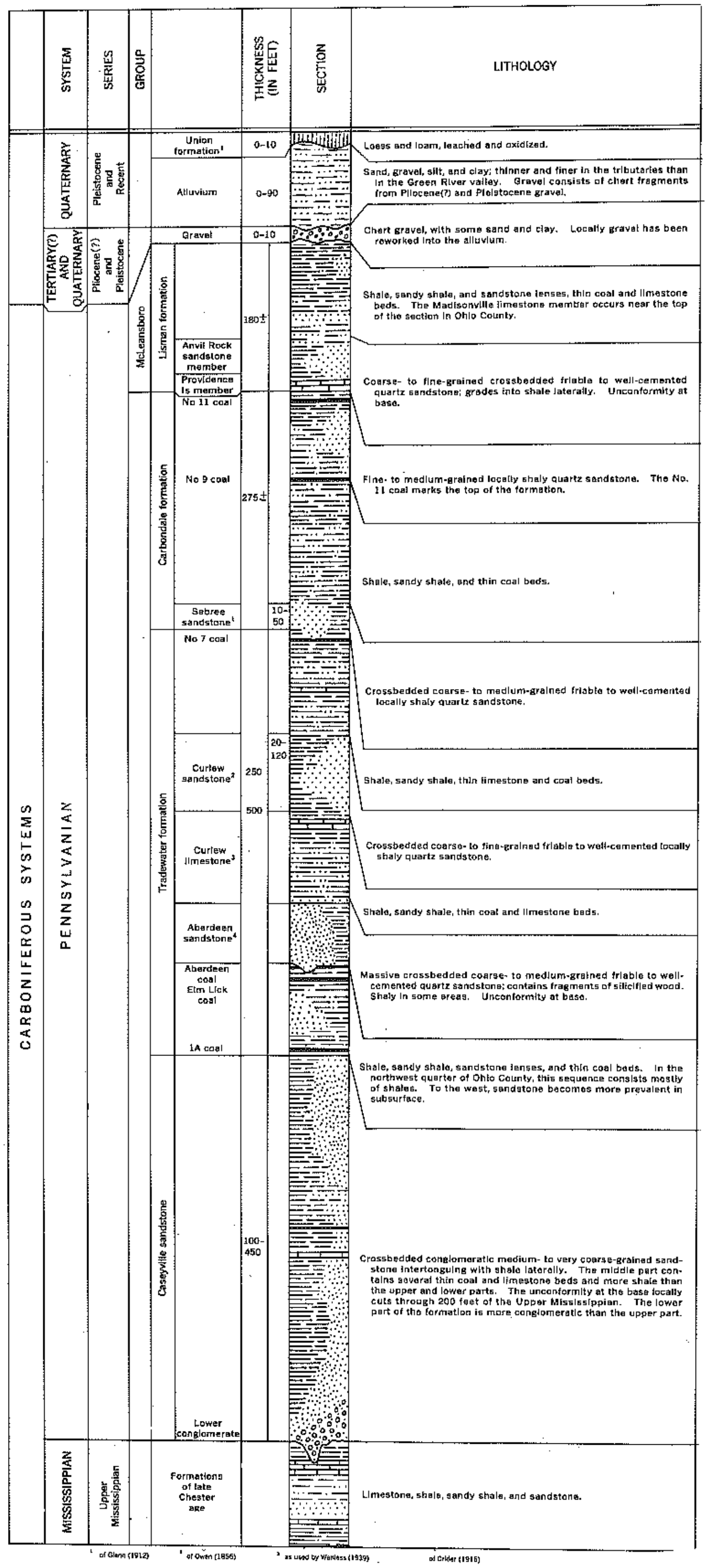


Figure 3-7. Generalized Columnar Section for Grayson County, Kentucky (from "Availability of Ground Water in Butler and Ohio Counties, Kentucky," B.W. Maxwell and R.W. Devaul, U.S. Geological Survey, 1962).

Strength and classification data obtained from laboratory testing of the Shelby tube samples taken from the left shoulder in February, 1966, were augmented with soundings by the Division of Maintenance along both shoulders and the median. Split-barrel samples were also obtained by the Division of Research in May from two bore holes at the toe of the embankment. This additional data proved valuable in locating the groundwater surface. The subsurface data are plotted in Figures 3-8 through 3-11.

Conventional, consolidated-undrained triaxial tests--with pore pressure measurements--were performed on two-inch diameter by three-inch long specimens trimmed from the Shelby tube samples. Mohr's circles and failure envelopes obtained from the triaxial tests are shown in Figure 3-12. A summary of the laboratory test data is given in Table 3-1.

A prepetually wet area in the shale cut, across from the slide area, indicates that seepage water is a contributing cause of the unstable condition. Observations of the groundwater level in drill holes is further verification that, in fact, seepage is likely the principle cause of the failure (see Figures 3-8 through 3-11). Water stands near the surface at the centerline, about 15 feet below the surface at the left (north) shoulder, and about 30 feet below the surface near the toe.

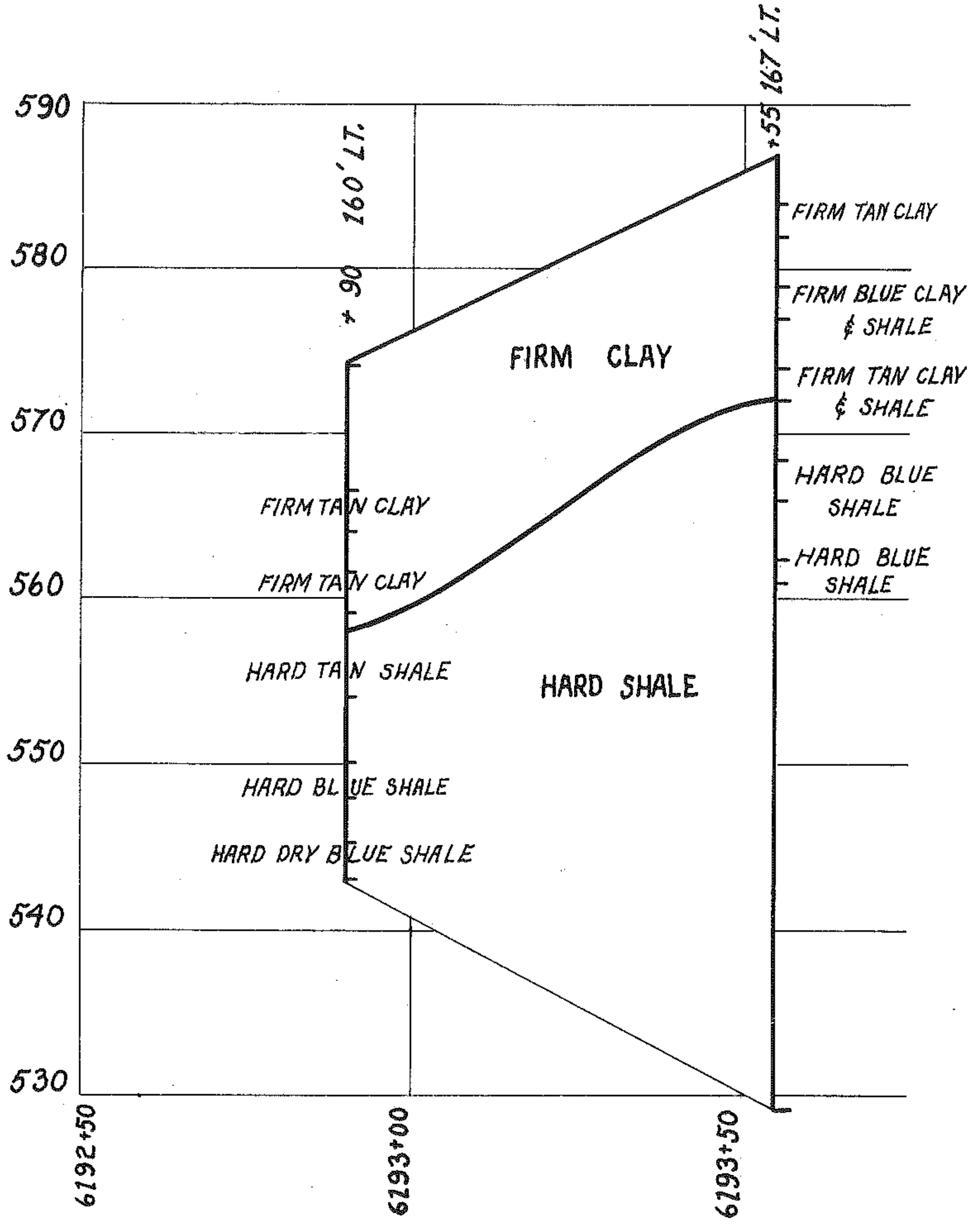


Figure 3-8. Soil Profile, Approximately 100' Left, Mile Post 83.

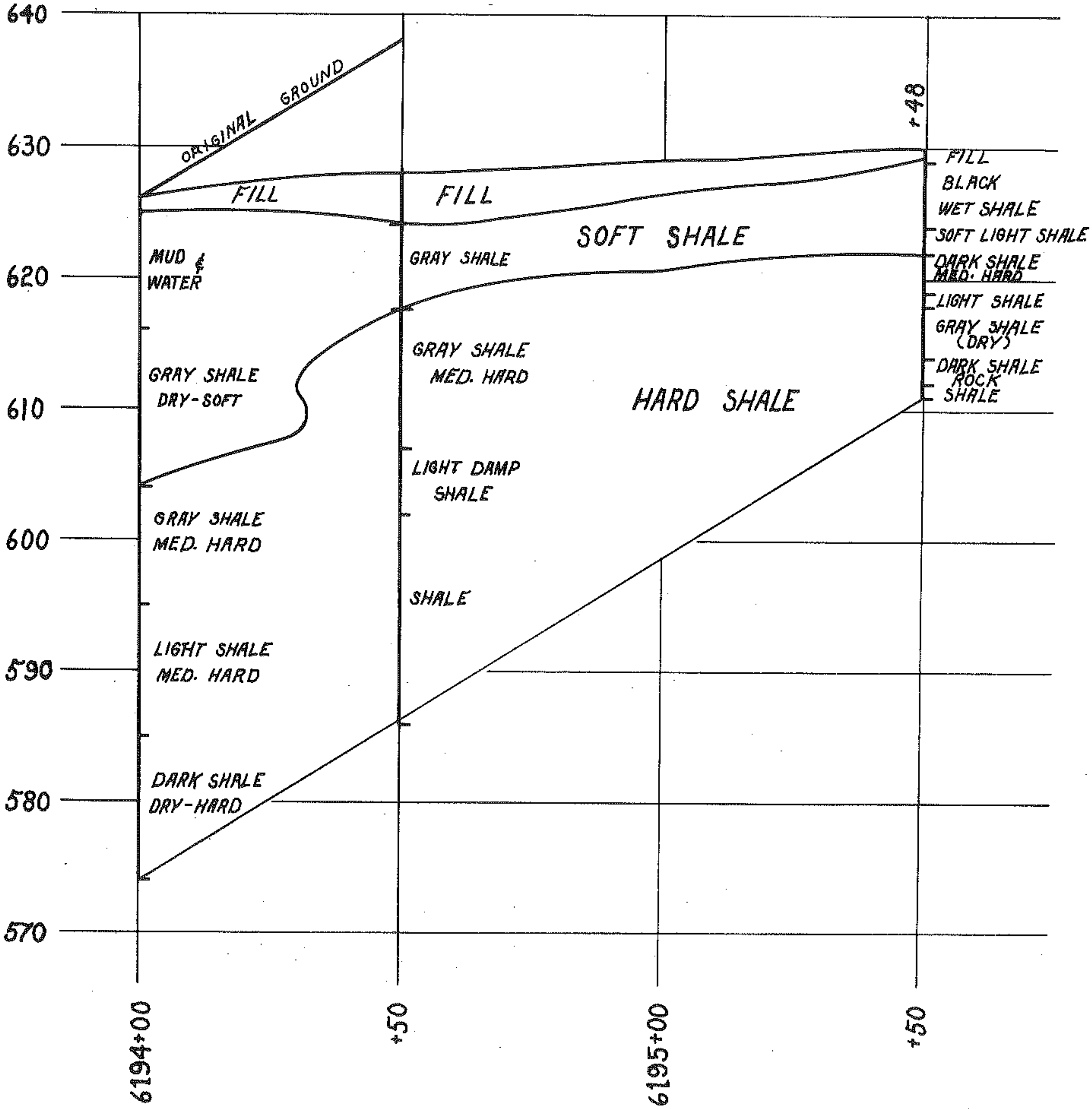


Figure 3-9. Soil Profile, Approximately 42' Left, Mile Post 83.

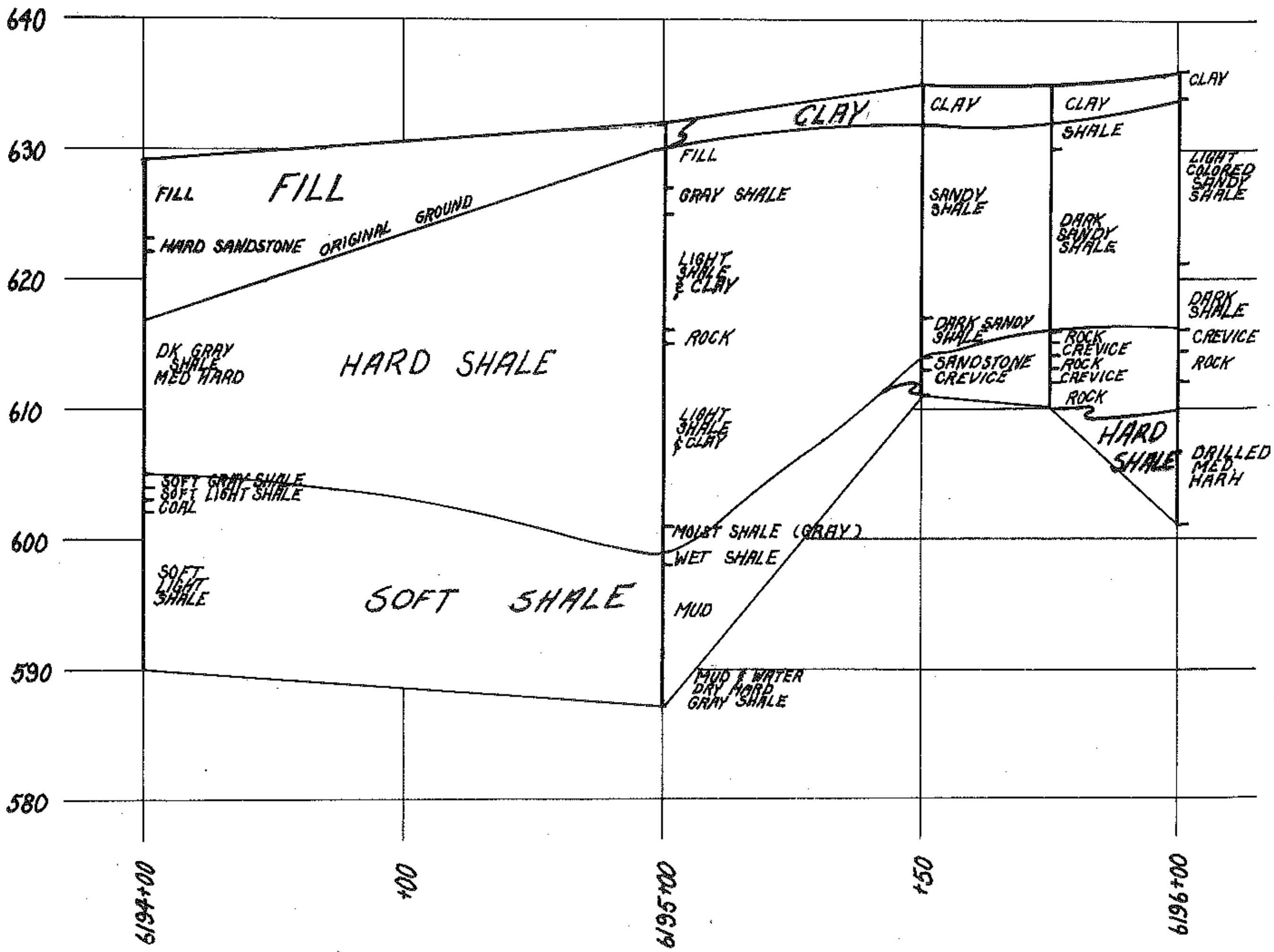


Figure 3-10. Soil Profile, Centerline, Mile Post 83.

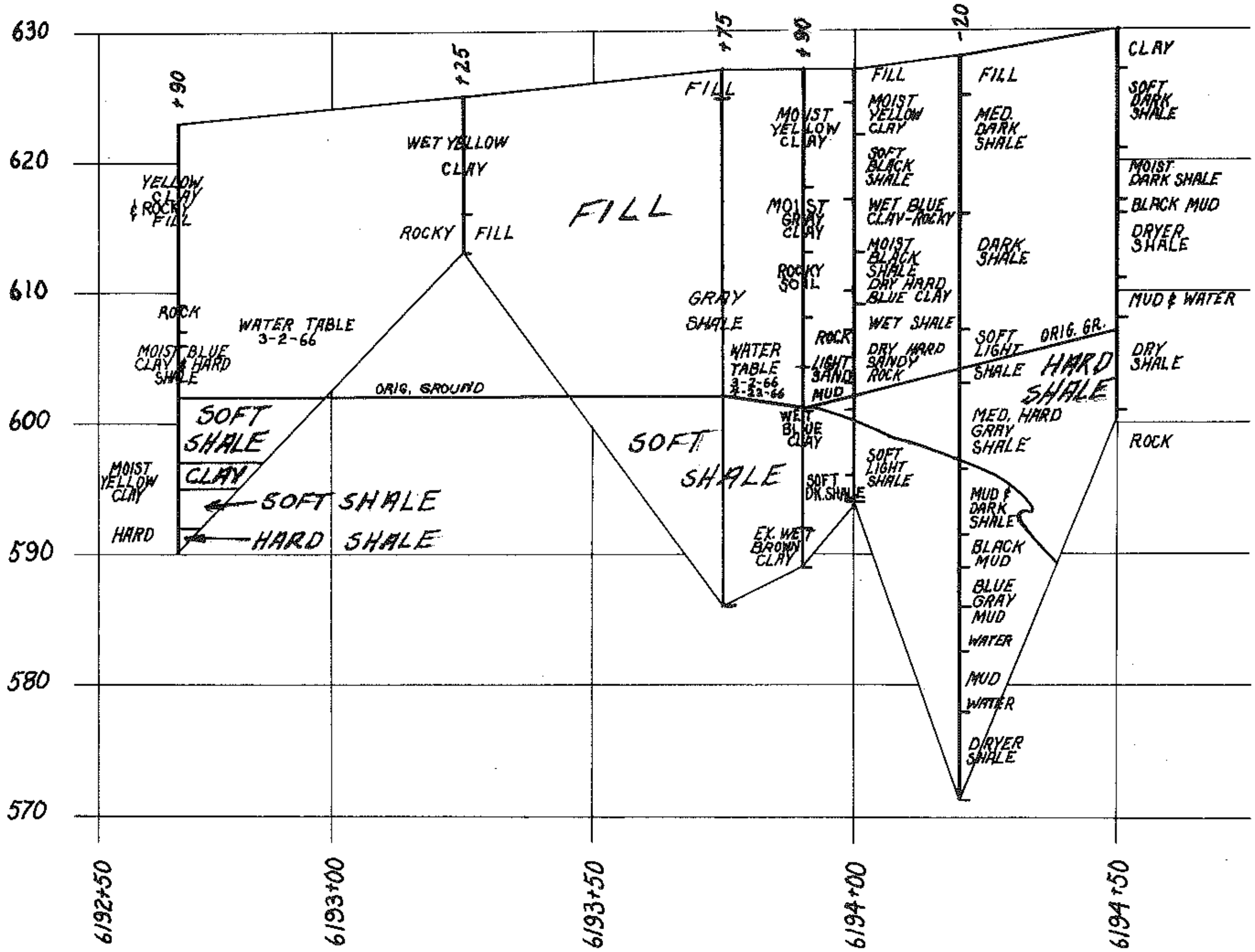


Figure 3-11. Soil Profile, Approximately 42' Right, Mile Post 83.

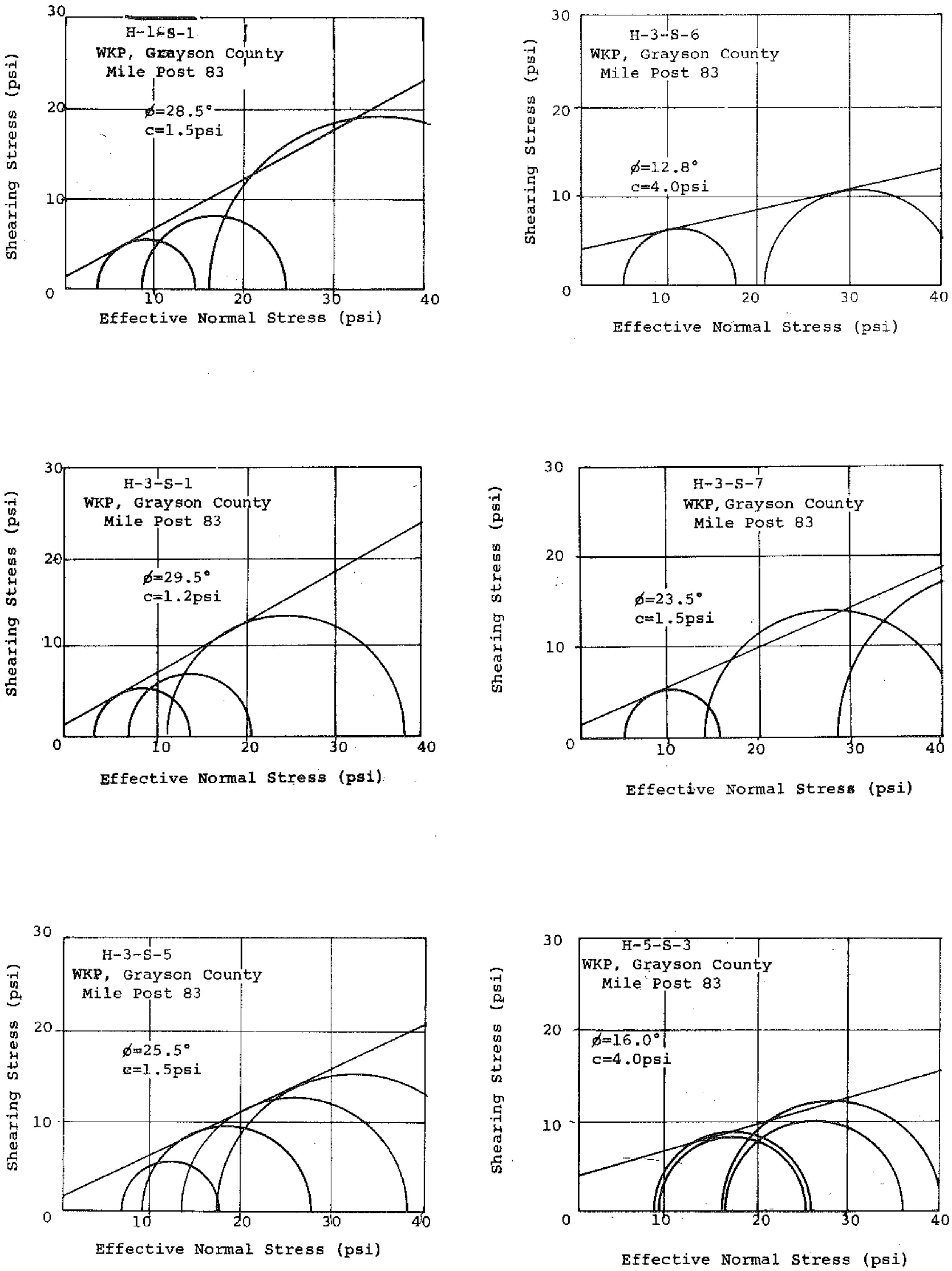


Figure 3-12. Mohr's Circles and Failure Envelopes.

TABLE 3-1. SUMMARY OF TEST RESULTS

Location	Sample No	Description	Depth (Feet)	Moisture Content (Shelby Tube Sample) (Percent)	Liquid Limit (Percent)	Plasticity Index (Percent)	Specific Gravity	Classification Unified	Classification AASHO	Dry Unit Weight (Lbs/CuFt)	Triaxial Test Data: Moisture Content (Percent) Before Test	Triaxial Test Data: Moisture Content (Percent) After Test	Triaxial Test Data: Effective Confining Pressure (Psi)	Triaxial Test Data: Cohesion (Psi)	Triaxial Test Data: Friction Angle (Degrees)
15' LT, STA 6193+25	H-1-S-1	Moist, Yellowish Brown Clay with Rock Fragments	5-7	Top - 15 Bottom-11	31	9	2.73	SC	A-2-4	112.8 106.7 107.4	18.6 17.7 18.5	18.8 18.3 20.7	3.5 8.5 16.1	1.5	28.5
30' LT, STA 6192+75	H-2-S-1	Moist, Yellowish Brown Clay with Rock Fragments	5-7	Top-16 Bottom-14											
20' LT, STA 6193+90	H-3-S-1	Moist, Yellowish Brown Clay with Rock Fragments	5-7	14	34	20	2.74	SC	A-2-4	112.2 114.0	17.8 19.2	18.4 19.9	3.5 7.3 11.6	1.2	29.5
	H-3-S-2	Moist, Rock Gray Clay	10-12	16											
	H-3-S-3	Rocky, Gray and Yellow Clay	15-17	16											
	H-3-S-4	Wet, Rocky, Sandy Clay	20-22	14	23	5	2.67	SC	A-2-4						
	H-3-S-5	Wet, Blue Clay and Weathered Shale	25-27	Top-12 Bottom-17	26	5	2.64	SC	A-2-4	101.8 101.2 106.6 104.0	20.5 17.0 19.0 20.4	21.2 17.0 16.0 19.1	6.5 8.7 13.0 17.0	1.5	25.5
	H-3-S-6	Extremely Wet, Rocky Brown Clay	30-32	Top-25 Bottom-19	31	9	2.72	SC	A-2-4	100.2 90.4	21.9 20.7	17.0 20.8	4.8 20.5	4.0	12.8
	H-3-S-7	Extremely Wet, Brown Clay	35-37	Top-25 Bottom-18	34	13	2.69	SC	A-2-6	123.0 116.0	17.6 18.6	19.5 19.6	4.8 13.6 28.4	1.5	23.5
20' LT, STA 6194+00	H-4-S-1	Moist, Rocky, Yellow Clay	5-7	12											
	H-4-S-2	Wet, Blue Clay	10-12	18											
	H-4-S-3	Hard, Blue Clay and Shale, Dry	15-17	Top-12 Bottom-9											
	H-4-S-4	Dry, Firm Sandy Loam	20-22	10											
15' LT, STA 6192+70	H-5-S-1	Rocky Fill	15-16												
	H-5-S-2	Wet, Yellow Clay at Top and Blue Clay and Shale at Bottom	19	Top-14 Bottom-10			2.70	GC							
	H-5-S-3	Moist, Yellow Clay	24-25	22	45	17	2.77	SC	A-2-7	105.2 107.6 110.6 105.1	21.5 20.1 20.8 21.9	22.3 21.9 21.0 21.8	8.3 9.0 15.8 16.3	4.0	16.0
	H-5-S-4	Solid Rock	30-32												

The available information strongly suggests that a free draining embankment or blanket would provide a suitable solution (see Figure 3-13). The stability analysis indicated that the fill would be safe against sliding if the groundwater level is maintained no higher than the level of the crushed stone blanket. The calculated factors of safety are 1.4 for a 2:1 slope and 1.5 for a 2-1/2:1 slope. A cutoff wall and drainage blanket of crushed stone, such as that shown in Figure 3-13, would insure that the water table would not rise above the blanket elevation along the westbound lanes. Actually, it is likely that the cutoff wall would be effective in lowering the water table below the crushed stone level so that the computed factors of safety represent minimum values.

The method of correction illustrated in Figure 3-13 does not include any means for lowering the groundwater level beneath the eastbound lanes. However, since this part of the roadway is in a cut and since the drill data indicate that the subsurface material is firm, there would not appear to be any cause for concern as to the stability of this area. Nevertheless, a cutoff drainage trench along the right (south) ditchline should be considered as an extra measure of safety along with reconstruction and

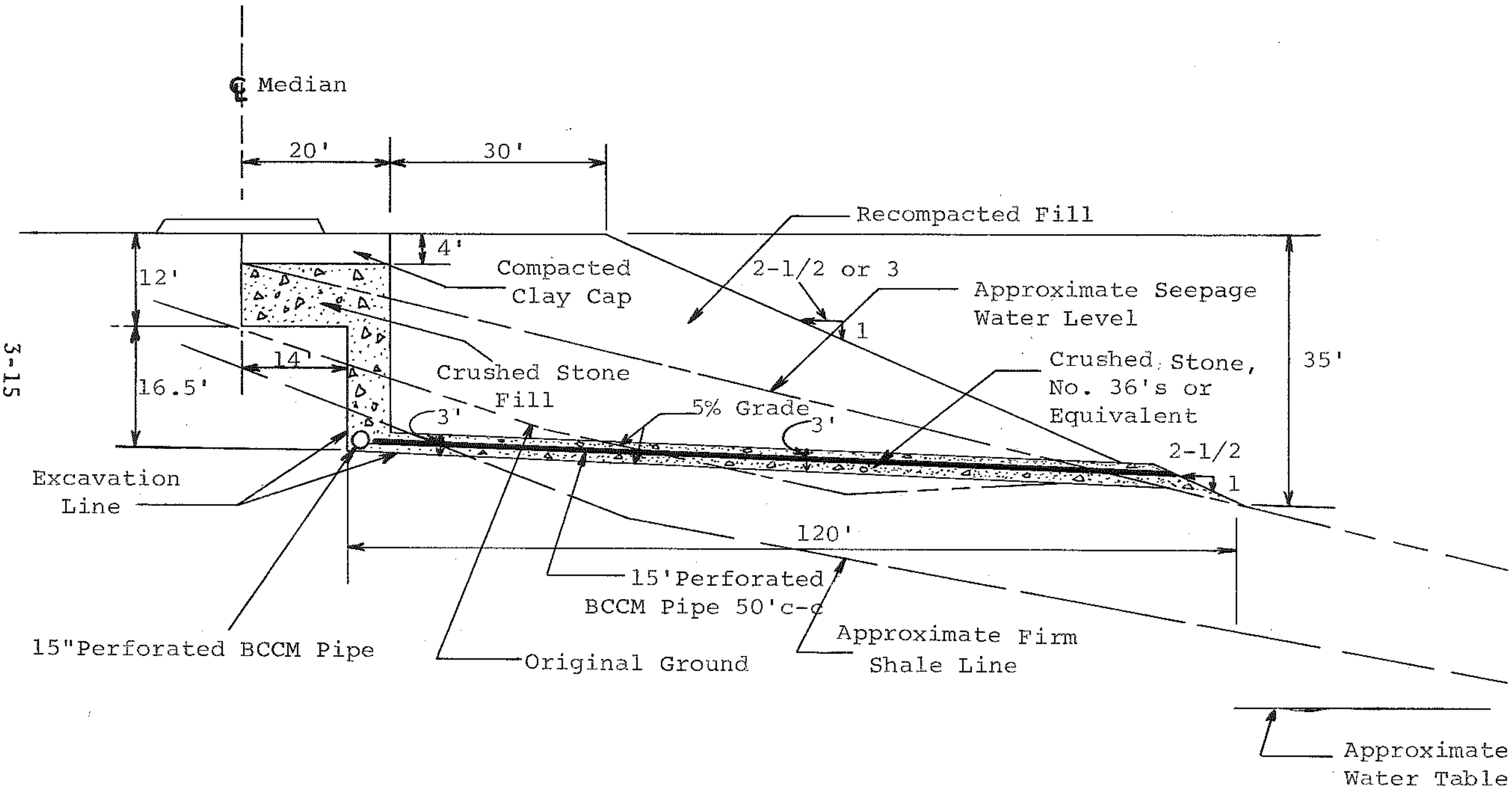


Figure 3-13. Proposed Method of Correction.

drainage of the failed portion.

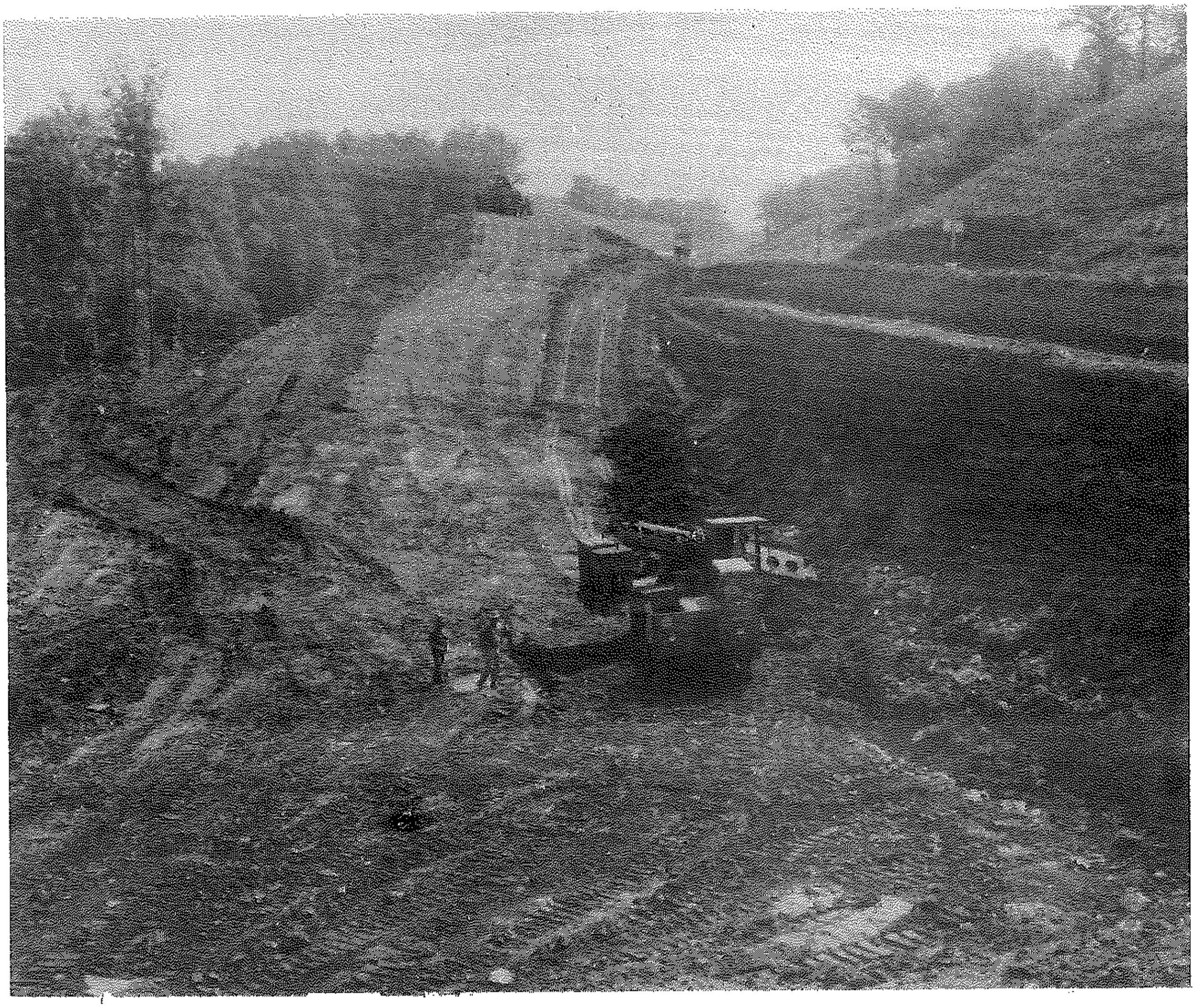


Figure 3-14. Failed Area Excavated Approximately to Firm Material.

SLOPE STABILITY ANALYSIS

US 23

Lawrence County, 1 Mile North of Louisa

SLOPE STABILITY ANALYSIS
US 23
Lawrence County, 1 Mile North of Louisa

In 1960, shortly after construction, settlement of the northbound lane of a side-hill fill was noticed on US 23 approximately one mile north of Louisa. At this time, no movement of the toe area was noticeable. Movement of the soil mass continued at a relatively slow pace until late 1963. At this time, buildings in the area of the toe of the embankment begin to carck and tilt (see Figures 4-1 through 4-3). This more rapid movement continued to the middle of 1964. The sliding mass appeared to be somewhat stabilized in the summer of 1965. The slope again continued to move gradually, and the Division of Research was requested to make a recommendation concerning the number of piles necessary to stabilize the embankment.

The site of the slide area is apparently located near the contact between the river alluvium in the valley and the Breathitt Formation which makes up the valley walls in this area. The Breathitt Formation consists of siltstones, sandstones, and claystones in this area. Minor contituents are coal, clay, ironstone, limestone, and chert (see Figgure 4-4). The numerous thin coal beds

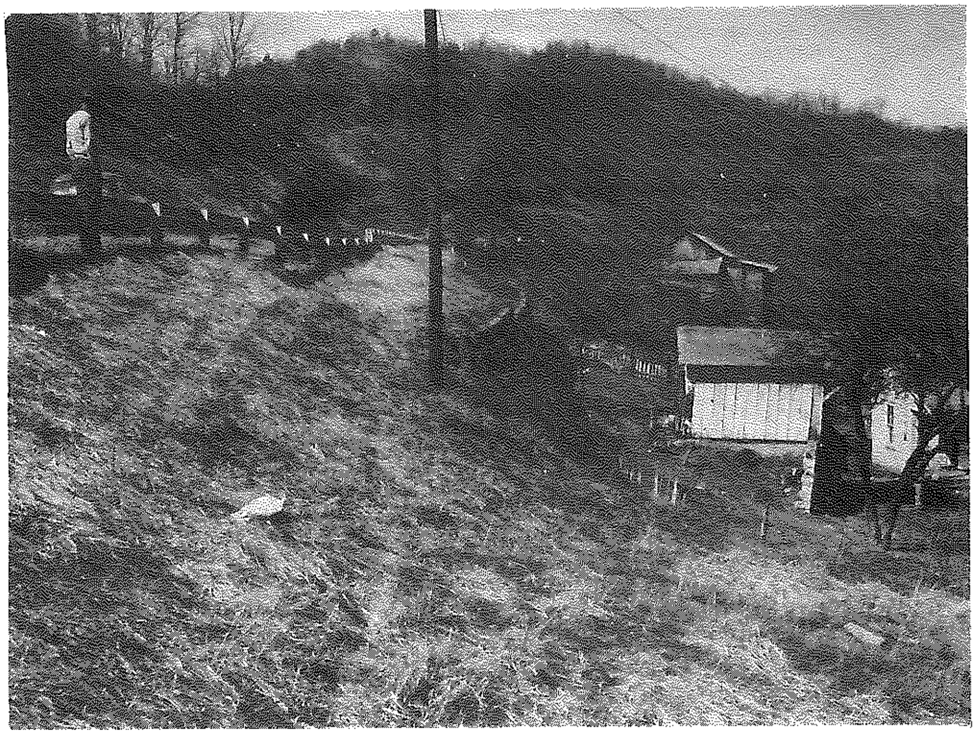


Figure 4-1. View of Slide Showing Involvement of Crib Wall.

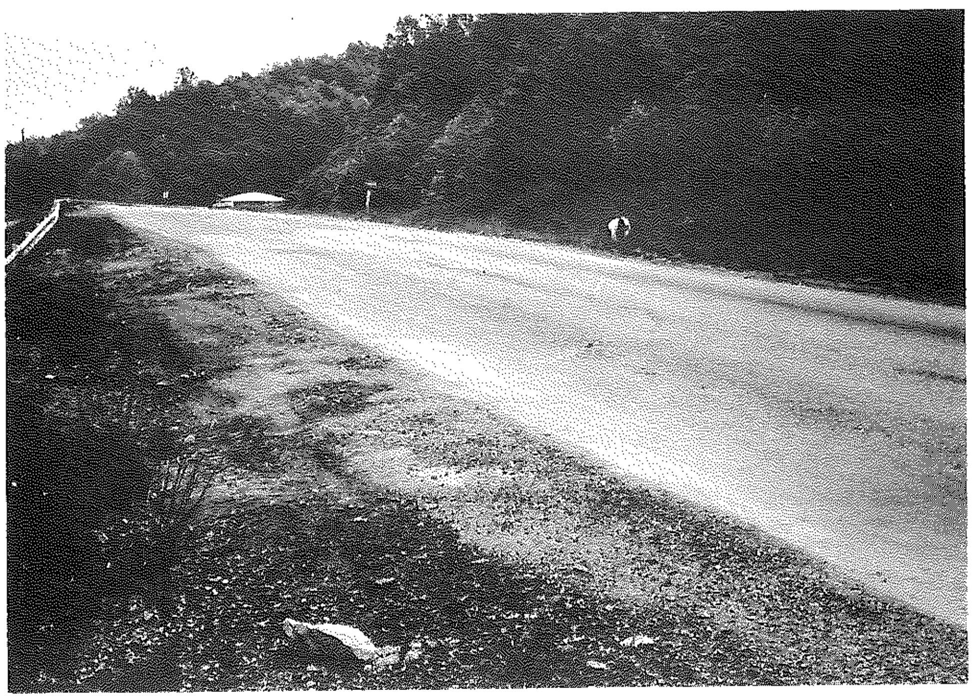


Figure 4-2. View of Slide Area Showing Displacement of Guard Rail.

Figure 4-3. Cracked and Tilted Walls at Toe of Slide.

