MEMORANDUM TO:  J. W. Spurrier
Assistant State Highway Engineer

SUBJECT:  "Unstable Embankment, US 119; Harlan-Pineville Road, Stations 1260 to 1265; MP 48-168J.

At a meeting last January, in which several landslide situations were reviewed, we were assigned principal responsibility for the investigation and analysis of the US 119 site near Wilhoit. The report submitted herewith presents in-depth analyses and recommendations.

The analyses indicate that the river bank was probably unstable before the roadway was constructed. This explains, to my satisfaction, why the two previous reconstructions were not successful. It also appears that the work done in the last reconstruction remains as an asset.

A channel change to accommodate a berm seemed an intuitively necessary remedy before the in-depth analyses were begun.

The report was delayed somewhat in order to include the results of the borings made in the channel by the Division of Materials.

Respectfully submitted,

[Signature]

Jas. H. Havens
Director of Research

JHH:dw
attachment

CC's:  W. B. Drake
       G. F. Kemper
       C. G. Grayson
       J. T. Anderson
       J. E. McChord (attn. Henry Mathis)
       A. R. Romine
       C. S. Layson
       A. B. Magee
       E. B. Gaither
       T. P. Mason
UNSTABLE EMBANKMENT, US 119
Harlan-Pineville Road
Stations 1260 to 1265
MP 48-168 J

by

T. C. Hopkins
Research Engineer Senior

Division of Research
DEPARTMENT OF HIGHWAYS
Commonwealth of Kentucky

July 1972
In January 1972, an in-depth investigation of an unstable embankment, Figure 1, on US 119 near Station 1262+00 was initiated. The site is approximately five miles southwest of Harlan and near Wilhoit. Figures 2 and 3 show a plan view and a typical cross-section, respectively. US 119 travels on a tangent from Station 1252+16 to 1255+54 in a northerly direction. From that point, the roadway gradually curves four degrees to the east, becoming tangent again near Station 1275+00. At about Station 1258+00, the roadway emerges from a cut, and near Station 1266+00 it reenters a small cut. Between the latter two stations the embankment is sidehill, resting partly on a bench of cut shale and partly on an alluvium and colluvium river bank. Maximum height of the unstable embankment is about 25 feet. The unstable portion of the embankment, Figure 2, is between Stations 1260+00 and 1265+00 (approximately). The roadway embankment is bounded on the north by a cliff varying from 50 to 100 feet in height and on the south by the Cumberland River.

Construction of US 119 (relocation of old US 119) in the area began in the spring of 1967. During construction, a slip occurred near Station 1259+00. At that time, the area was undercut, Figure 4, and refilled partly with stone. Shortly after completion of roadway paving in early 1969, the embankment began moving near Station 1262+00 as shown in Figure 5. Efforts made at that time to correct the failure consisted of 1) removing the unstable portion of the embankment to previous excavation limits, 2) benching the underlying shale, 3) installing drainage ditches and perforated pipe, 4) placing a one-foot granular blanket (Figure 6) and large toe fill stone, and 5) reconstructing the embankment. Apparently, it had then been concluded that the embankment failure was a result of ground water seeping into the embankment, thereby inducing failure by lowering shear resistance to a critical level.

Approximately one year after completion of reconstruction in the spring of 1969, appreciable subsidence of the pavement was observed. A comparison of Figures 1 and 5 shows the embankment began failing at approximately the same location. From 1970 to the present (June 1972), the roadway had been patched and built up on numerous occasions.

During the investigation, visible signs of imminent and impending failure were observed. As shown in Figure 1, the guardrail on the southern shoulder had been dropped and been displaced considerably. A failure scarpment on the southwestern flank of the slide, just below the guardrail in Figure 1, was visible. The pavement was cracked near Station 1264+00 where it has been patched numerous times (Figure 7). The cross drain near Station 1264+00 was broken; the inlet had been plugged. Another cross drain had been installed near Station 1266+20.

