Research Report
-531-

CORRECTIVE MEASURES FOR UNSTABLE BRIDGE-APPROACH EMBANKMENT

US 68, LICKING RIVER, BLUE LICKS

KYP-72-38, HPR-PL-1(15), Part III B

by

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MEMO TO: G. F. Kemper
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SUBJECT: Research Report 531; "Corrective Measures for Unstable Bridge-Approach Embankment; US 68, Licking River, Blue Licks; " KYP-72-38; HPR-PL-1(15), Part III-B

By prior arrangement with the Division of Materials, Research accepted responsibility for investigating and analyzing specific landslides. The one reported now has been awaiting a plan for correction for a long time. The pattern is not new; however, the investigative and analytical processes were nonetheless tedious and demanding. The corrections proposed are very straightforward, also. We recommend, of course, that the work be authorized and programmed without undue delay.

Respectfully submitted,

Jas. H. Havens
Director of Research

gd
Enc.
cc's: Research Committee
Henry Mathis

INTRODUCTION

In January 1979, an in-depth investigation of the unstable bridge-approach embankment, Figure 1, on US 68 over the Licking River in the north central portion of Nicholas County (MP 91-68-B0027) was begun by the Division of Research. This investigation was a continuation of a project initiated by the Division of Materials in 1967. This site is approximately 0.3 miles (0.5 km) north of Stony Creek Road and approximately 7.5 miles (12.0 km) north of Carlisle and 0.8 miles (1.3 km) southwest of Blue Licks Battlefield State Park. The bridge is a three-span, continuous, welded plate girder structure with span lengths of 160, 200, and 160 feet (48.8, 61.0, and 48.8 m) at a 0 degree skew. The bridge is aligned at north 47 degrees east. The pier foundations are spread footings on rock; the abutments utilize 12 point-bearing (53 steel-type) piles. The roadway emerges from a cut of approximately 20 to 40 feet (6.1 to 12.2 m) near Stations 217 + 50 and 221 + 00 on the right and left, respectively. The nearby bridge approach embankment rests partly on a bench of limestone and partly on an alluvium river bank. Maximum height of the embankment is about 100 feet (30.5 m).

Relocation of US 68 began in early 1967. Subsidence occurred near Station 226 + 50, opposite the embankment currently under investigation. The subsidence was corrected by adding a berm at the 603.5-foot (183.9-m) elevation. Construction was resumed when subsidence appeared to have

ceased. The bridge was completed in 1969. In December 1975, movement of the west abutment and approach embankment required extensive patching of the pavement and repair of the guardrail near the abutment (Figures 2 and 3). On the basis of displaced guardrail, the unstable embankment extended from approximately Station 220 + 00 to the bridge. This information was useful in establishing the configuration of a possible deep failure surface. Movement of the bridge abutment has also been evidenced by an extreme tilt of rocker arms and jamming of the bridge girders against the concrete abutment; this is shown in Figure 3. Figure 4 shows break up of one of the expansion dams on the west abutment. This was additional evidence that the abutment was moving toward the river because rocker arms on top of both piers were still vertical. Voids and cracks are shown in Figure 5 along the front of the soil-abutment interface. These openings were as large as 2 to 3 feet (0.6 to 0.9 m) deep and 3 feet (0.9 m) wide. This suggested the presence of a shallow slide developing directly in front of abutment.

Because the embankment partially rests against a hillside, seepage along the original foundation may have raised the water table into the fill, lowered the factor of safety, and caused the movements. Erosion of the toe of the embankment may have contributed to present movements. Upstream from the bridge, the lower end of a concrete-lined ditch collapsed because supporting soil eroded. Downstream from the bridge, eddy currents apparently caused the deep

cuts shown in Figure 6. The effects of erosion and a high water table may have been aggravated by a rise in the normal backwater curve due to debris piled up in front of the old US-68 bridge downstream from the present bridge (Figure 7.)

TOPOGRAPHY AND GEOLOGY

The site lies in the Outer Bluegrass physiographic region of Kentucky (Figure 8), generally a low-lying or low-relief area with rolling hills. Formations of the Inner Bluegrass outcrop briefly at the Licking River. The Knobs lie to the east. Figure 9 gives a stratigraphic section of the geology crossed by US 68 in the vicinity of the Licking River[1]. Recent alluvium covers the portion of the site near the Licking River while the roadway is located in an area overlaid in part by Ordovician rocks, which include the Clays Ferry and several members of the Lexington Limestone Formations. The Clays Ferry Formation is composed of about 50 percent limestone and has a high shale and clay content which can cause instability on steep slopes. The Lexington Limestone Formation includes the Tanglewood, Millersburg, and Grier Limestone Members. Tongues of the Clays Ferry Formation of intervals of as much as 10 feet (3.0 m) are found between the Millersburg and Grier Limestone Members. Members of the Lexington Limestone Formation vary between 70 percent (Millersburg) to 90 percent (Tanglewood) dark gray, fine crystalline, hard limestone. These members are irregularly imbedded with 10 to 15 percent dark gray shale.