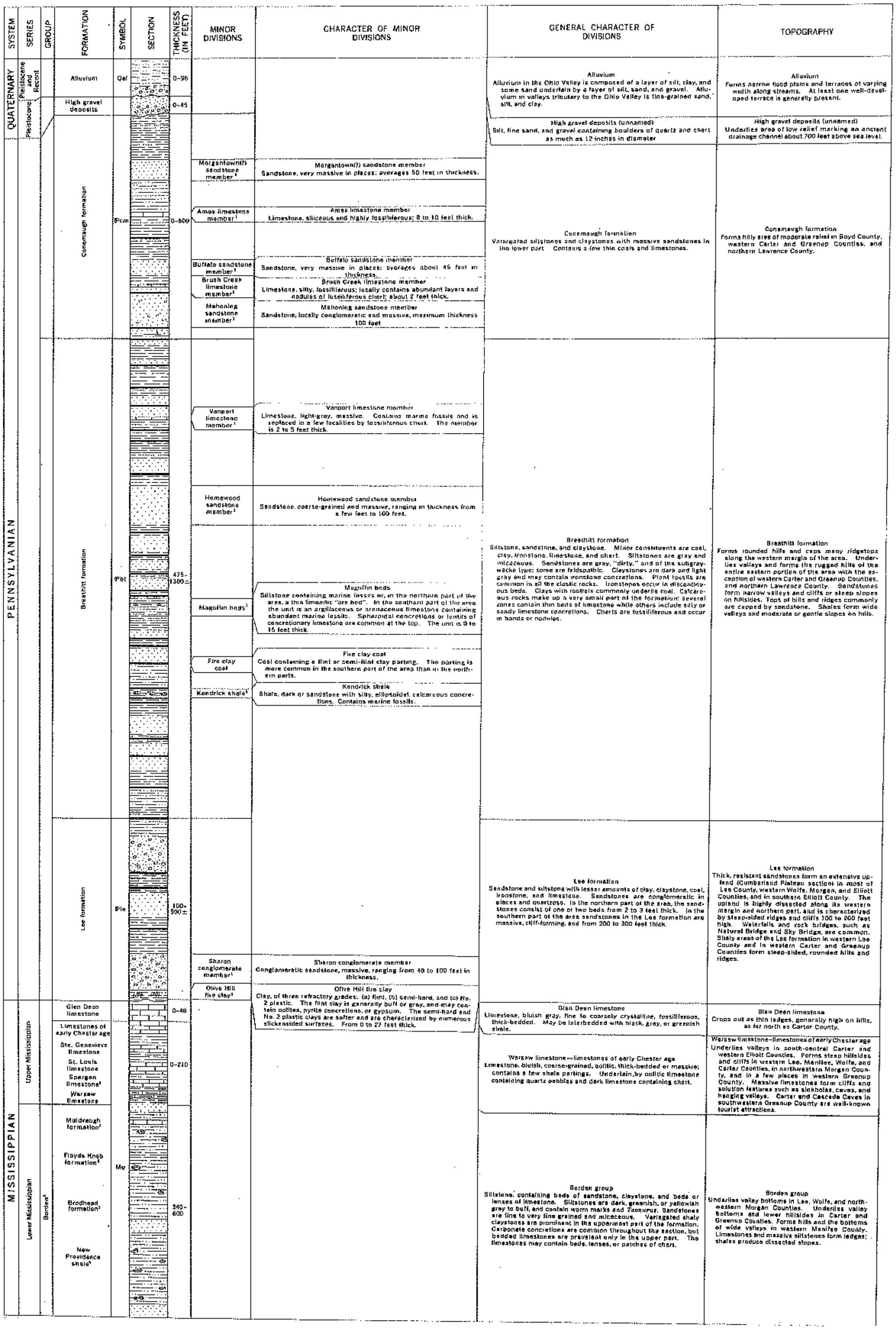


Figure 4-4. Generalized Columnar Section for Lawrence County, Kentucky (from "Availability of Ground Water in Boyd, Carter, Elliott, Greenup, Johnson, Lawrence, Lee, Menifee, Morgan, and Wolfe Counties, Kentucky," W.E. Price, Jr., C. Kilburn, and D.S. Mull, U.S. Geological Survey, 1962).

throughout the formation are associated with underclays along which seepage water may move. It is suggested that the principle cause of the slide is such seepage water which has been blocked by the side-hill fill--causing the fill to become saturated and therefore to lose strength and slip. The strata dip eastward and southward toward the roadway; seepage water is evident in the ditch.

Shelby tube samples were obtained from the slide area and returned to the Research Laboratory for testing. The samples were extruded from the tubes as soon as they were received from the field and any apparently distrubed material was discarded. The remaining undisturbed material was then cut into specimens approximately four inches long. These were immediately dipped in melted wax for storing and protection until time of testing. Conventional, consolidated-undrained triaxial test--with pore pressure measurements--were conducted in order to determine the strength parameters for use in slope stability analyses. The results of the triaxial tests are presented in Figure 4-5. A summary of pertinent test data is listed in Table 4-1.

At the time the slope stability analysis for this site was made, a computer program was available which

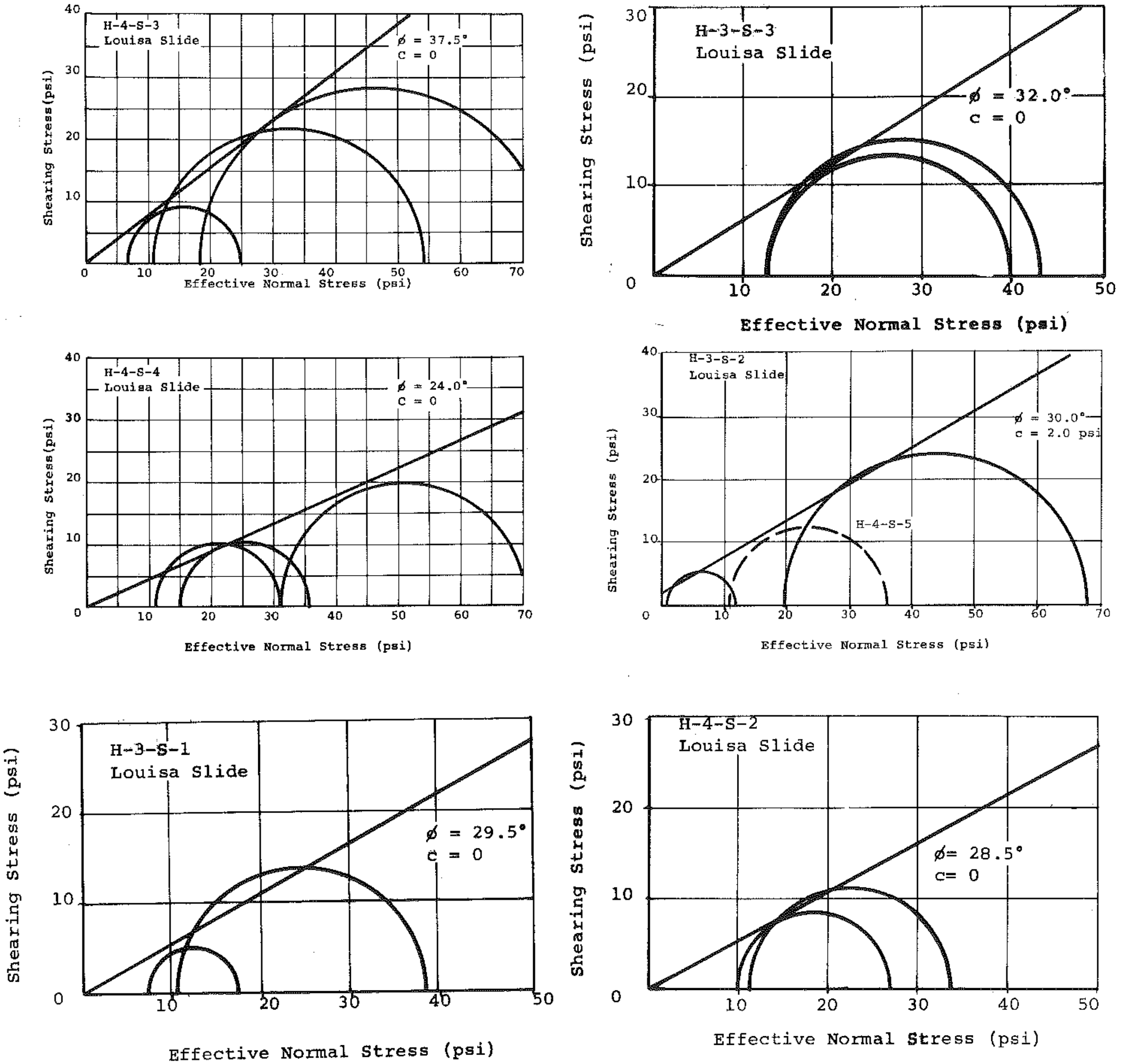


Figure 4-5. Mohr's Circles and Failure Envelopes.

Table 4-1. Summary of Test Results.

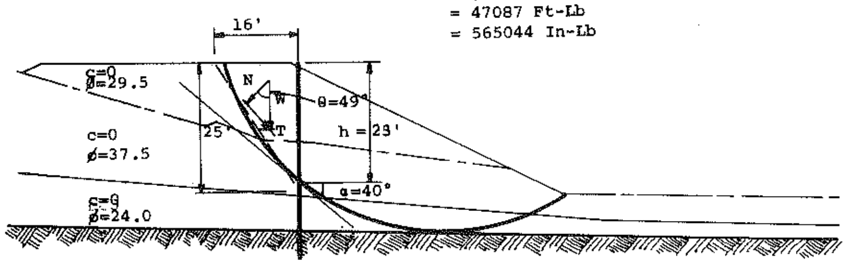
Location	Sample No.	Description	Depth (Feet)	Moisture Content (Shelby Tube Sample) (Percent)	Liquid Limit (Percent)	Plasticity Index (Percent)	Specific Gravity	Classification		Triaxial Test Data					
								AASHO	Unified	Dry Unit Weight (Lbs/CuFt)	Moisture Content (Percent) Before Test	Moisture Content (Percent) After Test	Effective Confining Pressure (Psi)	Cohesion (Psi)	Friction Angle (Degrees)
14'RT,STA 48+31	H-3-S-1	Light Brown Sandy Loam	5-7	19.0			2.64	A-3		118.5	17.5		7.3	0	29.5
										111.2	19.6	17.2	10.5		
												16.8	0.9		
	H-3-S-2	Light Brown Sandy Loam	10-12	15.0	34	10	2.76	A-2-4	SC	88.0		14.2	19.5	0	32.5
	H-3-S-3	Light Brown Sandy Loam	15-17	22.2	36	11	2.65	A-2-4	SC	103.0	20.0	21.3	12.8(12)	0	32.5
										99.2	22.6	22.2	12.8(25)		
	H-3-S-4	Light Brown Sandy Loam	20-21	22.0			2.70	A-3							
28'RT,STA 47+05	H-4-S-2	Tan Clay	10-12	16.6	32	11	2.72	A-2-4	SC	80.0	13.2	15.6	10.1	0	28.5
										103.8	15.4	18.4	11.5		
	H-4-S-3	Moist Tan Clay	15-17	21.2	31	9	2.66	A-2-4	SC	99.4	22.3	22.1	6.7		
											21.0	20.0	10.8	0	37.5
											21.0	20.0	18.4		
	H-4-S-4	Firm Tan Clay	20-22	22.0	39	12	2.67	A-2-6	SC	119.8	15.0	17.0	11.0		
										108.8	21.0	21.0	15.0	0	24.0
										115.5	19.0	19.0	31.4		
	H-4-S-5	Tan Clay with Shale Fragments	25-27	16.0	33	6	2.71	A-2-4	SC-SM	115.0	15.0	18.0	11.0		

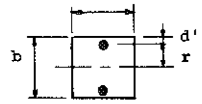
was capable of handling only a homogeneous cross section. Therefore, it was necessary to assume that the material involved at the site possessed the same strength parameters throughout. On the basis of this analysis, it was found that the factor of safety against slippage was on the order of 0.6.

An investigation was then made of the possibility of using piles as a means of stabilizing the moving mass. First, the effect of lateral loads on the pile imposed by the soil was investigated. The thurst P_D against the piling is determined from the equation which sums the effect of the tangential forces, the frictional forces and the cohesional forces along the slip surface uphill from the pile installation. The horizontal component of this force is assumed to be distributed linearally with depth such that the loading diagram on the pile is triangular and a cantilever loading situation exists. The total load is equal to the shearing force, the horizontal component of the thrust, on the piles. This cantilever loading of the pile induces certain bending moments in the pile which can be determined. The proportioning and spacing of the piles can then be made to withstand these bending moments. On the basis of this type of analysis (see Figure 4-6) it

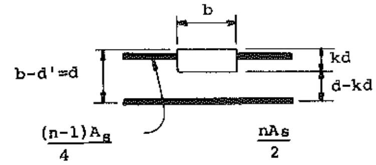
$W = \frac{1}{2} \times 16 \times 1 \times 125 = 25000$ Lb
$N = W \sin\theta = 18875$ Lb
$T = W \cos\theta = 16400$ Lb

$P_D = \Sigma T - \Sigma N \tan\phi' - c\ell$
$= 8018$ Lb
$M_D = \frac{h}{3} P_D \cos\alpha$
$= 47087$ Ft-Lb
$= 565044$ In-Lb



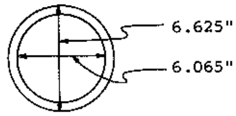


Equivalent Reinforced Square Pile

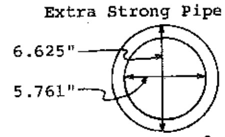


Transformed Section

Standard Pipe



Extra Strong Pipe



Standard Pipe		Extra Strong Pipe
5.58 in²	Area of Steel, A_s	8.41 in²
28.87 in²	Area of Concrete, A_c	26.06 in²
5.37"	b	5.11"
2.25"	r	2.20"
1.06"	d'	1.11"
4.31"	d	4.00"
10	n	10
27.90 in²	$nA_s/2$	42.03 in²
25.11 in²	$A_s(n-1)/2$	37.82 in²

$$\Sigma M_{(\text{Neutral Axis})} = 0$$

$$\frac{1}{2}b(kd)^2 + \frac{n-1}{2}A_s(kd-d') - \frac{n}{2}A_s(d-kd) = 0$$

Standard Pipe		Extra Strong Pipe
$2.69(kd)^2+53.0\,kd-146.9=0$		$2.56(kd)^2+79.9\,kd-210.1=0$
2.44"	kd	2.40"
1.87"	d-kd	1.60"
1.38"	kd-d'	1.29"

$$I = b\frac{(kd)^3}{3} + \frac{n}{2}A_s(d-kd)^2 + \frac{(n-1)}{2}A_s(kd-d')^2$$

Standard Pipe		Extra Strong Pipe
169.5 in⁴	I	194.0 in⁴
69.8 in³	$S_c = \frac{I}{kd}$	81.0 in³
9.9 in³	$S_s = \frac{I}{n(d-kd)}$	12.1 in³
209,400 in-lb	$M_c = f_c S_c$; $f_c = 3000$ lb/in²	243,000 in-lb
198,000 in-lb	$M_s = f_s S_s$; $f_s = 20000$ lb/in²	242,000 in-lb
3.56 Piles	$N_p = FS\frac{M_D}{M_s}$; N_p=No. of Piles/Ft; NS=Factor of Safety (Bending)	2.91 Piles
72,540 lb	$v = A_s f_v$; v=Shearing Resistance; $f_v = 13000$ lb/in²	109,330 lb
11.8 N_p	$FS_v = \frac{N_p v}{P_D \cos\alpha}$	17.8 N_p

Figure 4-6. Summary of Pile Design.

was noted that the number of piles required to adequately resist these bending moments was 3.6 piles per foot of length (standard strength pipe piles filled with concrete) to 2.9 piles per foot of length (extra strong pipe filled with concrete). An analysis was then made of the influence of these piles upon the factor of safety against shearing. It was found that in all cases the factor of safety against shearing through the concrete-filled pipe piles was more than adequate when a sufficient number of piles are used to resist the bending moments.

SETTLEMENT AND FOUNDATION STABILITY ANALYSIS

I 65

Barren County, SP 5-682-1L, I 65-2(1)37

SETTLEMENT AND FOUNDATION STABILITY ANALYSIS
I 65
Barren County, SP 5-682-1L, I 65-2(1)37

At the request of the Division of Materials, an analysis of the expected ultimate settlement and time-rate of settlement, with and without sand drains, was made for a section of I 65 in Barren County near Cave City. A stability analysis was also performed to check the adequacy of the foundation to support the embankment load. The embankment under consideration is approximately 40 feet high and the foundation consists of approximately 17 feet of wet, fine, silty sand. The water table may be expected to be near the surface during a large part of the year--the embankment traverses a sinkhole depression in the area.

In the northwest corner of Barren County, the corridor of I 65 crosses exposures of the Ste. Genevieve Limestone. The Ste. Genevieve is a white- to light-gray, thick-bedded, oolitic limestone. In the lower part there are tabular masses of dark chert. The Ste. Genevieve overlies the St. Louis Limestone conformably. The St. Louis is a cherty light- to dark-gray limestone consisting of thin- to medium-bedded deposits. The St. Louis is almost everywhere covered by a thick residum of red clay containing abundant chert fragments. The St. Louis and the Ste. Genevieve limestones are subject to solution by subterranean waters and have developed an extensive karst topography (see Figure 5-1).

Eleven Shelby tube samples were obtained by the Materials Division from four holes--three on the centerline at Stations 2082+00, 2083+00 and 2084+50 and one 30 feet left of Station 2084+50. The samples were extruded from the tubes as soon as they were received from the field and any apparently distrubed material was discarded. The remaining undisturbed material was then cut into specimens approximately four inches long and dipped in melted wax for storing and protection until testing. Detailed description of the appearance and consistency of the samples were noted at the time of extrusion to aid in subsequent selection of representative samples for testing. Moisture content samples were also obtained at the time of extrusion. All subsequent handling of the specimens, that is trimming and measuring, was done in a moist room to minimize the lost of natural moisture.

Four conventional consolidation tests and one special consolidation test were performed. The special test was arranged so that drainage was through a small

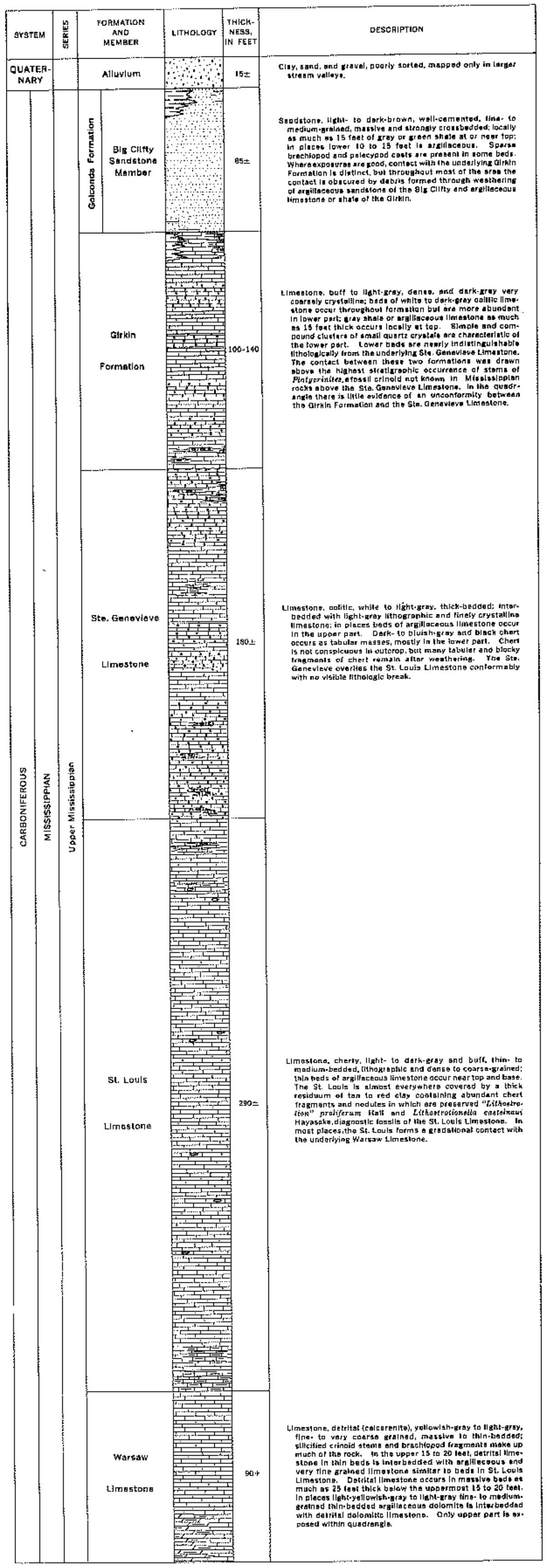


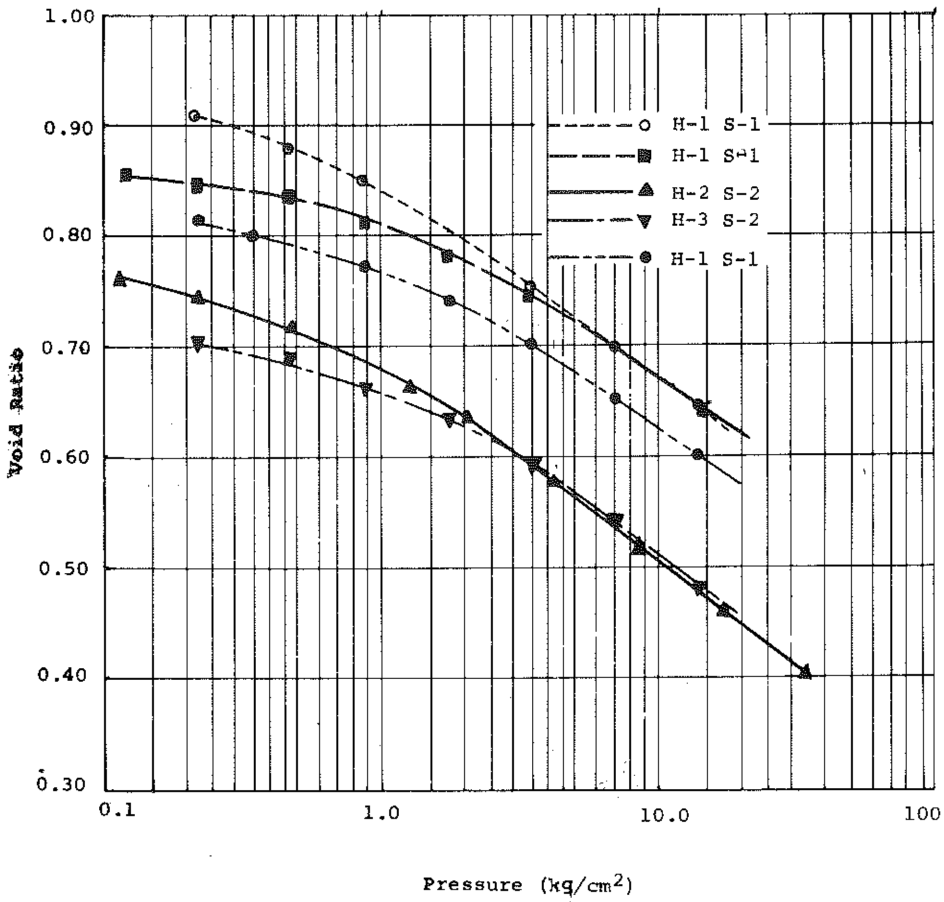
Figure 5-1. Generalized Columnar Section for Barren County, Kentucky (from "Geology of the Park City Quadrangle, Kentucky," D.D. Haynes, U.S. Geological Survey, 1962).

sand column in the center rather than to the top and bottom porous stones. The coefficient of consolidation for horizontal drainage, which is required for the design of sand drains, was calculated from data obtained using the special test procedure. The void ratio-log pressure curve and the coefficient of consolidation-log pressure curves are shown in Figure 5-2.

Analysis of the situation indicates that the ultimate settlement would be on the order of 12 inches (see Table 5-1). Calculations determining the time-rate of settlement (see Table 5-2) indicate that approximately 50 percent of the settlement will occur in four months and 90 percent of the settlement will take as long as 11 months. With sand drains on 15-foot centers, 90 percent of the consolidation would occur in approximately three months (see Figure 5-3).

Six unconfined compressive tests and five consolidated-undrained triaxial tests with pore pressure measurements were performed. A summary of the strength test results as well as the consolidation tests results are included in Table 5-3. Mohr's circles and the failure envelope are shown in Figure 5-4.

The unconfined compressive tests were performed and analyzed before the triaxial tests were undertaken.



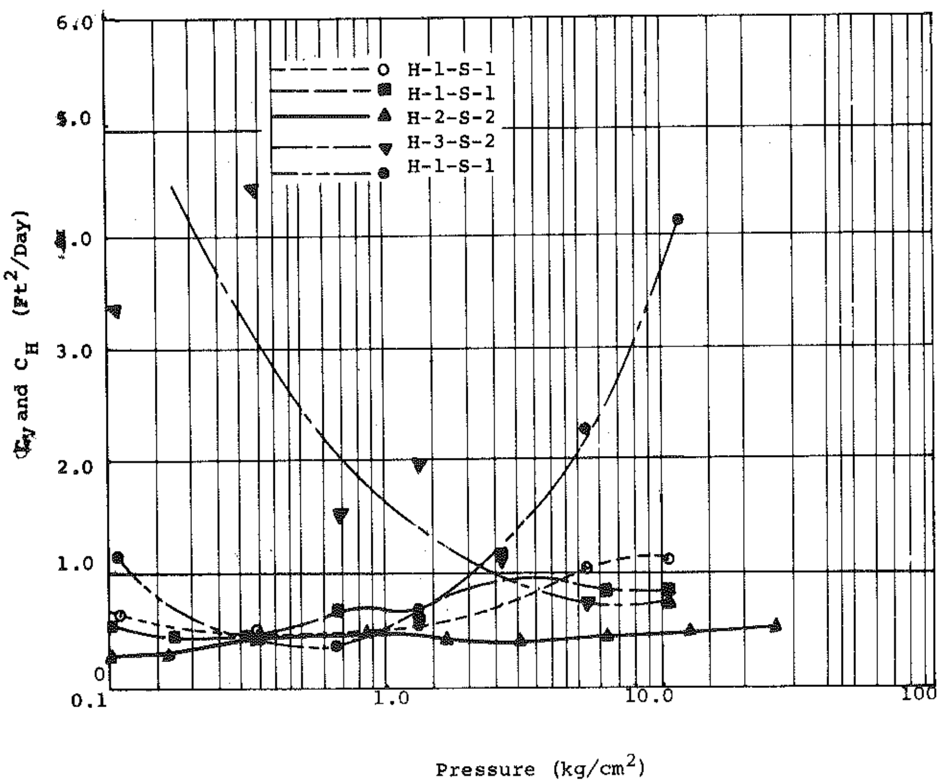


Figure 5-2. Void Ratio-Log Pressure Curves and Coefficient of Consolidation-Log Pressure Curves.

TABLE 5-1

CALCULATION OF ULTIMATE SETTLEMENT

Station Number	Assumption	Layer Description	Layer Thickness (H) (Feet)	Depth To Midpoint of Layer (D) (Feet)	Unit Weight (γ)[1] (Lbs/CuFt)	Overburden Pressure (P_o)[2] (Kg/Cm^2)	Influence Values (I)[3]	Vertical Stresses (ΔP)[4] (Kg/Cm^2)	Final Pressures (P_f)[5] (Kg/Cm^2)	Initial Void Ratio (e_1)	Final Void Ratio (e_2)	Settlement (ΔH)[6] (Inches)
2083	Water Table at Bottom of Layer	Tan Sandy, Silty Clay	17	8.5	118	0.490	1.0	2.443	2.934	.754	.661	9.0
2083	Water Table at Ground Surface	Tan Sandy, Silty Clay	17	8.5	118	0.231	1.0	2.443	2.674	.775	.669	12.2

1. $\gamma = \frac{1+w}{1+e_1} G\gamma_w$, where γ_w = Unit Weight of Water and G = Specific Gravity of Soil

2. $P_o = \gamma D$

3. From Influence Tables or Charts

4. $\Delta P = I\gamma_f H_f$, where γ_f = Unit Weight of Embankment Material (125 Lbs/CuFt) and H_f = Height of Embankment (40 Feet)

5. $P_f = P_o + \Delta P$

6. $\Delta H = \frac{H}{1+e_1}(e_1 - e_2)$

(Equilateral Sand Drain Spacing = 15 Feet and Sand Drain Radius = 10 Inches[1])

Radial Consolidation (U_r)[2] (with Sand Drains) (Percent)	Radial Time Factor[3] (T_r)	(Time (t)[4] (Days)	Vertical Time Factor[5] (T_v)	Vertical Consolidation (U_v)[3] (Without Sand Drains) (Percent)	$100-U_r$ (Percent)	$100-U_v$ (Percent)	$\frac{(100-U_r)(100-U_v)}{100}$ (Percent)	Average Total Consolidation (U_c)[6] (Percent)
10	.0192	4.8	.0149	13.6	90	86.4	77.8	22.2
20	.0404	10.1	.0314	20.0	80	80.0	64.0	36.0
30	.0644	16.1	.0501	25.0	70	75.0	52.5	47.5
40	.0928	23.2	.0722	30.3	60	69.7	41.8	58.2
50	.1258	31.5	.0980	35.3	50	64.7	32.4	67.6
60	.1654	41.4	.1289	40.4	40	59.6	23.8	76.2
70	.2122	53.1	.1653	45.9	30	54.1	16.2	83.8
80	.2912	72.8	.2267	53.7	20	46.3	9.3	90.7
90	.4180	104.5	.3254	63.5	10	36.5	3.7	96.3
95	.5422	135.6	.4222	71.3	5	28.7	1.4	98.6

1. Effective Drain Radius Taken as One-Half of Actual Radius to Account for Smear.

2. Selected Values

3. From Time Factor Tables

4. $t = \frac{(2R)^2 T_r}{C_{vr}}$ where 2R = Sand Drain Spacing and C_{vr}= Coefficient of Consolidation.

5. $T_v = \frac{C_v t}{H^2}$

6. $U_c = 100 - \frac{(100-U_v)\ (100-U_r)}{100}$

TABLE 5-2

SAMPLE CALCULATION FOR RATE OF CONSOLIDATION

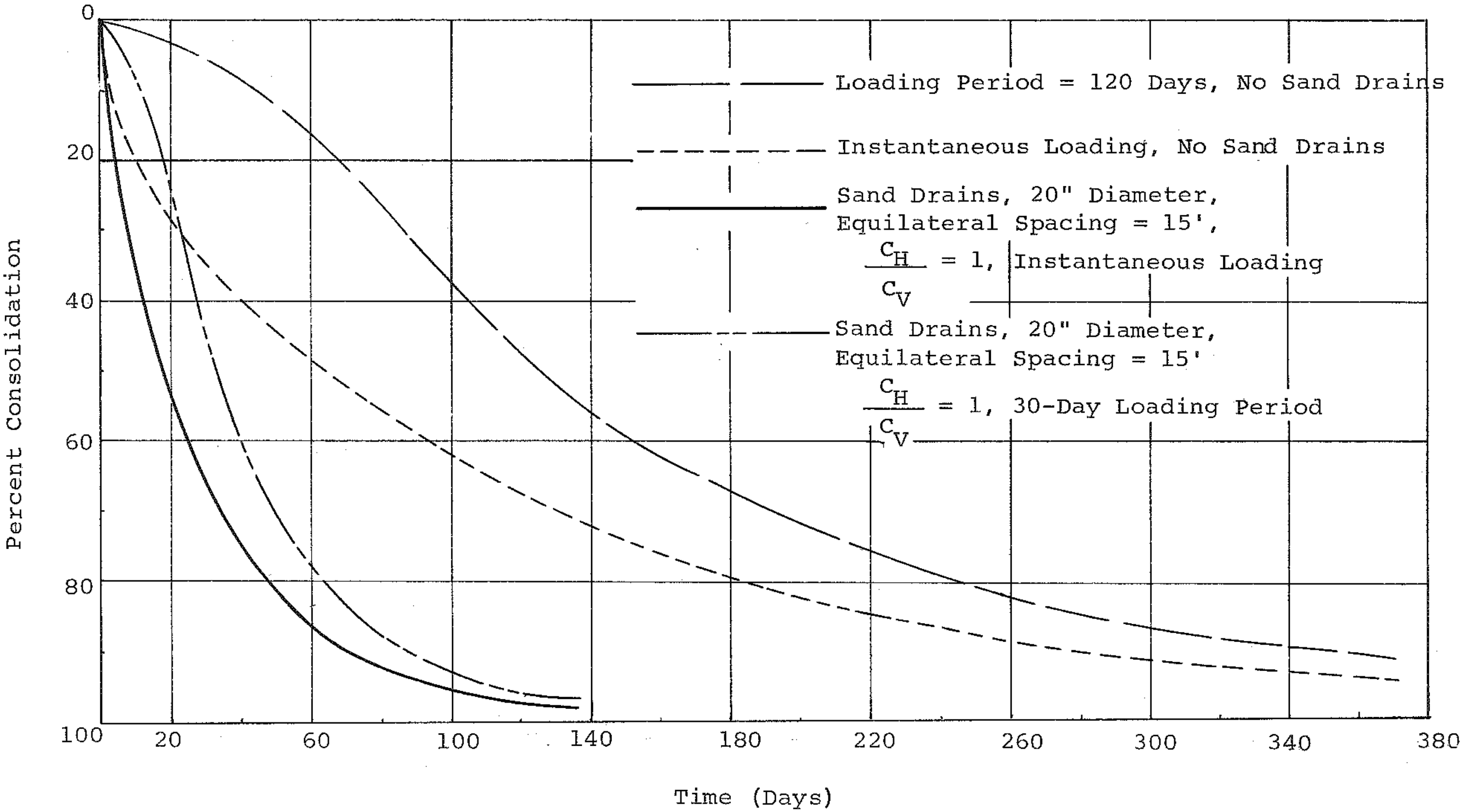


Figure 5-3. Rate of Settlement Curves.

TABLE 5-3. SUMMARY OF TEST RESULTS

Location	Sample No	Description	Depth (Feet)	Moisture Content (Shelby Tube Sample) (Percent)	Consolidation Parameters C_v (Ft^2/Day)	Consolidation Parameters C_c	Unconfined Compression Ultimate Strength (Psi)	Unconfined Compression Failure Strain (Percent)	Triaxial Test Data Dry Unit Weight (Lbs/CuFt)	Triaxial Test Data Moisture Content (Percent) Before Test	Triaxial Test Data Moisture Content (Percent) After Test	Effective Confining Pressure (Psi)
30' LT, STA 2084+50	H-1-S-1	Saturated, Silty Clay	5	32.7	0.56*	.105						
	H-1-S-2	Gray,Saturated,Silty Clay	10	31.4								
	H-1-S-3	Gray,Saturated,Sandy Clay	15	30.1			3.36 3.43	5.0 15.0	85.1			
	H-1-S-4	Gray,Saturated,Sandy Clay	20	30.0								
℄ STA 2084+50	H-2-S-1	Wet,Sandy Clay	5	29.6								
	H-2-S-2	Very Wet Sand	10	23.8	0.45	.125						
	H-2-S-3	Very Wet Sand	15	19.6								
℄ STA 2082+00	H-3-S-1	Firm,Moist,Sandy Clay	5	21.7			4.18	13.8	98.9	20.3 20.7 18.6	20.1 18.6 16.0	4.0 8.7 21.0
	H-3-S-2	Firm,Moist,Sandy Clay	10	22.1	1.80	.095	4.73	5.0				
℄ STA 2083+00	H-4-S-1	Firm,Moist,Sandy Clay	5	19.7			6.23	9.8	103.7			7.5
	H-4-S-2	Firm,Moist,Sandy Clay	10	21.5			3.85	3.8				

*C_h

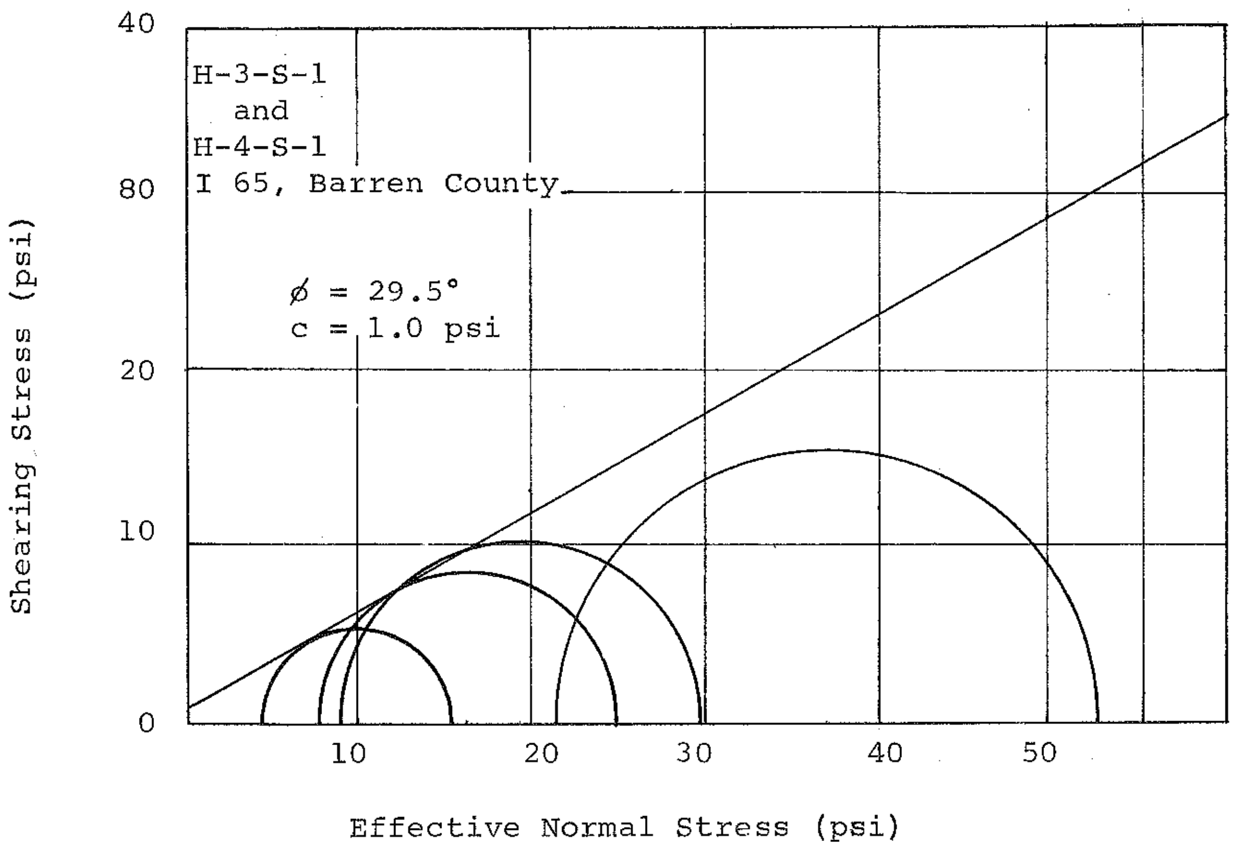
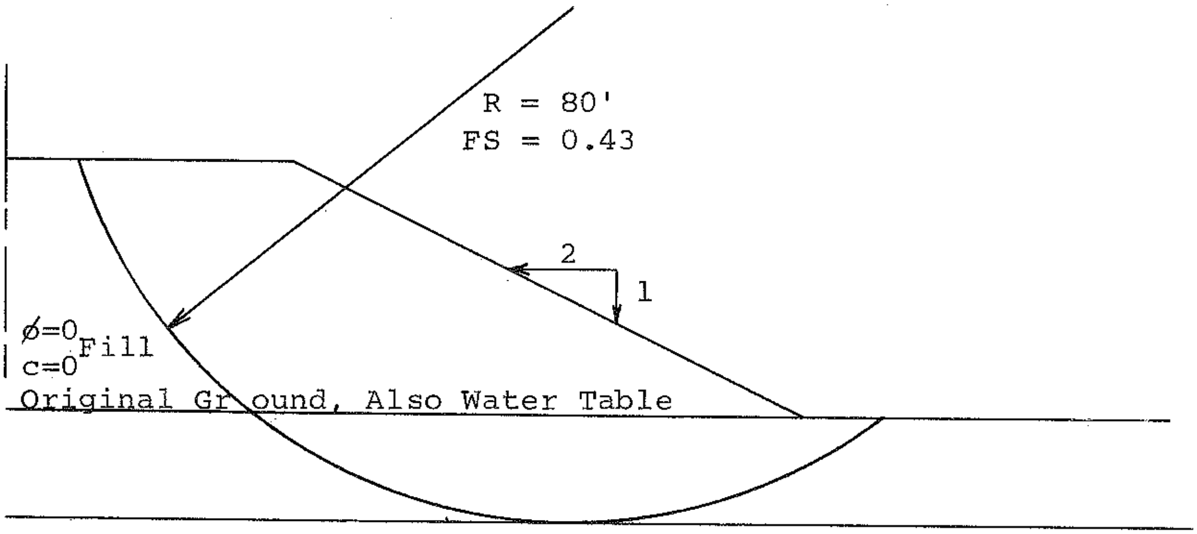


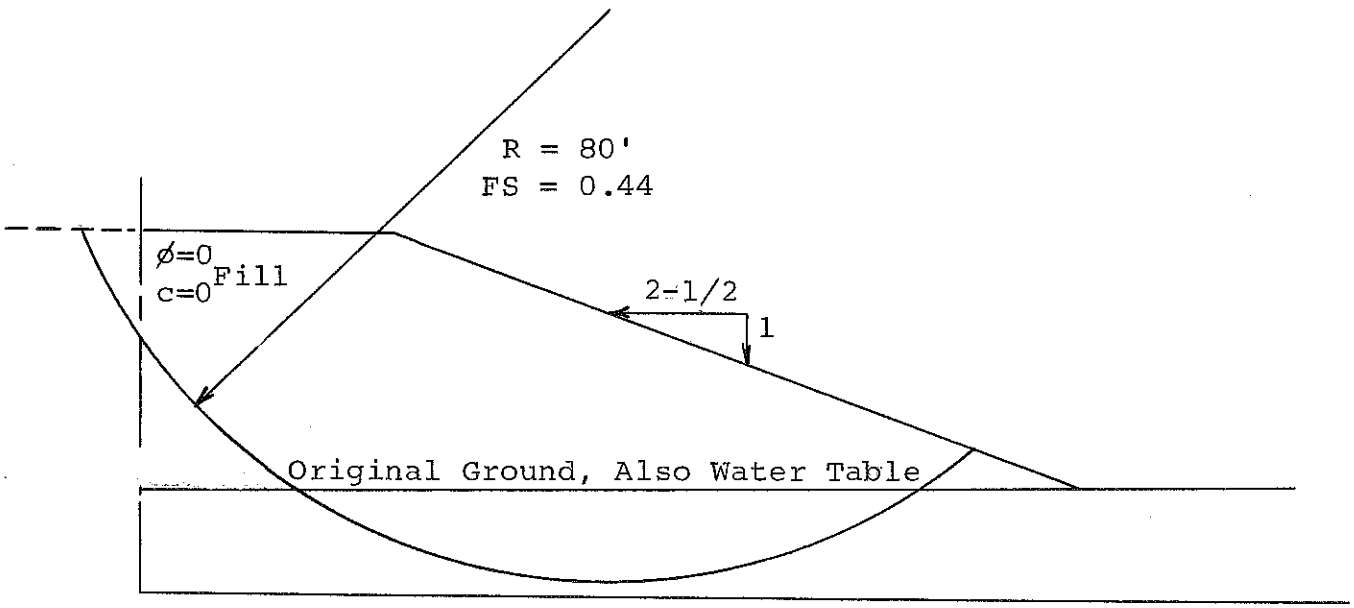
Figure 5-4. Mohr's Circles and Failure Envelope.