**TOPOGRAPHY AND GEOLOGY**

US 119 between Pineville and Harlan, Figure 8, generally follows the Cumberland River along the toe of Pine Mountain. Generally, rock strata of Pine Mountain dip 20 to 40 degrees toward the Cumberland River.

A stratigraphic section of the area, Figure 8, consists of stable, resistant cross-bedded sandstones of the upper part of the Lee Formation overlain by a less resistant sequence of interbedded shales, siltstones, sandstones, coal beds and underclays of the lower part of the Breathitt Group -- probably the Hance Formation. At the site, a massive slide formation (Lower Breathitt) is exposed in a cliff north of the roadway. The unstable portion of the embankment rests partly on benched shale and partly on a river bank composed of colluvium and alluvium.

The area near the site has been the scene of numerous rock and debris slides resulting from the undercutting of the steeply dipping strata of Pine Mountain. There also is evidence (Figure 8) that some ancient slides have occurred in the area, especially where the river has cut into the southern flank of the overthrust fault. During construction in 1967 between Stations 1243+00 and 1259+50, a massive ancient earth slide (Church slide) involving some 330,000 cubic yards of material was reactivated by roadway excavation. In the spring of 1968, additional slide material had to be removed. The present unstable embankment does not appear to be directly related to that massive earth slide, although both are located within the same vicinity. However further benching or undercutting at the present site would seem to involve unnecessary dangers.

**FIELD INVESTIGATION**

Subsurface exploration consisted of 12 borings. Locations of 11 of these borings are shown in Figure 2. Slope indicator casing was installed in eight of the boreholes to determine the slip zone, rate and direction of movement, and mode of failure of the unstable soil mass. The slope indicators were located in such a manner that shear zones could be detected at four sections, 1-1', 2-2', 3-3' and 4-4', shown in Figure 2 and cross-sectional view in Figures 9 and 10. At Station 1262+00, an attempt was made to place a slope indicator (No. 5, not shown) about halfway down the...
slopes, measured from the top of the slide. However, efforts were abandoned when difficulties were encountered during drilling. Three holes, Numbers 10, 11, and 12, were augered and cored by the Division of Materials. Three split-spoon samples were obtained from Hole 10. Cores were geologically logged by personnel of that division. All resultant horizontal movement-depth curves obtained from February 1972, when the slope indicator casing was initially installed, to June 1972 are presented in Figures 11 through 13.

Boring results are presented in Figures 14 through 18. Horizontal movement and slope indicator dial changes as a function of depth, as well as resultant horizontal movement-time curves, are shown plotted adjacent to the boring results in those figures. Table 7-1 shows water table elevations obtained from each slope indicator well during the period February-May 1972. The highest water table recorded during the study period is shown in Figures 9 and 10.

During the period July 1971 to January 1972, surface movements of the slide were obtained from three parallel lines of hubs, established along the face of the slip (Figure 2). Results of these measurements are shown in Figures 19 and 20. In Figure 20, surface movement-time curves are shown for a selected station.

Due to the rocky nature of the soils at the site, only three Shelby Tube samples were obtained for classification purposes and soil strength determinations. These samples were obtained in accordance with the method for thin-walled sampling of soils, ASTM Designation: D 1587, from Holes 5 and 6.

LABORATORY INVESTIGATION

Soil samples were extruded from the three Shelby Tubes, cut into four-inch lengths and identified according to the visual-manual procedure (ASTM Designation: D 2488T). Only two relatively undisturbed soil samples were obtained; triaxial consolidated-undrained (CU) tests were performed on these samples. A tabulation of soil test data is presented in Table 2. Results of the CU tests are shown in Figure 21. Although normally three or more tests are desired in order to define Mohr's failure envelope, it can be obtained from fewer samples by plotting the CU test results on a "p-q diagram". Near failure on that diagram, effective stress paths generally follow the K-failure line as shown on the bottom of Figure 21.