The unstable portion of the west embankment rests partly on a limestone-shale bench and partly on a river bank composed of alluvium.

FIELD INVESTIGATION

Subsurface exploration on the west approach embankment began with one boring in late 1966 at centerline at Station 221 + 60. Two 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: D1587. Three borings were made by Division of Materials in April 1976. The location of these and subsequent borings to be discussed are shown in Figure 10. The first three borings were:

Hole No. 1 - 33 feet (10 m) left of centerline at Station 221 + 43

Hole No. 2 - 98 feet (30 m) left of centerline at Station 221 + 43

Hole No. 3 - On the centerline at Station 222 + 46

A total of 15 thin-walled tube samples were taken from Holes 1 and 2. Rocky soils at Hole 3 prevented sampling with Shelby tubes. Slope inclinometer casings were installed at each hole to determine rates and directions of movement and the locations of any shear zones. Data from these holes showed that the embankment was moving toward the river in a northerly direction. Consequently, a slope inclinometer casing was installed along this direction of

movement in September 1977. This well was placed 108 feet (33 m) left of centerline at Station 222 + 27 or about 45 degrees skewed to the left of the centerline from Hole No. 1 while facing east. Four split-spoon samples were obtained from Hole 4 by the Division of Materials. Cores were geologically logged. Cross sections of the abutment embankment were taken to establish the existing groundline through holes along the centerline (Holes 1 and 3), holes perpendicular to the centerline (Holes 1 and 2), and holes skewed in a northerly direction from the centerline (Holes 1 and 4).

Table 1 gives the maximum and average water table elevations observed in Holes 1, 2, 3, and 4. Figure 11 gives the water-table elevation as a function of elapsed time in days. Results for Holes 1, 2, and 4 are presented in Figure 12A. Results for Hole 3 and subsurface explorations prior to construction are presented in Figure 12B. Attempts to obtain tube samples were unsuccessful at Hole 3 because of rocky soil. However, samples were successfully obtained from Holes 1 and 2 for classification purposes and strength determinations.

Slope inclinometer results for the four holes are shown in Figures 13, 14, 15, and 16. Resultant horizontal movements and dial changes are plotted as a function of depth. Horizontal movement parallel to centerline and toward the Licking River is shown in Figure 13B. Movement-time curves are given for all four holes in Figures 13

through 16. Relative magnitudes of these movements are shown in Figure 10. Hole 4 indicates less movement because it was installed 16 months later. Table 2 is a summary. A maximum rate of movement of 0.056 inches (1.4 mm) per month was found for the period between August 1977 and August 1978. This rate of movement has remained constant through August 1979 and is less than the 0.125 inches (3.2 mm) per month recorded in April 1976.

From May 1976 through September 1977, readings were taken at Holes 1, 2, and 3 using the Digitilt Indicator Machine, Model 50301. On September 20, 1977, readings were taken using a Mag-Tape Digitilt, Model 50308, and compared with those obtained before using Model 50301. The Digitilt Mag-Tape, Model 50308, was used thereafter. In October 1977, the Division of Materials discovered errors when data from both machines were combined. Those errors were attributed to differences in the orientation of the accelerometers in the probes. Also, data obtained from Hole 3 with the Model 50301 inclinometer were discarded because excessive settlement occurred in the backfill during the initial readings. Therefore, only data obtained with the Model 50308 were considered on Hole 3. Hole 1 also underwent some subsidence, but readings previous to September 1977 were retained because they closely resembled data from the new Mag-Tape machine. Readings previous to September 1977 were also retained for Hole 2. A final set of readings were taken on Holes 1 and 2 with the Model 50301

in February 1979 and combined with readings taken before September 1977. Readings taken during this period with the Model 50308 showed a constant rate of movement. Consequently, readings taken with the Model 50301 inclinometer have been reported because they spanned a longer observation period. All inclinometer readings for Hole 4 were obtained using the Model 50308.

LABORATORY INVESTIGATION

Soil samples extruded from Shelby tubes obtained in 1967 and 1976 were cut into 4-inch (100-mm) lengths and identified according to the visual-manual procedure (ASTM Designation: D 2488). A tabulation of soil index properties is presented in Table 3. Four relatively undisturbed soil samples were obtained in 1967. Triaxial consolidated-undrained (CU) test results are presented in Figure 17. Because time-deflection data were not recorded during the triaxial test performed in 1967, results of the variation of pore pressure and deviator stress as a function of ordinary strain (in percent) could not be presented. Eleven relatively undisturbed soil samples were obtained from Holes 1 and 2 in 1976; consolidated-undrained direct shear tests were performed on these samples. The consolidated-undrained direct shear test is something between the consolidated-drained and the consolidated-undrained triaxial tests. These tests were run to investigate the residual shear strength characteristics of

the embankment soils. Results of the direct shear tests are given in Figures 18, 19, 20, and 21. Table 4 summarizes shear strength properties as determined from laboratory tests performed on samples extruded from Shelby tubes and from empirical relationships[2]. Table 5 summarizes estimates of shear strength from standard penetration data using De Mello's Method[3].