If the total stress analysis using the unconfined compressive test results had indicated an adequate factor of safety, it would not have been necessary to perform the triaxial tests. A total stress analysis (ø-equal-zero analysis) yields the minimum factor of safety for the critical time--soon after construction. However, as shown in Figure 5-5, the total stress factor of safety is inadequate and triaxial tests were necessary to define the effective stress strength. The long-term factor of safety--after the strength gain due to consolidation---is shown in Figure 5-6, and Figure 5-7 illustrates the long-term factors of safety for two different berm sizes.

The factors of safety obtained in Figures 5-5 through 5-7 neglected the shear strength of the embankment in accordance with current recommended practice for embankments on weak foundations. Figures 5-5 and 5-6 show that neither the total stress nor the effective stress factor of safety is adequate, even for a 2-1/2:1 fill slope, but Figure 5-7 indicates that the 15-foot high by 50-wide berm is more than adequate. However, it seems likely that this method of analysis--that is, neglecting the strength of the fill--may be overly conservative for this case. The sandy foundation soils

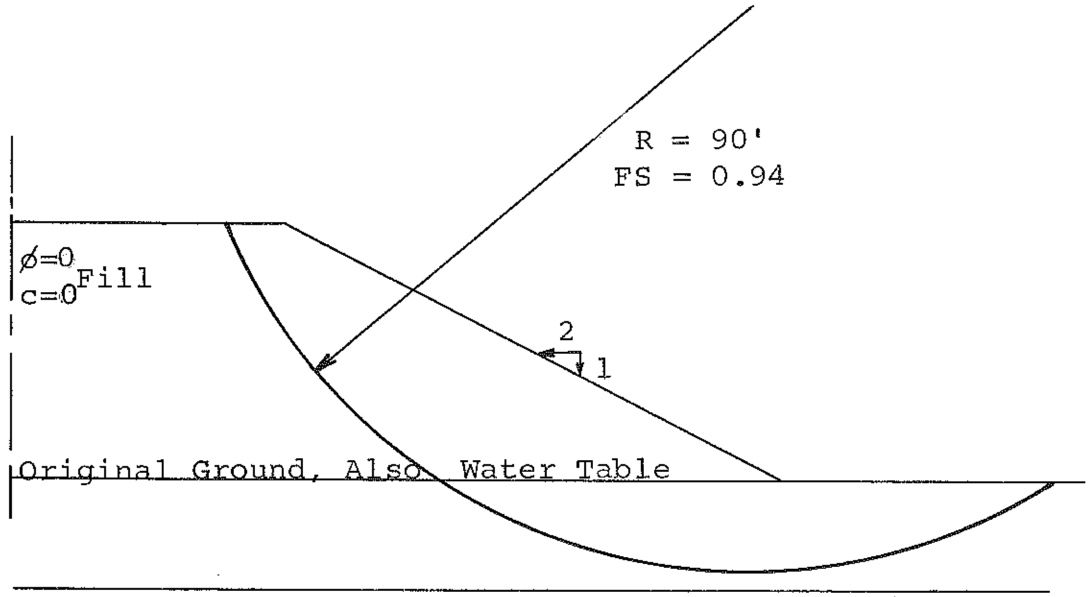


ø = 0 Analysis

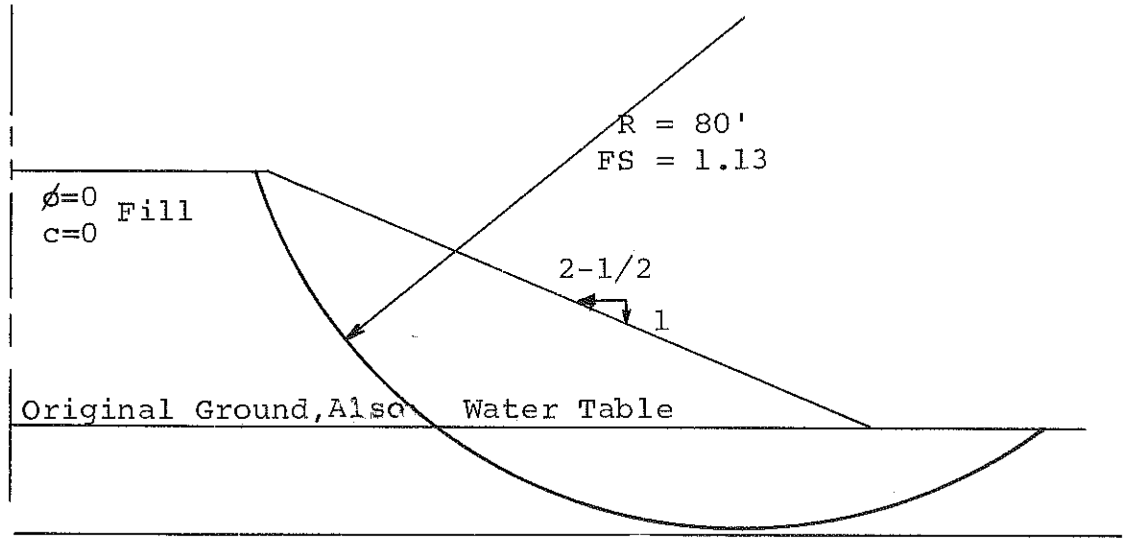


ø = 0 Analysis

Figure 5-5. Critical Surfaces for ø-Equal-Zero Analysis.

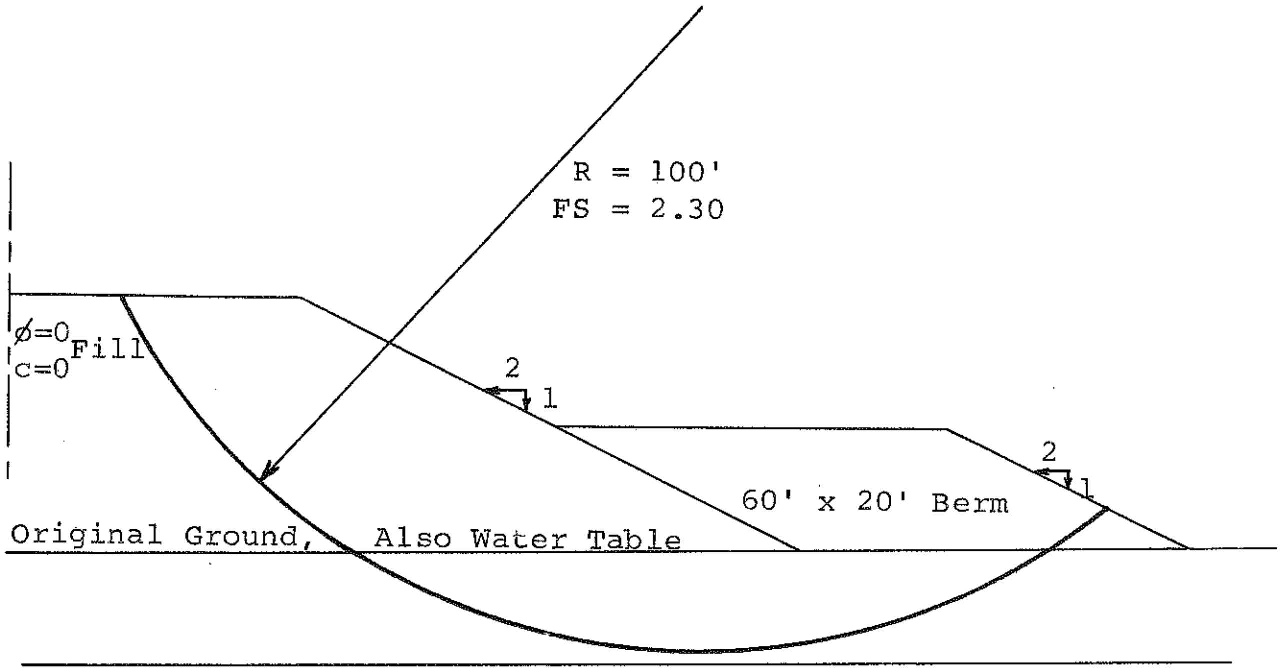


Effective Stress Analysis

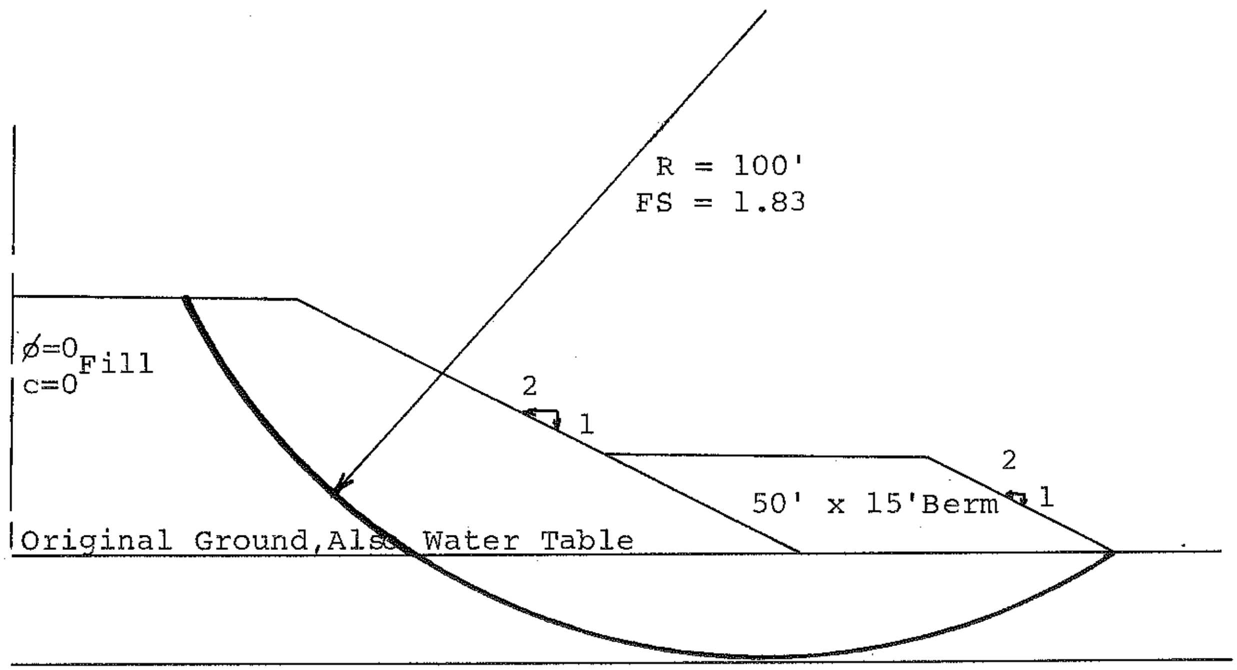


Effective Stress Analysis

Figure 5-6. Critical Surfaces for Effective Stress Analysis.



Effective Stress Analysis

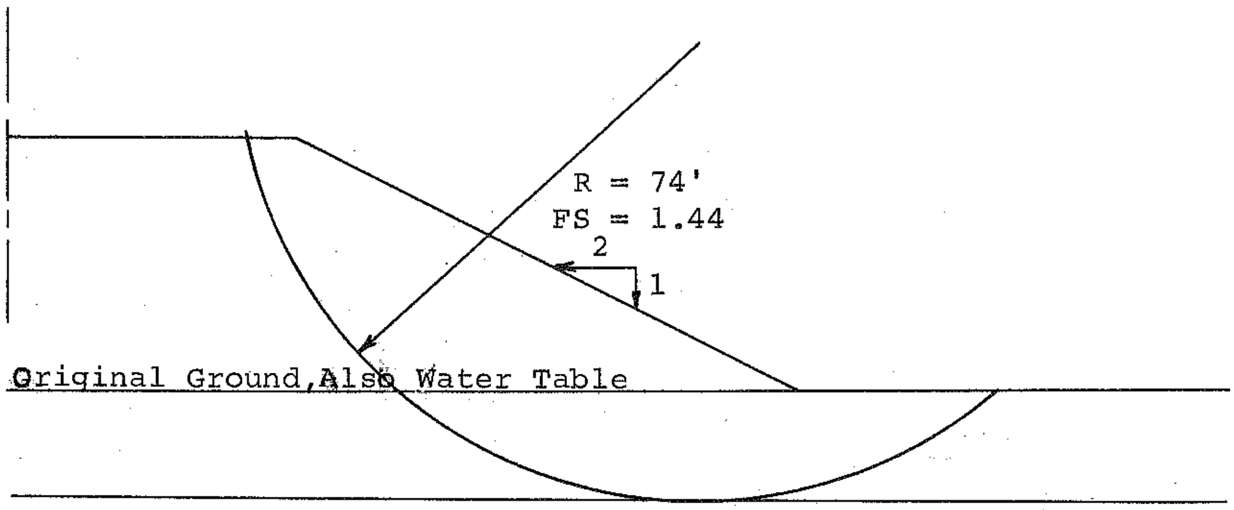


Effective Stress Analysis

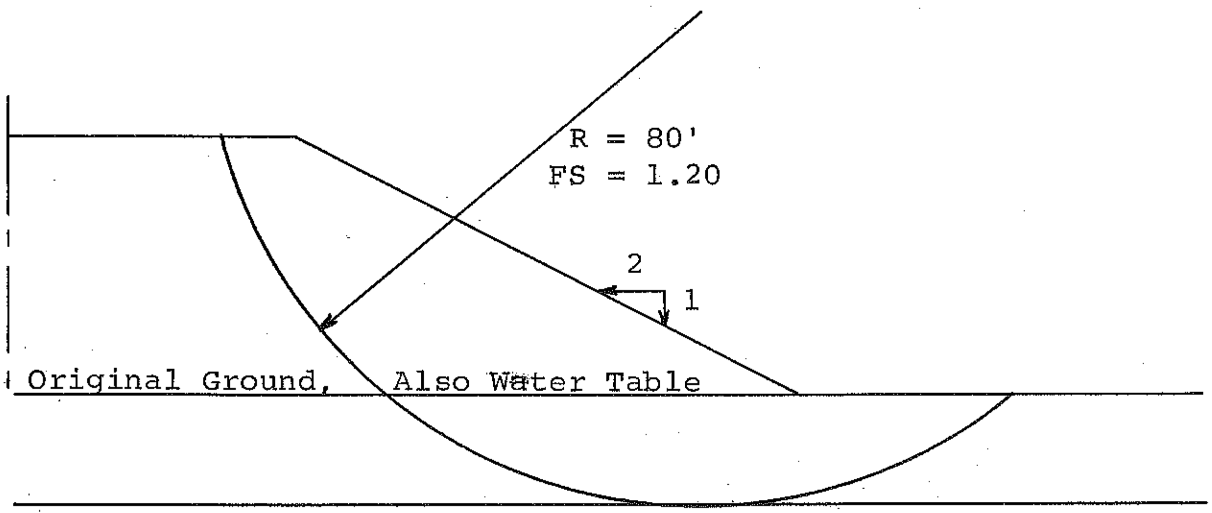
Figure 5-7. Critical Surfaces for Effective Stress Analysis and Berm Configurations.

consolidate relatively quickly and thereby increase their shear strength considerably during the construction period. Thus, at the end of construction, the strength and rigidity of the foundation may be little different from that of the embankment.

In Figure 5-8 are shown failure arcs and long-term factors of safety calculated on the assumption that the embankment and foundation soil are homogenous--that is, the shear strength of the embankment was not neglected and was assumed to be equal to that of the foundation material. The factors of safety are adequate and indicate that an embankment with 2:1 slopes may be safe if not constructed too rapidly. To insure that the foundation does not fail, it may be desirable to increase the rate of consolidation through sand drains or to use a slower-than-normal rate of construction.
Figure 5-3 shows that the sandy foundation materials consolidates relatively fast, even without sand drains. An extended construction period of approximately 120 days would permit a higher degree of consolidation than would be obtained with sand drains on 15-foot centers in a construction period of 30 days. Thus, there does not appear to be sufficient justification for the installation of sand drains.



ø' and c' Fill = ø' and c' Foundation



ø' and c' Fill = $\frac{1}{2}$ ø' and c' Foundation

Figure 5-8. Critical Surfaces for Effective Stress Analysis Assuming Different Strength Parameters for the Embankment.

Inasmuch as this investigation failed to indicate one single embankment configuration and construction procedure to be superior in all respects, three alternatives are worthy of consideration:

1. Construct the embankment and 15-foot high by 50-foot wide berms on 2:1 slopes at a normal rate.

2. Construct the embankment on 2-1/2:1 slopes, without berms, to a height of 20 feet at a normal rate and then construct the remainder at a reduced rate--approximately 120 days.

3. Construct the embankment on 2:1 slopes, without berms, to its full height and at a normal rate. If the embankment fails during construction, add 15-foot high by 50-foot wide berms.

SLOPE STABILITY ANALYSIS

US 119

Bell County, APD 151(18)

SLOPE STABILITY ANALYSES
US 119
Bell County, APD 151(18)

In reconstructing a portion of US 119 between Pineville and Harlan, near the community of Cardinal near the Bell-Harlan County line, a large cut was made through a toe ridge extending from the southeastern slope of the Pine Mountain Upthrust towards the Cumberland River. After the excavation had reached approximately to the planned grade line, a massive mantle of sandstone and topsoil began to slip from the slope above the high wall of the cut and to refill the excavated area (see Figure 6-1 and 6-2). The slide potentially involved more awesome quantities of tilted shales and sandstones lying beneath the sandstone mantle. The Division of Research was invited to study the site and to offer suitable suggestions.

The materials involved in the slide are of Pennsylvanian age and are associated with the well-known Pine Mountain Thrust Fault. The dip of the bedding planes of the sedimentary materials in this area is approximately 35°. The strike of the beds is essentially parallel to the roadway. When the excavation was made, support was removed and this permitted slippage along the bedding planes.

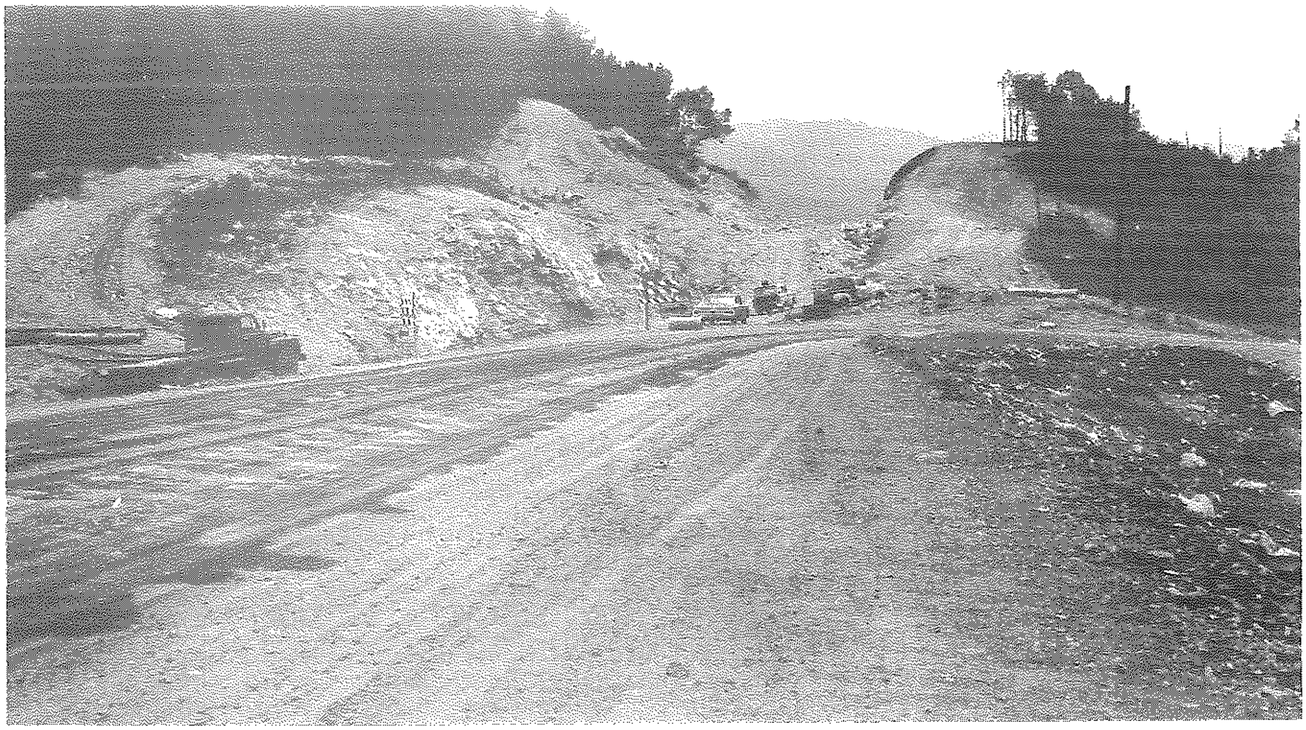


Figure 6-1. View of the Slide Area From the Southwest.

Figure 6-2. View of the Slide Area From the Northeast.

The area involved in the slide extends northwesterly from the roadway cut up a lower slope of the Pine Mountain ridge approximately 800 feet--where the sandstone ledge has broken loose and formed a chasm 15 to 75 feet in width and approximately 25 feet in depth, roughly paralleling the road (see Figure 6-3). Beyond this the terrain levels off for approximately 200 feet, forming a small bench (see Figure 6-4). The summit slope then rises an additional 1000 feet or more to the northwest. Geologists say that the lower slope and the strata tending the daylight at the bench are probably the Breathitt Formation; whereas the summit slope is thought to be the upper part of the Lee Formation. It is speculated, at least, that the slide area is of the upper Lee Formation and (or) of the lower Breathitt Formation (see Figures 6-5 and 6-6).

An investigation of the site indicates that the slope consists of approximately 18 to 25 feet of sandstone resting directly on a plastic clay approximately one foot thick. A thin coal seam approximately one foot thick lies under the clay. Below this is approximately 60 feet of dark, gray shale and then about 20 feet of light gray, shaley sandstone (see Figure 6-7). The material which is

Figure 6-3. View of the Crevasse above the Moving Mass.

Figure 6-4. View of Slide Area Showing a Portion of The Southeast Slope of Pine Mountain.

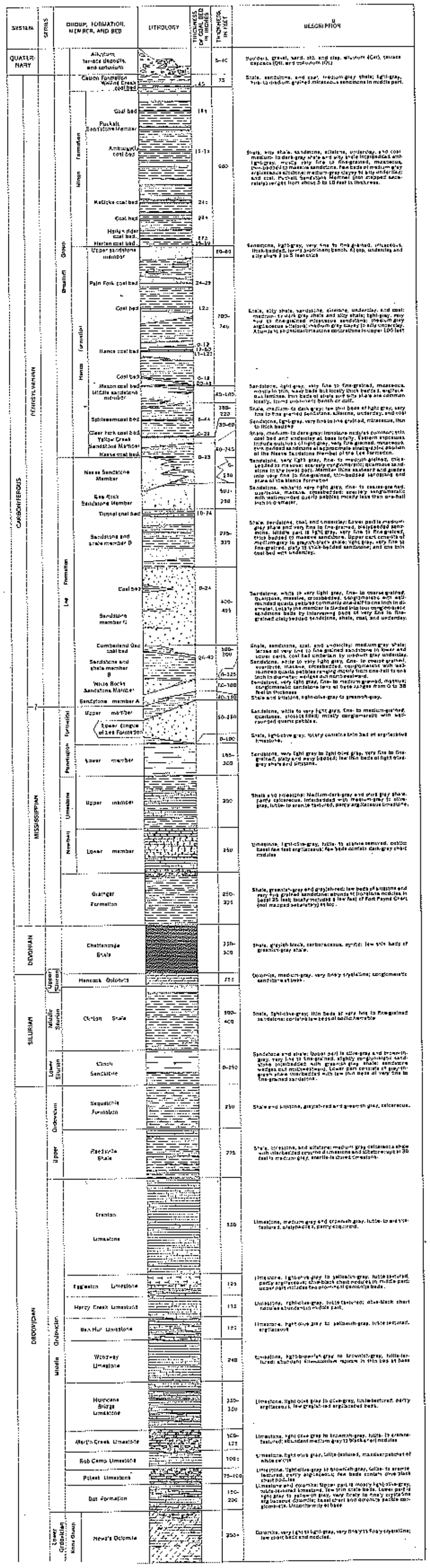


Figure 6-5. Generalized Columnar Section for Bell County, Kentucky (from "Geology of the Varilla Quadrangle, Kentucky-Virginia," K.J. Englund, E.R. Landis, and H.L. Smith, U.S. Geological Survey, 1963).

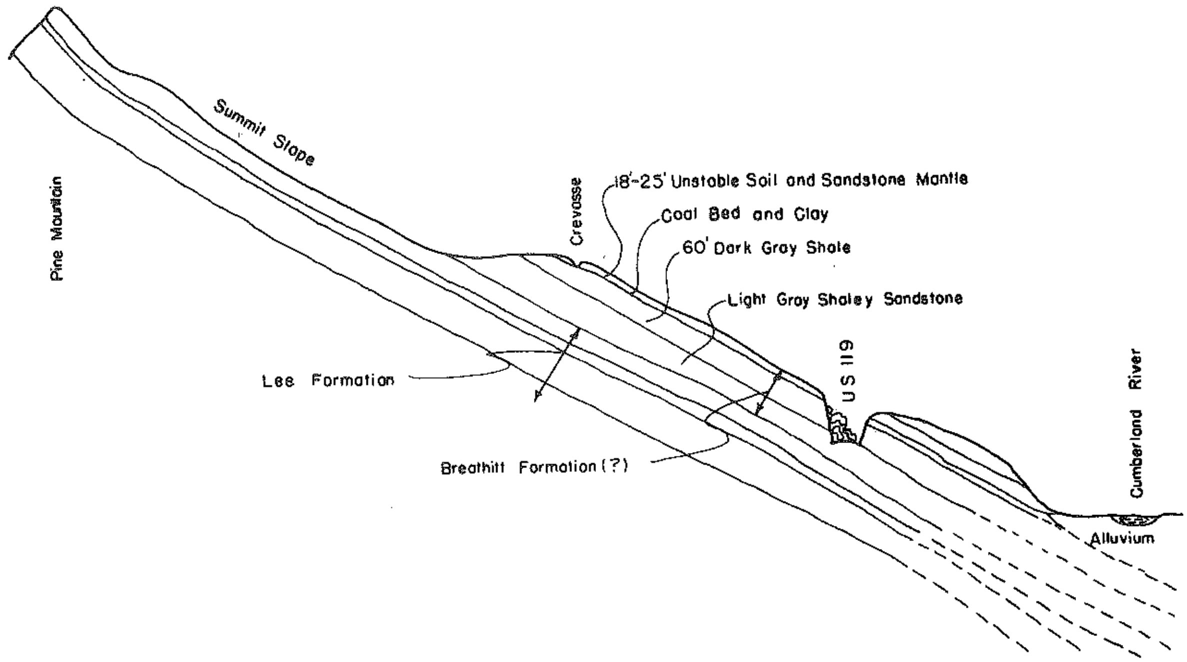


Figure 6-6. Typical Cross Section of Slide Area

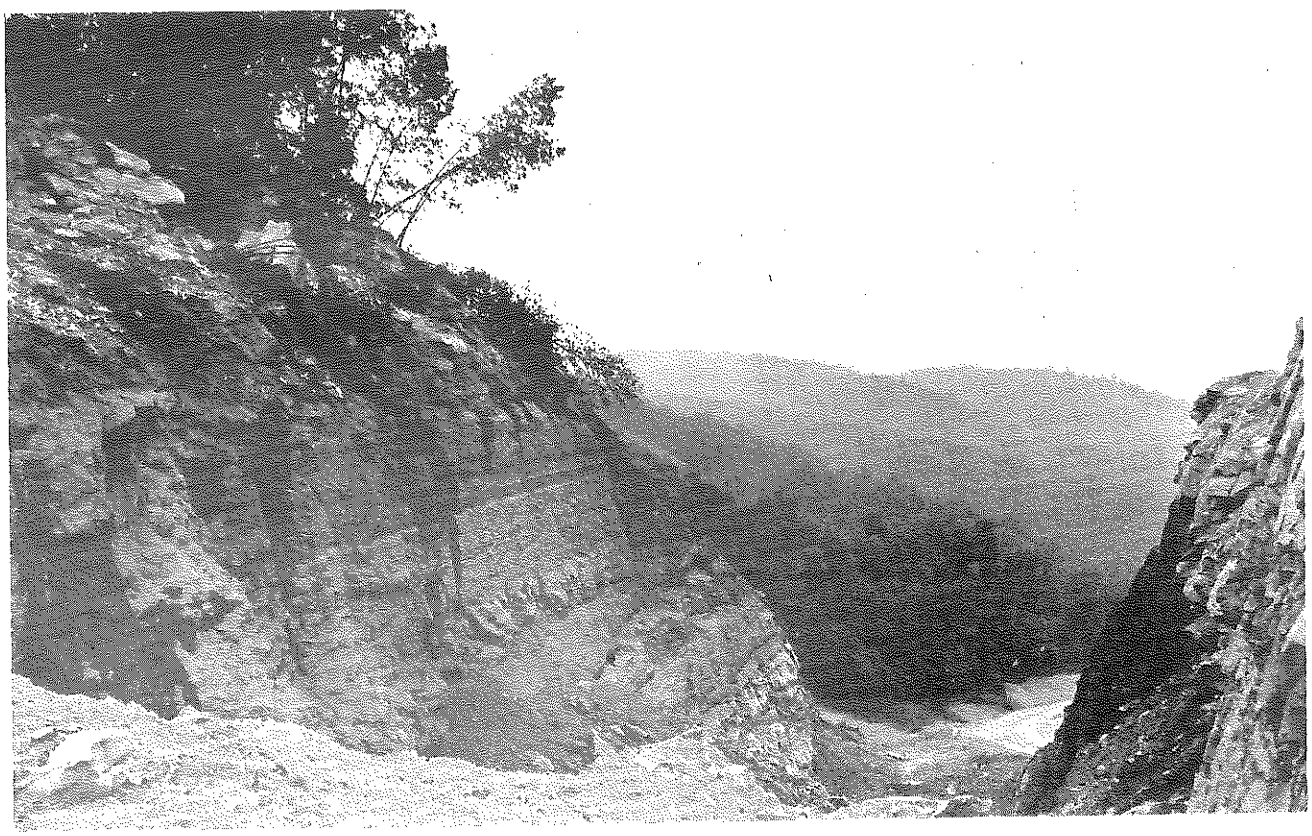


Figure 6-7. View of Slide Area Showing the Top Mantle Which is Presently Moving.

moving at the present is the upper 18 to 25 feet of sandstone. However, the dip of the beds is critical in this area and potential glide planes exist at all levels. Conceivably all of the shale laying above the base of the cut-slope could slide. Weathering and lubrication by water are likely to unbalance the static forces which now exist. It was suggested, therefore, that the entire mass of shale above a plane parallel to the bedding planes and intersecting the ditch line be considered untrustworthy as a backslope.

If the shale above a plane parallel to the bedding planes extending from the ditch line were removed, the resulting smooth surface could not be tolerated because rainwater runoff would be catastrophic. Diagonal flumes could be installed and the exposed shale could be benched and protected against weathering. However, the volumes of excavation and details of slope protection appear to be overwhelming. This suggests that a more positive measure would be a major relocation to avoid the type of situation that exists here and at numerous other locations on the west side of the Cumberland River at the foot of the Pine Mountain Thrust.

Another alternative would be to raise the grade as much as possible through the cut. Even so, all material above a plane parallel to the bedding plane from the higher ditch line would have to be removed. By raising the grade in the cut area, the quantity of material which must be removed from the backslope would be greatly reduced. This alternative, however, is merely a "stop gap" method. Tunnelling might have been feasible had foresight been sufficiently forewarning of the perils of open-cut excavation. The preferred solution, of course, is to avoid this type of geological situation.

RETAINING WALL ANALYSIS

US 23

Boyd County, SP 10-65-3L2, U 537(15)

RETAINING WALL ANALYSIS
US 23
Boyd County, SP 10-65-3L2, U 537(15)

At the request of the Divisions of Bridges and Materials, the Division of Research undertook to study the soil conditions at a site on US 23 in Cattletsburg where plans were being considered to construct a large retaining wall. Disturbed samples and split-spoon specimens were obtained by the Division of Materials and submitted to the Research Laboratory for evaluation.

Consideration was being given to the construction of two retaining structures in association with the improvement of US 23 through Cattletsburg. One retaining wall, approximately 500 feet in length, was to be constructed between Stations 25+50 and 30+50 and would be 30 to 40 feet in height. A second retaining wall, only about 15 feet in height, was being considered between Stations 13+00 and 15+00.

The sites of the proposed retaining walls are located at the foot of the slopes of Pennsylvanian deposits where they are overlapped by the alluvium deposits of the Ohio and Big Sandy Rivers (see Figures 7-1 and 7-2). The Princess No. 7 coal bed is known to outcrop or be located approximately at the contact between the Pennsylvanian deposits and the river alluvium material. The

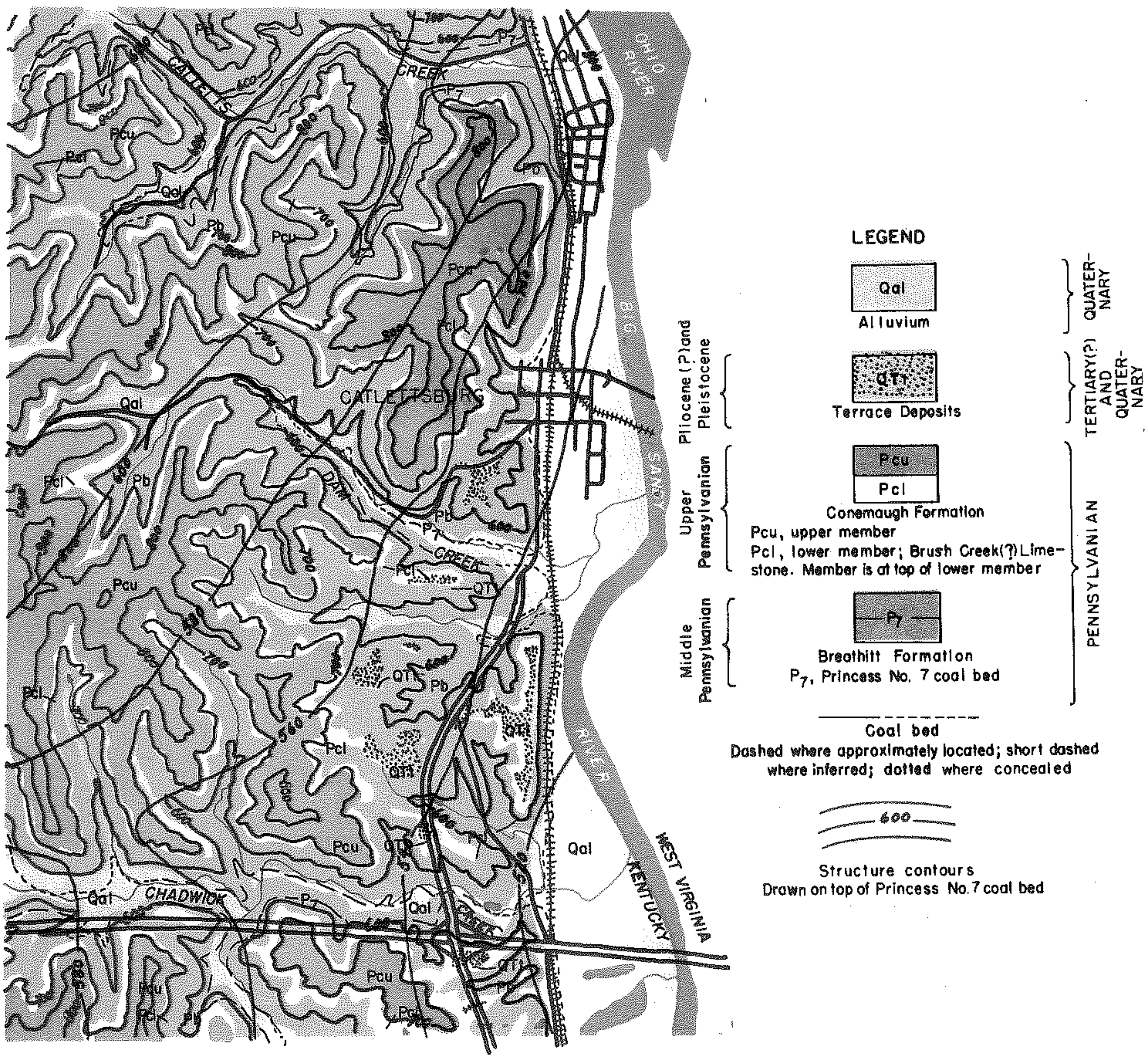
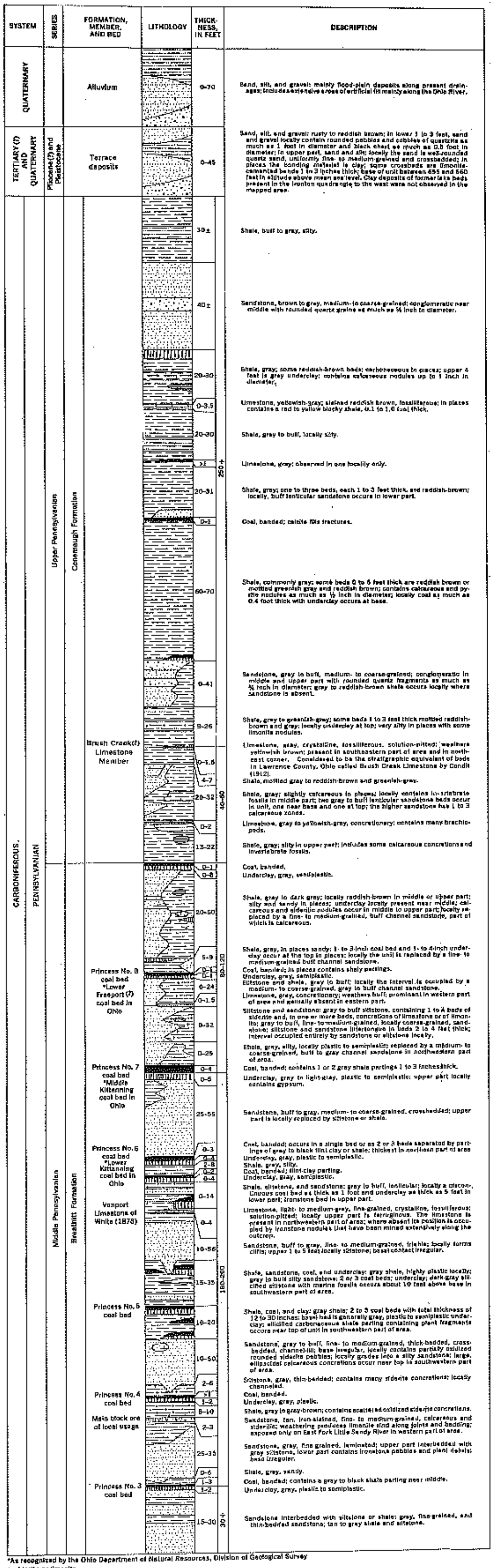


Figure 7-1. Geology of the Catlettsburg Area (from "Geology of the Ashland Quadrangle, Kentucky-Ohio and The Catlettsburg Quadrangle in Kentucky", E. Dobrovolny, J.A. Sharps, and J.C. Ferm, U.S. Geological Survey, 1963).



*As recognized by the Ohio Department of Natural Resources, Division of Geological Survey
+, siderite or limonite

Figure 7-2 Columnar Section for Boyd County, Kentucky (from "Geology of The Ashland Quadrangle, Kentucky-Ohio and The Catlettsburg Quadrangle in Kentucky", E. Dobrovolny, J.A. Sharps, and J.C. Ferm, U.S. Geological Survey, 1963).