SOILS

Based on boring and laboratory test results, the embankment soils consist of compacted, dry, gray, sandy clay with shale gravel. These soils classify as ML-CL, A-4(4) and clay and are moderately plastic. Fill materials contained about 16 percent gravel, 26 percent sand, 28 percent silt and 30 percent clay. Liquid limit and plasticity index were 31 and 10 percent, respectively. Natural moisture content was about 9 percent. Generally, these materials were relatively dry.

Materials located at the toe of the embankment are grayish-brown, moist, soft sandy clay and silty clay with sandstone gravel. The materials are composed of 10 to 14 percent coarse aggregate, 22 to 26 percent sand, 24 to 28 percent silt and 28 to 30 percent clay. Boring results indicated boulders were present in the soil matrix. Liquid limit and plasticity index were about 32 and 12 percent, respectively. The materials classify as A-4(4) and A-6(5), CL and ML-CL, and clay loam and clay.

Foundation materials consisted of fine-grained, gray, hard boulders (?) underlain by silty, black, medium hard to hard, fractured shale. Based on the results of Borings 10, 11, and 12, the strata in the river area are horizontal. Shale elevation was established at 1117 feet. Overburden was relatively shallow. The fractured shale was apparently a result of the faulted anticline. Such fracturing serves as avenues of percolation for groundwater.

Based on the Atterberg limits, these materials have a high coefficient of consolidation, C_v. It ranges from 0.5 feet square per day in the recompression range to a value greater than two feet square per day in the virgin compression range. Hence, when loaded, these materials would consolidate rapidly. The materials have a low coefficient of secondary compression, C_{q'} and a comparatively low compression index. Consequently, settlements of these materials, when loaded, are moderately small.

STABILITY ANALYSIS

Failure Mode of Unstable Embankment

Except at Wells 2 and 9, slope indicator data show the main shear zone of the unstable soil mass lies some 25 to 30 feet below present ground surface, as shown in Figures 9 and 10. At Well 2, the shear zone lies approximately 10 to 12 feet below the surface while at Well 9 only nominal movement was detected. Moreover, the failure zone generally lies near or slightly below the embankment and the colluvium-shale interface. The unstable soil mass is moving toward the river along a line parallel to the nose of a nearby faulted anticline. The failed mass involves some 50,000 cubic yards of material. Direction of movement of the slide at each slope indicator well is shown in Figure 2.
Based on average rates of movement during the study period, February-May 1972, the eastern and western top flanks of the slip are moving at a rate some six to seven times faster than the bottom of the slip; the top center is moving at a rate about one and one-half times faster than the toe. Average rates of movement of the top of the slip range from 1.5 to 3.7 inches per month; the toe of the failure, the alluvium-colluvium bank, is moving at a rate varying from about 0.4 to 1.0 inch per month. Total resultant horizontal movement of the top of the slip ranged from about three to nine inches and at the bottom the movement ranged from approximately one to three inches. Apparently, the unstable mass has broken into two parts. It appears that the slower toe movements of the slip are causing the top of slip, the embankment, to fail at a much faster rate.

**Calculated In-Place Strength Parameters**

Stability analyses were performed using the four sections (1-1', 2-2', 3-3' and 4-4') shown in Figures 2, 9 and 10 to determine the in-place strength parameter, effective angle of internal friction, \( \phi \), of the slide under conditions presently (mid 1972) prevailing at the site. In performing the analyses, it was assumed that 1) the failure plane was circular, 2) only frictional resistance was available along the failure plane; that is, since there has been considerable movement of the soil mass, then the effective cohesion was equal to zero, 3) material in the failure zone was homogeneous, and 4) the correct factor of safety of the slip was close to unity; that is, the driving forces causing failure are equal to the resisting forces. The highest water table elevations recorded during the study period were used in the analyses. These readings were recorded during the "wet" season (winter-spring). At each section, the major failure arcs were drawn through the detected shear zones as shown in Figures 9 and 10.

