COMMONWEALTH OF KENTUCKY
DEPARTMENT OF HIGHWAYS
FRANKFORT

HENRY WARD
COMMISSIONER OF HIGHWAYS

April 25, 1966

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

H-2-16

MEMORANDUM

TO: W. B. Drake, Assistant State Highway Engineer; Chairman, Kentucky Highway Research Committee

SUBJECT: Interim Report of Research; KYHPR-63-16, HPR-1(1), Part II; "Proposed Remedial Design for Unstable Embankment Foundation, I 64-6(6)117, Bath County".

Sidehill cut-and-fill sections are typical design features of roadways in mountainous or hilly terrain, sliver-shaped fills often provide the outer portion of the roadway shelf. Lateral seepage of ground waters into sidehill fills and the attendant damming and pressures have, historically speaking, been a major cause of slides. Of course, proper drainage effectually removes the cause of instability -- that is, if the offending water can be tapped or diverted so as to reduce hydrostatic pressures. Earth embankments which have stood for a time and then failed have presumably become weakened by some water condition. In this case-study, the embankment was built partly on water-laden, natural soil foundation; the burden of the embankment tended to squeeze the pore-water out of the foundation soil -- like a sponge; however, the soil was unusually impermeable; and, in effect, all avenues for relief of pressure and drainage were blocked. For example, if the pressure due to the embankment load is designated by P and the opposing pressures are designated by p_s and p_w (partial pressure borne by soil structure and partial pressure borne by water, respectively), then at equilibrum, $P = p_s+p_w$; p_w, of course, is fluid or lubricating pressure; as p_w increases,

p_s decreases, and the internal frictional resistance of the soil decays; failure occurs through loss of frictional resistance in the soil; eventually pw will dissipate; in normally permeable soils, it never becomes critical. Analyses of stability, involving settlement and shear, are infinitely more complex than the example given; the essential information is derived from consolidation tests and triaxial-shear tests. This report presents a rather exhaustive analysis or case-study of the slide on I 64 in Bath County, eastward of Owingsville, which occurred earlier this year and which is described more fully therein.

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 overlaying the bedrock is a weathered layer of the siltstone. The elevation of the top of the Garrard formation is about 700 feet; whereas, borings reached 670 feet before encountering bedrock. Bedrock appears to steepen downward from elevation 700 -- indicating the greater susceptibility of the siltstone to erosion or weathering. From 700 feet upward, the rock is the Maysville and Richmond Linestones, respectively; 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.

A slide of this magnitude commands practical, scientific, and perhaps some intuitive decisions. One approach, of course, is to abandon the site; another is to restructure the embankment. We have devoted our full attention to restructuring and offer three enabling plans -- mindful that they are based on sparingly few samples and tests. The plan listed as the second alternate is subject to precautionary measures during the installation of the sand drains -- that is, because the mass of the slide may not be adequately counterweighted at that time unless additional fill is placed at the toe before commencing; sloughing from the slope above could be troublesome if the work is done during a rainy season. Nevertheless, each plan merits consideration.

Messers Scott and Deen are specialists in embankment and foundation design. Dr. Deen is the senior scientist and progenitor, so to speak, of our soil mechanic staff. The authors were assisted by Mr. T. C. Hopkins, who designed the sand drains, and Mr. W. W. McGraw, who made the stability analyses; both are studying for their Master Degree in Civil Engineering and are specializing in soil mechanics; both have been recipients of the Departments' scholarship; H. F. Southgate also studying for his Masters Degree in soil mechanics, rendered general assistance. I am quite pleased to acknowledge their services and to submit this report in timely season.

Respectfully submitted,

Jas. H. Havens

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

JHH:em
Attachment
cc: Members, Research Committee
A. O. Neiser
R. O. Beauchamp
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Harold G. Mays
Frank Kemper
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Research Report

PROPOSED REMEDIAL DESIGN
for
UNSTABLE HIGHWAY EMBANKMENT FOUNDATION
I-64-6(6)117, Bath County

by
G. D. Scott
Research Engineer
and
R. C. Deen
Assistant Director

DIVISION OF RESEARCH
DEPARTMENT OF HIGHWAYS
COMMONWEALTH OF KENTUCKY

April, 1966

INTRODUCTION

Early in 1966, during the construction of a large embankment between Stations 1738+00 and 1745+00 on I 64 in Bath County (I 64-6(6)117, SP 6-404-5G1), 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.

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 1. Figure 2 illustrates the large upheavel 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 3.

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 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 3 and also in Figures 4 through 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