Princess No. 7 coal bed is underlain by approximately five feet of underclay, which is plastic to semiplastic. Below that is 25 to 55 feet of sandstone, locally replaced in the upper portion by siltstone or shale. Above the Princess No. 7 coal bed are several feet of interbedded shales, siltstones and sandstones. The hilltop is capped with limestones, shales, and sandstones of the Conemaugh Formation of upper Pennsylvanian age. It is suspected that the material to be confined by the retaining structures consist of talus slope materials derived from the Conemaugh and Breathitt Formations above the river valley terrace. The material consists of a matrix of clayey soil with many sandstone and shale fragment inclusions. Numerous sandstone fragments range in size up to that of an automobile.

Because time was not available to obtain proper samples, the investigation was based upon disturbed samples obtained from two test pits in the area of the large retaining structure, approximately 40 feet left (west) of the centerline, and split-spoon specimens obtained 60 to 95 feet left of the centerline in the area of the large retaining structure and approximately 50 feet left of the centerline in the area of the small retaining structure.

Remolded specimens were prepared from the pit samples by an extrusion process in which the soil is mixed in a vacuum and forced by augers through a die of the desired size and shape. Mixing in a vacuum produced a high degree of uniformity and saturation. The soil was first pulverized and passed through a No. 4 sieve, water was added to the entire sample to bring the moisture content to about 20 percent and mixed for a few minutes by hand. The mixture was then run through the extrusion machine at least twice to insure uniform mixing before making specimens for testing. The bar of extruded soil was cut into specimens, approximately four inches long, which were immediately dipped in melted wax for protection until testing. When the desired number of specimens, at least six, were obtained, more water was added to the remaining soil and the mixing and extrusion process was repeated. By this procedure, specimens having three different moisture contents ranging between 20 and 30 percent were made for each of the pit samples. The split-spoon specimens were treated as "undisturbed" and were prepared for testing in a moist room to minimize evaporation of the nature moisture.

The specimens obtained were subjected to a series of consolidated-undrained triaxial tests and unconfined compressive tests. A summary of the triaxial test results is contained in Tables 7-1 and 7-2. Mohr's

TABLE 7-1

SUMMARY OF TRIAXIAL TEST DATA FOR REMOLDED SPECIMENS

DESCRIPTION OF MATERIAL	LOCATION	CONFINING PRESSURE (PSI)	DRY UNIT WEIGHT (Lbs/Cu Ft)	MOISTURE CONTENT (Percent) Before Test	After Test	Average	COHESION (PSI)	ANGLE OF FRICTION (Degrees)
Blue and Brown Clay	40' Lt, Sta. 27+00	0	113.9	18.6	19.1	18.8	27.0	8.0
		15	114.2	18.5	19.1	18.8		
		30	-	17.6	-	17.6		
		45	114.2	18.0	18.6	18.3		
		0	-	22.5	-	22.5	12.5	7.0
		15	107.6	28.7	23.0	25.7		
		30	108.5	21.1	21.5	21.3		
		45	106.8	21.4	21.9	21.6		
		45*	106.2	22.3	22.0	22.2		
		0	95.0	27.9	28.8	28.3	5.5	2.5
		15	95.4	28.4	28.4	28.4		
		30	95.8	27.9	27.6	27.8		
		45	95.4	28.7	28.1	28.4		
Yellow and Brown Clay	40' Lt, Sta. 28+00	0	112.8	18.4	18.3	18.4	22.5	5.0
		15	111.9	18.8	20.5	19.6		
		30	113.2	18.3	19.7	19.0		
		45	112.6	18.5	19.4	18.9		
		0	102.8	24.7	24.9	24.8	9.0	5.0
		15	102.5	24.9	25.7	25.3		
		30	102.2	24.0	24.4	24.2		
		45	102.7	24.5	24.6	24.5		
		0	99.0	27.2	27.2	27.2	6.5	6.0
		15	98.9	26.8	27.3	27.0		
		30	98.7	27.2	26.3	26.8		
		45	97.9	27.3	26.5	26.9		

*Drained Test

TABLE 7-2

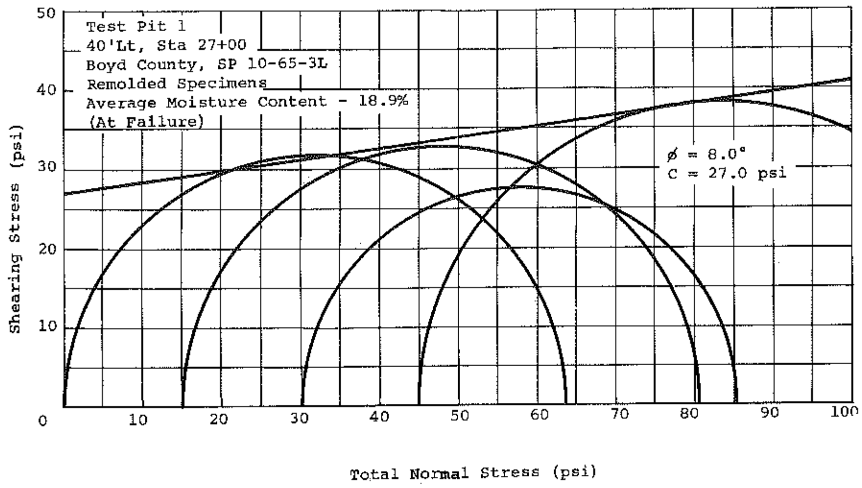
SUMMARY OF TRIAXIAL TEST DATA FOR SPLIT SPOON SPECIMENS

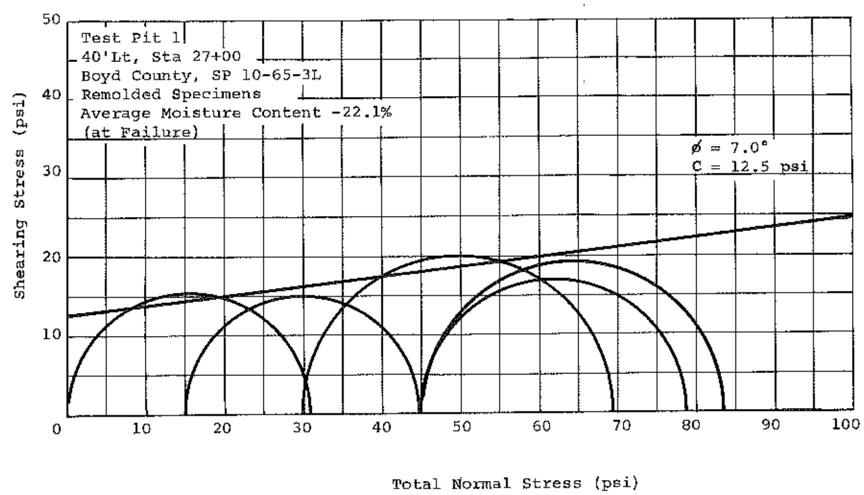
DESCRIPTION OF MATERIAL	LOCATION	SAMPLE DEPTH (Ft)	CONFINING PRESSURE (PSI)	DRY UNIT WEIGHT (Lbs/Cu Ft)	MOISTURE CONTENT (Percent)			COHESION (PSI)	ANGLE OF FRICTION (Degrees)
					Before Test	After Test	Average		
Yellow and Blue Clay, Shale	50'Lt, Sta.13+50	5	0	118.2	14.1	13.4	13.7	14.0	12.5
		10	20	118.5	13.4	16.4	14.9		
		10	40	123.1	13.1	15.4	14.3		
Yellow and Blue Clay, Shale	95'Lt, Sta.27+00	10	0	110.6	19.9	18.4	19.1	6.5	14.5
		10	20	112.9	19.9	17.0	18.4		
Yellow and Blue Clay, Shale	95'Lt, Sta.27+00	20	10	116.3	-	17.0	17.0	10.3	14.0
		20	40	112.0	-	18.5	18.5		
Yellow Clay, Shale, Sandstone	48'Lt, Sta.14+38	25	10	120.8	12.6	12.0	12.3	6.5	18.0
		20	25	121.4	12.5	15.0	13.7		
	60'Lt, Sta.30+00	25	40	113.0	15.7	19.1	17.4		

circles and failure envelopes are shown in Figures 7-3 through 7-5.

The friction angles obtained for the remolded specimens varied between 5° and 8°, with the exception of the wet samples for Pit 1, which had an angle of friction of 25°. These specimens were very wet and only 30 minutes were allowed for consolidation prior to shearing. The wet specimens for Pit 2 were also very wet, but two hours were allowed for consolidation, and an angle of friction of 6° was obtained. Hence, it is felt that the angle of friction for the wet series of Pit 1 would have been higher if more time had been allowed for consolidation. The cohesion intercept varied between 5.5 pounds per square inch and 27 pounds per square inch and was noted to be particularly sensitive to moisture content.

The friction angles obtained from triaxial tests on the split-spoon samples varied between 12° and 18°. This large difference in strength between the remolded and split-barrel specimens is evidently due, in part, to a very large difference in structure caused by the thorough mixing under vacuum and extruding of the remolded specimens. Also some gain in strength of the split-barrel specimens over the in situ conditions could be





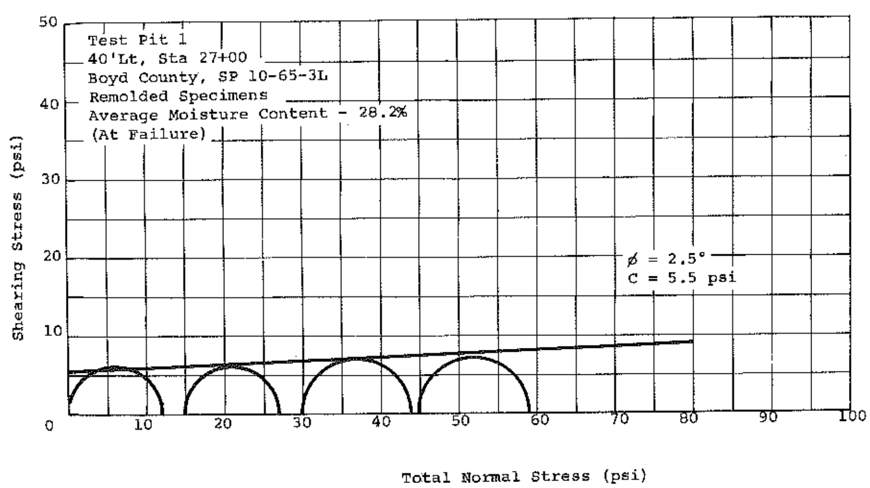
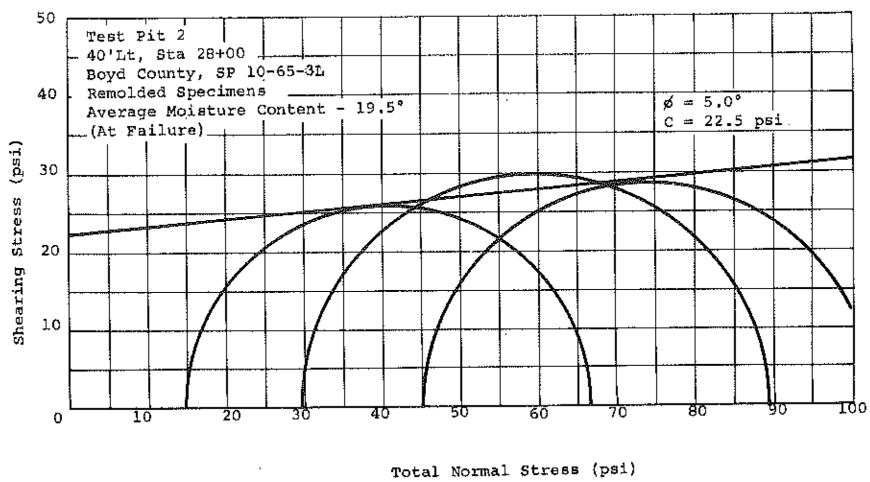
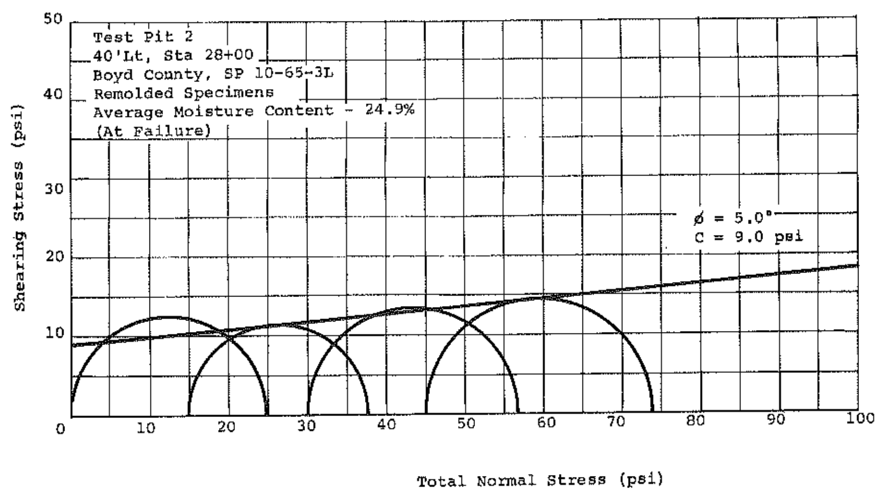


Figure 7-3. Mohr's Circles and Failure Envelopes.





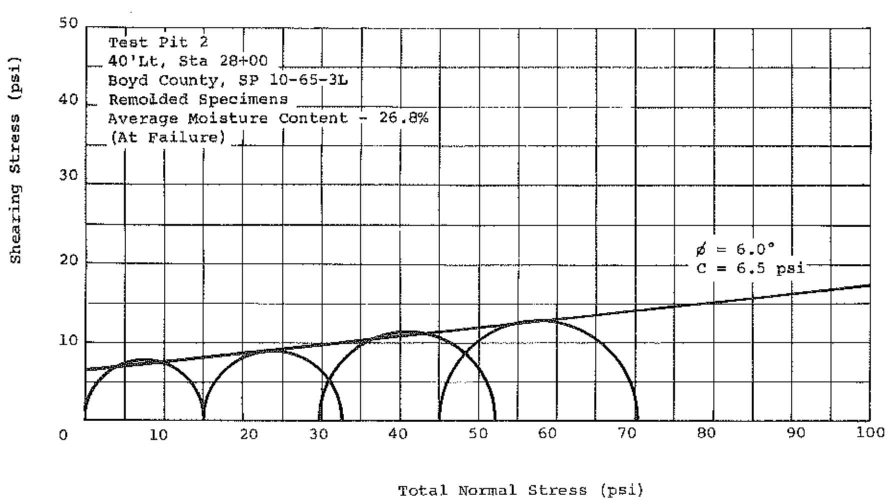


Figure 7-4 Mohr's Circles and Failure Envelopes.

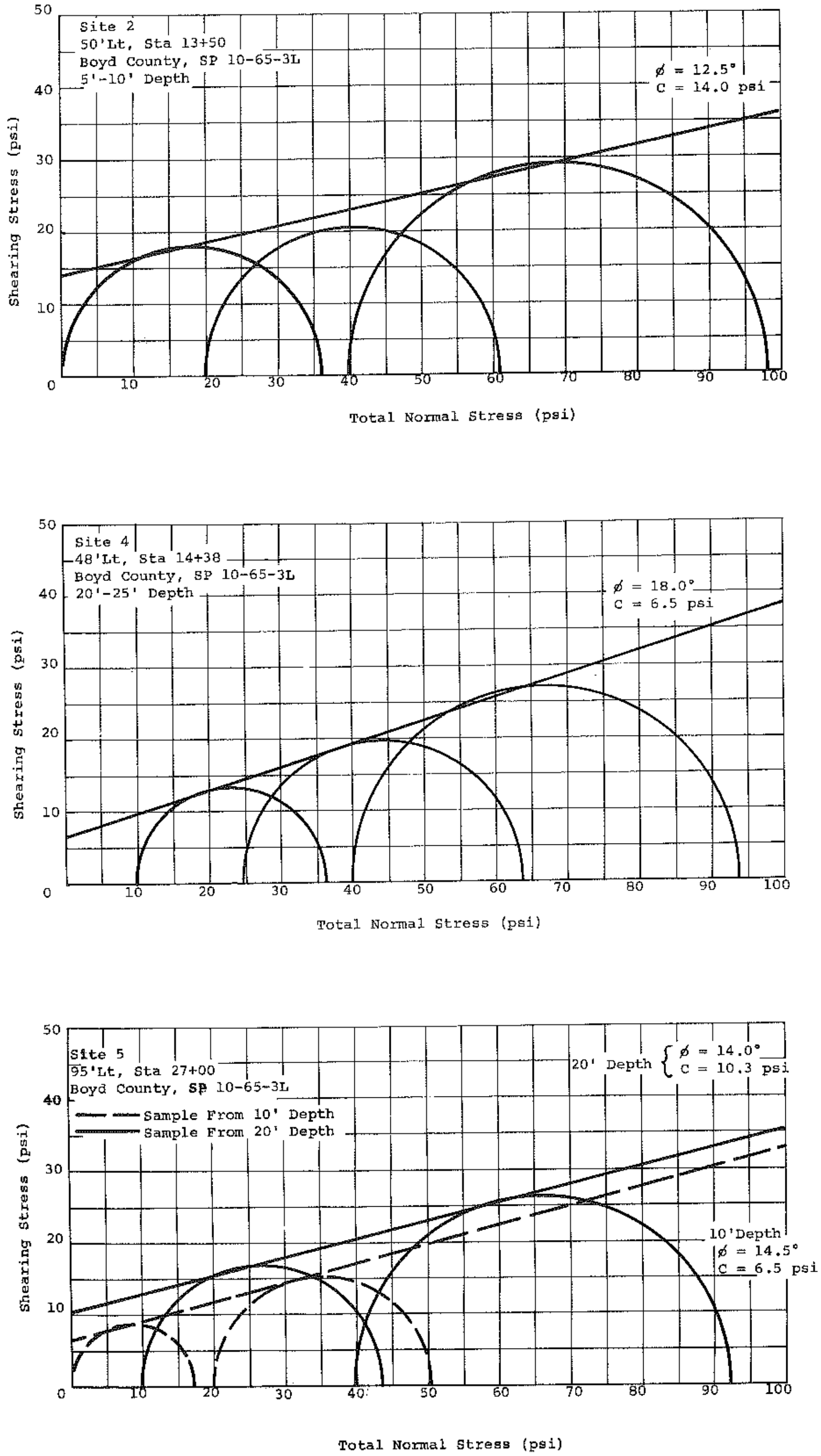


Figure 7-5. Mohr's Circles and Failure Envelopes.

expected due to compaction in the sampler when it was driven. Thus, it can be assumed that test results on the remolded samples may underestimate the in situ strength while tests on split-barrel specimens may over-estimate the strength.

Bishop and Bjerrum* have discussed the errors likely to arise in applying to the field situation the relationship between undrained strength and consolidation pressure obtained in the laboratory from the consolidated-undrained test. It is suggested that the value of the angle of friction, ϕ_{cu}, obtained from the conventional, consolidated-undrained test on saturated soil is applicable only to the passive earth pressure case. If the lateral pressure increases during the undrained loading (as in a foundation problem) the factor of safety obtained using the value of ϕ_{cu} from the conventional, consolidated-undrained test will be over-estimated. If the confining pressure decreases (as in the excavation of cuts) the use of the conventional ϕ_{cu} may lead to an underestimation of the factor of safety. It is recommended, therfore, that a value for the angle of friction ϕ_{cu} of 12° and a value of cohesion of 0 to 6 pounds per square inch may be appropriate for this situation on US 23.

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*A. W. Bishop and L. Bjerrum, "The Relevance of the Triaxial Test to the Solution of Stability Problems", Proceedings, Research Conference on Shear Strength of Cohesive Soils, ASCE, 1960.

The results of the unconfined compressive tests are summarized in Table 7-3. An average of 2.3 tons per square foot was obtained for the unconfined compressive strength. A value of the ultimate bearing capacity of a continuous footing, assuming a ø-equal-zero analysis, can be computed from the equation $q_d = 2.85\ q_u$, where q_u equals the unconfined compressive strength. This gives an average ultimate bearing capacity of 6.7 tons per square foot. Under normal conditions, a factor safety of three for footings on clay soils is recommended. The allowable bearing capacity would then be approximately 2.3 tons per square foot.

Table 7-4 summarizes the lateral earth pressures calculated by different methods and based upon the triaxial tests results summarized in Tables 7-1 and 7-2. The calculations were made for a vertical wall 30 feet high. Adhesion between the backfill and wall was neglected. A backfill slope of 2:1 was assumed for the methods (Items 1, 2, and 3) which allow the backslope to be taken into account. A unit weight of 130 pounds per cubic foot was used in all calculations.

The value of the lateral pressures range from 10,500 to 58,500 pounds per lineal foot of wall. The lower values represent fully active earth pressures while the high

TABLE 7-3

SUMMARY OF UNCONFINED COMPRESSIVE STRENGTH DATA
FOR SPLIT SPOON SPECIMENS

LOCATION	DESCRIPTION OF MATERIAL	DEPTH OF SAMPLE (Ft)	MOISTURE CONTENT (Percent)	UNCONFINED COMPRESSIVE STRENGTH (PSI)	UNCONFINED COMPRESSIVE STRENGTH (TSF)	FAILURE STRAIN (Percent)
46' Lt, Sta. 13+00	Yellow Clay, Shale	15	17.0	32.9	2.37	8.3
50' Lt, Sta. 13+50	Yellow Silty Clay	25	24.2	51.5	3.70	6.6
57' Lt, Sta. 14+00	Yellow and Blue Clay, Shale	20	16.9	23.4	1.69	6.7
48' Lt, Sta. 14+38	Yellow Clay, Shale	15	18.0	31.8	2.29	4.2
90' Lt, Sta. 28+50	Yellow and Blue Clay, Shale	45	16.4	21.3	1.54	13.5
90' Lt, Sta. 28+50	Yellow and Blue Clay, Shale	40	-	30.5	2.20	7.5
82' Lt, Sta. 29+00	Yellow and Blue Clay, Shale	40	-	41.4	2.98	9.2
60' Lt, Sta. 30+00	Yellow Silt and Sand	35	-	17.6	1.26	3.3
60' Lt, Sta. 30+00	Yellow and Blue Clay, Shale	30	-	43.2	3.11	9.5

TABLE 7-4

SUMMARY OF LATERAL EARTH PRESSURE CALCULATIONS

	METHOD OF ANALYSIS	REFERENCES	ASSUMPTIONS[1]	FORMULA[2]	LATERAL PRESSURE (Lbs/Lineal Ft of Wall)
1	Coulomb	Taylor[3], p 526	$\phi=6°$, c=800psf	Graphical	23,000
2	Coulomb	Taylor[3], p 526	$\phi=12°$, c=0	Graphical	45,000
3	Coulomb	Taylor[3], p 526	$\phi=12°$, c≈800psf	Graphical	14,200
4	ϕ=0 Analysis	Taylor[3], p 525 Leonards[4], p 459	$\phi=0$, q_u=1600psf	$P_a=(\gamma H^2/2)-q_u H$	10,500
5	Pressure at Rest		$K_o=1.0$	$P_a=K_o\gamma H^2/2$	58,500
6	Pressure at Rest	Leonards[4], p 46	$K_o=0.5$	$P_a=K_o\gamma H^2/2$	29,250
7	Trapezoidal Distribution	Leonards[4], p 459	See Figure 7-12		37,000

[1] γ = 130 Lbs/Cu Ft = Unit Weight
[2] H = 30 Ft = Height of Wall
q_u = Unconfined Compressive Strength
K_o = Coefficient of Lateral Earth Pressure
P_a = Lateral Earth Pressure
[3] D. W. Taylor, Fundamentals of Soil Mechanics, Wiley, 1962
[4] G. A. Leonards, Foundation Engineering, McGraw-Hill, 1962

value represents hydrostatic pressure. The intermediate values are for lateral earth pressures at rest calculated by various methods.

The first three entries in the table were obtained by graphically completing the force polygon for various trial failure wedges using Coulomb's method. The values obtained are active earth pressures and are not recommended for design in cohesive soils where large deflections cannot be tolerated. Item 4 is an active earth pressure based on a ϕ-equal-zero analysis and using the formula $P_a=(\gamma H^2/2)-q_u H$. Items 5 and 6 are lateral pressures for the at-rest condition. Item 5 was computed on the basis of lateral pressure being equal to the vertical pressure. Item 6 was computed on the basis of a coefficient of lateral earth pressure of 0.5. The value entered for Item 7 is based on a lateral pressure distribution recommended by Peck and illustrated in Figure 7-6.

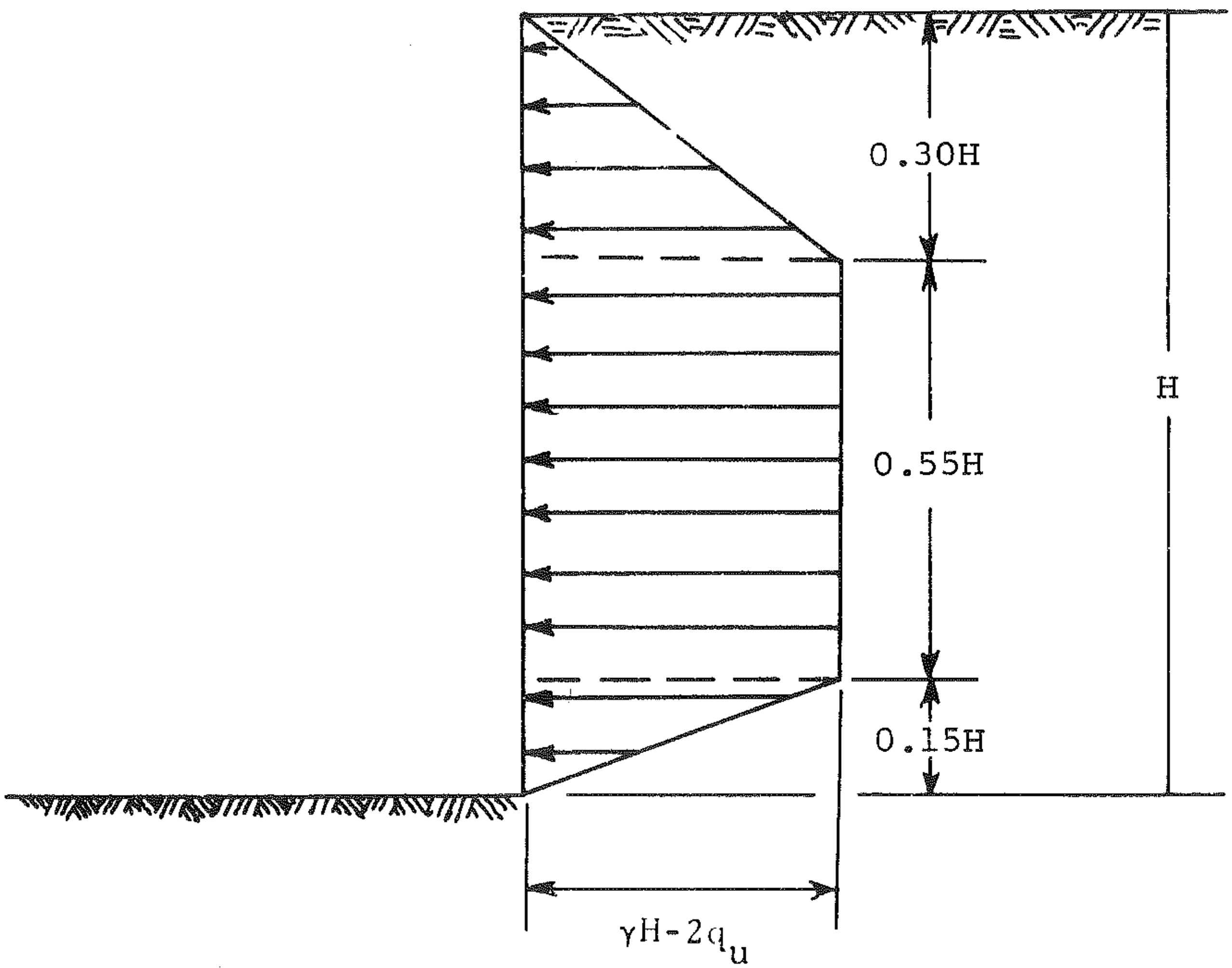


Figure 7-6. Trapezoidal Distribution of Lateral Earth Pressures.

SLOPE STABILITY ANALYSIS

I 64

Boyd County, SF 10-115-25C1

SLOPE STABILITY ANALYSIS
I 64
Boyd County, SF 10-115-25Cl

In June, 1966, the Division of Research undertook the investigation of a landslide located 0.7 miles west of the junction of US 23 on I 64 in Boyd County. The investigation was made at the request of the Office of Projects Mangement.

The side-hill embankment in the westbound lanes of I 64 near Station 440 was observed to have slipped in July, 1962, while the grade-and-drain contractor was still on the job. Provisions were made to remove the embankment material, bench the original ground, and to replace the embankment. A string of perforated pipe was also placed about five feet below the south ditch between Stations 429+02 and 443+20. After this work had been accomplished, the embankment again moved in late spring, 1963, in the vicinity of Station 445. The contractor was then required to rework and replace the embankment in this area. In the spring of 1965, after the road had been opened to traffic, the embankment began to slip downhill in the same location. Movement has continued up to the present time. Maintenance endeavors at the site have involved patching of the pavement in order to maintain two lanes of westbound traffic.

According to the cross sections for the grade and drain project, it is noted that a coal seam lies at or just below the finished grade in the area between Stations 439+00 and 442+00. The elevation of this coal seam is approximately 600 feet and is therefore probably the Princess No. 8 coal bed. The Princess No. 8 is a banded coal seam approximately 1 to 2 feet thick and in places contains shaly partings. Above the coal is a layer of gray shale approximately ten feet thick. In places this shale layer is relatively sandy. At the top of this ten-foot shale layer is a 1- to 3-inch coal bed with a 1- to 4-inch underclay. Above this thin coal layer is 20 to 60 feet of gray shale which is silty to sandy in many places. The shale, particularly in the upper protions, is reddish brown. The Princess No. 8 coal bed and the shale which overlie it are members of the Breathitt Formation of Middle Pennsylvanian age. Above the Breathitt Formation lies the Brush Creek Limestone, a member of the Conemaugh Formation of Upper Pennsylvanian age.

Immediately below the Princess No. 8 coal bed is an underclay approximately one foot thick. This clay is semiplastic to plastic and is relatively impermeable. Below the underclay there are several feet of siltstone and shale and, in places, channel sandstone. This type of material should present no particular foundation problem (see Figures 8-1 and 8-2).

The physical properties of the bedrocks in this area have a significant influence on landslides, which are present throughout the Ashland-Cattletsburg area. Most of the landslides occur along the underclays of the Breathitt Formation, but a few occur in the shales and shaly siltstones of the Conemaugh Formation where the hillsides are extremely steep. Landslides are so prevalent in and near the outcrops of the Princess coal beds 6 and 7 and to a lesser extent No. 8 that these beds can be located by failures of roads that cross them.

Groundwater is the most significant factor in causing these landslides. The water percolates downward through the sandstones and coal until it reaches the less permeable underclays. The water then migrates along the top of the clays until it outcrops, where its presence may be indicated by intermittent springs or seepage zones. When the soil material near the surface of the ground becomes saturated with this water, it may flow down the slope as a structureless and incompetent mass.

Many of these landslides can be controlled or prevented by removing the underclay and replacing it with a material with a higher permeability and bearing capacity.

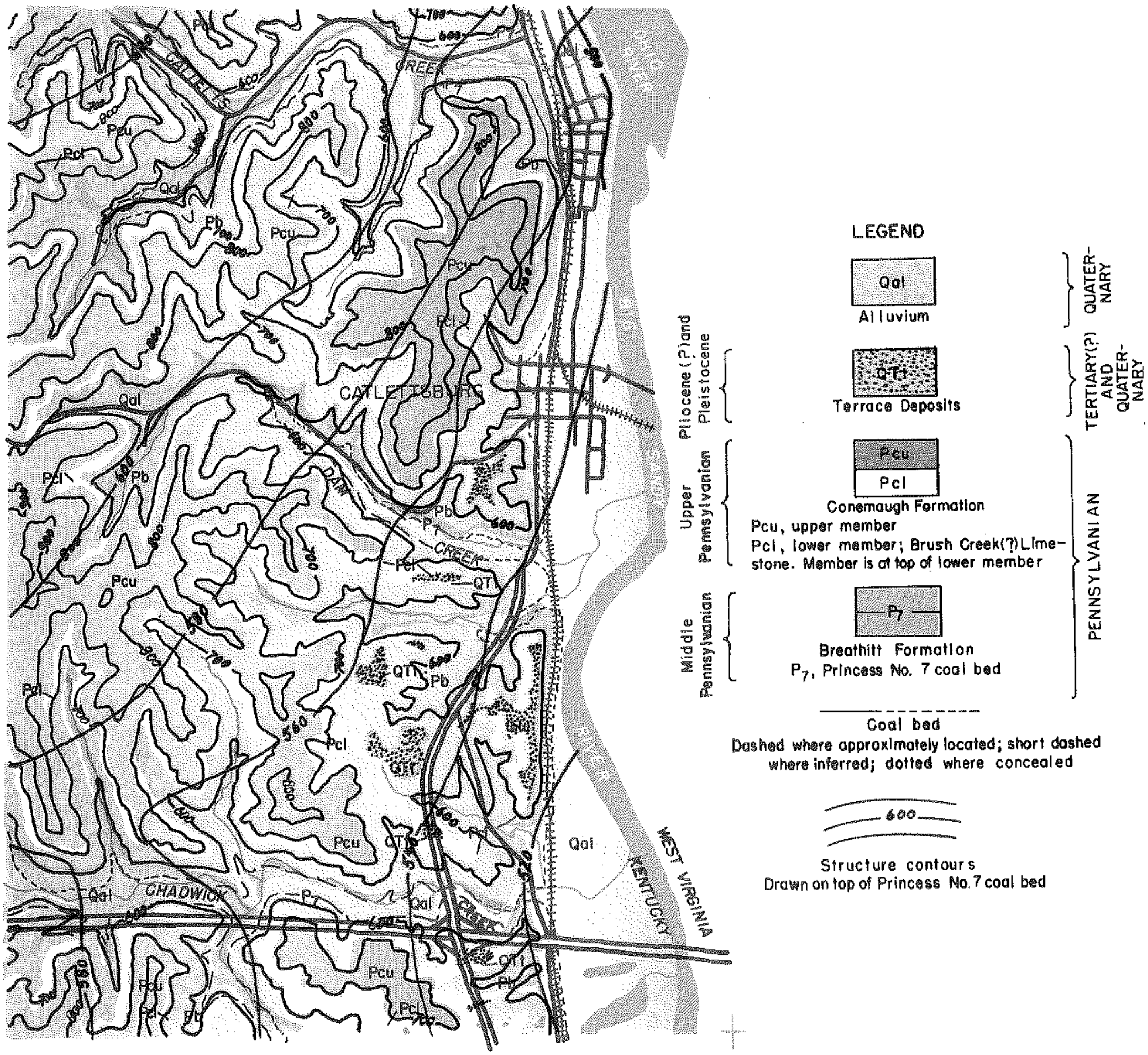
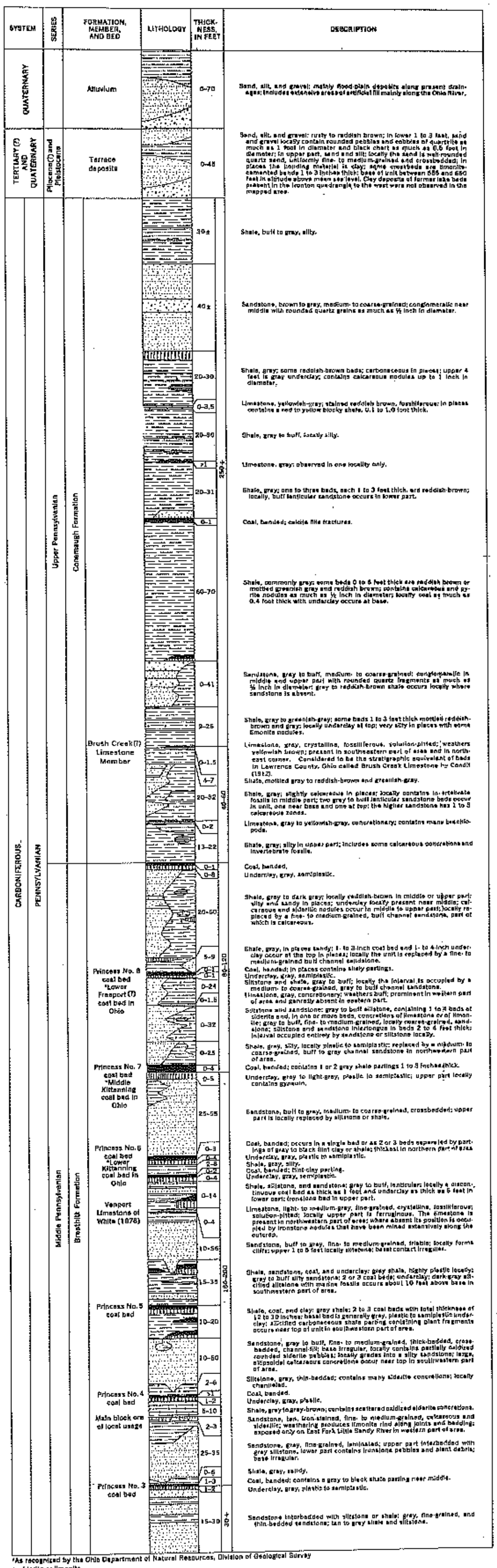


Figure 8-1. Geology of the Catlettsburg Area (from "Geology of the Ashland Quadrangle, Kentucky-Ohio and the Catlettsburg Quadrangle in Kentucky", E. Dobrovolny, J.A. Sharps, and J.C. Ferm, U.S. Geological Survey, 1963).



*As recognized by the Ohio Department of Natural Resources, Division of Geological Survey
+, siderite or limonite

Figure 8-2. Columnar Section for Boyd County, Kentucky (from "Geology of The Ashland Quadrangle, Kentucky-Ohio and The Catlettsburg Quadrangle in Kentucky", E. Dobrovolny, J.A. Sharps, and J.C. Ferm, U.S. Geological Survey, 1963).

Groundwater which may be moving along the interface between the clay and the coal beds could be intercepted by drains which lead the water away from the troublesome areas.

It is suggested that the principle contributing factor to the slide in the vicinity of Station 440+00 is the seepage of water along the underclay associated with the Princess No. 8 coal bed. Water apparently is seeping more or less horizontally along this underclay until it intercepts the side-hill embankment in the troublesome zone. The seepage water appears to be coming from a southwestwardly direction across the roadway rather than from directly across the roadway to the south. This embankment material has become saturated and, therefore, has lost much of its strength.

To correct the slide, it is suggested that the embankment material be removed to firm rock. A layer of free-draining granular material should then be placed over the foundation before the embankment material is replaced and compacted to the desired template (see Figure 8-3). In addition, the installation of transverse drains to the west of the slide areas may intercept seepage waters and keep it out of the slide area.

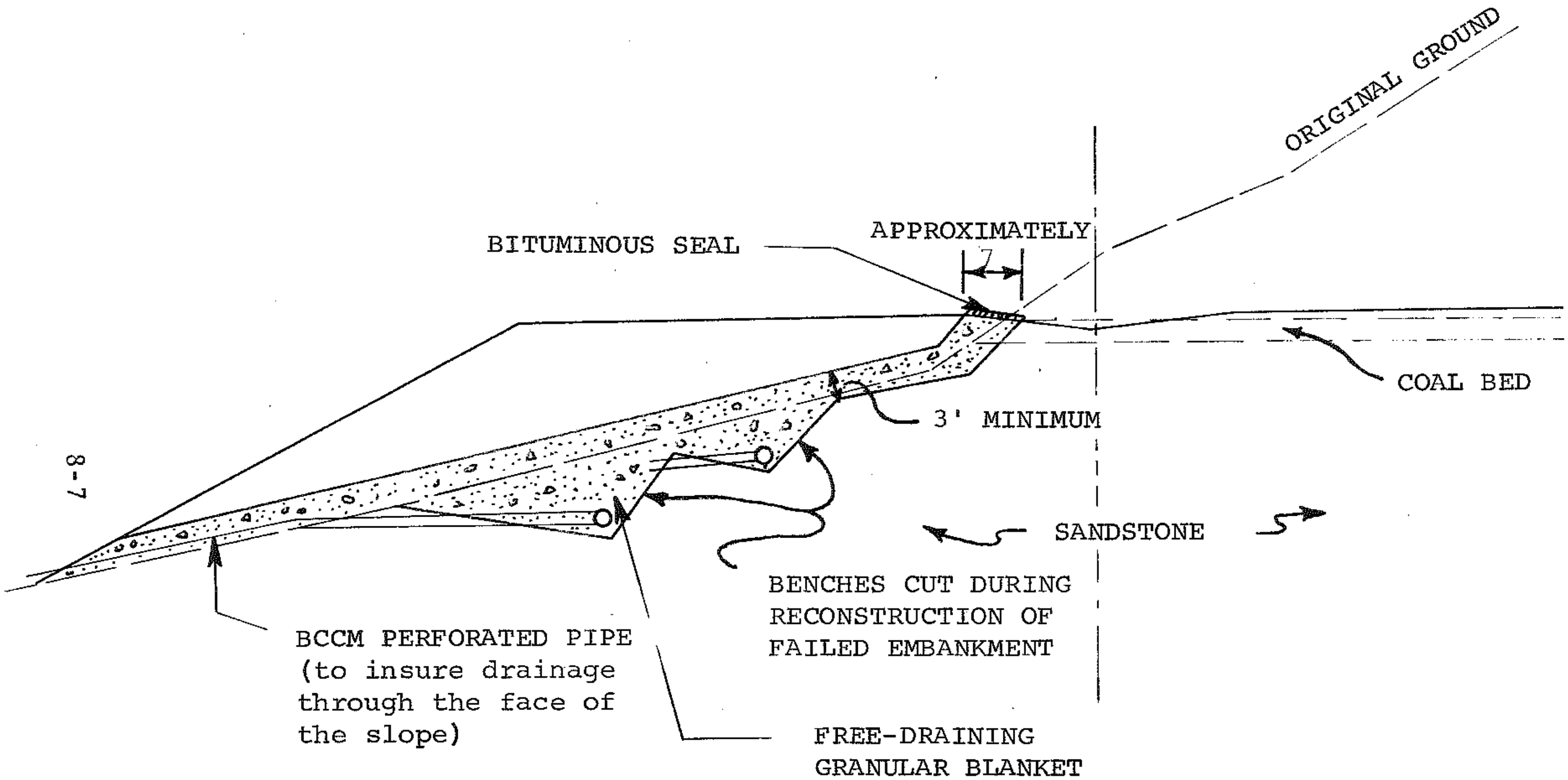


Figure 8-3. Proposed Corrective Measures

SETTLEMENT AND FOUNDATION STABILITY ANALYSIS

I 64

Bath County, SP 6-404-5G1, I 64-6(6)117

SETTLEMENT AND FOUNDATION STABILITY ANALYSIS*
I 64
Bath County, SP 6-404-5Gl, I 64-6(6)117

Early in 1966, during the construction of a large embankment between Stations 1738+00 and 1745+00 on I 64 in Bath County, a serious slide occurred involving large quantities of the embankment material. After a visit to the site and a review of the subsurface information available, it was assessed that the slide occurred as a result of a bearing-capacity failure of the foundation material.