Effective angles of internal friction of 10°, 18°, 23° and 30° were assumed and stability analyses were performed using each of the four sections; safety factors were computed using a computer program (Taylor's Modified Stability Equations) for each assumed \( \phi \) value. Results of these analyses are shown in a plot of assumed \( \phi \) values versus safety factor (Figure 22). For each assumed \( \phi \) value, a weighted \( \phi_w \) was determined based on the area (mass) of each section, and a \( \phi \)-safety factor curve was constructed. Assuming the factor of safety of the entire unstable mass equal to one, the weighted \( \phi_w \) value was projected to be 20.4°. Individual effective angles of shearing resistance for each section ranged from 18.3° to 23°. There was a reasonably close agreement between the weighted \( \phi_w \) (20.4°) determined in the above manner and \( \phi \) (23°) determined from consolidated-undrained tests.

**Remedial Stability Analyses**

The basic approach in developing a plan that would halt movement of the embankment consisted of determining stability of the unstable soil mass combined with various berm and water table configurations. An integral part of this plan also involves a partial change in the river channel; a berm would have to occupy part of the present channel area. Relocation of US 119 had previously been considered a possible solution; however, this approach was undesirable because it would have required some excavation of the shale cliff. There was some fear that such work might reactivate massive movements which occurred in 1967 and 1968. Furthermore, relocation of the roadway closer to the cliff would not prevent the likely total failure of the present unstable embankment. If total failure occurred after relocation, the failure might spread, enlarge, and endanger the relocated roadway.

Table 7-3 shows results of the remedial stability analyses. Various berm and embankment configurations investigated are shown in plan view, Figure 2, and cross-sectional view, Figures 9 and 10. Case 1 illustrates the effect on the safety factor when the observed phreatic surface is lowered about 18 or 20 feet in the lower portion of the slide. The weighted safety factor is increased from 1.00 to 1.10. The water table in the toe area might be lowered by constructing drainage ditches (backfilled with gravel) perpendicular to the roadway centerline (Figures 9 and 10). However, this measure would serve only as a short-term means of stabilizing the slide since such a system may not be capable of maintaining the phreatic surface at a lowered level. For instance, during flooding when the water level would rise in the lower portion and the river falls rapidly (rapid drawdown), benefits of the system would disappear. Hence, this method of stabilization would succeed only if the drainage system in the toe area could drain rapidly enough so that the phreatic surface level in the toe area fell at the same rate as the river surface. Successful application of this method would involve a trial and error procedure in the field.

Construction of Berms 1 and 2 (Cases 2 and 3) shown in Figures 9 and 10, combined with a lowered water table, would increase the weighted safety factor from unity to 1.26 and 1.42, respectively. However, when the observed pore pressures are used, the safety factor is reduced from 1.42 to 1.29 (Case 3a). Case 4 shows Berm 3 and the observed pore pressure would
increase the safety factor from 1.00 to 1.52.

Since slope indicator measurements showed that the upper portion of the slide was moving faster than the toe of the slide, stability of the top of the slide was investigated to determine the required change in the embankment slope. Failure arcs were drawn through the detected shear zones at each section as shown in Figures 9 and 10. A \( \phi \) of 20.4° was used in these analyses. The weighted safety factor for these assumed failure arcs was 1.22, Case 5. As shown by Case 5a, the weighted safety factor for the slope changes shown in Table 3 was increased from 1.22 to 1.58, which is considered sufficient to halt the assumed failure of the top of slip.

Case 6 involved checking the stability of the unstable soil mass using Berm 3, slope changes shown in Table 3, and observed pore pressures. The weighted safety factor was increased from unity to 1.47, slightly lower than the safety factor of 1.52 obtained when Berm 3 alone was used.

With regard to stability of the berms, the most critical case arises when rapid drawdown occurs. Unless pore pressures within the slope of the berm can adjust immediately to the falling water level, high pore pressures can exist, thereby lowering the safety factor. For example, Case 6a considers this condition. The weighted safety factor is reduced from 1.47 to 1.27.

A high (effective) angle of shearing resistance of 35 degrees was assumed for the berm in this analysis and the water table level in the toe area was located at an elevation of 1148 feet, the flood stage elevation of January 1946. In the analysis, the river level was dropped to normal pool.