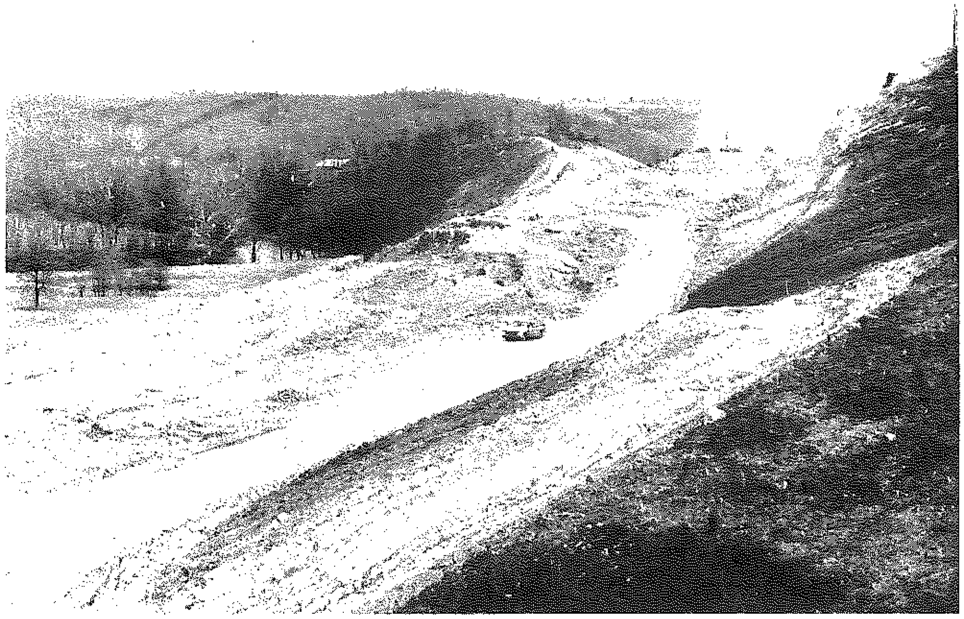


Figure 1. A General View of Failed Embankment on I 64 in Bath County

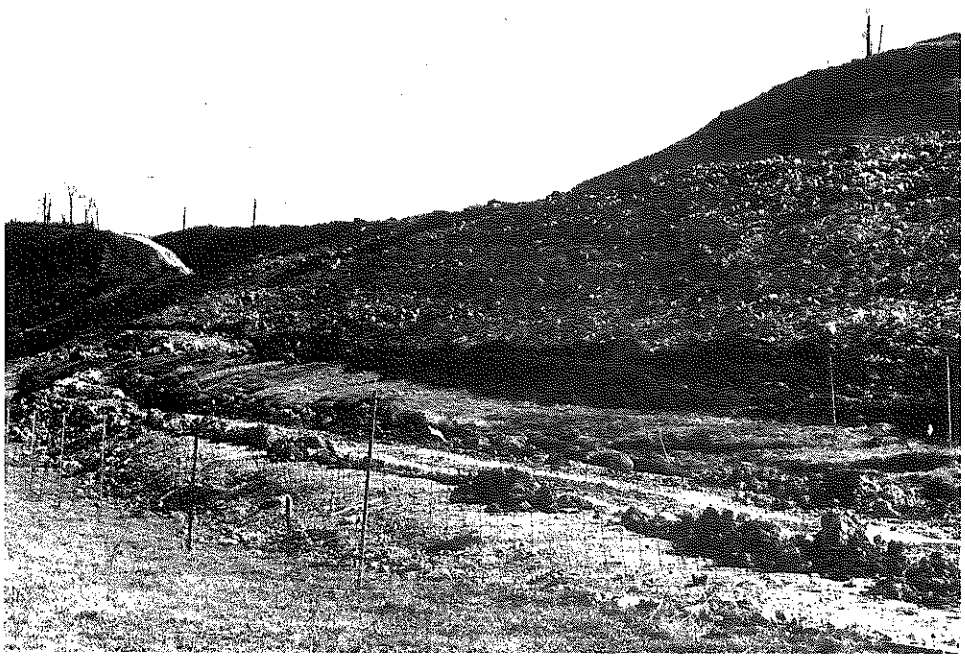
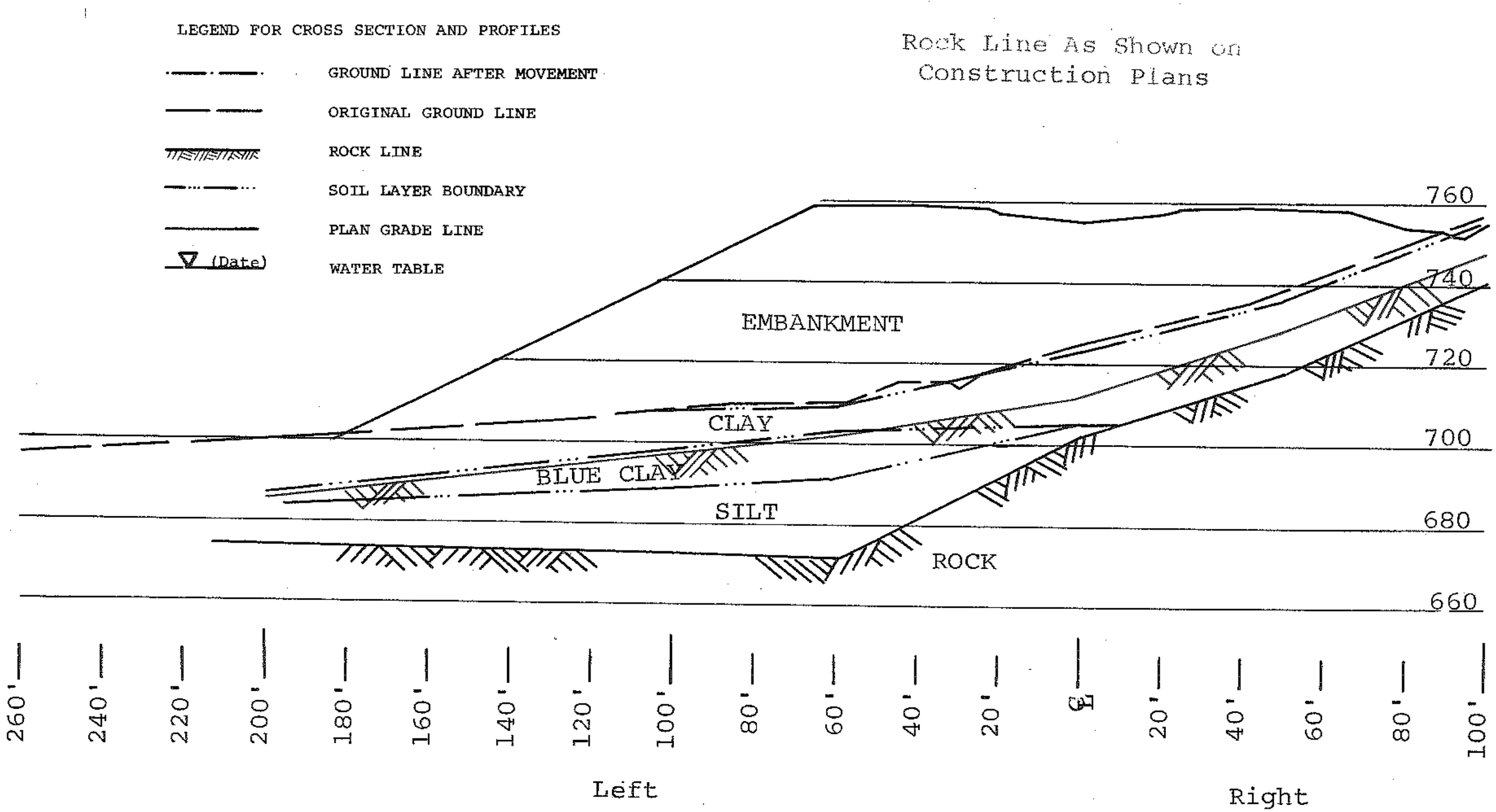


Figure 2. View of Upheaval at Toe of Failed Embankment.



STA. 1740+00
SCALE: 1"=40'

Figure 3. Typical Section Showing General Subsurface Conditions.

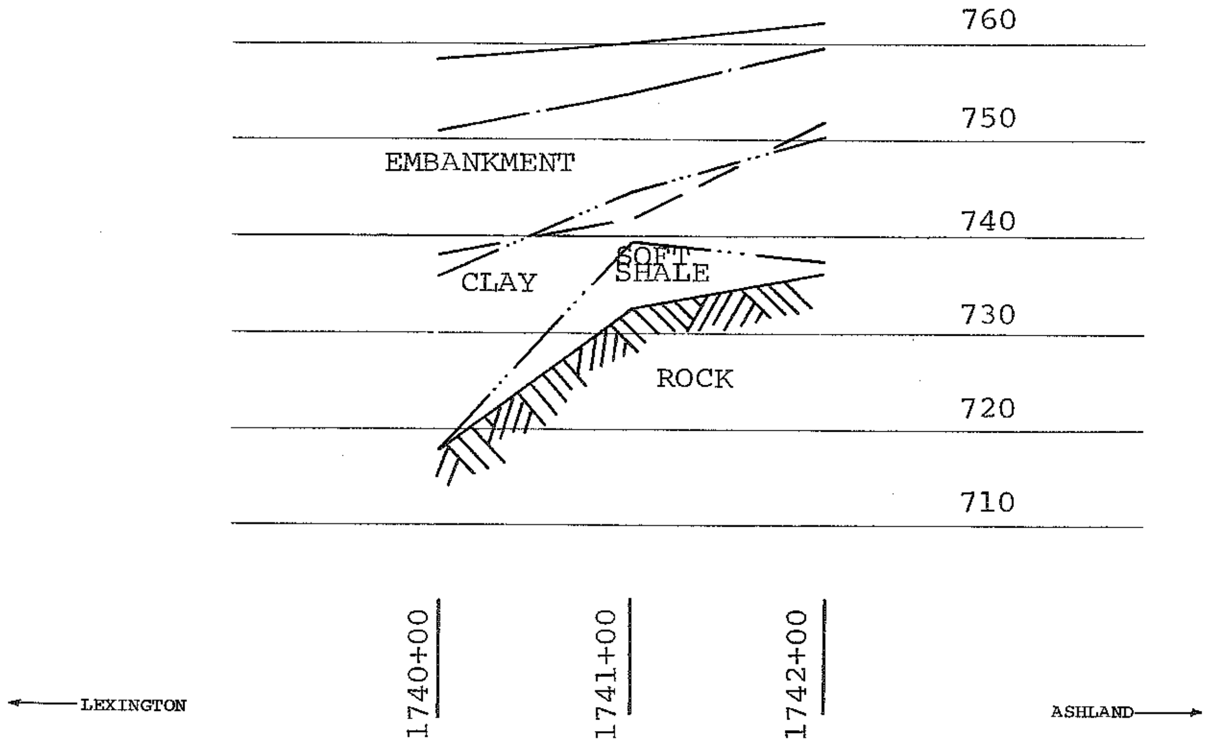


Figure 4. Soil Profile, Approximately 52 Feet Right of Centerline.