Although no geological reconnaissance was made in connection with the field borings at the site, it appears from published information that the borings penetrated the upper reaches of the Garrard Siltstone (Ordivician) and that the silt layer overlying the bedrock is a weathered layer of the siltstone. The elevation of the top of the Garrard formation is approximately 700 feet. Borings reached an elevation of 670 feet before encountering bedrock. From the 700-foot elevation upward, the rock is the

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*Scott, G. D. and Deen, R. C. "Proposed Remedial Design for Unstable Highway Embankment Foundation, I-64-6(6)117, Bath County" Division of Research, Kentucky Department of Highways, April, 1966.

Maysville and Richmond Limestones. Both are notably rubbly and shaly. These strata are probably the parent source of the blue clay overlying the silt. The clays are extremely calcareous or dolomitic, and the top layer of silt is quite oily. The valley fill is relatively flat.

The embankment is partly a side-hill type, but the slope of the original ground is quite gentle along the affected portion. From Station 1738 to Station 1743 the slope is downward from right to left on approximately 3:1 to 3.5:1 from a point to the right of the embankment to a point beneath the westbound traffic lane; from there to the toe, the slope is about 10:1 to 12:1. Beyond the toe is the level flood plain of Slate Creek. Between Stations 1743 and 1745 there is a transition zone where the side-hill slope of the original ground increases to approximately 2.5:1 and the embankment height decreases from 45 feet to 18 feet at the left shoulder line. At Station 1745 the major part of the section is in a cut area.

A view of the embankment after failure is shown in Figure 9-1. Figure 9-2 illustrates the large upheaval at the toe which, about a month after the slide occurred, was as high as 20 feet. A typical cross section of the central part of the embankment is shown in Figure 9-3.

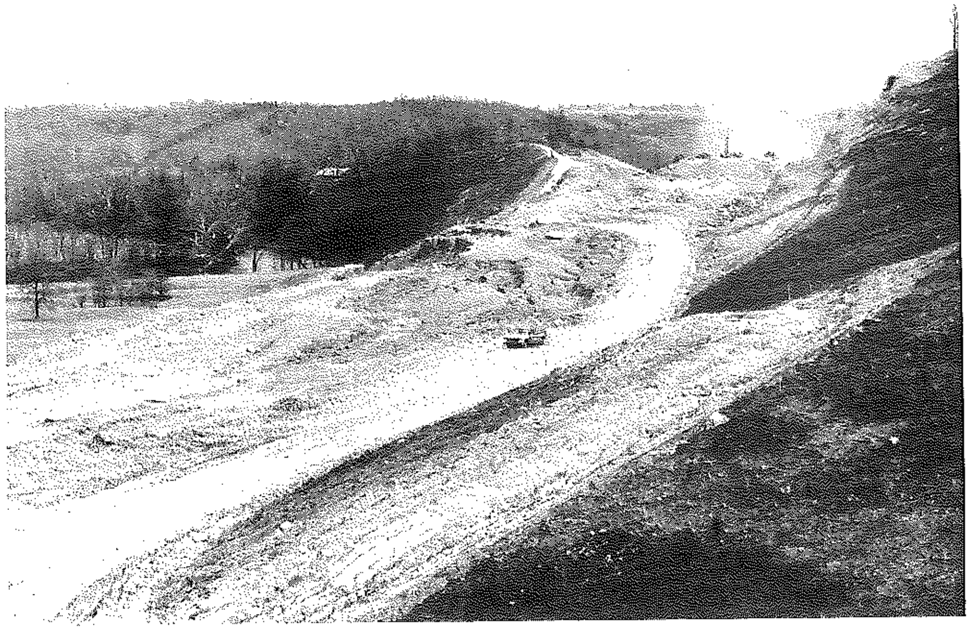


Figure 9-1. A General View of Failed Embankment on I 64 in Bath County.

Figure 9-2. View of Upheaval at Toe of Failed Embankment.

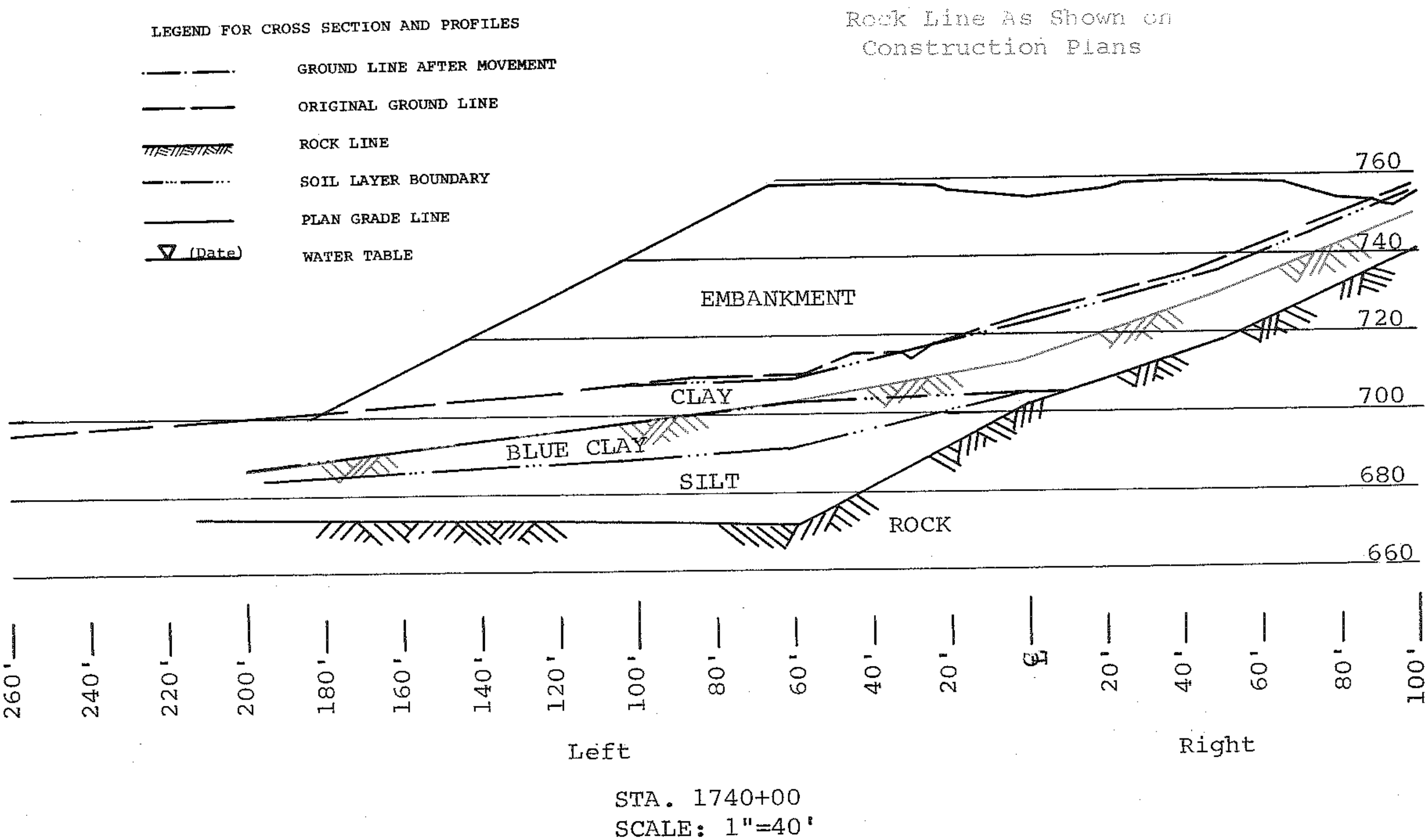


Figure 9-3. Typical Section Showing General Subsurface Conditions.

The subsurface exploration for design purposes involved borings made only along the centerline of the project at rather infrequent intervals. An erroneous interpretation of this rather limited data indicated firm rock at relatively shallow depths, as shown in Figure 9-3. Soundings made by the Division of Materials subsequent to failure define a firm yellow clay layer, a blue clay layer, and a wet silty clay layer of variable thicknesses ranging from about 10 to 40 feet in thickness as shown in Figure 9-3 and also in Figures 9-4 through 9-10. The depth to bedrock, according to the later soundings, increases from an average of 20 feet at the centerline to approximately 35 feet at the left shoulder and is reasonably uniform in depth to a point beyond the toe of the embankment.

The Materials Division's soundings were dry auger borings. Identification of layer boundaries and visual classifications of the various soils were made from the cuttings brought to the surface by the auger. Samples of the cuttings were obtained for moisture content and classification tests. The boring records showed that the lower portion of the soil profile was very wet and soft--indicating a soil having low shearing strengths. This along with other considerations, such as the relatively flat slopes of the original ground beneath the high part of the fill

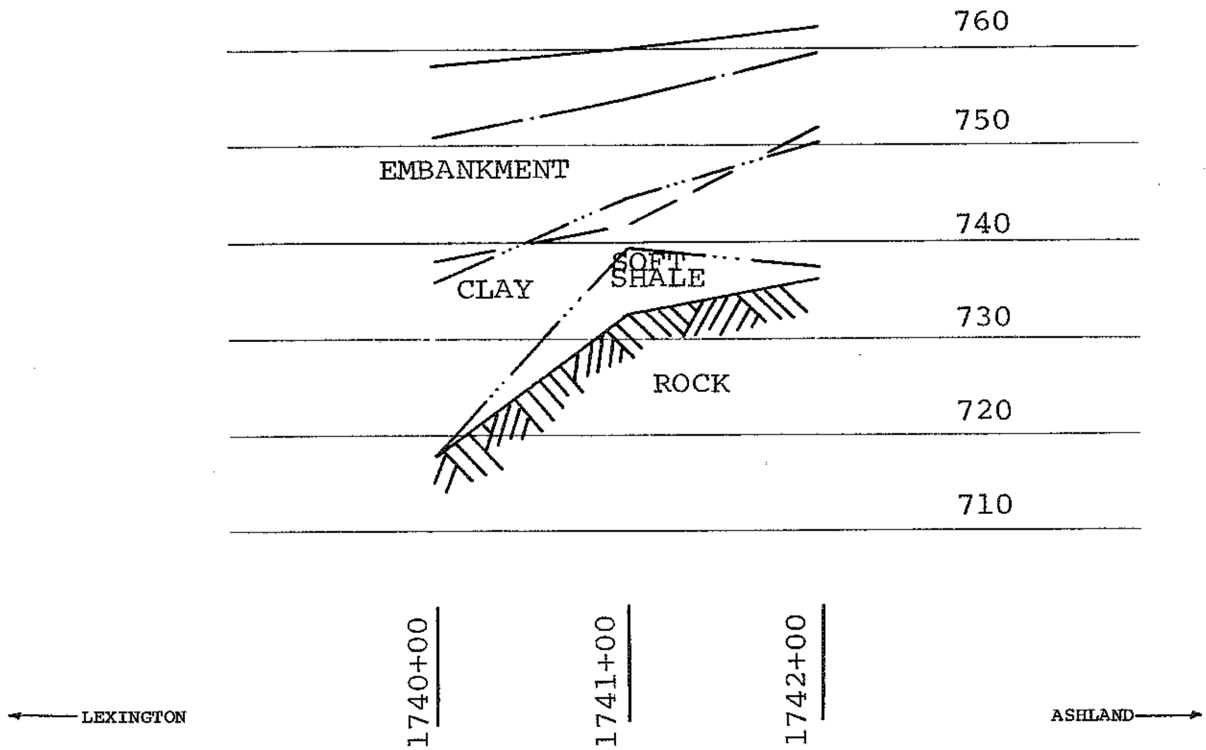


Figure 9-4. Soil Profile, Approximately 52 Feet Right of Centerline.

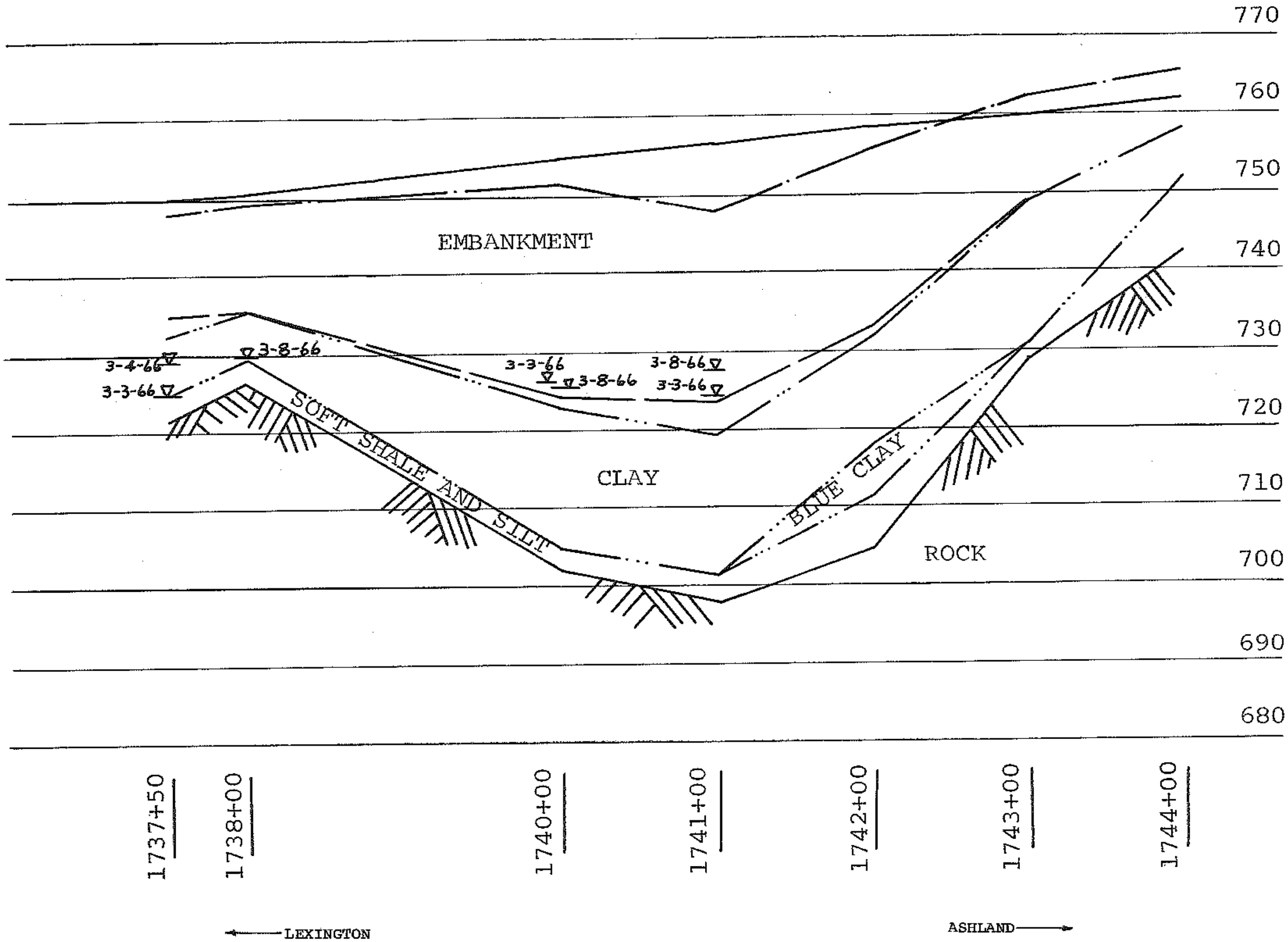


Figure 9-5. Soil Profile, Approximately 2 Feet Right of Centerline.

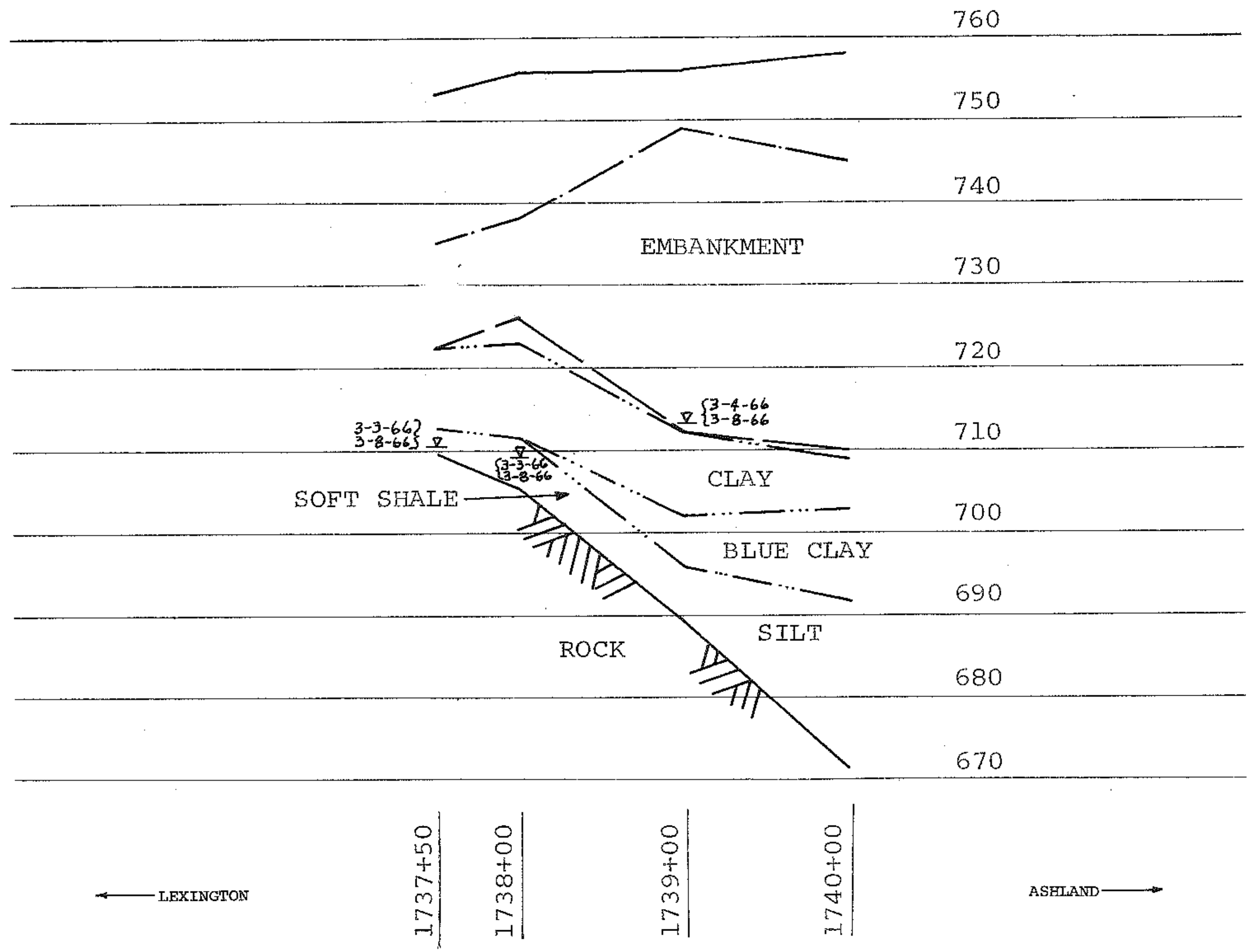


Figure 9-6. Soil Profile, Approximately 59 Feet Left of Centerline.

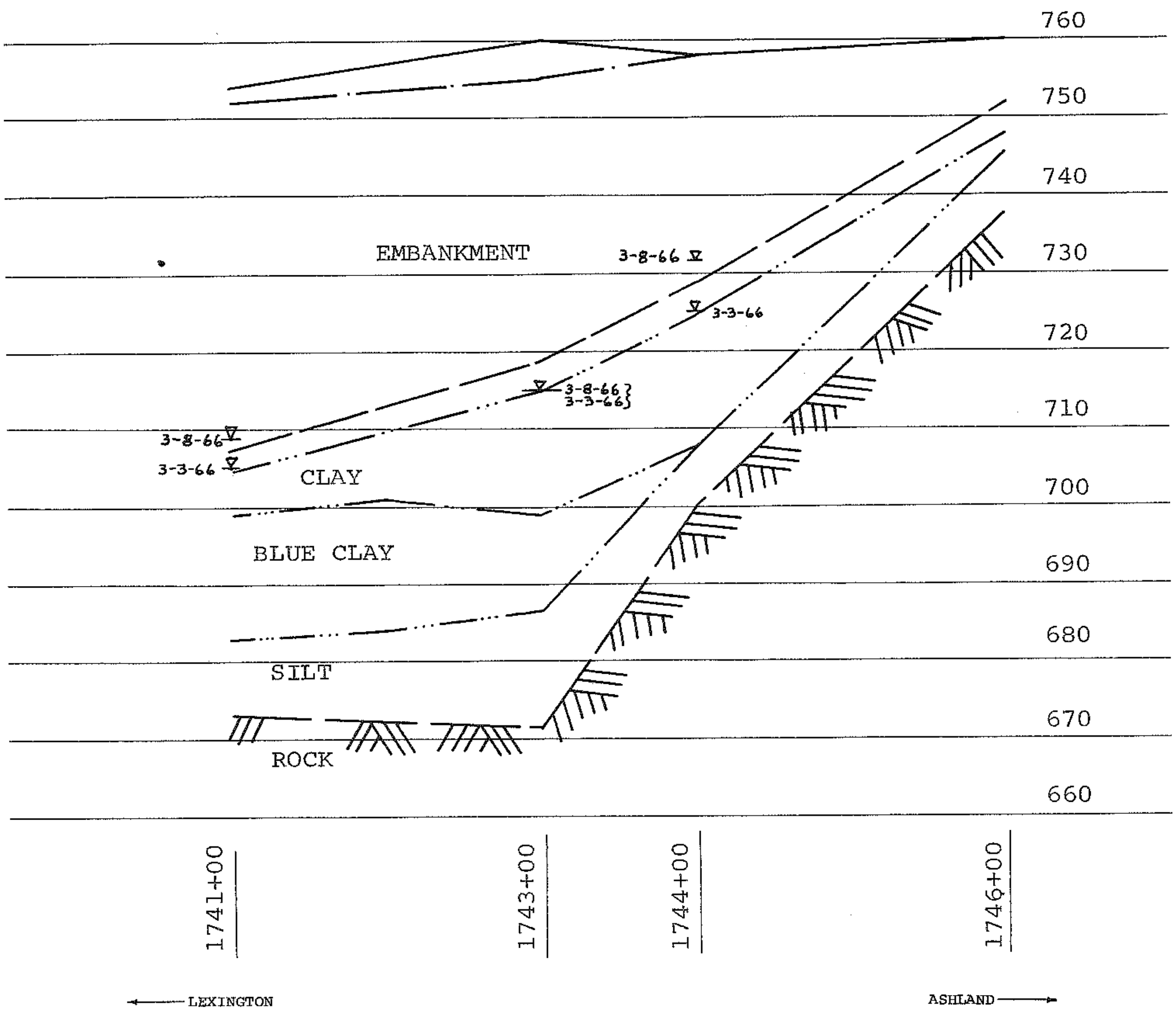


Figure 9-7. Soil Profile, Approximately 77 Feet Left of Centerline.

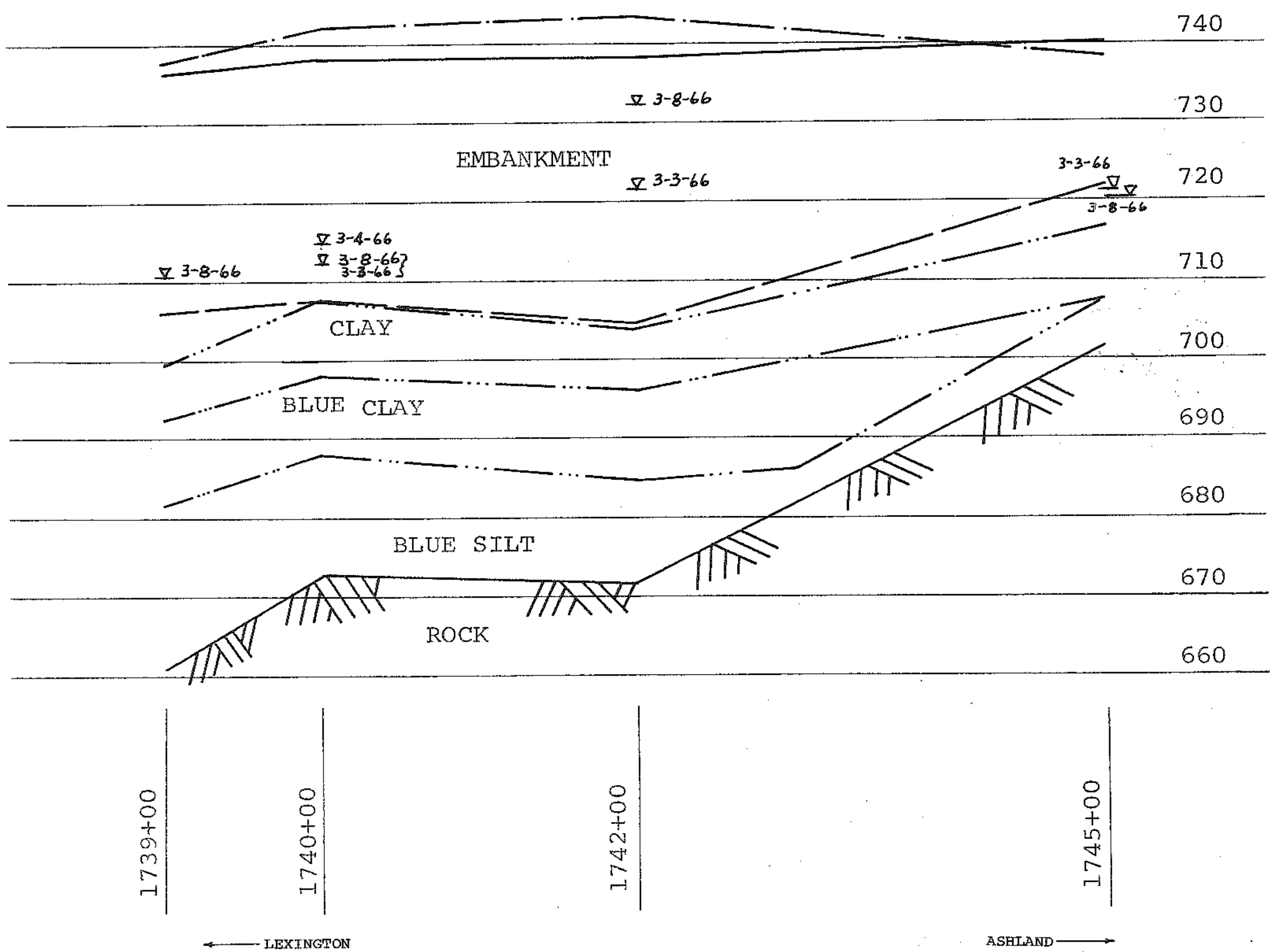


Figure 9-8. Soil Profile, Approximately 112 Fee Left of Centerline.

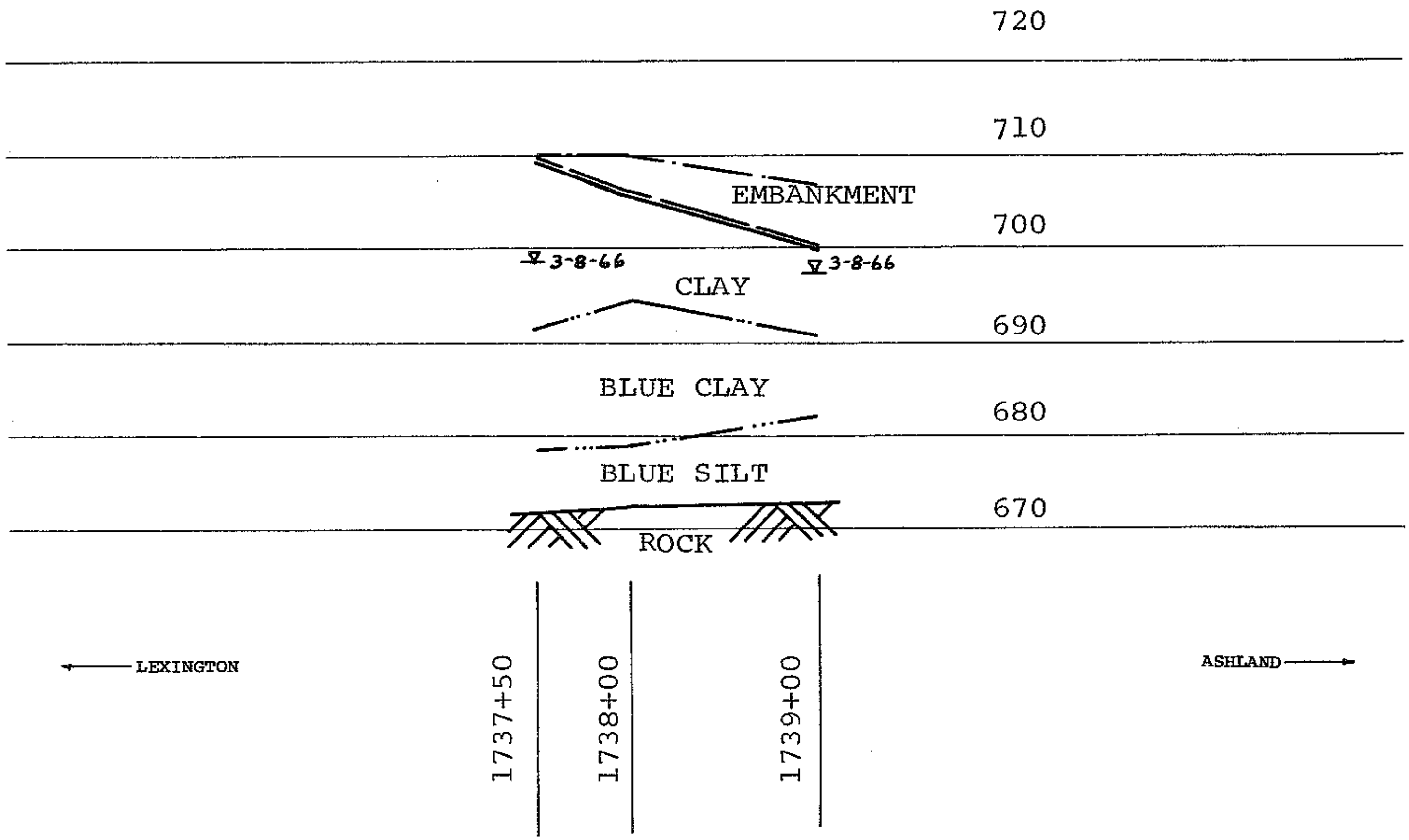


Figure 9-9. Soil Profile, Approximately 179 Feet Left of Centerline.

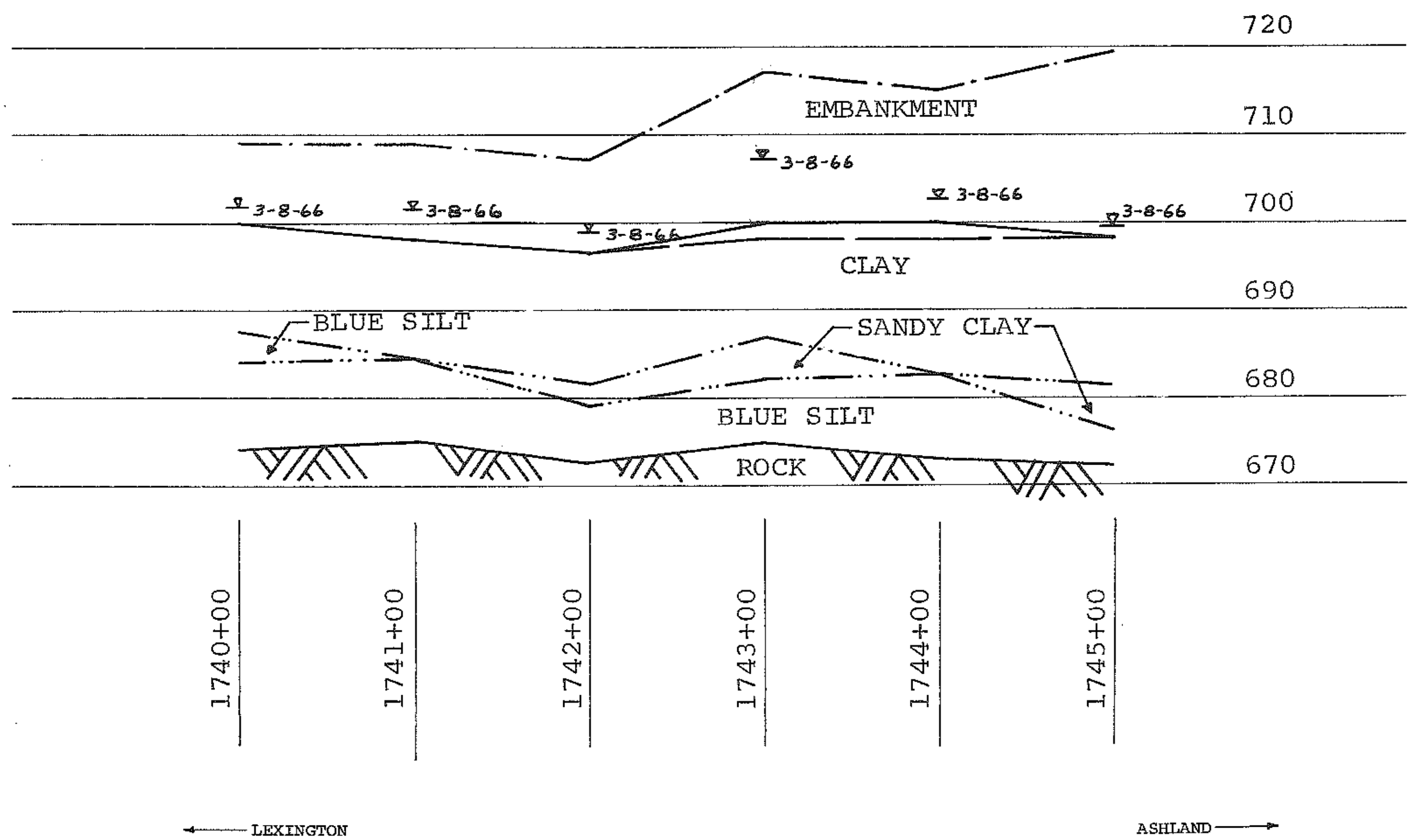


Figure 9-10. Soil Profile, Approximately 197 Feet Left of Centerline.

and lack of seepage from above the slide, indicated that the failure of the embankment was a result of exceeding the bearing capacity of the foundation soil rather than a failure of the embankment itself.

There are two general methods for the correction of a bearing capacity and(or) sliding failure--that is: 1) to reduce shearing stresses or overturning moments and 2) to increase the shearing resistance. It was suggested that the Bath County slide could be corrected by loading the toe of the slope with a berm--which would reduce the overturning movement--and by installing sand drains beneath the berm--which would increase the rate of consolidation under the load of the berm and thus increase the rate of increase of shearing resistance. Since this slide is not complicated by seepage or steep bedding planes, no other remedial action would appear to be necessary.

The Division of Research was requested to analyze the slide from the standpoint of determining a berm size and sand drain spacing. Undisturbed Shelby tube samples were obtained from two drill holes 175 feet left of the centerline at Station 1730+50 and Station 1741+00. Samples were obtained at 5-foot intervals of depth in each hole. This sampling procedure yielded only one sample of the blue clay. This was unfortunate because classification and consolidation tests later showed the sample to be a highly compressible organic clay. The extent of the organic clay is somewhat uncertain although the Division of Materials' borings define the blue clay layer adequately (See Table 9-1). The one sample of organic material is not conclusive evidence that the entire blue clay layer is also organic. In fact, the wide variation of the liquid limits of the blue clay would indicate that this layer may not be organic. However, settlement calculations, which are included in the analysis, were made for two assumed extremes: 1) the case of the maximum thickness of the organic blue clay and 2) the case of no organic clay. This was done to show the magnitude of possible error due to incomplete data concerning the organic clay.

The undisturbed samples were extruded from the Shelby tubes when they were received in the laboratory. To remove soil which may have been seriously disturbed during sampling, material was trimmed from each end of the tube specimens and discarded. The remainder of the sample was cut into specimens approximately four inches long and dipped in melted wax for protection and to maintain the moisture contents at natural conditions. Triaxial and unconfined compression tests were performed to define the shear strength of the embankment foundation and consolidation tests were performed to define the settlement characteristics.

Consolidated-undrained triaxial tests, with pore pressure measurements, were performed. Two-inch diameter by three-inch long specimens were trimmed from the undisturbed samples. This work was done in a moist room to minimize the evaporation of the natural moisture in the specimens. The strain rate used for testing was one to two percent per hour and failure occurred in about seven hours.

Samples for the unconfined compression tests were trimmed in the same manner as the triaxial specimens. The testing strain-rate, however, was 1/2 of a percent per minute.

Summary data from the triaxial and unconfined compression tests are shown in Table 9-2. The Mohr circles and failure envelopes for the triaxial tests are shown in Figure 9-11. The average unconfined compressive strength, including the triaxial test data for the smallest confining pressures and disregarding two tests on specimens from Hole 2, Sample 2, which were considerably higher than the average, was 15.5 pounds per square inch. The average effective angle of friction was 29° and the average effective cohesion was approximately 200 pounds per square foot.