Cases 6b and 6c are similar in every respect, except Case 6c assumed the berm was to be constructed of a material having a \( \phi \) of 20.4°. The weighted safety factor decreased from 1.27 to 1.15 when the latter condition was imposed.

The safety factor for Case 7 using the observed pore pressures, Berm 4, and the same slope changes shown in Case 6 was 1.59. For a partial rapid drawdown condition, Case 7a, the safety factor was lowered to 1.40. In the case of total rapid drawdown, the safety factor was 1.37. Case 7b was checked using the computed \( \phi \) values obtained from each individual section. The weighted safety factor was the same as obtained for Case 7b, 1.37. For these cases, the berm was assumed to be constructed of materials having a \( \phi \) of 30°.

The rapid drawdown condition considered in Cases 6a, 6b, 7a and 7b would occur if the berm was constructed of a slowly draining material. It does not account for the increase in pore pressures in the berm due to the change in water load against the slope. The latter situation arises with slow draining material when the drawdown time is much less than the time in which consolidation adjustments can occur. Hence, safety factors would be slightly less than those shown for the above cases -- depending, however, on the type of materials used.

High pore pressures can be avoided almost completely by constructing the berm of free draining material and by a liberal use of perforated pipe dispersed throughout the berm. Consequently, consolidation time will be less than the actual drawdown time if the berm is constructed of rock or gravel having a \( \phi \) of 35°. The safety factor would be maintained at a value close to 1.47. In the case of Berm 4, the safety factor would be maintained near a value of 1.59.

Hand computations were performed using Bishop's simplified method of slices for Section 2-2' and the conditions shown in Table 7-3 for Cases 6a and 7b. Also a computation was made using the Bishop method, the present cross-sectional area of 2-2' and observed pore pressures. The intent of these analyses was to check results obtained from the computer program, which uses Taylor's modified slope stability equation. For the latter case above, using a \( \phi \) of 23°, the computer program gave a safety factor of 1.00 compared to 1.20 obtained from Bishop's equations. To obtain a safety factor of 1.00 for Section 2-2', using Bishop's method, a \( \phi \) of about 19.5° would be required. For Case 6a, Bishop's method gave a safety factor of 1.19 compared to 1.24 obtained from the computer program. For Case 7b (BERM 4), using a \( \phi \) of 19.5°, Bishop's method yields a safety factor of 1.28. If a \( \phi \) of 23° (computed in-place shear strength parameter from Taylor's equations) had been used in the latter case, the safety factor would have been slightly higher. Hence, the above analyses generally show that, for a given shear strength and conditions imposed in the above cases, Bishop's method generally yields safety factors which are some 15 percent higher than those obtained from Taylor's equations. Consequently, computed, in-place shear strengths resulting from Bishop's method are lower than those from Taylor's equations. However, significant errors do not arise in the analyses using strength parameters from Taylor's equations. For example, using Taylor's equations and the computed in-place shear strength parameter (23°), BERM 4 (Case 7b) increases the safety factor from 1.00 to 1.24. When \( \phi \) (19.5°) from Bishop's equation is used, BERM 4 increases the safety factor from 1.00 to 1.28. However, when comparing laboratory strengths to field strengths, the Bishop method would be preferred because it has been demonstrated to give results closer to results obtained from methods incorporating slope stability equations which are mathematically "correct."
SUMMARY

The failure extends from Station 1260+00 to 1265+00 and from centerline to the outer edge of the river bank located on the north side of the Cumberland River. In plan view, the slide is about 500 feet in length and 170 feet in width. The unstable soil mass consists of approximately 50,000 cubic yards of material. The major failure surface is located near or slightly below the embankment and colluvium-shale rock (medium hard) interface. Movement is toward the Cumberland River in a southeasterly direction along a line paralleling the river bank located on the north side of the Cumberland River. In plan view, the slide is about 500 feet in length and 170 feet in width. The unstable soil mass consists of approximately 50,000 cubic yards of material. The major failure surface is located near or slightly below the embankment and colluvium-shale rock (medium hard) interface. Movement is toward the Cumberland River in a southeasterly direction along a line paralleling the nose of a faulted anticline. The left and right upper portions of the failure are moving about six to seven times faster than the bottom, center portion. Apparently the mass has broken into two parts. Movement of the mass as a function of time is linear — indicating movement will continue into the future until total failure occurs. However, the rate of movement may slow during the summer and fall of 1972 as the phreatic level lowers.