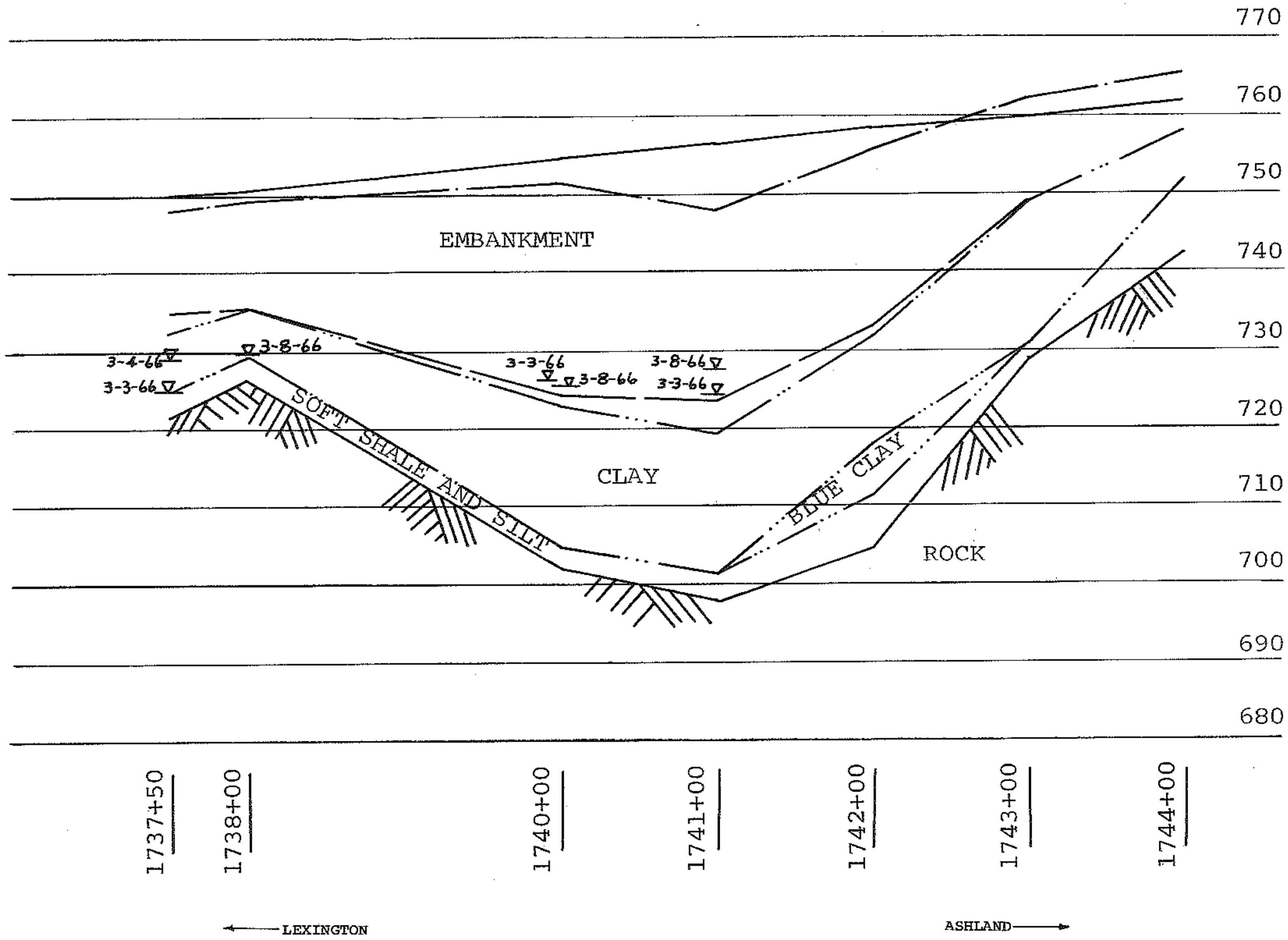


Figure 5. Soil Profile, Approximately 2 Feet Right of Centerline.

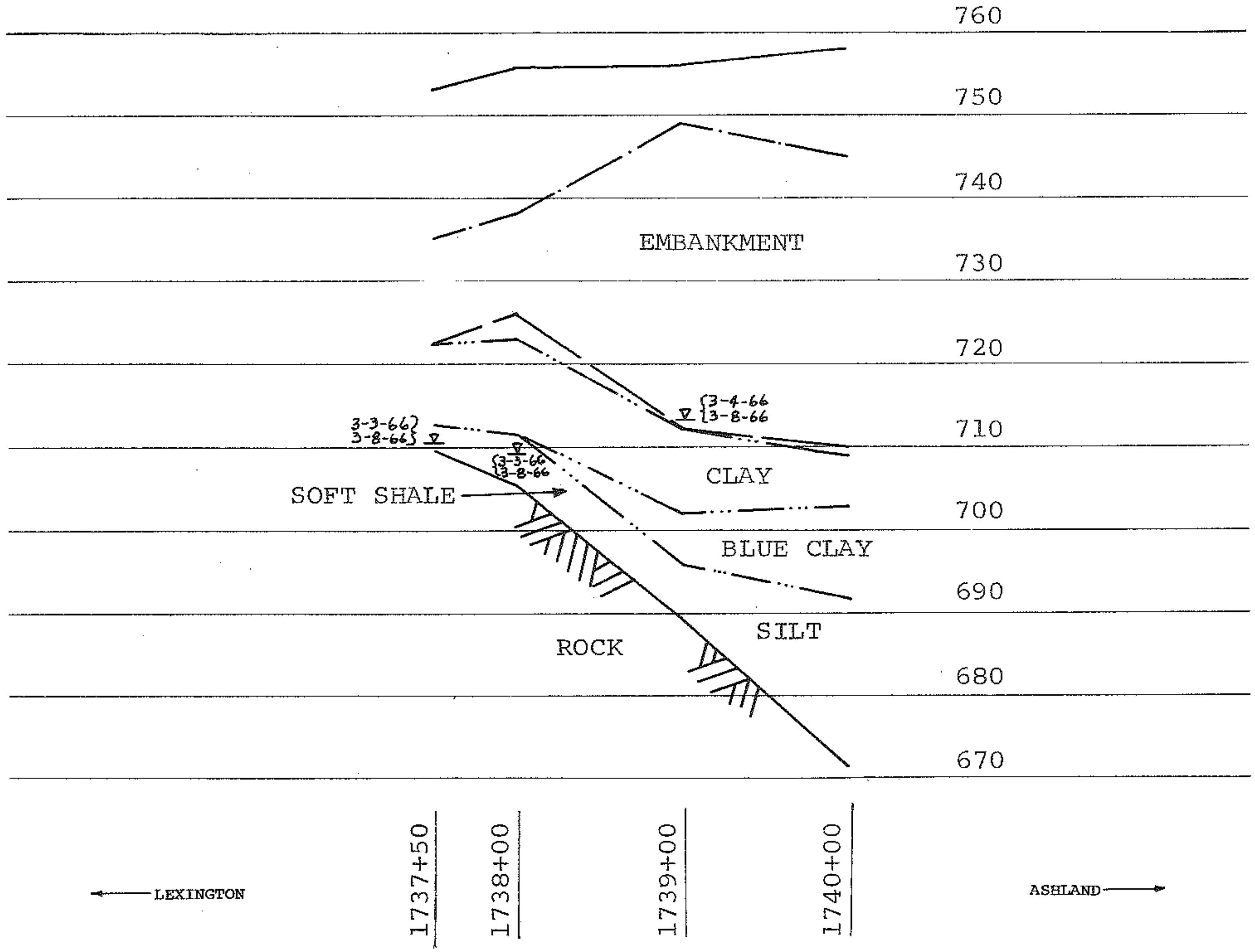


Figure 6. Soil Profile, Approximately 59 Feet Left of Centerline.