Specimens 2-1/4 inches in diameter by one inch nominal thickness were trimmed using a cylindrical cutter in the

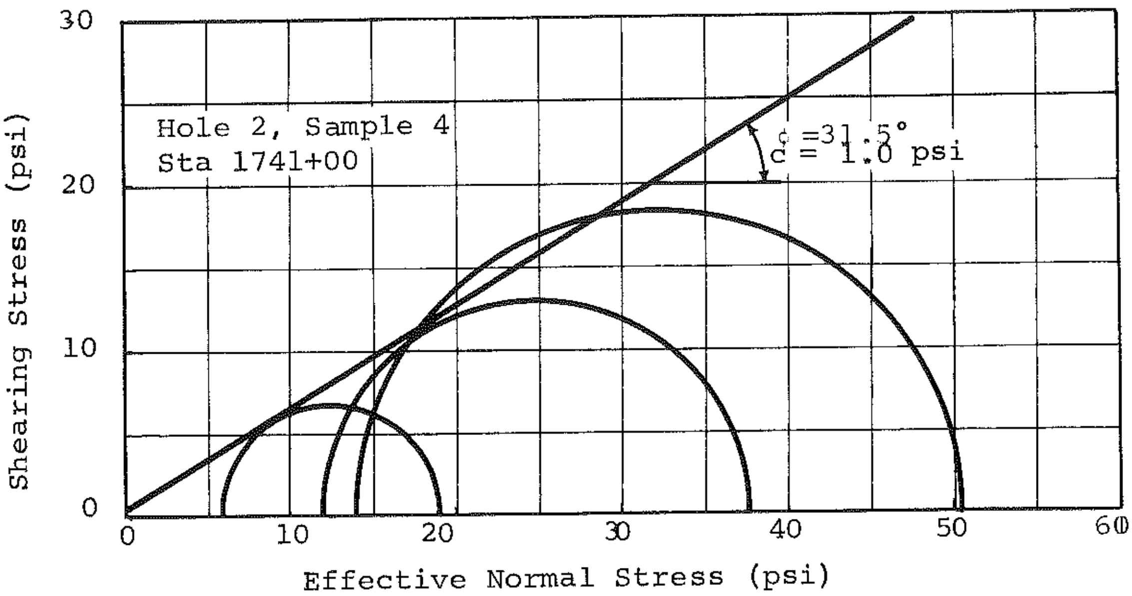
TABLE 9-1. SUMMARY OF LABORATORY TEST DATA (DIVISION OF MATERIALS)

Location	Depth (Feet)	Description	Liquid Limit (Percent)	Plasticity Index (Percent)	Specific Gravity	Compaction Data: Maximum Dry Unit Weight (Lbs/CuFt)	Compaction Data: Optimum Moisture Content (Percent)
STA 1737+50		Silt	28	12	2.70		
185'LT, STA 1739+00	16.3-24.8	Blue Clay	65	38	2.65	97	24.8
	24.8-34.1	Silt	32	14	2.70		
198'LT, STA 1740+00	21.7-25.2	Blue Clay	46	25	2.74		
	25.2-35.1	Silt	28	9	2.67		
203'LT, STA 1741+00	24.6-34.2	Silt	29	12	2.66		
204'LT, STA 1742+00	27.8-30.1	Clay	34	15	2.68		
	30.1-36.6	Silt	31	13	2.70		
191'LT, STA 1743+00	27.3-31.9	Sandy Clay	35	17	2.69		
	31.9-39.5	Silt	29	11	2.68		
192'LT, STA 1744+00	31.1-41.2	Silt	26	10	2.72		
204'LT, STA 1745+00	28.2-33.3	Sandy Clay	33	15	2.70		
	33.3-37.4	Silt	29	12	2.70		
246'LT, STA 1746+00	15.3-16.4	Silt	40	22	2.76		
	16.4-22.3	Silt	29	11	2.69	114	15.2

TABLE 9-2. SUMMARY OF LABORATORY TEST DATA(DIVISION OF RESEARCH)

Location	Sample No	Description	Depth (Feet)	Moisture Content (Shelby Tube Sample) (Percent)	Liquid Limit (Percent)	Specific Gravity	Consolidation Parameters C_v (Ft^2/Day)	Consolidation Parameters C_c	Triaxial Test Data: Unconfined Compression Ultimate Strength (Psi)	Unconfined Compression Failure Strain (Percent)	Dry Unit Weight (Lbs/CuFt)	Moisture Content (Percent) Before Test	Moisture Content (Percent) After Test	Effective Confining Pressure (Psi)	Cohesion (Psi)	Friction Angle (Degrees)
175' LT, STA 1738+50	H-1-S-1	Yellow Clay	10-12	26.0		2.81	.090[4]	.055[4]								
	H-1-S-2	Yellowish-Brown Clay	17-19	29.7		2.79			13.9	2.0						
	H-1-S-3	Blue Organic Clay	20-22	60.2	66[1] 48[2]	2.50	.050[3]	.385[3]								
	H-1-S-4	Moist Blue Sandy Clay	27-29	23.7		2.69			13.6	12.7	102.0	27.7		0		
											100.7	25.2	24.9	9.0		
											99.9	26.4	24.5	10.0	1.5	27.0
											103.1	23.0	22.5	12.5		
175' LT, STA 1741+00	H-2-S-1	Yellow Clay	5-7	26.2					17.8	3.3	98.9	26.0		0		
	H-2-S-2	Yellow Clay	20-22	28.8		2.83			26.4	6.7	102.4	26.1		0		
									27.7	9.2	101.2	25.1		0		
	H-2-S-3	Yellow Silty Clay	25-27	22.9		2.70			14.0	9.2	104.4	24.8		0		
									18.8	9.0	102.1	22.8		0		
	H-2-S-4	Moist Blue Silt	30-32	25.4							99.7	25.1	24.2	6.0		
														12.0	1.0	31.5
											101.6	24.9	22.2	14.0		

1. Tests on Air Dried Soil
2. Tests on Oven Dried Soil
3. Range of Loading-P_o= 1.14 kg/cm^2 , P_f= 2.61 kg/cm^2
4. Range of Loading-P_o= 0.36 kg/cm^2 , P_f= 1.89 kg/cm^2



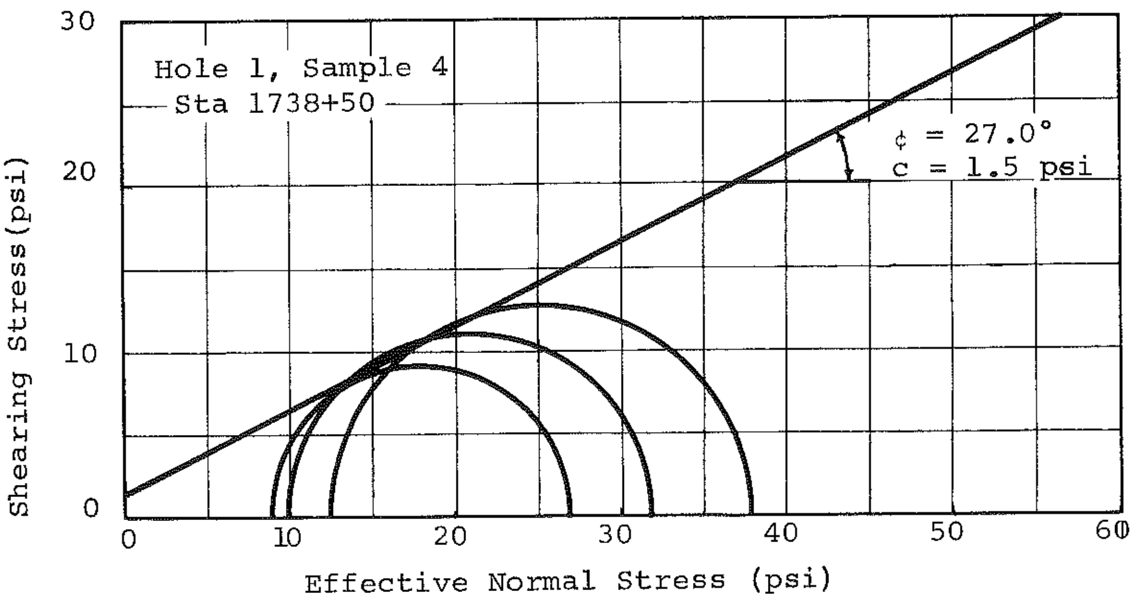


Figure 9-11. Triaxial Test Results

moist room. The loading procedure was the generally accepted one in that the load-increment ratio was one and the load was increased once each day. Specimens were trimmed and tested with the structure and stratifications oriented both horizontally and vertically so that drainage was in some cases parallel to the strata and in others perpendicular to the strata. This was done in order to assess the effect of stratification on the permeability and thus on the rates of consolidation.

In order to design the sand drain spacing, information was required for two cases of loading and drainage. The first of these cases, vertical loading and vertical drainage, fits exactly the boundary conditions of the standard consolidation test. The second case, vertical loading and horizontal drainage, is more difficult to duplicate in the standard one-dimensional consolidation test. Thus a compromise condition was used in the laboratory--that is, horizontal loading and horizontal drainage. Since the loading condition does not correspond to what is expected in the field, ultimate settlement computations based on this particular consolidation test would not be reliable. Since the direction of the drainage path in this test does correspond to that in the field with sand drains, it was expected that the special test would give a fair approximation of the rate of settlement due to horizontal drainage.

Void ratio-log pressure curves and coefficient of consolidation-log pressure curves are shown in Figure 9-12. The values of the compression index, C_c, and the coefficients of consolidation, C_v and C_h*, used in the computations of the time rate of settlements and the ultimate amount of settlement are given in Table 9-2.

The triaxial test data were used in a computer analysis to determine the minimum factor of safety for stability of the embankment under various conditions of berm size and pore pressure. For the initial phase of the analysis, because of limitations of the computer program, it was necessary to assume that the soil was homogeneous--that is, the strength, unit weight, etc., of the fill and the foundation soil were equal. Average values of the angle

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$$*C_v = \frac{k_v\ (1+e)}{a_v\ \gamma_w}; \quad C_h = \frac{k_h\ (1+e)}{a_v\ \gamma_w}$$

where k_v and k_h = vertical and horizontal coefficients of permeability, respectfully,

$$a_v = \frac{e_1 - e_2}{P_2 - P_1}$$

e_1 and e_2 = initial and final void ratios, respectfully, and

P_1 and P_2 = initial and final pressures, respectfully.

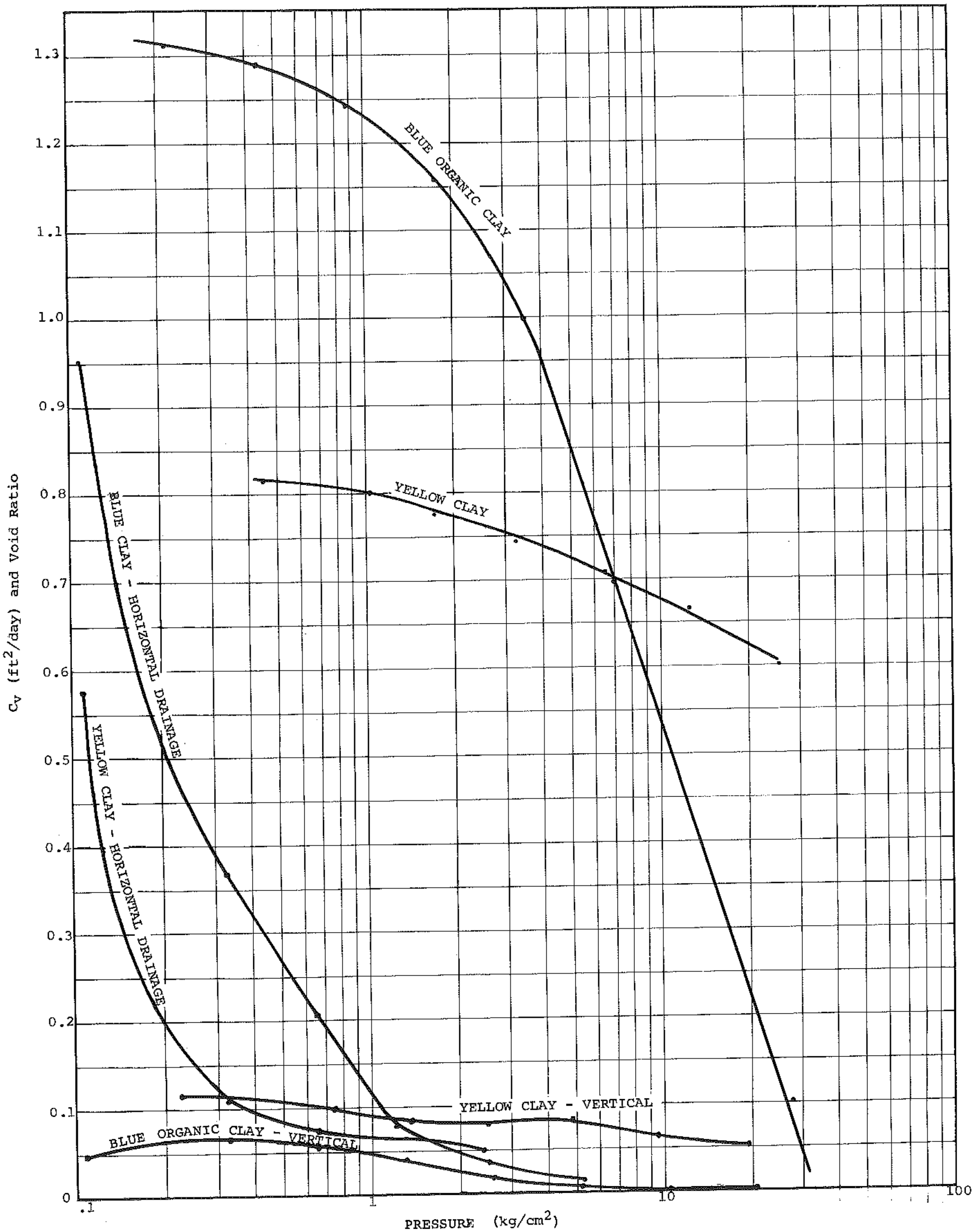

Figure 9-12. Consolidation Test Results.

of friction of 29°, cohesion of 200 pounds per square foot, and unit weight of 125 pounds per cubic foot obtained from tests on the foundation soil were used.

The embankment, being more rigid than the foundation, may develop cracks rather than deforming plastically, and obviously there can be little resistance to shear if there is not intimate contact between the shearing surfaces. For the case of large foundation settlement, which may cause cracking of the embankment, the resistance to shear provided by the embankment may be expected to be very small and the stability of the fill will depend upon the resistance to shear provided by the foundation soil only. Thus the problem reduces to determining the stability of a system composed of two layers--the fill assumed to have no shear strength and the foundation which contributes the only resistance to failure. This is in accordance with current recommended practice for the case of an embankment constructed on a weak foundation*.

The output of the computer program(assuming homogeneous soil conditions) was therefore examined to select the critical circle--neglecting those that did not penetrate

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*A. W. Bishop and L. Bjerrum. "The Relevance of the Triaxial Test to the Solution of Stability Problems", Research Conference on Shear Strength of Cohesive Soils, ASCE, 1960.

into the foundation soil. This critical circle was then analyzed using hand computation methods and considering the fact that the embankment and foundation materials have different strength and rigidity properties. The factors of safety shown in Table 9-3 were determined by hand computations in which the shear resistance of the embankment was neglected.

The long-term stability is represented by the factors corresponding to the water table at the surface where it will be during part of the year due to the proximity of Slate Creek. The factors of safety, based on unconfined compression test data, for the short-term case, represent the stability for the critical time--soon after construction. After the embankment is completed, pore pressures built up in the foundation soil by the additional weight of the embankment begin to dissipate at a rate dependent upon the soil permeability. This results in an increase of stability, which ultimately reaches the long-term value.

Suitable berm dimensions can be selected by studying Table 9-3. It is desirable for the factor of safety for the total stress or short-term condition to be greater than one in order to eliminate the necessity to control the rate of construction. Considering that the berm height should not exceed half the embankment height, or

TABLE 9-3. FACTORS OF SAFETY FOR EMBANKMENT WITH BERM

Unit Weight (Lbs/CuFt)	Berm Height (Feet)	Berm Width (Feet)	Short Term Factor of Safety (Total Stress Analysis)	Effective Stress Analysis: Long Term Factor of Safety	Effective Stress Analysis: Pore Pressure Condition
125	30	75	1.45		Pore Pressures Equivalent To Static Water Table at Ground Surface.
		85	1.58	5.29	
	20	78	1.28	1.98	
	10	52	0.99	1.59	
125	30	85		1.57	Pore Pressures Equivalent To Static Water Table 30 Feet Above Ground Surface
	20	75		1.21	
	10	53		0.71	
135	30	75	1.34		
		85	1.47		
	20	75	1.08		
	10	53	0.91		

the stability of the berm itself would be critical, the optimum berm size is about 25 feet high by 80 feet wide. The long-term factor of safety, from Table 9-3, would be between 2 and 5. The initial factor of safety would be 1.3 to 1.5, depending upon the actual unit weight of the soil. These factors appear to be adequate without allowing pore pressures to dissipate by drainage--even considering an expected error of ± 15 percent.

An 80-foot wide berm, however, would require the acquisition of additional right of way whereas a berm 65 feet wide would not. A berm 65 feet wide by 20 feet high would provide a long-term factor of safety approaching 2. On the other hand, the initial factor of safety provided by the smaller berm is only 1.1 or 1.2. Again considering a likely error of ± 15 percent, it is apparent that the initial factor of safety is inadequate. It is not recommended that a 20-foot by 65-foot berm be constructed without providing for rapid dissipation of the induced pore pressures through sand drains.

It should be noted that the addition of any berm weighing more than the existing bulged material at the toe of the embankment would increase the factor of safety, which now appears to be in the order of one--as no recent movement has been detected. The most critical stage of construction, however, would be during the installation of sand drains, after leveling the excess material at the toe, and prior to construction of the berm. The sand drain and berm construction should proceed with all haste in order to provide the additional support at the toe as quickly as possible. Reconstruction of the main embankment, on the other hand, should be delayed three or four months to allow for pore pressure dissipation (consolidation) and for the shear resistance in the foundation to increase beneath the berm. If a berm of sufficient dimensions as to nullify the need of sand drains was to be constructed, this critical stage of construction would be avoided. In that case, it would be necessary only to level the excess material at the toe and to construct the berm.

The consolidation test data were analyzed both in terms of expected settlement under the weight of a 25-foot high berm (see Table 9-4) and the rate of settlement and consequent gain of shear resistance for various sand drain spacings (see Table 9-5 and Figure 9-14). Table 9-4 is a sample calculation for the rate of consolidation with sand drains, and Figure 9-14 shows the percent consolidation as a function of time for various sand drain spacings.

The dashed lines in the figure show percent consolidation as a function of time for the assumption that the

TABLE 9-4. SAMPLE CALCULATION OF ULTIMATE SETTLEMENT

Station Number	Layer Description	Layer Thickness(H) (Feet)	Depth To Midpoint of Layer(D) (Feet)	Unit Weight(γ)[1] (lbs/CuFt)	Overburden Pressure(P_o)[2] (Kg/cm^2)	Influence Values(I)[3]	Vertical Stresses(ΔP)[4] (kg/cm^2)	Final Pressures(P_f)[5] (kg/cm^2)	Initial Void Ratio(e_1)	Final Void Ratio(e_2)	Settlement (ΔH)[6] (Inches)
1738+00	Yellow Clay	12.0	6.0	124	0.364	0.997	1.522	1.886	0.815	0.774	3.3
	Blue Clay	15.0	19.5	112	0.758	0.965	1.474	2.232	1.257	1.113	11.5
	Silt	7.0	30.5	124	1.046	0.925	1.413	2.459	0.800	0.762	1.8
										Total Settlement	16.6
1741+00	Yellow Clay	22.0	11.0	124	0.844	0.990	1.512	2.056	0.812	0.770	6.1
	Silt	10.0	27.0	124	1.025	0.940	1.435	2.461	0.800	0.762	2.5
										Total Settlement	8.6

1. $\gamma = (\frac{1+w}{1+e_1}) G\gamma_w$, where γ_w = Unit Weight of Water and G= Specific Gravity of Soil
2. $P_o = \gamma D$. Water Table Assumed 7 Feet Below Original Ground Elevation.
3. From Influence Tables or Charts
4. $\Delta P = I\gamma_b H_b$, where γ_b = Unit weight of Berm Material (125 lbs/CuFt) and H_b = Height of Berm (25 Feet)
5. $P_f = P_o + \Delta P$
6. $\Delta H = \frac{H}{1+e_1}(e_1 - e_2)$

TABLE 9-5. SAMPLE CALCULATION FOR RATE OF CONSOLIDATION

(Equilateral Sand Drain Spacing = 7.5 Feet and Sand Drain Radius = 9 inches[1])

Radial Consolidation (U_r)[2] (with Sand Drains) (Percent)	Radial Time Factor[3] (T_r)	Time(t)[4] (Days)	Vertical Time Factor[5] (T_v)	Vertical Consolidation (U_v)[3] (Without Sand Drains) (Percent)	100-U_r (Percent)	100-U_v (Percent)	$\frac{(100-U_r)(100-U_v)}{100}$ (Percent)	Average Total Consolidation(U_c)[6] (Percent)
20	.026	20.9	.0013	3.8	80	96.2	77.0	23.0
30	.042	33.7	.0022	5.4	70	94.6	66.2	33.8
50	.081	65.1	.0042	7.1	50	92.9	46.5	53.5
70	.137	110.1	.0071	9.5	30	90.5	27.2	72.8
90	.270	217.0	.0139	13.1	10	86.9	8.7	91.3

1. Effective Drain Radius Taken as One-Half of Actual Radius to Account for Smear.
2. Selected Values
3. From Time Factor Tables
4. $t = \frac{(2R)^2 T_r}{C_{vr}}$ where 2R = Sand Drain Spacing and C_{vr} = Coefficient of Consolidation.
5. $T_v = \frac{C_v t}{H^2}$
6. $U_c = 100 - \frac{(100-U_v)(100-U_r)}{100}$

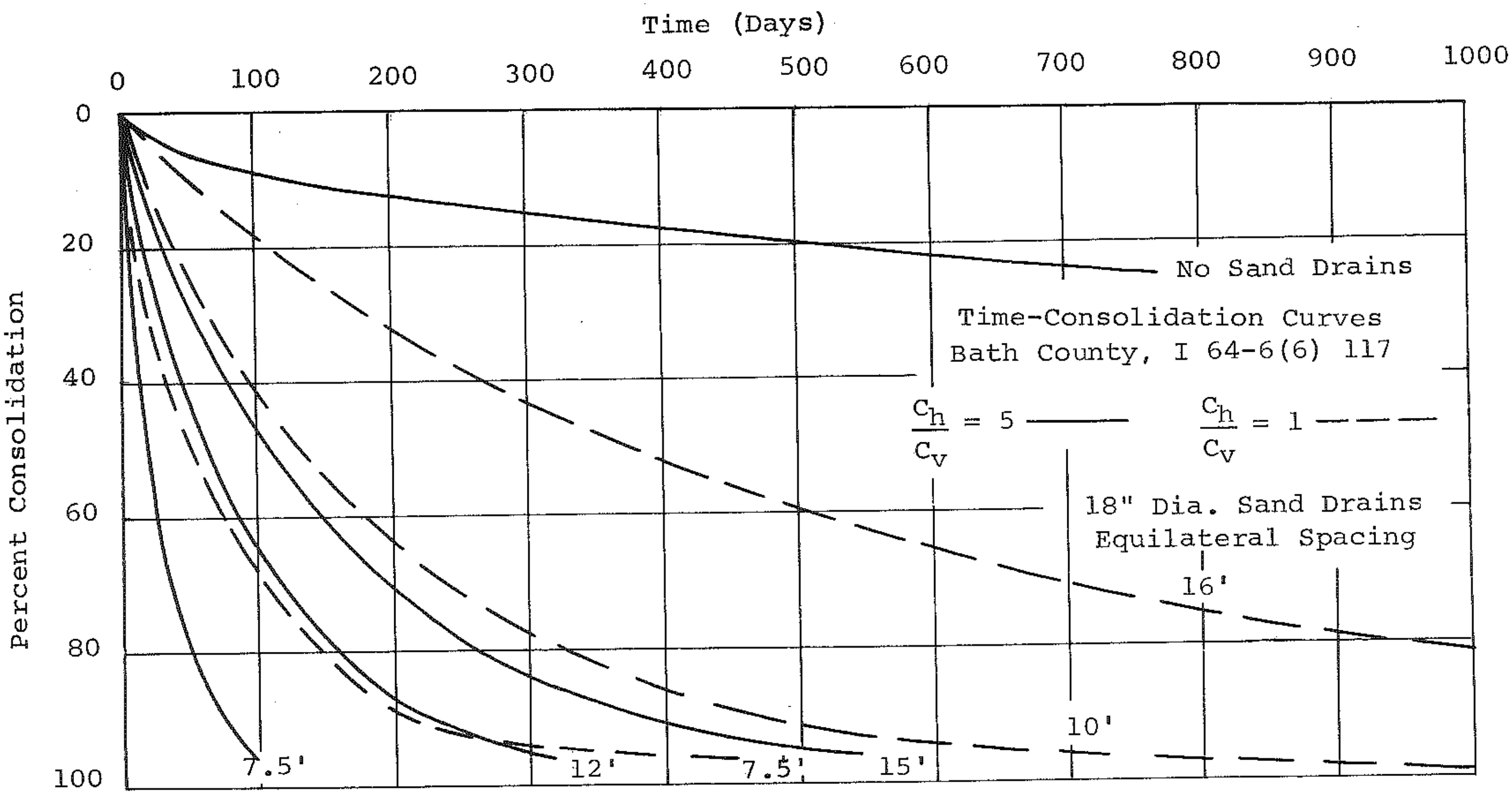

Figure 9-14. Time-Consolidation Curve

coefficient of premeability is the same in all directions. However, the curves of coefficient of consolidation in Figure 9-12 indicate that the coefficient of permeability in the horizontal direction may be five times or more greater than the corresponding value for vertical drainage. This is in agreement with experience. The rapid decrease of the coefficient of consolidation for horizontal drainage (C_h) with pressure is throught to be due to the unrealistic test conditions wherein the load is applied horizontally, in order to effect horizontal drainage, rather than to an inherent property of the soil.

The solid lines in Figure 9-14 are curves for percent consolidation as a function of time for the assumption that drainage in the horizontal direction is five times greater than in the vertical direction, that is, $k_h/k_v=5$. The curve of percent consolidation vs time without sand drains is for one-way drainage only--since the borings did not indicate a previous stratum below the compressible material. However, a very thin seam or seams of pervious soils could very well have gone undetected, in which case the rate of settlement would be increased at least four-fold.

The spacing of sand drains required to effect a given degree of consolidation in a given time can be determined from Figure 9-14. It is recommended that the solid curves be used. The dashed lines are shown only to illustrate the effect of the ratio C_h/C_v and to indicate the maximum possible inaccuracy of settlement rate predictions.

There is another consideration, namely settlement, in addition to stability that should perhaps be considered. In order to prevent possible objectionable long-term settlement of the main embankment, it would be desirable to accelerate consolidation of the foundation beneath the main embankment through the installation of sand drains there. This would necessitate the removal of a major portion of the existing embankment, the installation of sand drains, and reconstruction of the embankment with proper control of the rate of construction through the use of piezometers.

The test results and analysis show that stabilization of the slide by counterbalancing with a berm at the toe, with or without sand drains, is feasible and practical. It is also evident that stabilization can be effected by various combinations of berm size and sand drain configuration. Only in the case of a very large berm are sand drains not required to accelerate shear strength gain to an adequate value for safety within reasonable time limits. In all cases, sand drains are desirable from the standpoint of minimizing long-term settlements.

Three alternative designs were reviewed, and any of the three is recommended as a suitable solution. The alternatives are as follows:

1. Level the excess material at the toe and construct a berm 25 feet high by 80 feet wide. Piezometers and settlement gauges should be installed so that pore pressures and settlements can be monitored. It is not anticipated that control will need be exercised over the rate of construction of the berm. It may be desirable, however, to delay reconstruction of the main embankment if the piezometers indicate unexpectedly high pore pressures. A piezometer at the original ground surface under the berm could indicate pore pressures as high as thirteen pounds per square inch without causing concern. If gauge readings exceed thirteen pounds per square inch, reconstruction of the main embankment should be temporarily discontinued to permit dissipation of excess pore pressures.

2. Level the excess material at the toe and construct approximately two feet of embankment from the toe to the right-of-way line. Install sand drains on ten-foot centers and construct without undue delay a berm 20 to 25 feet high by 65 feet wide. Piezometers and settlement gauges should be installed. The pore pressures as measured by a piezometer under the berm located at the original ground elevation should not be allowed to exceed five pounds per square inch before discontinuing reconstruction of the main embankment. It is unlikely, however, that even these small excess pore pressures will be developed because of the relatively rapid drainage to the sand drains. Even so, reconstruction of the main embankment should be delayed three months after completion of the berm to insure that the foundation soil will have gained sufficient strength through consolidation to maintain the stability of the embankment.

3. Remove the main embankment to within approximately two feet of the original ground from the toe to a point beneath the westbound traffic lanes from Station 1738+50 to 1743+50. Construct sand drains on 15-foot centers within the area described and install lateral pipe drains from the toe to the right-of-way line. Install piezometers and settlement gauges. Construct the main embankment and 20-foot by 65-foot berm concurrently, using the piezometers to control the rate of construction of the final stages of the main embankment. Construction should be temporarily discontinued if readings of five pounds per square inch are obtained from a piezometer located under the embankment at original ground elevation.

The berm recommended in all three alternatives should extend from approximately Station 1737+50 to Station 1746+00, and material for the berm should be obtained, as far as possible, by lowering the finished grade in the vicinity of the slide. Figure 9-15 shows a typical berm and sand drain configuration. A detail of a drainage blanket that would serve as a suitable alternate to a two-foot thick sand blanket is also shown in the figure.

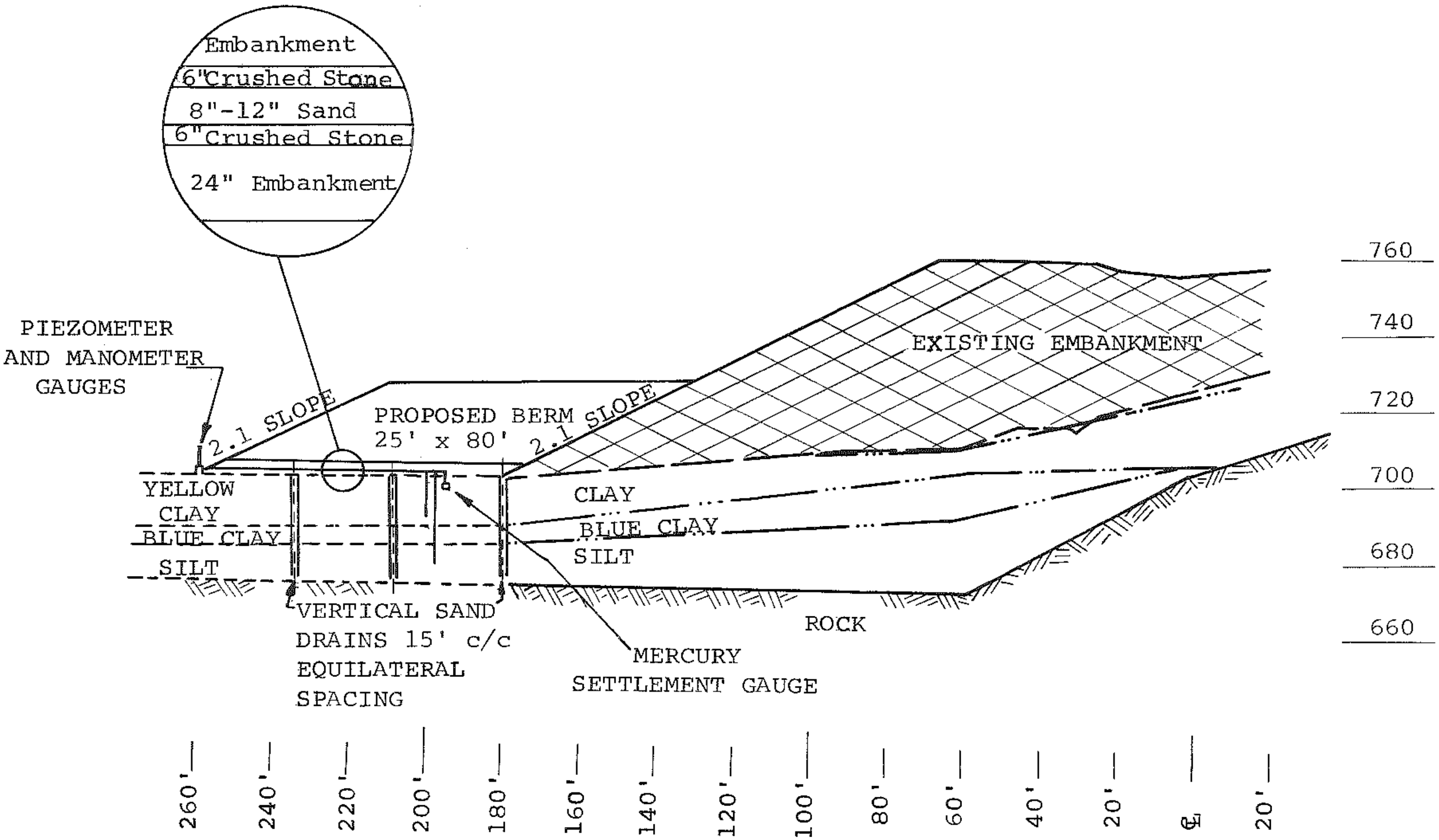

Figure 9-15. Typical Berm and Sand Drain Configuration.

SLOPE STABILITY ANALYSIS

Bluegrass Parkway

Nelson County, Mile Post 20

SLOPE STABILITY ANALYSIS
Bluegrass Parkway
Nelson County, Mile Post 20

Just prior to the completion of construction, a slide occurred in the eastbound lanes of the Bluegrass Parkway approximately 1.5 miles west of the US 31E interchange. The pavement, which was damaged, was patched just before the parkway was opened to traffic. Movement continued in this area until the eastbound lanes had to be closed to traffic.

It is interesting to note that two other slips have occurred on the Bluegrass Parkway within two miles of the US 31E interchange. Both of these slips, as the one described above, first occurred during the construction period. These two slides, one located approximately two miles west of the US 31E interchange in the eastbound lanes and the other located approximately one mile east of the US 31E interchange in the eastbound lanes, were noted during the grade and drain operation. The fill material was removed and replaced at that time. Since the parkway has been opened to traffic, there has been no serious inconvenience to the traveling public at these two slides. However, there has been indications of continued movement manifested by cracked shoulders and settlement of the guardrails. At the slide approximately two miles west of the interchange, the pavement has settled to such an extent that it has been necessary to patch in order to maintain a reasonably safe and smooth riding surface*.

The geologic materials which outcrop in this portion of Nelson County are predominately of Silurian age. Some re-manents of the New Albany Shale overlying a thin layer of Sellersburg Limestone has been noted along some of the higher ridges in the area. These materials are of Devonian age. As one progresses downward in the columnar section (see Figure 10-1), thin outcrops of the Louisville Limestone and Waldron Shale may be observed. Below that is a rather thick section of Laurel Dolomite, a thick- to medium-bedded, calcareous dolomite. Near the middle portion of the Laurel Dolomite, the deposit is thick-bedded and massive and weathers to a porous network of rock. In the lower portion of the unit, the Laurel lacks this cavernous nature of weathering, and contains a number of very thin interbeds of shale. Beneath the Laurel Dolomite is the Osgood Formation, consisting of shales and dolomites. Near the top and the bottom of the unit, dolomite occurs in thin beds interbedded with shale. In the central portion of the unit, the predominate material is a greenish-gray shale composed of a silty, dolomitic clay. The shales in this unit weather to a plastic clay and form the valley slopes at the base of escarpments in the overlying dolomites. The

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*Intra-departmental Communication, Research Division File H.2.16

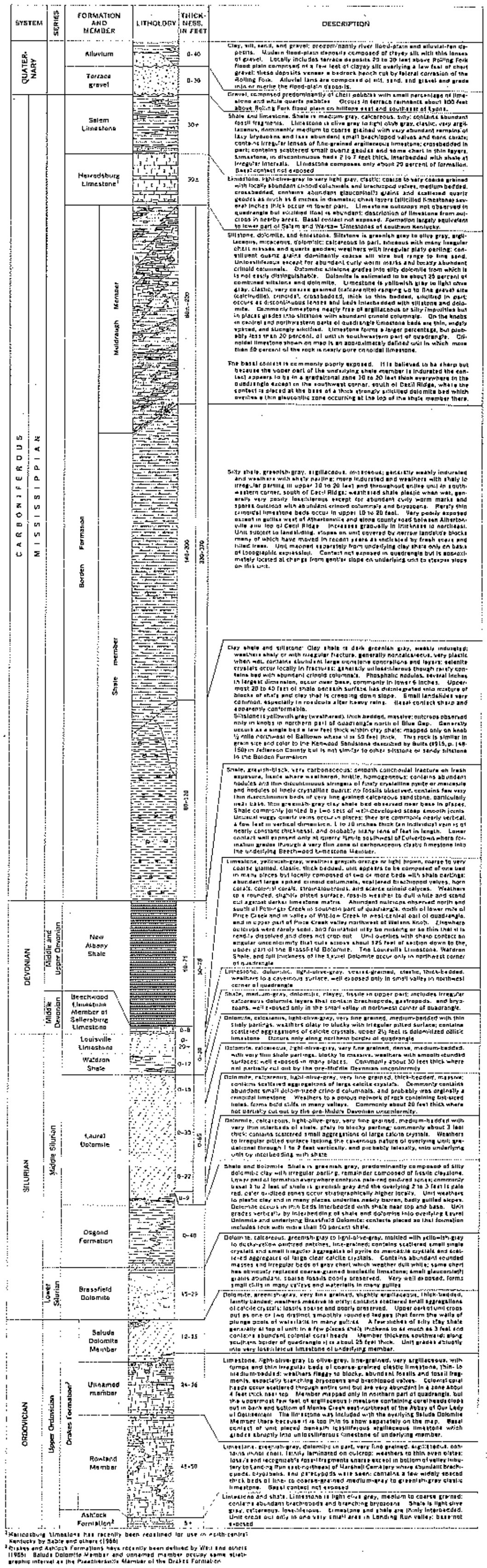

Figure 10-1. Generalized Columnar Section for Nelson County, Kentucky (from "Geology of the New Haven Quadrangle, Nelson and Larue Counties, Kentucky," W. L. Peterson, U. S. Geological Survey, 1966).

Osgood Formation grades vertically by interbedding of shale and dolomite into the overlying Laurel Dolomite and the underlying Brassfield Dolomite. The Brassfield is a greenish-gray, calcareous dolomite containing some chert. This unit is probably the lowest geologic unit which outcrops to any extent in this area of Nelson County. It overlies the lower Ordovician deposits in the area.

It is suspected that the material involved in the slide at Mile Post 20 consists predominately of portions of the Osgood Formation. The site is a side-hill cut-fill section. Outcrops of thinly bedded shales, known to be paths along which subsurface waters seep, have been covered with the embankment portion of the section. It is felt that the subsurface seepage water has been dammed by the embankment--the embankment becoming saturated and losing strength and therefore slipping down the original slope.

In an attempt to correct the situation, it was decided to remove the unstable material from the embankment (see Figures 10-2 through 10-4). Once firm and stable material was reached, several horizontal holes were augered into the original ground near the bottom of the excavation. Pipe were inserted in these holes to provide drainage (see Figure 10-5). A blanket of free-draining, granular material was then laid over the entire base of the excavated area (see Figures 10-6 and 10-7).

Figure 10-2. View of Slide Area, Looking East, Showing Removal of Unstable Material.

Figure 10-3. View of Slide Area, Looking West, in Late July, 1966.

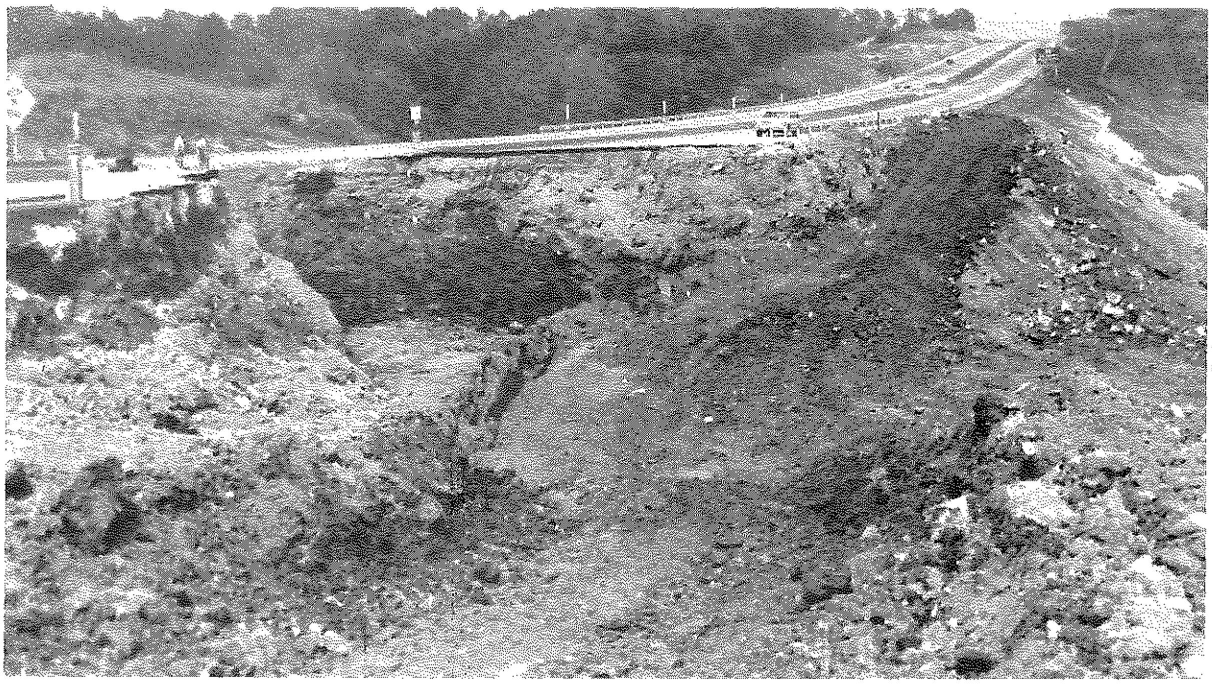

Figure 10-4. Slide Area with Unstable Material Removed.

Figure 10-5. Excavated Slide Area Showing Benched Foundation and Horizontal Pipes.

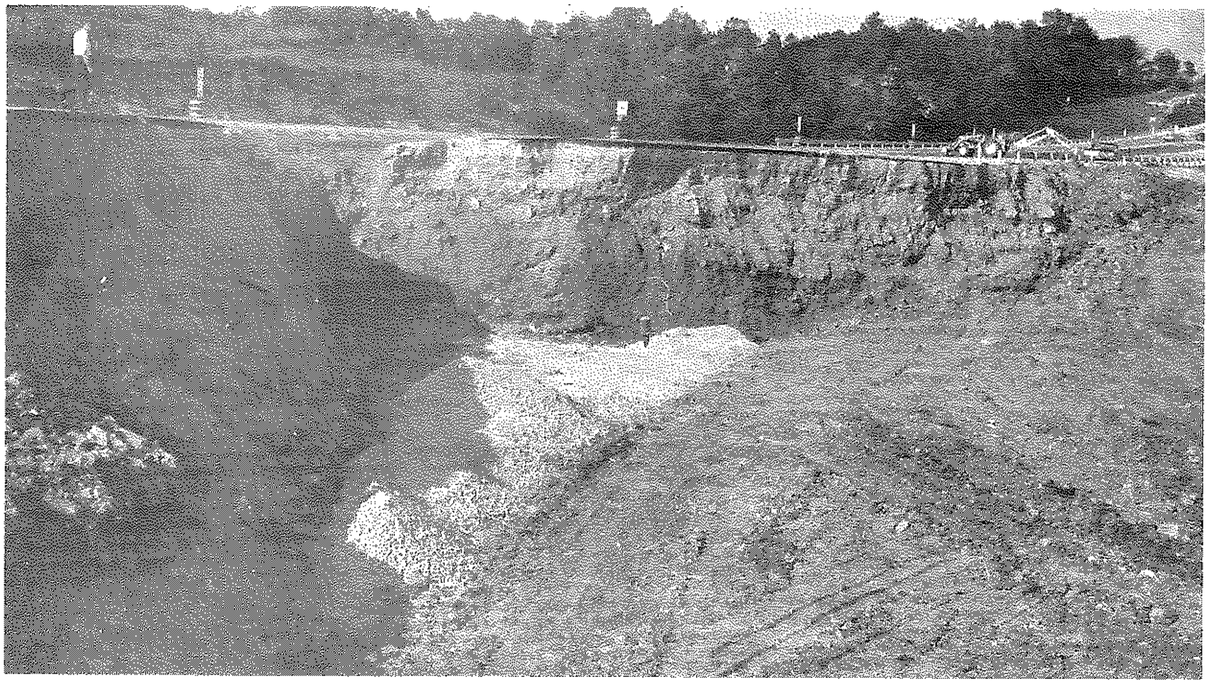

Figure 10-6. Free-draining, Granular Blanket and Horizontal Pipe.