Failure of the embankment on both previous occasions, 1969 and 1970, was a direct result of overloading the colluvium-alluvium soil on the bank of the river. Shear stresses were increased by embankment loading to a level that equalled or exceeded the bearing capacity (available shear resistance) of materials located in the embankment and colluvium-weathered shale interface, the slip zone.

Stability of the colluvium-alluvium mass existing at the site prior to construction was precarious; that is, the safety factor of the mass was close to unity. Stability analyses of this soil formation, using Sections 2-2', 3-3' and 4-4', the computed in-place angles of shearing resistance (\(\phi\)), and assuming the phreatic level is located near ground surface, a condition occurring immediately after flooding (rapid drawdown), gave a weighted safety factor of 0.85. If observed pore pressures are used, the computed safety factor of the original soil mass is only 0.95. In these analyses, maximum shear stresses were assumed to be concentrated along the embankment and colluvium-shale interface. Failure of the river bank near Station 1259+00 in 1967 during construction when small proportionate loads were applied further exemplifies the unstable nature of the existing ground.

Certain other factors influenced failure of the embankment. Erosion at the toe of the failed mass by the Cumberland River caused an increase in shear stresses along the slip zone. The presence of a high water table in the toe area resulted in low values of effective normal stress; available shear strengths were low along this portion of the slip zone. Dipping strata increased shear stresses. Cyclic changes in the river level also abetted the failure (rapid drawdown). The combined effects of these factors and overloading caused local overstressing of the toe of the slide and initiated progressive failure. Once the major slip zone developed and movement started, shear strengths of the materials in the slip zone were lowered from their peak strength to a residual value.

Settlements resulting from consolidation of the foundation under embankment loading were an unlikely cause of the failure. Consolidation of materials of the type located in the foundation at this site normally occurs rapidly and is moderately small. Computations indicated consolidation was probably completed almost immediately after completion of the embankment. Neither was settlement of the embankment a likely cause of the failure. Boring results showed that materials in the embankment were "dry" and well compacted. Such dry and compacted materials are not likely to settle appreciably.

Findings of the study indicated that, even if efforts had been made during reconstruction to extend the drainage system longitudinally beyond its present limit, failure of the embankment would have occurred. Such a system would not have been capable of draining the toe area of the slide. During the study period, the water table level never appeared above the drainage blanket; nor did it appear about the top of the shale at Well No. 7, which was located outside the drainage system. Moreover, failure occurred in areas where embankment heights were maximum. In areas located on either side of the slide where embankment heights were nominal, failure did not occur.

RECOMMENDATIONS

In order to stabilize the present slide and increase the safety factor to an acceptable limit, it is proposed that Berm 4 of dimensions shown in Figures 2, 9 and 10 be constructed. The berm should be constructed of a free draining material (sound sandstone or a comparable material), having a minimum \(\phi\) of 30°. If a good free-draining material is used, the use of perforated pipe is optional. The face of the berm should be protected with a rip rap blanket measuring at least two feet in thickness. This blanket should extend several feet upstream and downstream from the slide area in order to prevent excessive erosion in those areas.

A partial channel change will be necessary in order to accommodate the berm and to maintain the present cross-sectional area of the channel. The proposed cut section of the channel should parallel the toe of Berm 4 (Figure 2). It is proposed that the present slopes of the embankment be decreased to dimensions shown in Figures 9 and 10. Unclassified roadway material could be used to construct these changes, although
free-draining material would be preferred if it is economically available.

Presence of the drainage system constructed in 1969 is considered an important part of the plan proposed above. This system gives some assurance that captive water will not load and weaken the embankment at some future date. A high water table in the embankment would lower the safety factor of the proposed berm.