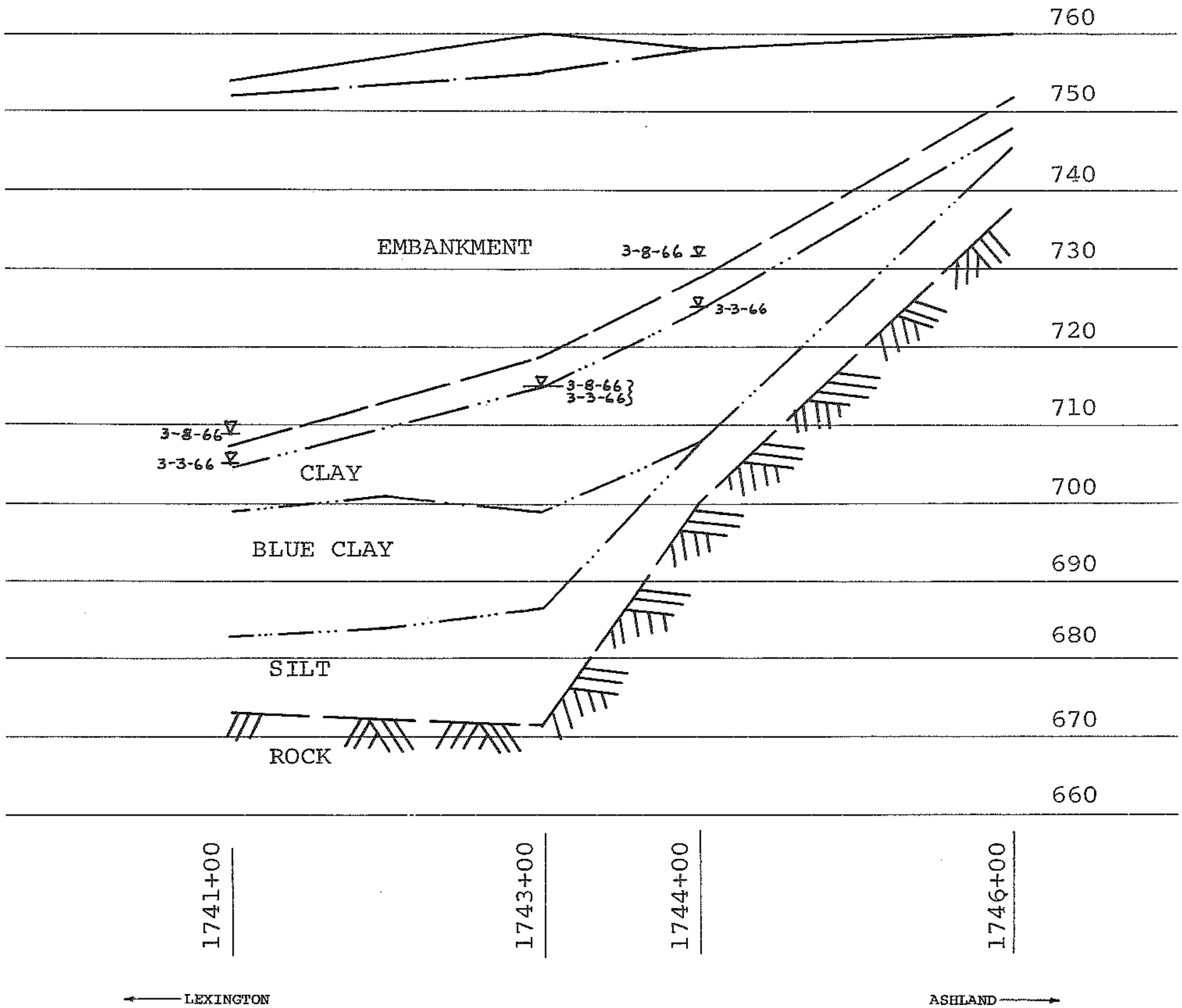


Figure 7. Soil Profile, Approximately 77 Feet Left of Centerline.

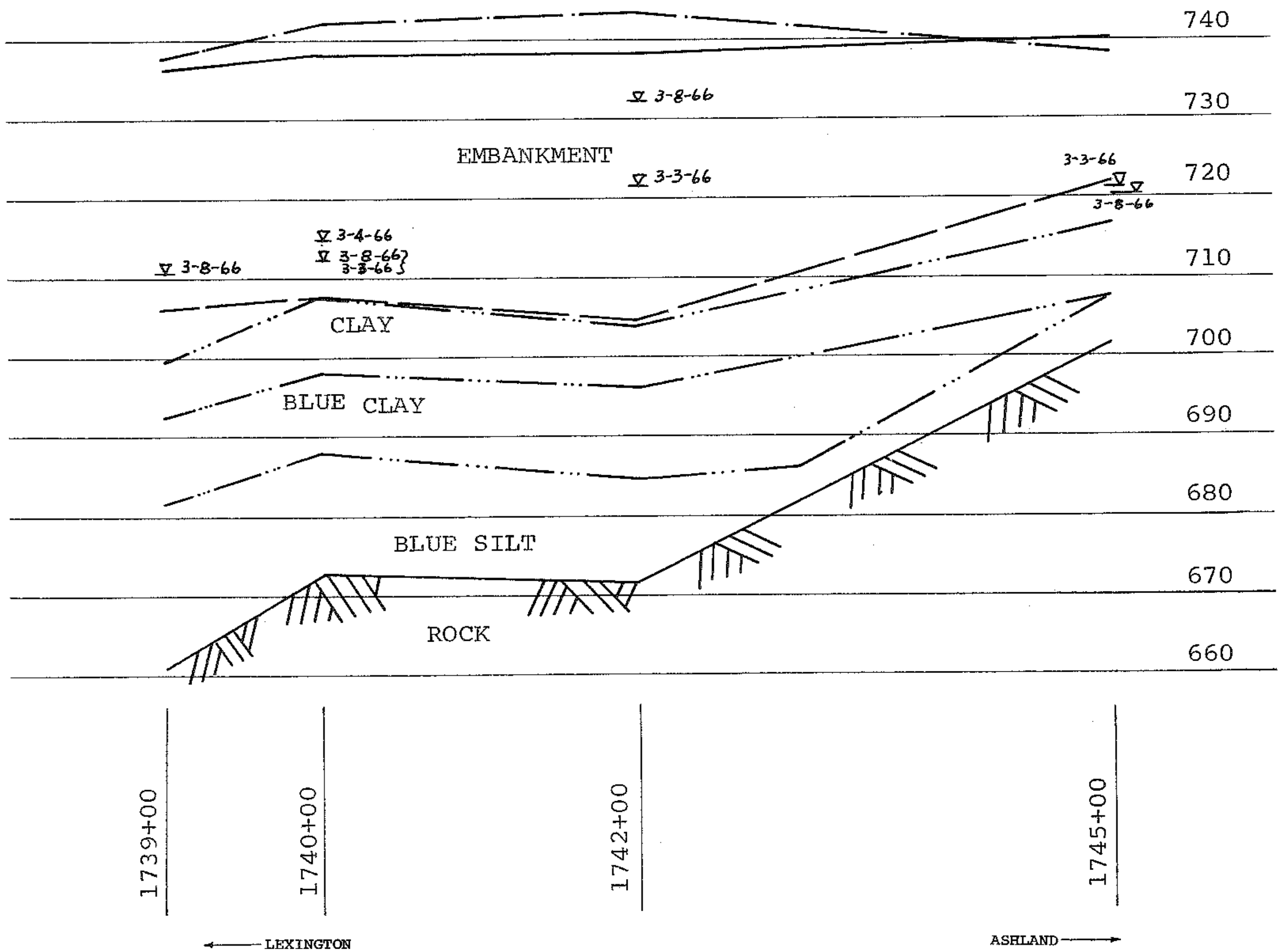


Figure 8. Soil Profile, Approximately 112 Feet Left of Centerline.