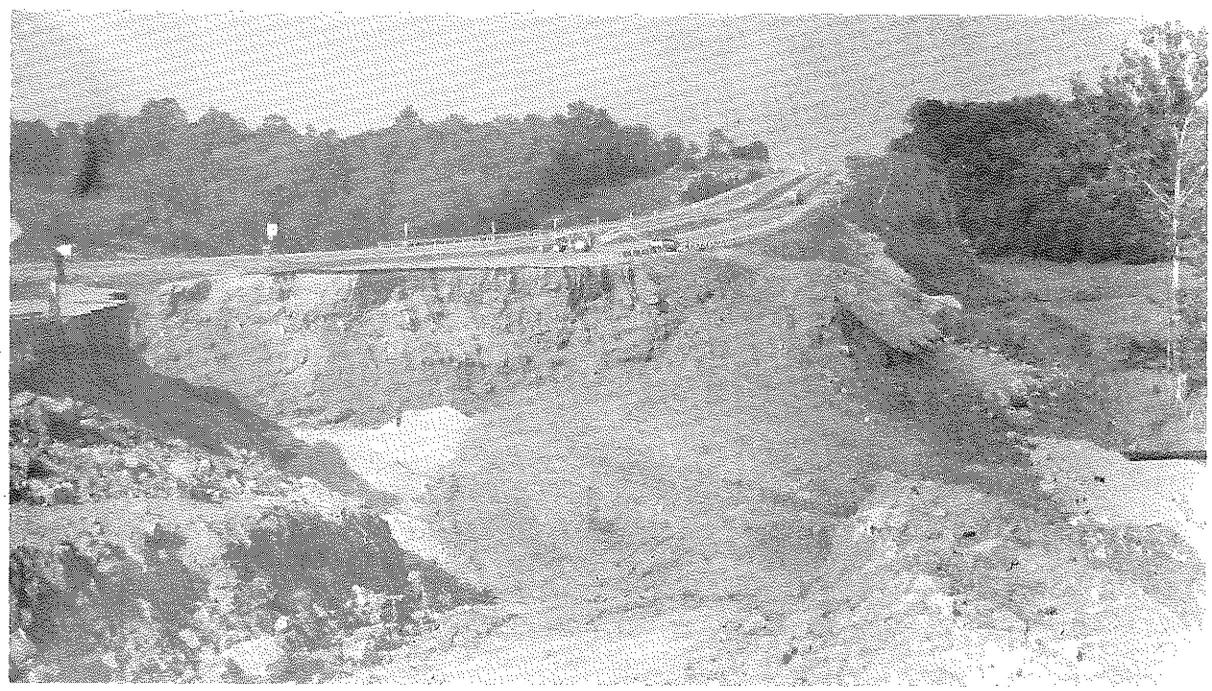

Figure 10-7. Free-draining, Granular Blanket and Pipe Outlet.

The embankment material was then replaced, compacted, and shaped to the desired cross section. At the time of this writing in late September, 1966, significant quantities of water were continuously discharging from the drainage system installed at the base and toe of the embankment (see Figure 10-8).

Figure 10-8. Effluent from Outlet Pipe, September, 1966.

SETTLEMENT AND FOUNDATION STABILITY ANALYSIS

Ky 55

Adair County, SP 1-10-4L, F 534(3)

SETTLEMENT AND FOUNDATION STABILITY ANALYSIS
Ky 55
Adair County, SP 1-10-4L, F 534(3)

In planning for the reconstruction of Ky 55 between Campbellsville and Columbia, a question arose concerning the stability of an embankment near the Taylor-Adair County line. A high side-hill embankment was to be constructed so that it would be founded on a relatively flat stream bottom and butting against rather steep escarpments of the valley wall. The embankment would be founded on the Fort Payne Chert (Osage Series), a sandy textured shale or siltstone. This formation yields significant quantities of water and is often used for domestic water supplies. A spring has been noted to feed a small pond and would be covered by the embankment. The walls of the valley are relatively steep and are formed by the St Louis Limestone and the Warsaw Limestone of the Meramec Series. The Warsaw Limestone, overlying the Fort Payne Chert, is a coarse-grained limestone with some interbeds of siltstone. The St Louis Limestone is a fine-grained and dense thick-bedded, cherty limestone. Both the St Louis and Warsaw limestones form steep bluffs in the area and contain sufficient amounts of water to be used for domestic water supplies. Both the Osage and Meramec Series materials are of Mississippian age (see Figure 11-1).

System	Series	Formation or Group	Thickness (in feet)	Lithology	Topography
QUATERNARY	Pleistocene and Recent	Alluvium	0-50	Silt, clay, and some gravel and sand in major stream valleys. Silt, clay, and some sand in tributary valleys.	Terraces and flood plains of Green River, Cumberland River, and tributaries.
CARBONIFEROUS / PENNSYLVANIAN		Lee	0-600±	Sandstone, yellowish-brown, medium-grained, massive. Yellowish-brown medium-grained conglomerate and sandstone contain quartzite pebbles; conglomerate is lenticular. Shale and coal found in places.	Forms dissected ridges and caps mountains in Clinton, Wayne, and Pulaski Counties; forms major escarpment of marginal area of eastern mountains. Underlies dissected upland in Pulaski County. Channel-fills mantle rocks of Meramec age in Taylor and Adair Counties.
CARBONIFEROUS / MISSISSIPPIAN	Chester	Pennington shale	0-250±	Shale, red and green, clayey, and minor amounts of limestone and sandstone. In places removed by pre-Pennsylvanian erosion. Thickness varies greatly owing to the pre-Pennsylvanian erosion.	Forms moderate to steep slopes in mountain margin area where capped by massive sandstone of Lee formation.
	Chester	Glen Dean limestone	15-25	Limestone, dark- to bluish-gray, fine- to medium-grained, shaly beds near top; very fossiliferous in places.	Forms steep hillsides or underlies dissected uplands in marginal area of mountains of Pulaski, Wayne, and Clinton Counties.
	Chester	Hardinsburg sandstone	10	Clay shale, green to dark-gray, soft. Conspicuous marker. Known by drillers as the "Pencil Cave".	Forms moderate to steep slopes and discontinuous benches.
	Chester	Haney limestone	15	Limestone, gray, fine- to medium-grained, medium crystalline, oolitic in places; beds ½ to 2 feet in thickness, contrasting to massive limestone below.	Forms steep hillsides or underlies dissected upland in marginal area of mountains of Pulaski, Wayne, and Clinton Counties.
	Chester	Beech Creek and Reelsville limestones	35	Limestone, light-gray to white, fine- to medium-grained, oolitic to coarsely crinoidal, massive. Cobbly or conglomeratic limestone at base.	Form foot of mountains along base of escarpment of eastern mountains. Underlie broad rolling karst areas and dissected uplands of Pulaski and Wayne Counties.
	Chester	Beaver Bend and Paoli limestones	60	Limestone, light-gray, fine-grained, dense. Oolitic in places.	
	Meramec	Ste. Genevieve limestone	85	Limestone; breccia present usually at top of formation. Limestone, light-gray to white, oolitic, fine-grained in places, crossbedded, minor amount of chert.	Forms steep bluffs along Lake Cumberland. Underlies dissected karst areas on uplands. Forms steep slopes on hills in Casey County.
	Meramec	St. Louis limestone	100	Limestone, dark-gray to black, fine-grained, dense, medium- to thick-bedded, cherty. Top 10 feet marked by black chert nodules and stringers containing coral, *Lithostrotion*. Argillaceous and oolitic in places. Medium- to dark-gray massive geodiferous siltstone. In places only siltstone is present.	Forms steep bluffs along Lake Cumberland. Underlies rolling karst areas in uplands; dissected close to Lake Cumberland and tributaries. Siltstone and shale layers form discontinuous minor benches on hillsides.
	Meramec	Spergen limestone	50	Limestone, gray, granular to oolitic, shaly, argillaceous, and siltstone beds in places. Light-brown medium-grained sandstone. Dark-gray to black gritty calcareous shale fossiliferous in places; grades into geodiferous limestone.	Underlie moderately to highly dissected rolling uplands. Form steep bluffs along Lake Cumberland. In some areas numerous small sinkholes occur in the Warsaw.
	Meramec	Warsaw limestone	0-100	Limestone, light- to dark-gray, granular, crinoidal, massive, crossbedded crosslaminated, argillaceous in places. Medium- to dark-gray brittle, geodiferous siltstone. In places only siltstone present. Light-brown medium-grained flaggy sandstone; occurs at top of formation in Wayne County.	
	Osage	Borden group: Muldraugh	60-95	Limestone, gray, siliceous; calcareous siltstone in places. Yellowish-brown argillaceous chert; contains impure limestone, small geodes.	Fort Payne chert: Siltstone, light- to dark-gray, brittle, argillaceous, geodiferous in places. Medium- to dark-gray shale. Light- to bluish-gray fine- to coarse-grained crinoidal siliceous limestone. — Underlie moderately to highly dissected rolling uplands. Form "knobs". Form major escarpment in Taylor County and in Casey County where it is modified by faults. Form steep bluffs along rivers.
	Osage	Borden group: Floyds Knob	5	Limestone, siliceous, crinoidal, glauconitic zones. Calcareous siltstone in places.	
	Osage	Borden group: Brodhead	135-140	Siltstone, gray, and impure siliceous limestone grading into siltstone. Gray fossiliferous patches of siltstone and shale; grade into shale. Gray to drab silty shale; abundant worm marks.	
	Osage	Borden group: New Providence shale	100-150±	Shale, green- to steel-gray, clayey, crinoidal, stringers of limestone. Thick massive boulders of dense fine-grained bluish-gray limestone in Wayne County. Variable facies occur.	Forms moderate to steep slopes near base of eastern mountain margin escarpment and "knobs".
DEVONIAN		Chattanooga shale	20-40	Shale, black, fissile.	Forms steep slopes near base of eastern mountain margin escarpment and "knobs". Underlies small round hills near base of escarpment.
		Sellersburg limestone	60	Limestone, gray, dolomitic, crinoidal, fossiliferous, and large amounts of light- to bluish-gray chert. Conglomeratic locally.	Forms gentle slopes on hillsides under capping Chattanooga shale in southern Casey County.
ORDOVICIAN		Maysville group: McMillan	25-90	Limestone, gray, fine-grained, dense, hard, argillaceous. Gray fossiliferous shale.	Form bluffs and dissected valley sides adjacent to Cumberland River area. Exposed in faulted areas in Casey County.
		Maysville group: Fairview	100-150	Limestone, thin-bedded, and shale.	

Figure 11-1. Generalized Columnar Section For Adair County, Kentucky (From "Availability of Ground Water in Adair, County, Clinton, Cumberland, Pulaski, Russell, Taylor and Wayne Counties, Kentucky," T. W. Lambert and R. F. Brown, U. S. Geological Survey, 1963).

Specimens were obtained from Shelby tube samples for the purpose of performing unconfined compressive tests and consolidation tests. In the unconfirmed compressive strength tests, the strain rate was 1/2 percent per minute. The results of the compressive tests are summarized in Table 11-1. The average unconfined compressive strength, excluding the high value obtained for the specimen from Station 338+20, is 33.1 pounds per square inch (2.37 tons per square foot). A ø-equal-zero stability analysis was performed using the unconfined compressive strength and a typical cross section at Station 367+00. The minimum factor of safety was calculated assuming that the embankment offered no resistence to sliding. The minimum factor of safety was determined to be 10.2, a very stable situation.

The void ratio-log pressure curves and the coefficient of consolidation-log pressure curves, obtained from the consolidation tests, are shown in Figure 11-2. Using these results in a settlement analysis, it was determined that the maximum ultimate settlement would be on the order of 6.4 inches. Fifty percent of this settlement would be obtained within approximately 29 days after construction and 90 percent would be realized within 122 days.

On the basis of the stability and settlement analyses for this site, it is not anticipated that there should be

TABLE 11-1. SUMMARY OF TEST RESULTS

Description of Material	Location	Sample Depth (Ft)	Moisture Content (Percent)		Unconfined Compressive Strength Tests	
			Shelby Tube	Unconfined Compressive Test Specimens	Strength (Psi)	Failure Strain (Percent)
Firm, yellow, moist clay	Sta 388+20	8-10	21.4	23.2 27.1	59.6 34.9	9.5 7.7
Firm to slightly firm, dark brown, clayey silt	Sta 367+00	4-5.5	22.1		31.6 32.6 33.3	8.0 10.8 9.7

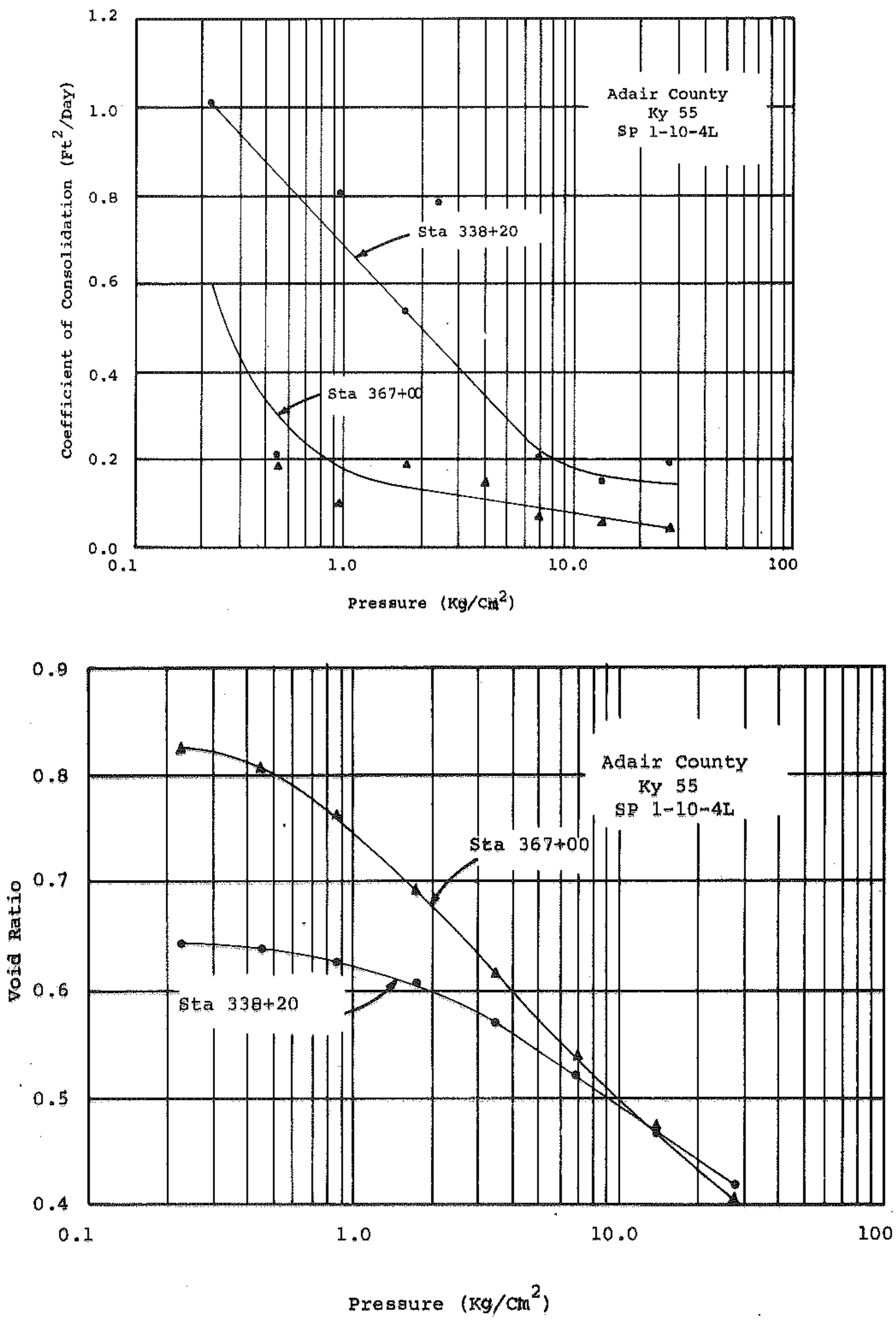

Figure 11-2. Void Ratio-Log Pressure Curves and Coefficient of Consolidation-Log Pressure Curves.

any significant difficulties with regard to the construction of the embankment in this area. However, positive measures must be taken to make sure that the embankment does not become saturated with waters that may enter it from both the Osage deposits at the base as well as from the Meramec Limestones that form the valley walls.

SUMMARY AND DISCUSSION

SUMMARY AND DISCUSSION

It is felt that a review of the seven landslide sites discussed in this report and an eighth site on the Mountain Parkway in Clark County reveals some very signigicant points. The reoccurrence of certain features concerning the slides suggest that the difficulties in the state can be readily classified. Because of this repetitive nature, it is possible to be alerted to the possibility of slides in certain situations early in the planning and design stages of the facility to be constructed.

Enough cannot be said concerning the necessity for having adequate soils and geological information concerning possible routes for highway locations. A review of the discussions on the individual slides contained in this report suggests that there are certain troublesome geologic formations. The slides on Western Kentucky Parkway at Mile Post 75 and Mile Post 83 both occurred in materials of the Tradewater Formation. The slides seem to be associated with subsurface seepage waters moving along impervious shales and underclays. Landslide conditions appear to correlate extremely well with the underclays of the Breathitt Formation as suggested by the discussions on the slides on US 23 in Lawrence County, I 64 in Boyd County, and US 119 in Bell County. The slide at Mile Post 20 on the Bluegrass Parkway occurred in shales of the Osgood formation. Many geologists correlate the Osgood in Western Kentucky with the Crab Orchard Formation, which is mapped in the eastern portion of the State. The slide under study on the Mountain Parkway in Clark County is thought to have occurred in the Crab Orchard Formation and therefore to be associated with the same material involved in the Bluegrass Parkway slides. It appears, then, that if an engineering structure is to involve these three geological formations--the Tradewater, the Breathitt, and the Osgood or Crab Orchard--the planning and design engineers should be extremely cautious in the location and design of the facilities. Proper consideration of the geological and topographical conditions can minimize to a great extent the possibility of trouble at future times. A geological mapping program, unparalleled in the United States, is in progress in the State. This program is providing information, in the form of geological quadrangle maps, concerning the geology of Kentucky which can be extremely useful to engineers in planning and designing engineering structures.

Seven of the eight slides under study have occurred in sidehill, cut-fill sections. The slides have been of the slip type and suggest the extreme difficulty involved with such a cross section. It is evident in six of these situations that the embankment portion of the section has in effect acted as a dam to significantly alter the groundwater seepage patterns. As illustrated in Figure 1, seepage zones along certain geologic

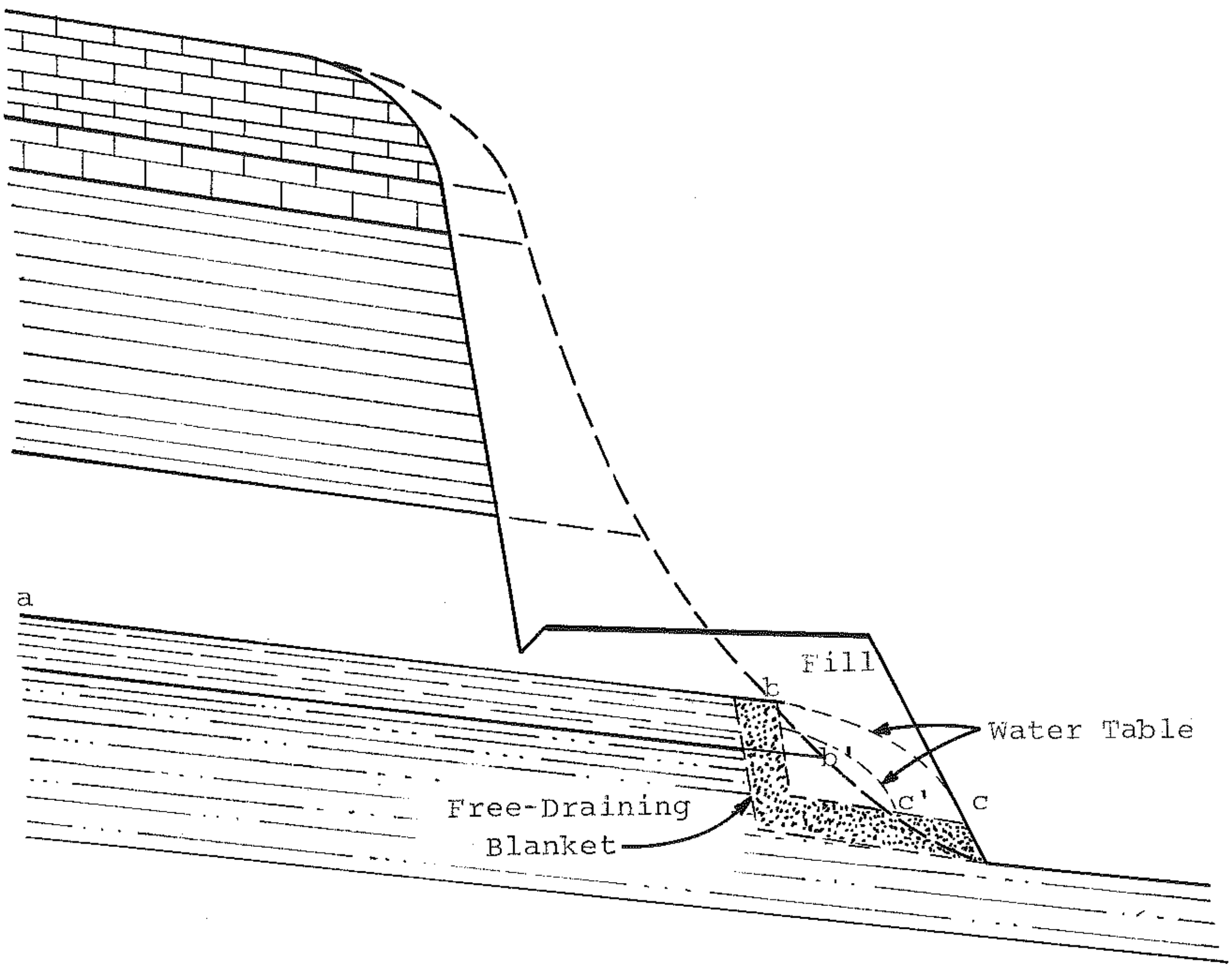

Figure 1. Side-Hill Cut-Fill Section Showing Damming Effect of Fill.

formations, illustrated by line $\overline{ab}$, normally outcrop on the natural slope and cause no signigicant difficulty. However, when the fill section of the highway is placed, this seepage water may be blocked and the water table in the fill could possibly rise with time to a position indicated by line $\overline{bc}$. The rise of the water table in the embankment causes the fill material to become saturated and to lose strength and causes excess pore pressures to develop, which also tends to decrease the strength of the fill material. The zone at the toe of the embankment slope may be weakened to such a point that it is unable to withstand the forces involved and thereby cause a slope failure. It such situations are anticipated, as in the Tradewater, Breathitt, and Osgood or Crab Orchard Formations, some type of corrective action should be provided in the original design. A possible solution is to design and construct a free-draining blanket beneath the embankment as indicated in Figure 1. Such a blanket would prevent the water table from rising above a line indicated by $\overline{b'c'}$ and would protect and maintain the shearing resistance of the toe of the embankment, and thus critical situations with regard to slope stability would not be likely to develop.

In other situations, as on I 64 in Bath County, the source of the difficulty was a weak foundation which was overloaded by a high embankment. If such weak foundation materials are overstressed, a bearing capacity failure will occur and thereby leave the embankment unsupported. The otherwise safe embankment would then crack and fail causing trouble at the roadway elevation. Failure of the embankment may also be caused by excessive settlements occurring within foundation materials. If such settlements are sufficiently large, the embankment may be distorted to such an extent that it will fail.

The importance of locating the highway facility properly with respect to the slope of the bedding planes of the geological deposits is illustrated in Figure 2. The difficulty encountered on US 119 in Bell County is illustrated in Figure 2a. In many topographical situations, the natural ground slopes often exist at near-equilibrium conditions. If material is excavated, as shown in Figure 2a, there is a natural tendency for the material to slip along the bedding planes into the excavated area. This situation may be further aggravated by the presence of seepage water on certain bedding planes, causing a reduction in strength in these areas. If the tilt of the bedding planes is sufficient and (or) the strength of material is not adequate, the material in the highwall of the embankment must fail and move into the excavated area. If, on the other hand, the highway had been located on the other side of the slope, as illustrated in Figure 2b, such difficulties may have been avoided. If there is any tendency for movement in Figure 2b, this

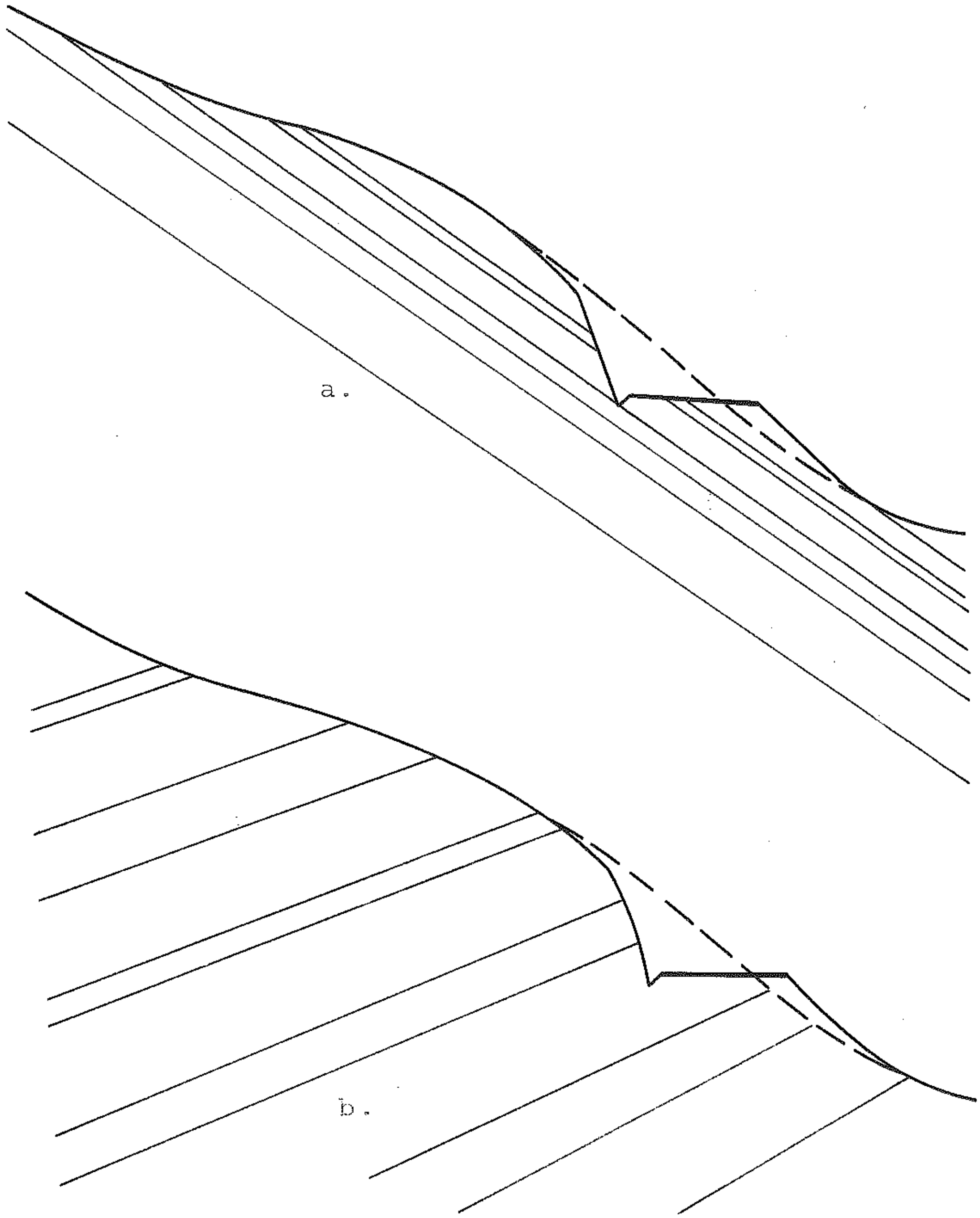

Figure 2. Cut-Fill Section Showing Effect of Dip of Bedding Planes.

movement is along the bedding planes and into the hill side. Since the slope is in natural equilibrium, the construction of the highway may not upset this equilibrium and cause a landslide problem.

A knowledge of the slope of the bedding planes is also important in designing a drainage system to adequately divert subsurface waters and to protect the embankment. If the bedding plane slopes toward the highway location, as shown in Figure 1 and Figure 2a, the quantities of water to be collected and discharged by any drainage system can be enormous. If the slope of the bedding planes is away from the highway location, as in Figure 2b, subsurface waters have a tendency to drain away from the embankment and therefore the drainage system required to protect side-hill embankments need not necessarily be elaborate.

It is interesting to note that many of the slides which have occurred as a result of the damming effect of a side-hill embankment have been located in the transition zone between full cut and full fill sections. In addition to the natural subsurface waters which may be impounded by the side-hill fill, additional waters from the cut section may be finding a way into the embankment in this transition area. Figure 3 is a photograph obtained from a construction project on I 75 south of the Richmond Bypass. Although specifications require that any depressions requiring refilling to the designated rock line will be

Figure 3. Photograph of Rock Cut Construction on I 75 Near Richmond, Kentucky.

provided with positive drainage, the figure suggests some doubt that this is being accomplished. The coarse refill material apparent in the photograph should be choked with a fine-grained impervious material to produce a smooth grading line upon which to place the dense graded aggregate base. The quantity of water which can be collected from a pavement section in a long rock cut could be enormous. In addition, waters moving down the ditches may actually be lost from the ditches and enter the subgrade or embankment beneath the pavement. This situation has been noted to exist in the rock cut adjacent to the slide in Clark County. Surface waters are disappearing from the side ditches through cracks or joints in the bedrock and apparently are moving beneath the pavement and into the side-hill fill.

The importance of situational surveys for highway locations cannot be over emphasied. It is desirable to prepare columnar sections and to identify each of the horizons or layers geologically. Much geological and soil date are available for Kentucky. As a result of the geological mapping program, there is already a rather widespread coverage of the State by geological quadrangle maps. Eventually, as the mapping program comes to a conclusion, there will be complete coverage of the State. Use should also be made of the soils and geology information contained in the soil survey reports and maps prepared by the

Soil Conservation Service and published by the U.S. Department of Agriculture. In addition, the Division of Research has prepared a summary of the engineering properties of soils mapped in Kentucky on the basis of date contained in the files as of this time (see Research Report "Engineering Properties of Kentucky Soils" by R. C. Deen, August, 1966). It is extremely important that design and location engineers refer to geological and soils information for guidance and forewarning signs of potential landslides. Failure to do so in the early planning and design stages of a highway compounds construction and maintenance problems. It is recommended that complete files of the geological maps and the soils maps and reports be maintained by the Division of Materials and the Division of Design and that certain persons in both of these divisions be responsible for a critical review of all geological and soils information concerning any highway construction at an early stage of planning and design.

APPENDIX

THE COMPUTER AS A TOOL IN SLOPE STABILITY ANALYSES

Consider the stability of the slope-forming material contained within the boundary abcdefgh (see Figure 1). The resultants of the forces acting on the vertical sides of the slice, bcfg, are assumed to be equal, opposite, and coincident. Using the condition of equilibrium $\Sigma F_N = 0$ for a slice---that is, the summation of the forces acting normal to the surface, abcd, is zero---it is found that

$$W \cos \alpha = (\sigma' + u)A = (\sigma' + u)b \sec \alpha \qquad 1$$

where W = total weight of the slice,

α = angle between the vertical and the normal to the surface bc,

σ' = effective stress acting on the surface bc,

u = pore pressure acting on the surface bc,

A = cross-sectional area along the surface bc, and

b = width of the slice.

Solving for the effective stress,

$$\sigma' = \frac{W \cos\alpha - ub \sec\alpha}{b \sec \alpha} = \frac{W}{b} \cos^2 \alpha - u \qquad 2$$

Turning attention to the entire mass under investigation, it is noted that the sum of the moments due to forces which

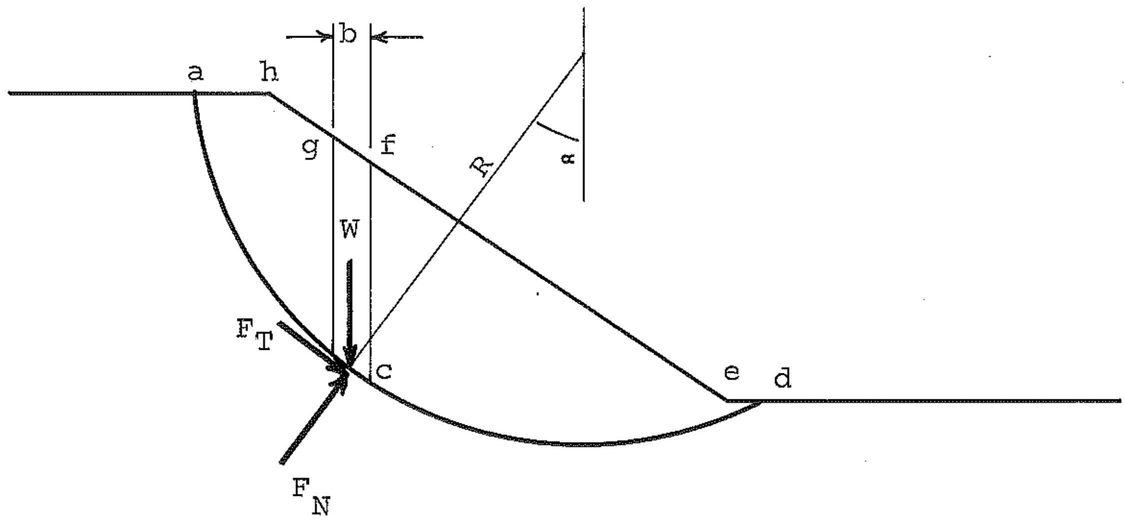

Figure 1. Slope Stability Analysis By Circular Arc Technique and Method of Slices.

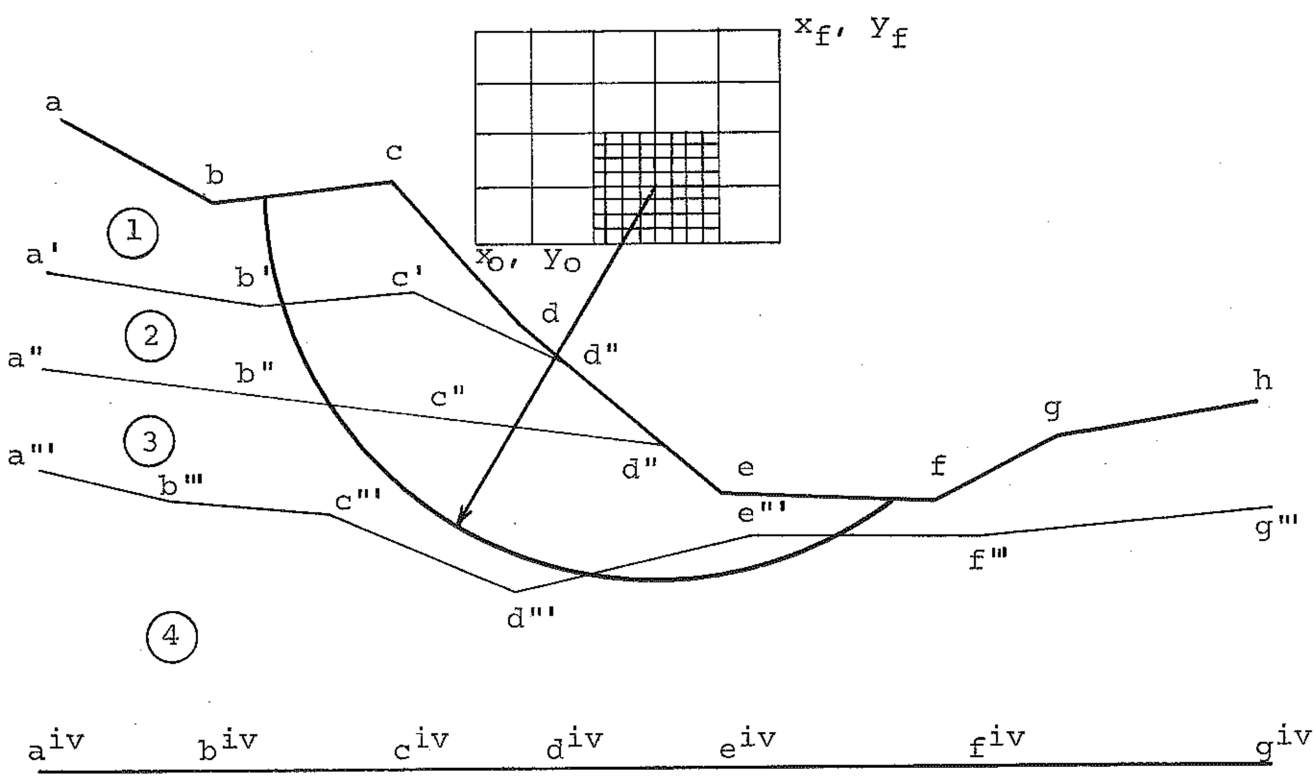

Figure 2. Typical Section for Slope Stability Analysis.

tend to cause rotation around the center of the arc, ΣM_D, are given by

$$\Sigma M_D = R \sum_{i=1}^{n} W_i \sin \alpha_i \qquad 3$$

where R = radius of the circular arc abcd and

n = number of slices into which the mass has been divided.

The summation of the moments due to forces tending to resist this rotation, ΣM_R, is given by

$$\Sigma M_R = R \sum_{i=1}^{n} s_{r_i} A_i \qquad 4$$

where s_r = shearing strength available to resist movement along the circular arc.

Assuming that the shear strength is described by Coulomb's Law,

$$s_{r_i} = c'_{a_i} + \sigma'_i \tan \phi'_{a_i}, \qquad 5$$

where c'_a = effective cohesion, and

ϕ'_a = effective angle of internal friction,

Equation 4 becomes

$$\Sigma M_R = R \sum_{i=1}^{n} (c'_{a_i} + \sigma'_i \tan \phi'_{a_i}) b_i \sec \alpha_i . \qquad 6$$

Considering the equilibrium of the mass with respect to rotation about the center of the arc, the factor of safety, F, is given by

$$F = \frac{\Sigma M_R}{\Sigma M_D} \qquad 7$$

or

$$F = \frac{\sum_{i=1}^{n} \left[c' a_i b_i + (W_i \cos^2 \alpha_i - u_i b_i) \tan \phi' a_i \right] \sec \alpha_i .}{\sum_{i=1}^{n} W_i \sin \alpha_i} \qquad 8$$

Equation 8 is an explicit solution for the factor of safety. If other assumptions had been made concerning the forces acting on the vertical sides of the slices, the solution for F would not have been so direct. The result would have been a transcendental equation in F.

Since the location of the critical or potential failure surface is not known a priori, it is necessary to solve Equation 8 for many circular arcs in order to determine the surface with the minimum factor of safety. Significant application has been made of electronic computers in such problems involving a large number of repetitive-type calculations. Accordingly, a program has been prepared for the IBM 7040 to do much of the tedious work of solving Equation 8 for numerous trial locations of the circular arc and to search for the conditions for which the minimum factor of safety was obtained.

The program is extremely versatile in that it is capable of handling a multi-layered soil section bounded by a ground surface boundary of any configuration. The boundaries between the various layers and the water table level can also assume any configuration. All of these lines--that is, the ground surface, the layer boundaries, and the water table--must be approximated by a series of straight line segments. The number of layers which can be handled by the program and the number of points defining these straight line segments is essentially unlimited---being restricted only by the storage capacity of the computer. The extreme coordinates on the ground surface line, the layer boundaries, and the water table must be sufficiently far to the left and right of the slope area to insure that the circular arc assumed by the program will intersect all of these lines within these extremes. The lowest layer boundary must be at such a depth that it will not be intersected by any of the trial circular arcs that the program will attempt to evaluate.

Figure 2 illustrates some of the requirements of the computer program. For example, the layer boundary between Layers 1 and 2 is noted to intersect the slope at point d". Since the computer program requires 1) that the coordinates of the extremities of each boundary layer be such that trial circles will intersect the layer boundary and 2) that each layer boundary be defined by the same number of coordinate points, it is necessary that the boundary between Layers 1 and 2 be assumed to continue along line d"efgh. Figure 3 illustrates how lenses of materials or cores within a dam may be treated. The lense indicated as Layer 6 is bounded on the upper side by line $a^{iv}b^{iv}c^{iv}d^{iv}e^{iv}f^{iv}g^{iv}h^{iv}$ while on the lower side it is bounded $a^{iv}b^{iv}c^{v}e^{iv}f^{iv}g^{iv}h^{iv}$. The material indicated as Layer 2, comprising a portion of the core, is bounded on the upper side by line abc"d"e"f"g"h"i"j" and on the lower side by abc"e"'e"f"f"'h"i"j". This line also represents the upper boundary for Layer 3 with the lower boundary of Layer 3 being given by line abc"e"' f"'h"i"j".