No attempt should be made to relocate the highway closer to the cliff. Relocation is considered too dangerous since it would involve new cuts into the same shale formation which is contiguous to the base of the old Church slide. Further removal of material in this base area might possibly reactivate the larger slide. The precarious nature of this old slide is illustrated by the off-set boreholes in the shale cut.

Slope indicator wells at the site should be fully protected during reconstruction. Data from these wells could be used to evaluate the short- and long-term effectiveness of the proposed plan.

### TABLE 7-1

**WATER TABLE ELEVATIONS OBSERVED DURING THE PERIOD FEBRUARY-MAY 1972**

<table>
<thead>
<tr>
<th>DATE</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Feb. 2</td>
<td>1130.9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Feb. 8</td>
<td>1131.1</td>
<td>1151.3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>DRY</td>
</tr>
<tr>
<td>Feb. 9</td>
<td>1147.2</td>
<td>1142.3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>DRY</td>
</tr>
<tr>
<td>Feb. 10</td>
<td></td>
<td>1141.3</td>
<td>1148.3</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Feb. 15</td>
<td>1138.0</td>
<td>1148.2</td>
<td>1142.3</td>
<td>1149.3</td>
<td>1133.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Feb. 16</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Feb. 22</td>
<td>1135.5</td>
<td>1148.7</td>
<td>1142.3</td>
<td>1150.3</td>
<td>1134.4</td>
<td>1165.1</td>
<td>DRY</td>
</tr>
<tr>
<td>Mar. 22</td>
<td>1134.7</td>
<td>1147.0</td>
<td>1142.0</td>
<td>1151.2</td>
<td>1139.1</td>
<td>1161.3</td>
<td>1157.0</td>
</tr>
<tr>
<td>Apr. 11</td>
<td>1136.2</td>
<td>1145.2</td>
<td>1142.3</td>
<td>1151.1</td>
<td>1140.9</td>
<td>1161.3</td>
<td>1167.4</td>
</tr>
<tr>
<td>May 3</td>
<td>1135.2</td>
<td>1144.7</td>
<td>1142.3</td>
<td>1150.3</td>
<td>1141.4</td>
<td>1159.1</td>
<td>1166.5</td>
</tr>
</tbody>
</table>
# Table 7-2

## Summary of Laboratory Test Data

<table>
<thead>
<tr>
<th>Hole Number</th>
<th>Sample Number</th>
<th>Depth (Feet)</th>
<th>Moisture Content (Percent)</th>
<th>Description</th>
<th>Unit Weight (Pounds/Foot³)</th>
<th>Liquid Limit (Percent)</th>
<th>Plasticity Index (Percent)</th>
<th>Coarse Aggregate</th>
<th>Sand</th>
<th>Silt</th>
<th>Clay</th>
<th>Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>H-4</td>
<td>6</td>
<td>6</td>
<td>13.0</td>
<td>Brownish-Gray, Mottled, Medium Sandy Clay With Gravel</td>
<td>130</td>
<td>24</td>
<td>7</td>
<td>5</td>
<td>43</td>
<td>24</td>
<td>28</td>
<td>CL-ML A-4(61) Sandy Clay Loam</td>
</tr>
<tr>
<td>H-3</td>
<td>6</td>
<td>17</td>
<td>20.8</td>
<td>Grayish-Brown, Moist, Fine Sandy Clay With Sandstone Gravel</td>
<td>129</td>
<td>32</td>
<td>14</td>
<td>26</td>
<td>22</td>
<td>24</td>
<td>28</td>
<td>CL A-4(65) Clay Loam</td>
</tr>
</tbody>
</table>