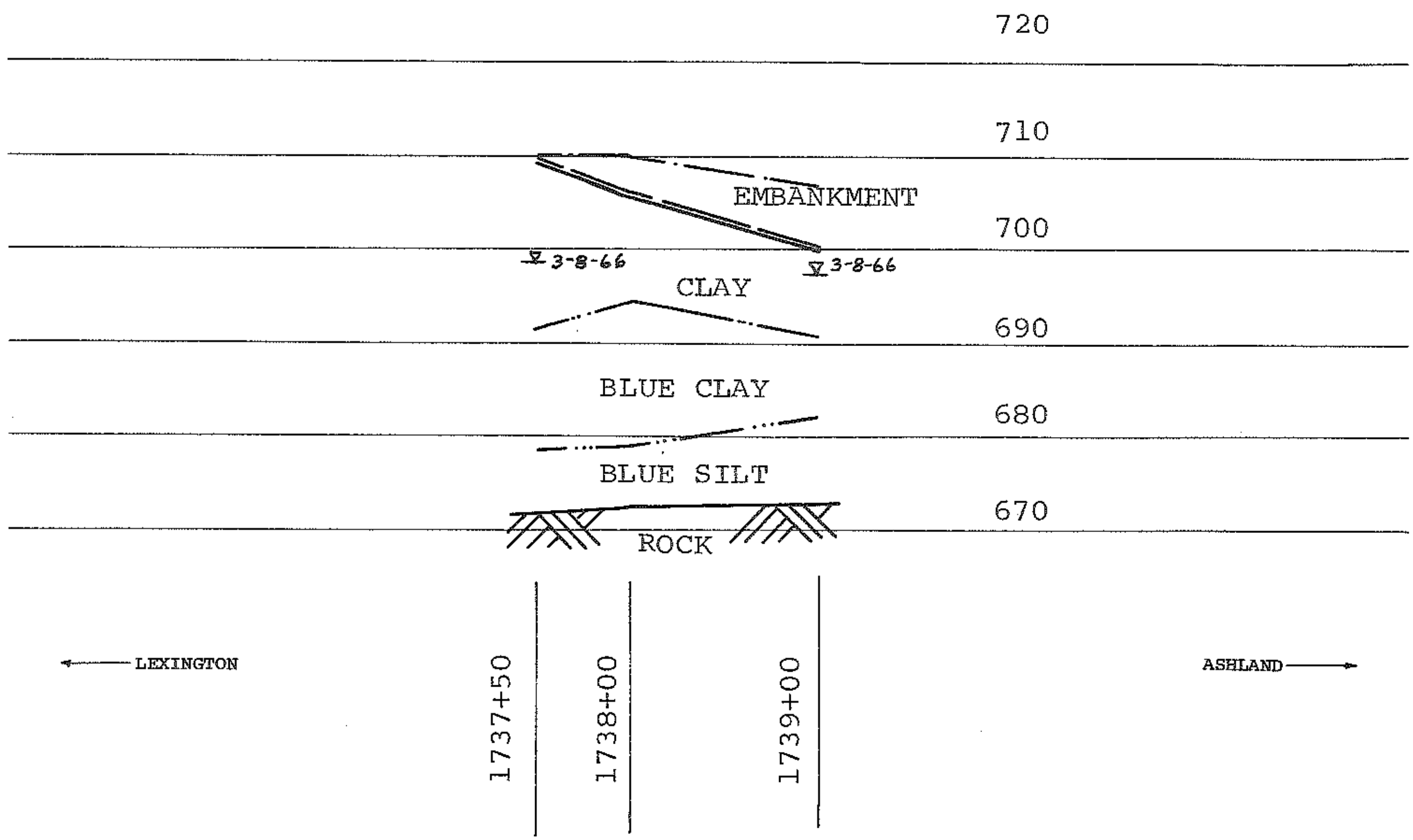


Figure 9. Soil Profile, Approximately 179 Feet Left of Centerline.

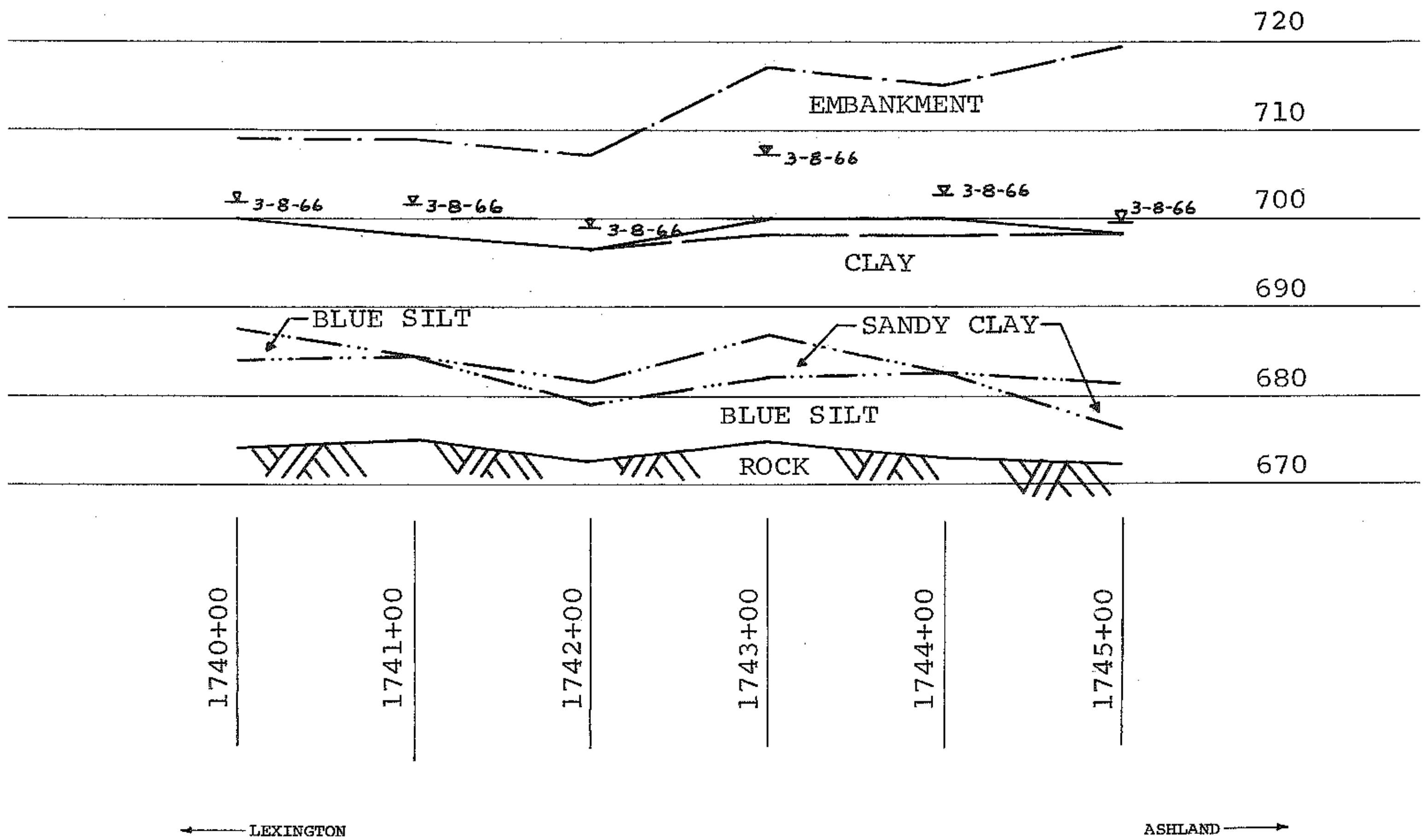


Figure 10. Soil Profile, Approximately 197 Feet Left of Centerline.