In order to use the computer program, it is necessary to limit the extreme coordinates of the grid system which defines the centers of various trial circular arcs. The extreme coordinates and the size of the grid system in both the x and y directions is read into the program as input data. The maximum and minimum radii which are to be investigated are also specified as well as the increment by which the radius is to be changed going from the minimum to the maximum length. The program has been prepared so that it will either 1) print out the x and y coordinates and the raduis of each trial circle and the calculated factor of safety or 2) will investigate each center point until a least factor of safety for that point is obtained and then print the center point

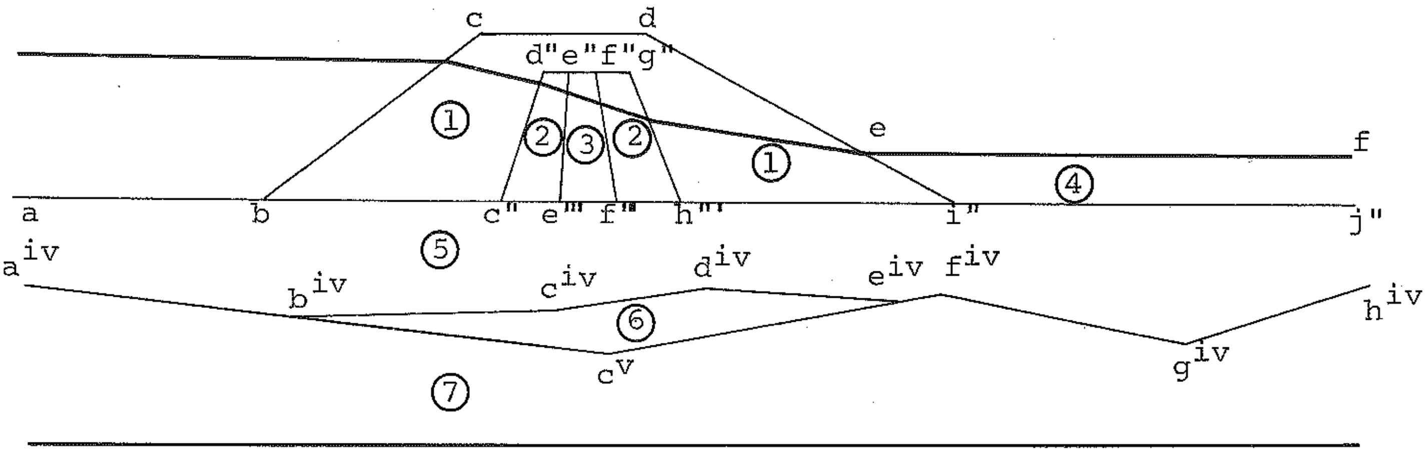

Figure 3. A Section Amenable to Computer Analysis.

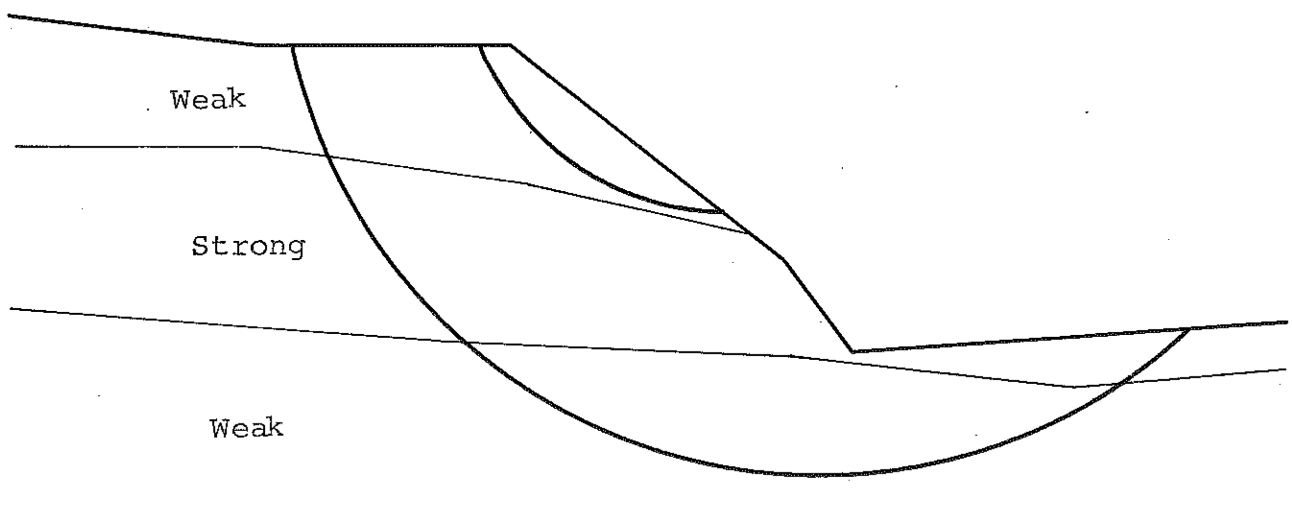

Figure 4. Limitation of Computer Problem.

coordinates and the radius for the minimum factor of safety conditions.

The most efficient way to use the program is to select a rather large grid initially. Large increments are used to vary the radius and the x and y coordinates of the centers of the circular arcs. After the computer has indicated the general location of the critical arc, a smaller grid system, using smaller increments, is used to more precisely define the critical surface and its factor of safety.

A situation is illustrated in Figure 4 which may not be adequately evaluated by the computer program. In this situation, the computer program will determine a minimum factor of safety for a circle which is probably located entirely within the upper and weak layer when, in fact, the critical surface may be more deeply located. If this situation exists, it is necessary to ask the computer program to print out all safety factor computations and then to manually review these and select the surface which is known to be more nearly the critical surface of primary concern.

Another major limitation of the program is that it is not capable of accounting for excess pore pressures directly. In order to account for excess pore pressures to some degree, it is necessary to adjust the water table elevation to give an equivalent height of water equal to the excess pore pressures. This is not a completely desirable situation, and it is hoped that future modifications of the program will permit the direct accounting of the excess pore pressures existing in the cross section.

The Fortran source statements for the computer program as it is currently developed, is given on the following pages. Examples of the input data and the output follow the program.

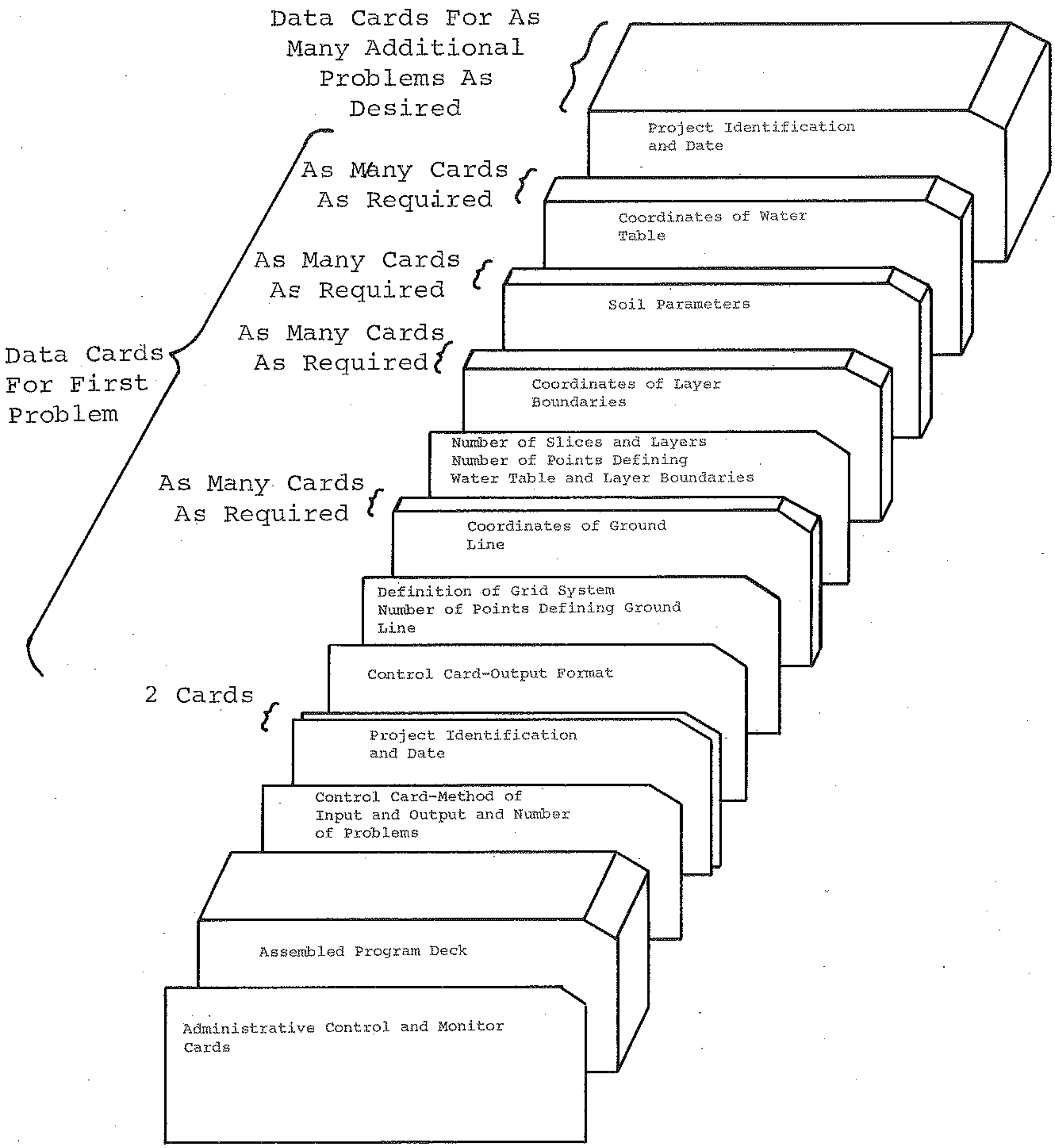

Figure 5. Assembly Order For Slope Stability Program with Data.

```
C     PROGRAM NO. 17-7040-F4
C     TITLE - PROGRAM FOR SLOPE STABILITY ANALYSIS USING MODIFIED TAYLOR
C         EQUATIONS - LAYERED SYSTEM
C     SPECIAL MACHINE REQ. - NCNE
C     SUBROUTINES REQUIRED - SQRT (SQUARE ROOT)
C     KEY WORDS - SLOPE STABILITY, MULTI-LAYERS, MODIFIED TAYLOR
C
C
C     THIS PROGRAM IS DESIGNED TO MAKE A SLOPE STABILITY ANALYSIS OF A
C     MULTI-LAYERED SOIL SYSTEM WITH THE GROUND SURFACE, LAYER BOUNDARY
C     AND WATER TABLE LINES OF ANY CONFIGURATION CONSISTING OF STRAIGHT
C     LINE SEGMENTS. A LAKE RETAINED BY A FILL CAN BE HANDLED BY TREAT-
C     ING THE WATER AS ANCTHER LAYER WITH UNIT WEIGHT= 62.4 LBS/CU.FT.
C     EXCESS PORE PRESSURES CAN BE HANDLED BY ADJUSTING THE WATER TABLE
C     ELEVATICN UPWARD TO AN EQUIVALENT HEIGHT OF WATER. EACH LAYER MUST
C     BE GIVEN A VALUE FOR COHESION, TANGENT MODULUS PHI, AND UNIT
C     WEIGHT. ALL LINES MUST BE GIVEN AS INPUT DATA IN THE FORM OF X AND
C     Y COORDINATES. THE RADIUS IS INCREMENTED FROM AN INITIAL LENGTH
C     UNTIL THE COMPUTED FACTOR OF SAFETY FOR THAT CENTER POINT BEGINS
C     TO INCREASE. THE CENTER POINT IS THEN INCREMENTED IN THE Y DIREC-
C     TION FRCM THE INITIAL TO FINAL GRID POINTS, THEN INCREMENTED IN
C     THE X DIRECTION TO ITS FINAL GRID POINT. THE CENTER POINT COORDI-
C     NATES, RADIUS LENGTH, AND THE LEAST FACTOR OF SAFETY FOR EACH CEN-
C     TER POINT ARE PRINTED OUT. AFTER THE ENTIRE CENTER POINT GRID SYS-
C     TEM HAS BEEN INVESTIGATED, THE SMALLEST FACTOR OF SAFETY IS PICKED
C     OUT AND PRINTED OUT ALCNG WITH THE CORRESPONDING RADIUS LENGTH,
C     AND CENTER POINT CCORDINATES. AFTER THE SMALLEST FACTOR OF SAFETY
C     IS PRINTED, THE GROUND POINT COORDINATES, WATER TABLE COORDINATES,
C     THE LAYER NUMBER, SOIL PROPERTIES FOR EACH LAYER, AND THE LAYER
C     BOUNDARY LINE COORDINATE POINTS ARE PRINTED OUT. ALL WEIGHT COMPU-
C     TATIONS ARE MADE ASSUMING A ONE FOOT CROSS-SECTIONAL THICKNESS
C     INTO THE PAGE OF THE DIAGRAM. ANY FORM OF INPUT MAY BE USED BY
C     PUNCHING A 0 THROUGH 4 FOR TAPE, 5 FOR CARDS, IN COLUMN 4 OF THE
C     FIRST DATA CARD. ANY FORM OF OUTPUT MAY BE USED BY PUNCHING 0 THRU
C     4 FOR TAPE, 6 FOR PRINT, AND 7 FOR PUNCHED CARDS IN COLUMN 8 OF
C     THE FIRST DATA CARD. THE NUMBER OF DATA DECKS IS PUNCHED IN COL-
C     UMNS 9 THRU 12 (RIGHT HAND JUSTIFIED) OF THE FIRST DATA CARD. ANY
C     NUMBER OF SITUATIONS MAY BE INVESTIGATED IN ONE COMPUTER SUBMIS-
C     SION BY MAKING A COMPLETE DATA DECK FOR EACH SITUATION. THE VARI-
C     ABLE NAMED SOIL IS A GIMIC FOR ROUTING THE FACTOR OF SAFETY STOR-
C     AGE AND PRINTING SYSTEM. IF SOIL HAS A VALUE OF 1., ONLY THE MINI-
C     MUM FACTOR OF SAFETY IS FOUND AND PRINTED FOR EACH GRID CENTER
C     POINT. IF SOIL HAS A VALUE OF 0., THE FACTOR OF SAFETY FOR EACH
C     RADIUS INCREMENT FOR EACH GRID CENTER POINT IS COMPUTED AND
C     PRINTED.
C
C     RESTRICTIONS**********************************
C     1. ALL LINES MUST BE DEFINED OR APPROXIMATED BY STRAIGHT LINE
C        SEGMENTS.
C     2. GROUND SURFACE AND WATER TABLE LINES MAY HAVE ANY DESIRED NUM-
C        BER OF STRAIGHT LINE SEGMENTS.
C     3. ALL LAYER BOUNDARY LINES MUST CONTAIN THE SAME NUMBER OF
C        COORDINATE POINTS.
C     4. THERE MUST BE THE SAME NUMBER OF LAYER BOUNDARY LINES AS THERE
C        ARE LAYERS OF SOIL WITH THE LOWEST LAYER BOUNDARY BEING SO LOW
C        IN THE Y DIRECTION THAT THE LARGEST RADIUS CAN NEVER BE TAN-
C        GENT, NOR INTERSECT, THE LAYER BOUNDARY LINE.
C     5. THE TERMINI FOR THE GROUND SURFACE, WATER TABLE, AND ALL LAYER
C        BOUNDARY LINES SHOULD HAVE THE SAME X COORDINATE VALUES AND
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C         MUST EXTEND FAR ENOUGH IN THE X DIRECTION ON EACH SIDE OF THE
C         CENTER POINT GRID SYSTEM SO THAT THE LARGEST RADIUS FROM ANY
C         CENTER POINT WILL BE WITHIN THE ZONE OF THE X TERMINI.
C
C
C      PRCGRAM FOR SLOPE STABILITY ANALYSIS
C
C                        USING
C
C          MCDIFIED TAYLCR EQUATIONS
C
      DIMENSION X(50),Y(50),XC(50),YC(50),XOS(1000),YOS(1000),ROS(1000),
     1FSS(1000),XWT(50),YWT(50),YG(50),YT(50),YTT(50,50),W(1000),WT(50),
     2SLCPE(50),YINT(50),SINA(50),COSA(50),CO(50),TANPHI(50),LC(50),
     3XLS(20,20),YLS(20,20),WW(50),WE(50)
      F=0.
C     IN=METHCD OF INPUT
C     IOUT= METHCD CF CUTPUT
C     NOP= NUMBER OF SETS CF PROBLEMS
      READ(5,1)IN,ICUT,NCP
    1 FORMAT(3I4)
      DO9C0 INC=1,NOP
      READ(IN,910)ID1,ID2,ID3,ID4,IRT1,IRT2,ICO1,ICO2,IAN,IPNF1,IPNF2,IP
     1NF3,IPNS1,IPNS2,IPNS3
  910 FORMAT(4A6,2A5,2A6,A6,2A6,A2,2A6,A2)
      READ(IN,912)MCNTH,KDAY,KYEAR
  912 FORMAT(3A2)
      WRITE(IOUT,911)ID1,ID2,ID3,ID4,IRT1,IRT2,ICO1,ICO2,IPNF1,IPNF2,IPN
     1F3,IPNS1,IPNS2,IPNS3,IAN,MONTH,KDAY,KYEAR
  911 FORMAT(1H1,////,21X,4A6,//,21X,2A5,//,21X,2A6,7H COUNTY,//,21X,11H
     1PRCJECT NO.,2/6,A2,/,32X,2A6,A2,//,21X,12HANALYSIS NO.,A6,//,21X,6
     2HCATE  ,A2,1H/,A2,1H/,A2)
C     SOIL IS A SAFETY FACTOR ROUTING GIMIC.
      READ(IN,905)SCIL
  905 FORMAT(F5.0)
C     ISTART= BEGINNING X CCORDINATE CF CENTER OF CIRCLE
C     JSTART= BEGINNING Y CCORCINATE OF CENTER OF CIRCLE
C     IFIN= ENDING X CCORDINATE OF CENTER OF CIRCLE
C     JFIN= ENDING Y CCCRDINATE OF CENTER OF CIRCLE
C     IRS= BEGINNING RACIUS LENGTH
C     IRF= ENDING RADIUS LENGTH
C     NO= NUMBER OF POINTS ON GROUND SURFACE
C     FS1=BEGINNING FACTOR OF SAFETY VALUE
C     IDEL1=INCREMENT FOR X AXIS FOR CENTER OF CIRCLE
C     IDEL2=INCREMENT FCR Y AXIS FOR CENTER OF CIRCLE
C     IDEL3=INCREMENT FCR RADIUS OF CIRCLE
      READ(IN,2)ISTART,JSTART,IFIN,JFIN,IRS,IRF,NO,FS1,IDEL1,IDEL2,IDEL3
    2 FORMAT(6I10,I3,F6.0,3I3)
C     READ IN GROUND COORDINATE POINTS.
C     X(I)=X CCORDINATE CF POINT ON GROUND SURFACE
C     Y(I)=Y COORDINATE OF POINT ON GROUND SURFACE
      READ(IN,3)( X(I),Y(I) ,I=1,NO)
    3 FORMAT(2F10.3)
      WRITE(ICUT,4)
    4 FORMAT(1H1,18X,24HSLCPE STABILITY ANALYSIS,//,27X,5HUSING,//,18X,2
     15HMODIFIED TAYLCR EQUATIONS,///,11X,41HCOORDINATES OF THE   RADIUS
     2 OF  FACTOR OF,/,10X,40HCENTER OF THE CIRCLE THE CIRCLE   SAFETY,/
     3/,12X,27HX(FEET)    Y(FEET)     (FEET),//)
C     NSLICE= NUMBER OF SLICES
```

```
C      NOWT= NUMBER OF POINTS DEFINING THE WATER TABLE
C      JJ= COORDINATE POINT NUMBER ON GROUND SURFACE WHICH IS USED AS A
C      RADIUS CHECK.
C      NL= NUMBER OF LINES MARKING THE BOUNDARIES BETWEEN LAYERS OF SOIL
C      NOPL= NUMBER OF POINTS DEFINING LAYER BOUNDARY LINE
       READ(IN,5)NSLICE,NOWT,JJ,NL,NOPL
     5 FORMAT(5I4)
C      READ IN LAYER BOUNDARY COORDINATE POINTS.
C      XLS(K,M)=X COORDINATE ON A LAYER BOUNDARY LINE.
C      YLS(K,M)=Y COORDINATE ON A LAYER BOUNDARY LINE.
       READ(IN,16)(( XLS(K,M),YLS(K,M) ,K=1,NOPL),M=1,NL)
    16 FORMAT(2F6.0)
C      CO(M)= COHESION VALUE FOR A PARTICULAR LAYER.
C      TANPHI(M)=VALUE OF TANGENT MODULUS PHI FOR A PARTICULAR LAYER.
C      WT(M)=UNIT WEIGHT OF SOIL FOR A PARTICULAR LAYER.
       READ(IN,15)(CO(M),TANPHI(M),WT(M),M=1,NL)
    15 FORMAT(F5.0,F5.4,F5.0)
       FS=FS1
       I=JJ
       KKK=0
       XE=X(I)
       YE=Y(I)
C      READ IN WATER TABLE COORDINATE POINTS.
C      XWT(II)=X COORDINATE FOR A PARTICULAR WATER TABLE POINT.
C      YWT(II)=Y COORDINATE FOR A PARTICULAR WATER TABLE POINT.
       READ(IN,3)( XWT(II),YWT(II) ,II=1,NOWT)
       DO899 IXO=ISTART,IFIN,IDEL1
       DO899 IYO=JSTART,JFIN,IDEL2
       DO888 IRO=IRS,IRF,IDEL3
C       BEGIN INITIALIZING SUBSCRIPTED VARIABLES TO 0.
       DO 1005 J=1,NSLICE
       DO 1005 M=1,NL
C      YTT(M,J)= COORDINATES OF INTERSECTION OF CENTER OF SLICE WITH A
C      BOUNDARY LAYER LINE
  1005 YTT(M,J)=0.
       DO 690 J=1,NSLICE
C      YG(J)= Y COORDINATE OF CENTER OF SLICE AND GROUND SURFACE
       YG(J)=0.
C      YT(J)= Y COORDINATE OF CENTER OF SLICE AND WATER TABLE
       YT(J)=0.
C      LC(J)= STORAGE GIMIC
       LC(J)=0.
C      W(J)= WEIGHT OF WET SOIL IN SLICE
       W(J)=0.
C      WE(J)= EFFECTIVE WEIGHT OF SOIL
       WE(J)=0.
C      WW(J)= WEIGHT OF HEIGHT OF WATER IN A SLICE
   690 WW(J)=0.
C       END  INITIALIZING SUBSCRIPTED VARIABLES TO 0.
       XO=IXO
       YO=IYO
       RO=IRO
       Z=SQRT((XO-XE)**2+(YO-YE)**2)
       IF(RO.LE.Z) GO TO 888
       N=NC-1
       DO101 I=1,N
C      SLOPE(I)= SLOPE OF GROUND SURFACE LINE
       SLOPE(I)=(Y(I+1)-Y(I))/(X(I+1)-X(I))
C      YINT(I)=INTERSECTION OF GROUND SURFACE LINE EXTENDED TO Y AXIS
       YINT(I)=Y(I)-SLOPE(I)*X(I)
```

```
  101 CONTINUE
      DO102 I=1,N
      A=1.+SLOPE(I)**2
      B=SLOPE(I)*(YINT(I)-YO)-XO
      C=(YINT(I)-YO)**2+XO**2-RO**2
      TEST=B**2-A*C
      IF(TEST.GT.0.)GO TO 103
      GO TO 888
  103 XP=(-B+SQRT(TEST))/A
      XM=(-B-SQRT(TEST))/A
      IF(.NOT.(XM.GE.X(I).AND.XM.LE.X(I+1)))GO TO 105
C     XA= X COORDINATE OF INTERSECTION OF CIRCLE WITH GROUND SURFACE
C     NEAREST Y AXIS
      XA=XM
C     YA= YCOORDINATE CORRESPONDING TO XA
      YA=SLOPE(I)*XA+YINT(I)
  105 IF(XP.GE.X(I).AND.XP.LE.X(I+1))GO TO 106
  102 CONTINUE
      GO TO 109
C     XB = X COORDINATE OF INTERSECTION OF CIRCLE WITH GROUND SURFACE
C     FURTHEST FROM Y AXIS
  106 XB=XP
C     YB= YCOORDINATE CORRESPONDING TO XB
      YB=SLOPE(I)*XB+YINT(I)
  109 SLICE=NSLICE
C     BB= WIDTH OF SLICE
      BB=(XB-XA)/SLICE
C     BEGIN CALCULATIONS TO DETERMINE X COORDINATE VALUES FOR THE INTER-
C     SECTION OF EACH SLICE WITH SLIP CIRCLE.
C     XC(J)= X COORDINATE OF INTERSECTION OF CENTER OF SLICE AND CIRCLE
      XC(1)=XA+BB/2.
      DO 201 J=2,NSLICE
  201 XC(J)=XC(J-1)+BB
C      END  CALCULATIONS TO DETERMINE X COORDINATE VALUES FOR THE INTER-
C     SECTION OF EACH SLICE WITH SLIP CIRCLE.
C     BEGIN CALCULATIONS TO DETERMINE Y COORDINATE VALUES FOR THE INTER-
C     SECTION OF EACH SLICE WITH SLIP CIRCLE.
      DO 202 J=1,NSLICE
C     YC(J)= Y COORDINATE OF INTERSECTION OF CENTER OF SLICE AND CIRCLE
      YC(J)=YO-SQRT(RO**2-(XC(J)-XO)**2)
C      END  CALCULATIONS TO DETERMINE Y COORDINATE VALUES FOR THE INTER-
C     SECTION OF EACH SLICE WITH SLIP CIRCLE.
      SINA(J)=(XO-XC(J))/RO
  202 COSA(J)=(YO-YC(J))/RO
      DO204 J=1,NSLICE
      DO 208 I=1,N
      IF(XC(J).GE.X(I).AND.XC(J).LE.X(I+1))GO TO 301
  208 CONTINUE
C     BEGIN CALCULATIONS TO DETERMINE THE INTERSECTION OF THE SLIP
C     CIRCLE WITH THE GROUND SURFACE.
  301 YG(J)=SLOPE(I)*XC(J)+YINT(I)
  204 CONTINUE
C      END  CALCULATIONS TO DETERMINE THE INTERSECTION OF THE SLIP
C     CIRCLE WITH THE GROUND SURFACE.
      NN=NOWT-1
      DO205 J=1,NSLICE
      DO206 II=1,NN
      IF(XC(J).GT.XWT(II).AND.XC(J).LE.XWT(II+1))GO TO 302
      IF(XC(J).GT.XWT(NOWT))GO TO 305
  206 CONTINUE
```

```
C       BEGIN CALCULATIONS TO DETERMINE THE INTERSECTION OF THE SLIP
C       CIRCLE WITH THE WATER TABLE.
  305 YT(J)=YG(J)
      GO TO 205
  302 YT(J)=((YWT(II+1)-YWT(II))/(XWT(II+1)-XWT(II)))*(XC(J)-XWT(II))+
     1YWT(II)
  205 CONTINUE
C        END  CALCULATIONS TO DETERMINE THE INTERSECTION OF THE SLIP
C       CIRCLE WITH THE WATER TABLE.
      NLPL=NCPL-1
      DO 1001 J=1,NSLICE
      DO 1001 M=1,NL
      DO 1001 K=1,NLPL
      IF(XC(J).GT.XLS(K,M).AND.XC(J).LE.XLS(K+1,M))GO TO 303
      GO TO 1001
C       BEGIN CALCULATIONS TO DETERMINE THE INTERSECTION OF THE SLIP
C       CIRCLE WITH THE LAYER BOUNDARY LINES.
  303 YTT(M,J)=((YLS(K+1,M)-YLS(K,M))/(XLS(K+1,M)-XLS(K,M)))*(XC(J)-XLS
     1(K,M))+YLS(K,M)
 1001 CONTINUE
C        END  CALCULATIONS TO DETERMINE THE INTERSECTION OF THE SLIP
C       CIRCLE WITH THE LAYER BOUNDARY LINES.
      EFF=0.
C       TOP = RESISTING WEIGHT OR MOMENT
      TOP=0.
C       BOT= OVERTURNING WEIGHT OR MOMENT
      BOT=0.
C       BEGIN CALCULATIONS TO DETERMINE THE EFFECTIVE WEIGHT FOR EACH
C       SLICE.
      DO 600 J=1,NSLICE
      DO 601 M=1,NL
  601 IF(YTT(M,J).NE.0.)GO TO 608
  608 MT=M
      SL=YG(J)-YC(J)
      DO 602 M=1,NL
      IF(YTT(M,J).EQ.0.)GO TO 602
      SL1=YG(J)-YTT(M,J)
      IF(SL1.GE.SL)GO TO 603
  602 CONTINUE
  603 IF(M.EQ.1)GO TO 604
      IF(YTT(M-1,J).EQ.0.)GO TO 604
      MB=M-1
      M=MT
      W(J)=W(J)+BB*(YG(J)-YTT(M,J))*WT(M)
      SL2=YTT(M,J)-YTT(M+1,J)
      SL3=YTT(M,J)-YC(J)
      IF(SL2.GE.SL3)GO TO 606
      MB=MB-1
      DO 605 M=MT,MB
  605 W(J)=W(J)+BB*(YTT(M,J)-YTT(M+1,J))*WT(M+1)
      M=MB+1
  606 W(J)=W(J)+BB*(YTT(M,J)-YC(J))*WT(M+1)
      LC(J)=M+1
      GO TO 641
  604 W(J)=W(J)+BB*(YG(J)-YC(J))*WT(M)
      LC(J)=M
  641 IF(YT(J).LT.YC(J))GO TO 642
      WW(J)=(YT(J)-YC(J))*62.4*BB
      WE(J)=W(J)-WW(J)
      GO TO 600
```

```
  642 WE(J)=W(J)
  600 CONTINUE
C       END  CALCULATIONS TO DETERMINE THE EFFECTIVE WEIGHT FOR EACH
C      SLICE.
C      BEGIN CALCULATIONS TO DETERMINE THE FACTOR OF SAFETY.
       DO 612 J=1,NSLICE
       EFF=WE(J)*COSA(J)**2
       M=LC(J)
       TOP=TOP+(CO(M)*BB+EFF*TANPHI(M))/COSA(J)
  612 BOT=BOT+WE(J)*SINA(J)
C      F= COMPUTED FACTOR OF SAFETY FOR PARTICULAR CIRCLE
       F=TOP/BOT
       IF(SOIL.EQ.0.)GO TO 903
  902 IF(F.LT.FS)GO TO 903
       GO TO 889
  903 FS=F
       IF(SOIL.EQ.0.)GO TO 904
       GO TO 888
  904 WRITE(IOUT,6)XO,YO,RO,FS
    6 FORMAT(10X,3F10.3,F11.3)
       IF(IXO.EQ.IFIN.AND.IYO.EQ.JFIN.AND.IRO.EQ.IRF)GO TO 703
       IF(IRO.EQ.IRF)GO TO 899
C       END  CALCULATIONS TO DETERMINE THE FACTOR OF SAFETY.
C      NOW INCREMENT RADIUS.
  888 CONTINUE
  889 DEL=IDEL3
       RO=RO-DEL
       WRITE(IOUT,6)XO,YO,RO,FS
       KKK=KKK+1
C      NOW STORE LEAST FACTOR OF SAFETY AND RELATED ANSWERS FOR GRID PT.
       XOS(KKK)=XO
       YOS(KKK)=YO
       ROS(KKK)=RO
       FSS(KKK)=FS
       FS=FS1
C      NOW INCREMENT TO THE NEXT GRID CENTER POINT.
  899 CONTINUE
       KK=KKK
C      NOW SEARCH FOR THE SMALLEST FACTOR OF SAFETY FOR ALL GRID POINTS.
       DO 700 K=1,KK
       IF(FSS(K).GE.FSS(1))GO TO 700
  701 FSS(1)=FSS(K)
       XOS(1)=XOS(K)
       YOS(1)=YOS(K)
       ROS(1)=ROS(K)
  700 CONTINUE
  702 WRITE(IOUT,7)XOS(1),YOS(1),ROS(1),FSS(1)
    7 FORMAT(1H1,20X,33HDATA FOR MINIMUM FACTOR OF SAFETY,/// 9X,62HX(FE
     1ET)             Y(FEET)              R(FEET)              FS(MINIMUM),///, 5X
     2,F11.2,6X,F11.2,6X,F11.2,6X,F11.2)
  703 WRITE(IOUT,8)
    8 FORMAT(1H1,10X,34HCOORDINATES OF ORIGINAL GROUNDLINE,//,16X,7HX(FE
     1ET), 5X,7HY(FEET),//)
       WRITE(IOUT,9)(X(I),Y(I),I=1,NO)
    9 FORMAT( 9X,F15.3,7X,F15.3)
       WRITE(IOUT,10)
   10 FORMAT(1H1,15X,25HCOORDINATES OF WATERTABLE,//,16X,7HX(FEET),15X,7
     1HY(FEET),//)
       WRITE(IOUT,9)(XWT(II),YWT(II),II=1,NOWT)
       DO 704 M=1,NL
```

```
      WRITE(IOUT,11)M,CO(M),TANPHI(M),WT(M)
 11   FORMAT(1H1,10X,24HLAYER NUMBER           ,I5,///,11X,20HCOHESION(
     1LB/SQFT)    ,F15.5,/,11X,20HTANGENT PHI         ,F15.5,/,11X,20HUNI
     2T WEIGHT(LB/CUFT),F15.5)
      WRITE(IOUT,12)
 12   FORMAT(/////,11X,34HCOORDINATES OF LAYER BOUNDARY LINE,//,13X,7HX(
     1FEET),15X,7HY(FEET),//)
704   WRITE(IOUT,13)(XLS(K,M),YLS(K,M),K=1,NOPL)
 13   FORMAT(6X,F15.3,7X,F15.3)
900   CONTINUE
      CALL EXIT
      END
```

EXAMPLE OF INPUT DATA

```
   5    6    1  ——— Control Card-Method of Input and Output and Number of Problems
1275                       3TC1          CAMPBELL          0 } ——— Project Identification and Date
092966
   1 ——— Control Card-Output Format
     5067        5020        5167        5100 ——— Definition of Grid System     40        180   610000.  10  10  10
                                                    Number of Points Defining Ground Line
      0.        5000. )
   5000.        5000. |
   5047.        5000. |
   5197.        4953. } ——— Coordinates of Ground Line
   5258.        4949. |
   7000.        4839. )
  10    6    3    3    6 ——— Number of Slices and Layers
                            Number of Points Defining Water Table and Layer Boundaries
      0   5060 )
   4955   4961 |
   5135   4957 }
   5197   4953 |
   5259   4949 |
   7000   4839 )
      0   5040 )
   4955   4941 |
   5047   4939 }      Coordinates of Layer Boundaries
   5153   4937 |
   5256   4933 |
   7000   4831 )
      0   1000 )
   1000   1000 |
   3000   1000 }
   5000   1000 |
   7000   1000 |
   9000   1000 )
   300.5317   127 )
   425.2679   127 }     Soil Parameters
     0.4663   127 )
      0.        5040. )
   4955.        4941. |
   5047.        4939. }     Coordinates of Water Table
   5153.        4937. |
   5256.        4933. |
   7000.        4831. )
```

EXAMPLE OF OUTPUT DATA

I275

3T01

CAMPBELL COUNTY

PROJECT NO.

ANALYSIS NO. 0

DATE 09/29/66

SLOPE STABILITY ANALYSIS

USING

MODIFIED TAYLOR EQUATIONS

COORDINATES OF THE CENTER OF THE CIRCLE X(FEET)	Y(FEET)	RADIUS OF THE CIRCLE (FEET)	FACTOR OF SAFETY
5067.000	5020.000	40.000	3.723
5067.000	5030.000	50.000	3.850
5067.000	5040.000	60.000	3.979
5067.000	5050.000	80.000	4.067
5067.000	5060.000	90.000	4.153
5067.000	5070.000	130.000	2.816
5067.000	5080.000	140.000	2.827
5067.000	5090.000	150.000	2.849
5067.000	5100.000	160.000	2.856
5077.000	5020.000	50.000	3.265
5077.000	5030.000	60.000	3.302
5077.000	5040.000	70.000	3.385
5077.000	5050.000	80.000	3.470
5077.000	5060.000	90.000	3.553
5077.000	5070.000	100.000	3.650
5077.000	5080.000	110.000	3.743
5077.000	5090.000	150.000	2.560
5077.000	5100.000	160.000	2.561
5087.000	5020.000	50.000	2.958
5087.000	5030.000	60.000	2.943
5087.000	5040.000	70.000	3.002
5087.000	5050.000	80.000	3.065
5087.000	5060.000	90.000	3.144
5087.000	5070.000	100.000	3.232
5087.000	5080.000	110.000	3.316
5087.000	5090.000	120.000	3.396
5087.000	5100.000	130.000	3.477
5097.000	5020.000	80.000	2.067
5097.000	5030.000	90.000	2.048
5097.000	5040.000	70.000	2.810
5097.000	5050.000	80.000	2.827
5097.000	5060.000	90.000	2.881
5097.000	5070.000	100.000	2.946
5097.000	5080.000	110.000	3.008
5097.000	5090.000	120.000	3.081
5097.000	5100.000	160.000	2.159
5107.000	5020.000	80.000	1.929
5107.000	5030.000	90.000	1.910
5107.000	5040.000	100.000	1.911
5107.000	5050.000	110.000	1.917
5107.000	5060.000	120.000	1.930
5107.000	5070.000	130.000	1.942
5107.000	5080.000	140.000	1.965
5107.000	5090.000	150.000	1.988
5107.000	5100.000	160.000	2.011
5117.000	5020.000	80.000	1.840
5117.000	5030.000	90.000	1.817
5117.000	5040.000	100.000	1.794
5117.000	5050.000	110.000	1.801
5117.008	5060.000	120.000	1.813
5117.000	5070.000	130.000	1.827
5117.000	5080.000	140.000	1.841
5117.000	5090.000	150.000	1.865
5117.000	5100.000	160.000	2.021
5127.000	5020.000	90.000	1.964
5127.000	5030.000	90.000	1.756
5127.000	5040.000	100.000	1.733
5127.000	5050.000	110.000	1.729
5127.000	5060.000	120.000	1.727
5127.000	5070.000	130.000	1.742
5127.000	5080.000	140.000	1.874
5127.000	5090.000	150.000	1.895
5127.000	5100.000	160.000	1.915
5137.000	5020.000	100.000	1.920
5137.000	5030.000	100.000	1.878
5137.000	5040.000	100.000	1.715
5137.000	5050.000	110.000	1.681
5137.000	5060.000	120.000	1.782
5137.000	5070.000	130.000	1.787
5137.000	5080.000	140.000	1.794
5137.000	5090.000	150.000	1.814
5137.000	5100.000	160.000	1.839
5147.000	5020.000	110.000	1.910
5147.000	5030.000	110.000	1.872
5147.000	5040.000	120.000	1.807
5147.000	5050.000	130.000	1.818
5147.000	5060.000	120.000	1.766
5147.000	5070.000	130.000	1.753
5147.000	5080.000	140.000	1.764
5147.000	5090.000	150.000	1.778
5147.000	5100.000	160.000	1.794
5157.000	5020.000	120.000	1.934
5157.000	5030.000	120.000	1.903
5157.000	5040.000	120.000	1.830
5157.000	5050.000	130.000	1.807
5157.000	5060.000	140.000	1.817
5157.000	5070.000	150.000	1.829
5157.000	5080.000	140.000	1.789
5157.000	5090.000	150.000	1.784
5157.000	5100.000	160.000	1.800
5167.000	5020.000	140.000	2.089
5167.000	5030.000	130.000	1.963
5167.000	5040.000	130.000	1.943
5167.000	5050.000	140.000	1.908
5167.000	5060.000	140.000	1.837
5167.000	5070.000	150.000	1.829
5167.000	5080.000	160.000	1.842
5167.000	5090.000	160.000	1.866
5167.000	5100.000	160.000	1.841

DATA FOR MINIMUM FACTOR OF SAFETY

X(FEET)	Y(FEET)	R(FEET)	FS(MINIMUM)
5137.00	5050.00	110.00	1.68

COORDINATES OF ORIGINAL GROUNDLINE

X(FEET)	Y(FEET)
0.000	5000.000
5000.000	5000.000
5047.000	5000.000
5197.000	4953.000
5256.000	4949.000
7000.000	4839.000

COORDINATES OF WATERTABLE

X(FEET)	Y(FEET)
0.000	5040.000
4955.000	4941.000
5047.000	4939.000
5153.000	4937.000
5256.000	4933.000
7000.000	4831.000

LAYER NUMBER 1

COHESION(LB/SQFT)	300.00000
TANGENT PHI	0.53170
UNIT WEIGHT(LB/CUFT)	127.00000

COORDINATES OF LAYER BOUNDARY LINE

X(FEET)	Y(FEET)
0.000	5060.000
4955.000	4961.000
5135.000	4957.000
5197.000	4953.000
5259.000	4949.000
7000.000	4839.000

LAYER NUMBER 2

COHESION(LB/SQFT)	425.00000
TANGENT PHI	0.26790
UNIT WEIGHT(LB/CUFT)	127.00000

COORDINATES OF LAYER BOUNDARY LINE

X(FEET)	Y(FEET)
0.000	5040.000
4955.000	4941.000
5047.000	4939.000
5153.000	4937.000
5256.000	4933.000
7000.000	4831.000

LAYER NUMBER 3

COHESION(LB/SQFT)	0.00000
TANGENT PHI	0.46630
UNIT WEIGHT(LB/CUFT)	127.00000

COORDINATES OF LAYER BOUNDARY LINE

X(FEET)	Y(FEET)
0.000	1000.000
1000.000	1000.000
3000.000	1000.000
5000.000	1000.000
7000.000	1000.000
9000.000	1000.000