## TABLE 7-3

### SUMMARY OF SAFETY FACTORS

<table>
<thead>
<tr>
<th>CASE NUMBER</th>
<th>COMPUTED SAFETY FACTORS</th>
<th>EFFECTIVE ANGLE OF SHEARING RESISTANCE, θ</th>
<th>WATER TABLE CONDITION</th>
<th>REMARKS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1-1'</td>
<td>2-2'</td>
<td>3-3'</td>
<td>4-4'</td>
</tr>
<tr>
<td>1</td>
<td>1.10</td>
<td>1.11</td>
<td>1.11</td>
<td>1.07</td>
</tr>
<tr>
<td>2</td>
<td>1.45</td>
<td>1.26</td>
<td>1.24</td>
<td>1.20</td>
</tr>
<tr>
<td>3</td>
<td>1.61</td>
<td>1.46</td>
<td>1.38</td>
<td>1.35</td>
</tr>
<tr>
<td>3a</td>
<td>1.41</td>
<td>1.23</td>
<td>1.26</td>
<td>1.26</td>
</tr>
<tr>
<td>4</td>
<td>1.73</td>
<td>1.49</td>
<td>1.48</td>
<td>1.47</td>
</tr>
<tr>
<td>5</td>
<td>0.96</td>
<td>1.19</td>
<td>1.19</td>
<td>1.33</td>
</tr>
<tr>
<td>5a</td>
<td>1.41</td>
<td>1.43</td>
<td>1.57</td>
<td>1.76</td>
</tr>
<tr>
<td>6</td>
<td>1.47</td>
<td>1.53</td>
<td>1.44</td>
<td>1.48</td>
</tr>
<tr>
<td>6a</td>
<td>1.12</td>
<td>1.29</td>
<td>1.28</td>
<td>1.29</td>
</tr>
<tr>
<td>6b</td>
<td>0.98</td>
<td>1.10</td>
<td>1.23</td>
<td>1.22</td>
</tr>
<tr>
<td>7</td>
<td>1.73</td>
<td>1.52</td>
<td>1.49</td>
<td>1.66</td>
</tr>
<tr>
<td>7a</td>
<td>1.39</td>
<td>1.25</td>
<td>1.43</td>
<td>1.48</td>
</tr>
<tr>
<td>7b</td>
<td>1.26</td>
<td>1.24</td>
<td>1.45</td>
<td>1.47</td>
</tr>
<tr>
<td>7c</td>
<td>1.31</td>
<td>1.38</td>
<td>1.47</td>
<td>1.37</td>
</tr>
</tbody>
</table>

*Hand Computation Using Bishop's Simplified Method of Slices*
Figure 1. View of Unstable Embankment, Looking Southwest, June 1972.
Figure 3. Typical Cross Section of Site.

Figure 4. View of Site at the Beginning of Construction, May 1967.
Figure 5. View of Failure in February 1969 after Completion of Embankment.

Figure 6. View of Site during Reconstruction, May 1969.
Figure 7. View of Cracked and Patched Pavement.
Figure 8. Geology of the Area near the Site. (from Geological Setting of Landslides Along South Slope of Pine Mountain, Kentucky, by A. J. Froelich, Research Record 323, Highway Research Board, 1970).
Figure 9. Sections 1-1' and 2-2'.
Figure 10. Sections 3-3' and 4-4'.
Figure 11. Slope Indicator Curves, Wells 1 and 2.

Figure 12. Slope Indicator Curves, Wells 3 and 4.
Figure 13. Slope Indicator Curves, Wells 6, 7, 8, and 9.
Figure 14. Boring Results and Slope Indicator Data, Holes 1 and 9.

Figure 15. Boring Results and Slope Indicator Data, Holes 2 and 3.
Figure 16. Boring Results and Slope Indicator Data, Holes 4 and 6.

Figure 17. Boring Results and Slope Indicator Data, Holes 7 and 8.
Figure 18. Boring Results, Holes 10, 11, and 12.
Figure 19. Surface Movement Data, Lines "B" and "C".
Figure 20. Surface Movement Data, Line "A", and Surface Movement for a Selected Station as a Function of Time for Lines "A", "B", and "C".
Figure 21. Consolidated-Drained, Triaxial Test Results.
Figure 22. Assumed Angle of Shearing Resistance-Safety Factor Curves.