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 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 the effectiveness of 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 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

TABLE 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	2.87		
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 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

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 emersed 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.

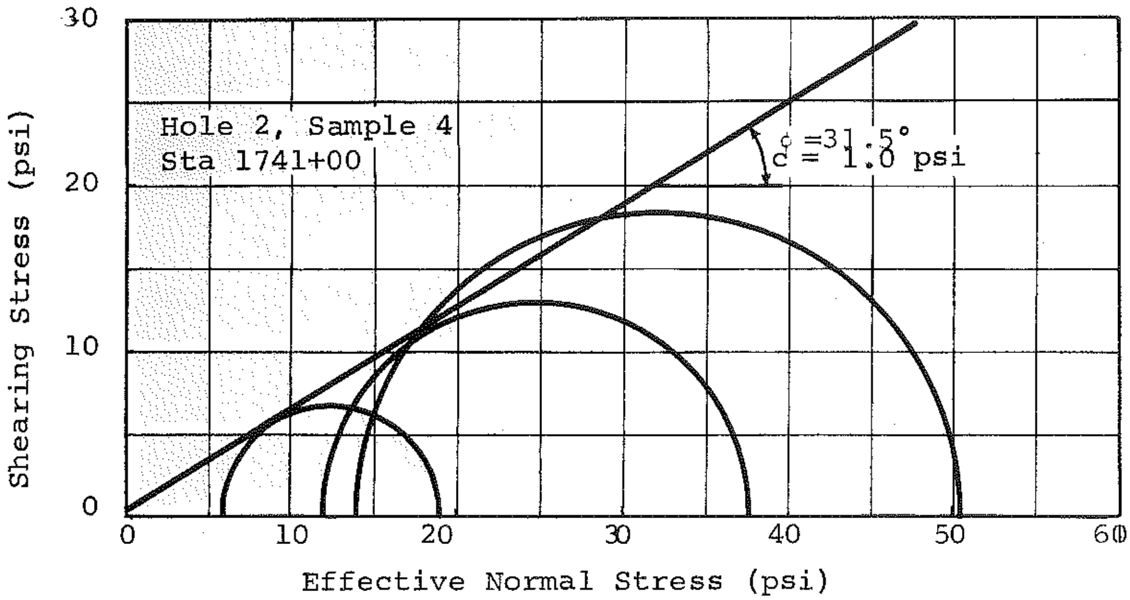
LABORATORY TESTING

Triaxial Test Procedure and Results

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 2. The Mohr circles and failure envelopes for the triaxial tests are shown in Figure 11. The average unconfined compressive strength, including the triaxial test data for the smallest confinding 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



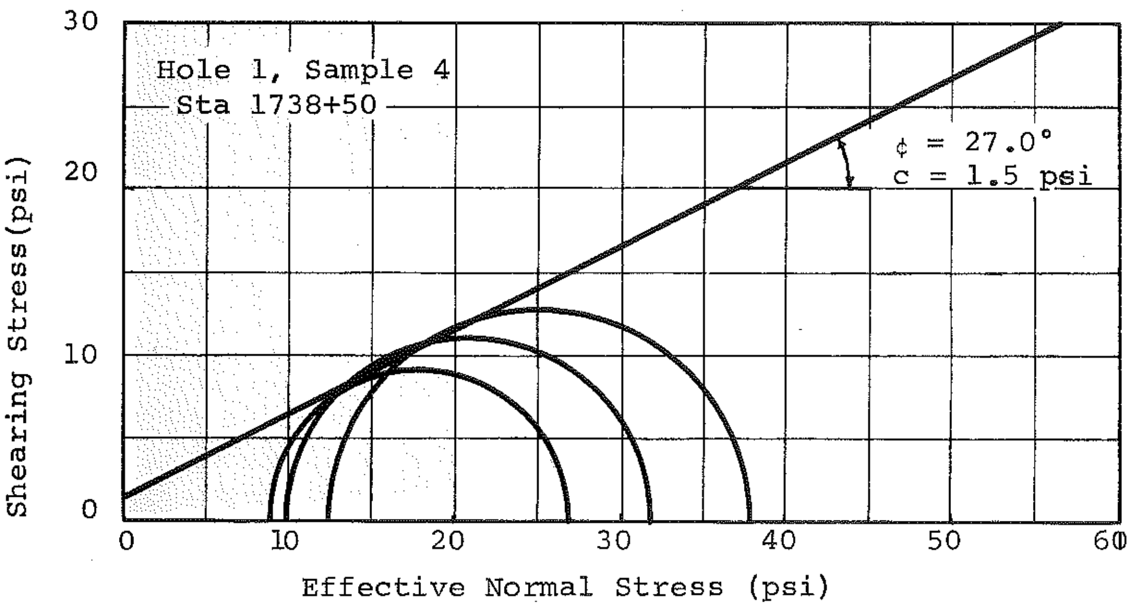


Figure 11. Triaxial Test Results

average effective cohesion was approximately 200 pounds per square foot.

Consolidation Test Procedure and Results

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

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

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.

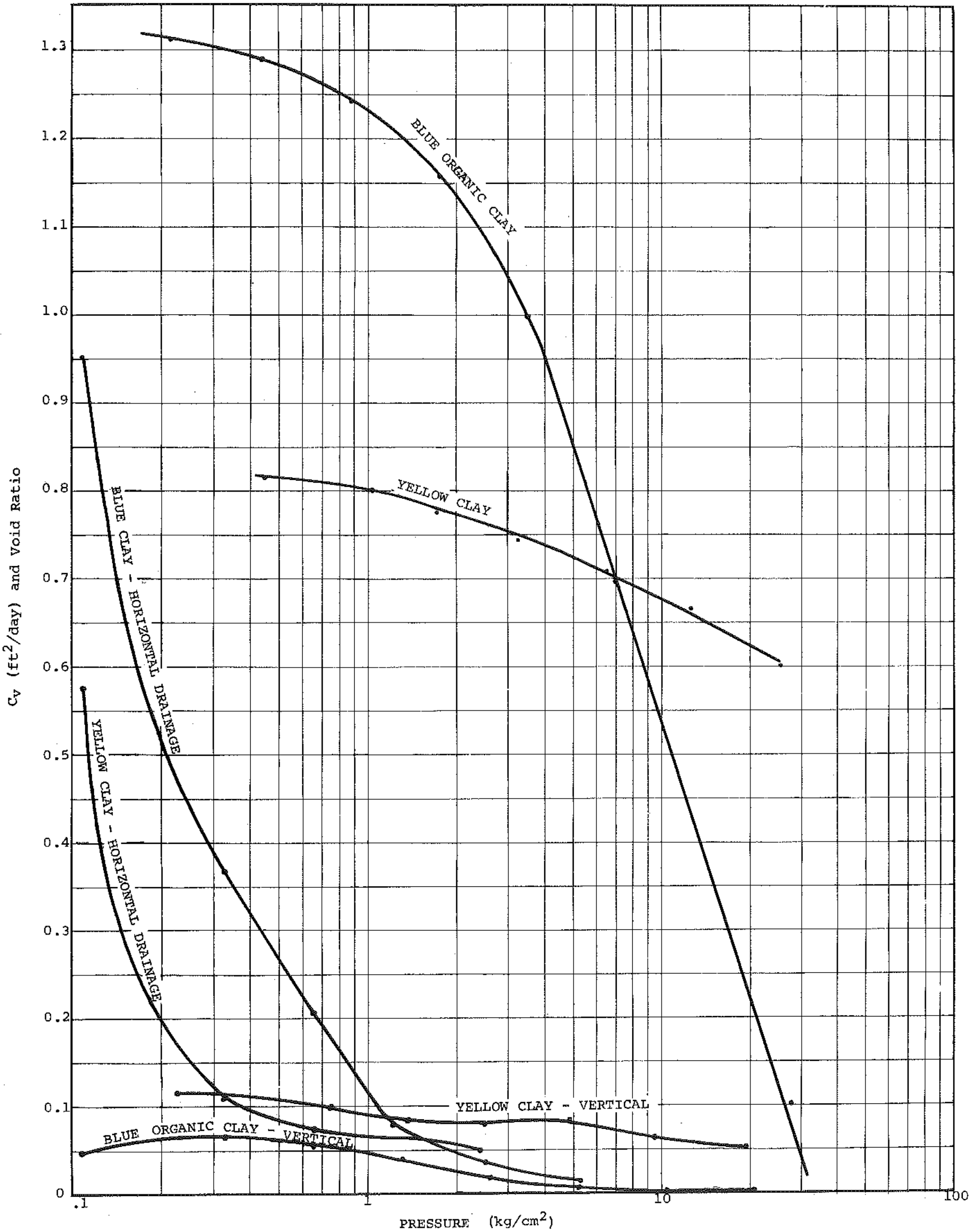


Figure 12. Consolidation Test Results

DATA ANALYSIS AND DISCUSSION

Shear Strength Data

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 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. A factor of safety was obtained.

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 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. Figure 13 shows the critical circles determined as described above for three different berm sizes. The factors of safety shown in Table 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

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

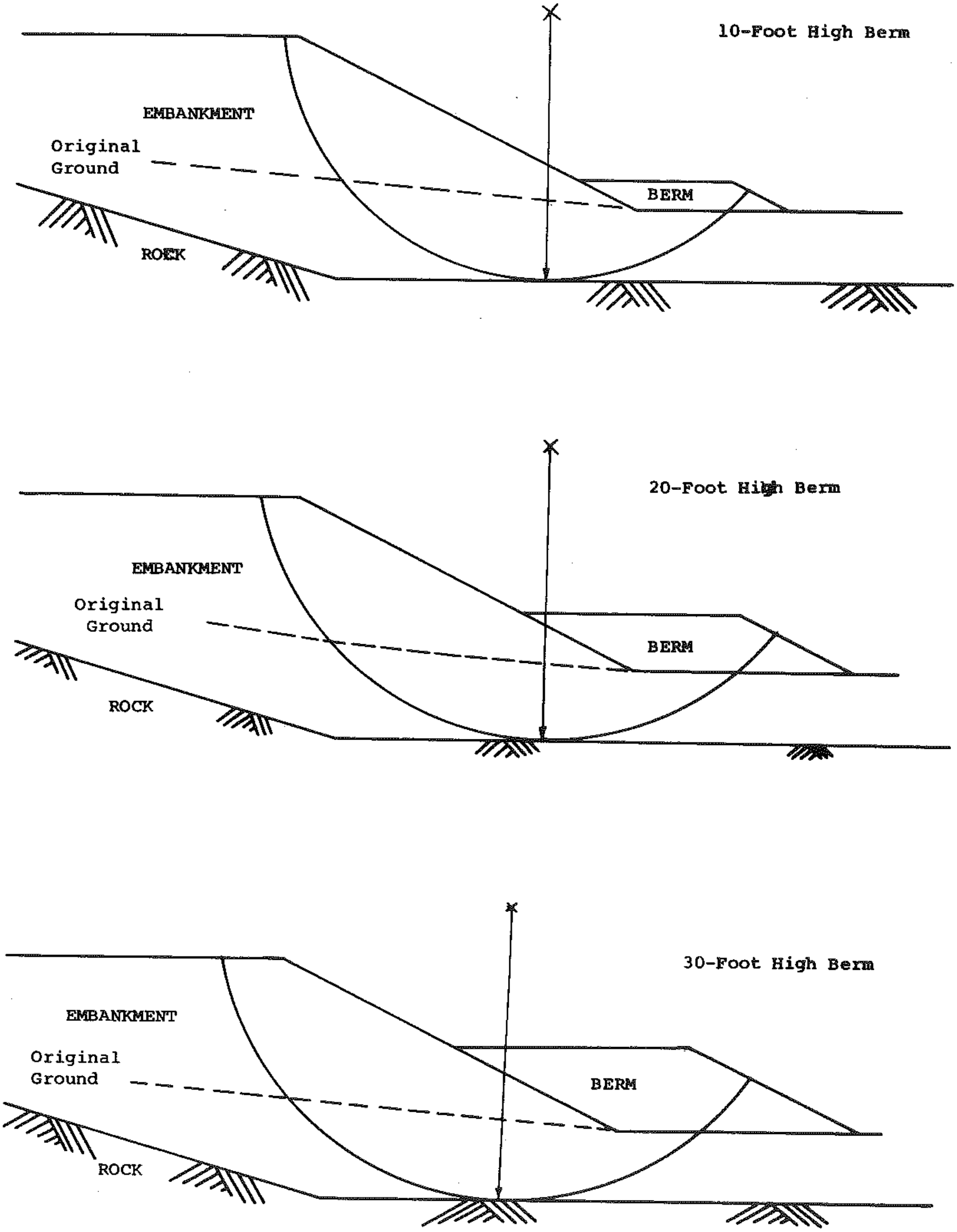

Figure 13. Critical Failure Circles for Various Berm Sizes.

TABLE 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	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		

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 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 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 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 to gain shear resistance in the foundation 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.

Consolidation Data

The consolidation test data were analyzed both in terms of expected settlement under the weight of a 25-foot high berm (See Table 4) and the rate of settlement and consequent gain of shear resistance for various sand drain spacings (See Table 5 and Figure 14). Table 4 is a sample calculation for rate of consolidation with sand drains, and Figure 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 coefficient of premeability is the same in all directions. However, the curves of coefficient of consolidation in Figure 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

TABLE 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	2.489	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 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. $U_c = 100 - \frac{(100-U_v)\ (100-U_r)}{100}$

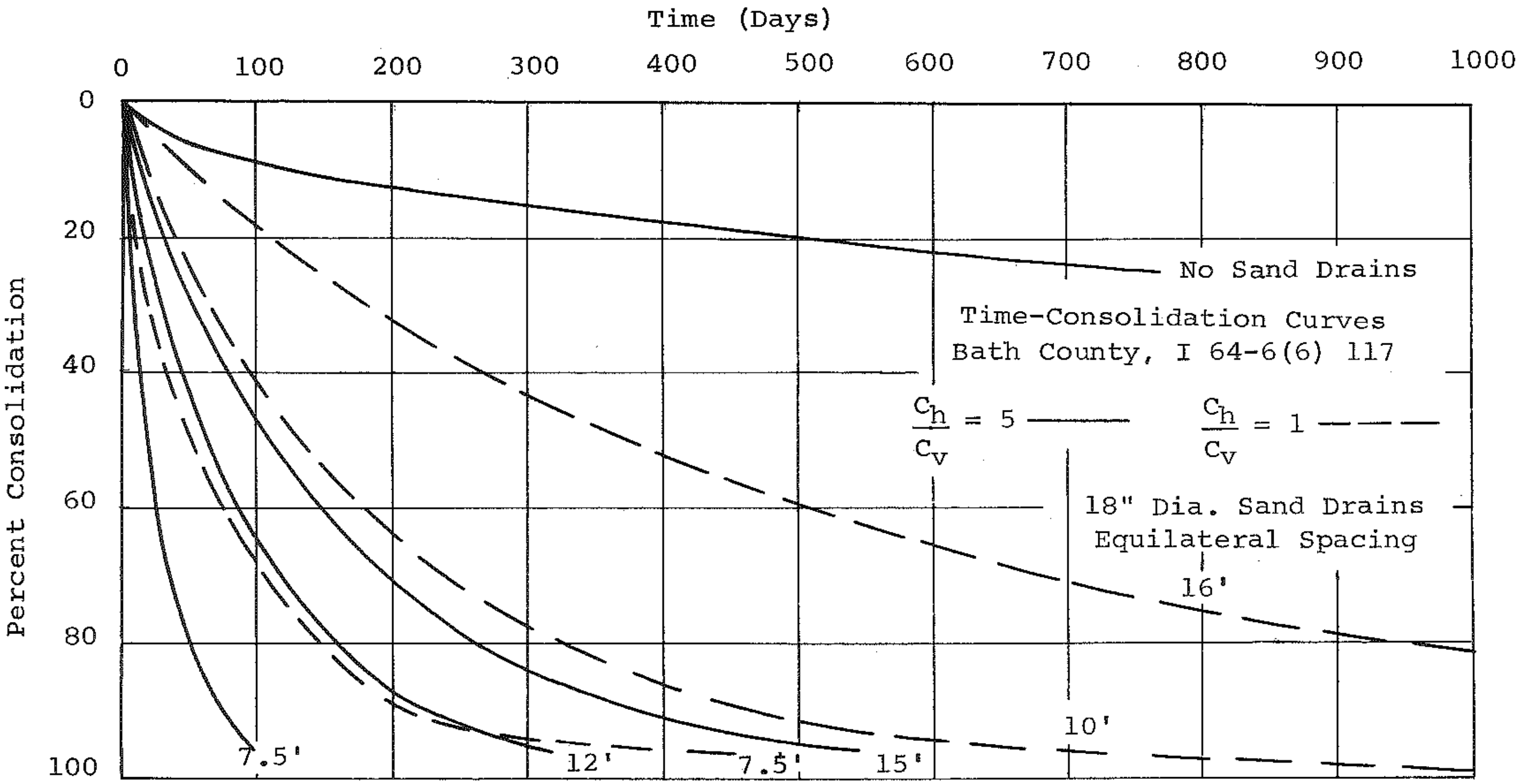

Figure 14. Time-Consolidation Curves.

is thought 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 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 pervious stratum below the compressible material. However, a very thin seam or seams of pervious soil could very well have gone undetected, in which case the rate of settlement would be increased at least fourfold.

The spacing of sand drains required to effect a given degree of consolidation in a given time can be determined from Figure 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.

RECOMMENDATIONS

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 twenty to twenty-five feet high by sixty-five 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 west bound traffic lanes from Station 1738+50 to 1743+50. Construct sand drains on fifteen-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 a twenty-foot by sixty-five-foot berm concurrently, using the piezometers to control the rate of construction of the final stages of the main embankment. Construction should be temporaily 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 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.

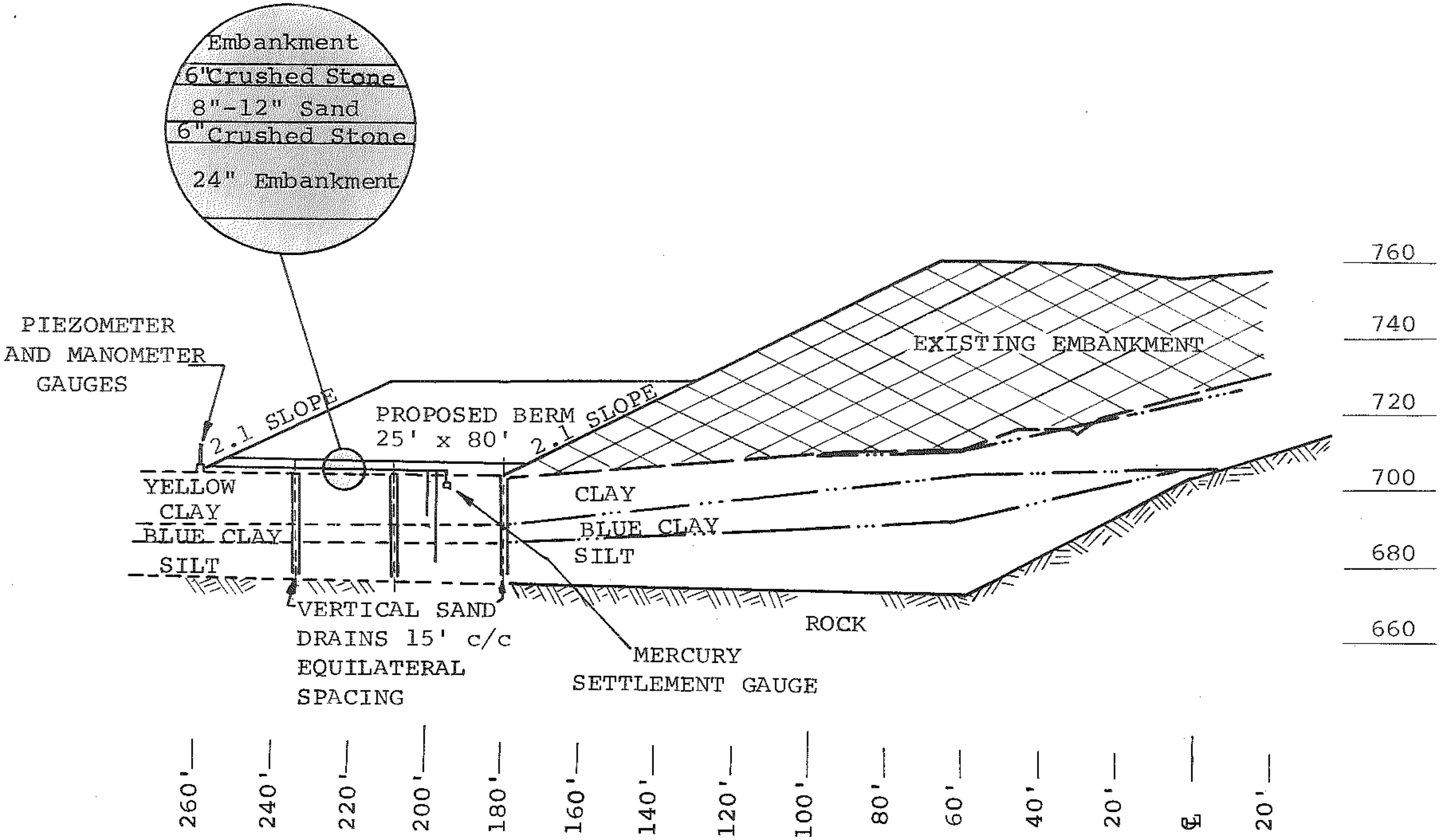

Figure 15. Typical Berm and Sand Drain Configuration.