Based on borings and laboratory tests, the embankment soil consists of dry, dark gray clays containing considerable amounts of silt mixed with shale rock. These soils classify as clay, CL, and A-6(7) through A-6(19) and are low to moderately plastic. Embankment soils above the 580-foot (177-m) elevation contain between 34 and 41 percent silt and 48 and 55 percent clay with traces of fine sand of about 2 percent. Soils below the 580-foot (177-m) elevation contain appreciable amounts of silt but still plot above the A-line of liquid limit-plasticity chart. These soils have the textural classifications of silty clay and silty clay loam and are located in the vicinity of Holes 2 and 4. Foundation material from Hole 4 contains substantial quantities of fine sand (23 percent) with traces of coarse sand (2 percent). Natural water contents of all 14 soil samples was around 20 percent. The liquid limit and plasticity indices ranged from 35 to 40 percent and 14 to 20 percent, respectively.

STABILITY ANALYSIS

Figures 13, 14, 15, and 16 suggest the presence of both circular and wedge shaped shear failures. Three cross sections were analyzed: 1) perpendicular, 2) parallel, and 3) about 45 degrees skew to centerline. Failure circles were found in each using ICES Lease . Wedges were investigated using the SWASE program[5]. The water table representing normal conditions was defined using the highest observed water levels in Holes 1, 2, 3, and 4. These elevations are given in Table 1. Normal pool elevation of Licking River was taken to be 567 feet (173 m). This is 7 feet (2 m) higher than normal pool elevation of 560 feet (171 m) observed in 1967. Flood stage was taken to be 602 feet (183 m); that is 1 foot (0.3 m) above highwater elevation of the 1963 flood.

CIRCULAR FAILURE MODES

The three sections shown in Figures 22, 23, and 24 were analyzed in terms of a long-term effective stress. Results are summarized in Tables 6, 7, and 8. The effective angle of internal friction, $\emptyset'$, was 23 degrees, which was obtained from the triaxial tests performed in 1967. The effective cohesion, c', was set to zero. Several grid searches were performed to find the failure circles. The circles which best explained the slope indicator data are shown in Figures 22, 23, and 24. In-place (backed-in) values of $\emptyset'$ were found for each circle by determining the $\emptyset'$ which would yield a factor of safety of one. The in-place values of $\emptyset'$

were averaged for each section. The smallest in-place average value was 23 degrees.

Using the critical circles given in Figures 22, 23, and 24, several berm configurations were investigated with and without the rapid drawdown. Rapid drawdown was from the 602-foot (183-m) elevation. Tables 6, 7, and 8 summarize the results from these analyses and include results for the existing slope geometry (that is, without berms) and highest observed water table elevations. The most critical section for a circular-type failure was the left-of-centerline cross section shown in Figure 22. Those stabilities are summarized in Table 6. Circle 1 fits the slope indicator data of Holes 1 and 2 extremely well. It also explains much of the observed displacement of guardrail near the bridge abutment. A berm with a 2:1 slope beginning at the 635-foot (194-m) elevation would give a factor of safety of 1.381; this reduces to 1.257 with rapid drawdown. This berm would have two 20-foot (6-m) benches at the 635- and 615-foot (194- and 187-m) elevations. The effect of rapid drawdown is much less if a berm with a 3:1 slope is extended from a 20-foot (6-m) bench at the 635-foot (194-m) elevation. This would give a factor of safety of 1.524 which reduces to 1.398 under conditions of rapid drawdown. All berms discussed herein are assumed to be free draining and composed of material having a minimum angle of internal friction of 30 degrees.

The critical circles for the cross secton parallel to the centerline are shown in Figure 23 along with possible berm configurations. These circles seem to explain the slope inclinometer movements at Holes 1 and 3. However, Figure 10 shows that Holes 1 and 3 are not moving parallel to the centerline, but movement is skewed in a northerly direction. The retaining-wall effect of Pier 1 probably prevents or forestalls a major slide along the easterly direction. However, this effect could not be accounted for in the ICES Lease program. Therefore, the results given in Table 7 are on the low side of a range of possible safety factors. The toe circle in Figure 23 (Circle 1) has the lowest factor of safety, 0.861, because a substantial portion of the circle is below the water table. However, field inspections have revealed no evidence of tension cracks or any other signs of movement at the toe. Pier 1 apparently has prevented this circle from developing.

Circle 2 seems to explain the guardrail and slope inclinometer movement fairly well. Figure 23 shows only the A-axis movement (parallel to centerline) for Hole 1. In Figure 23, the shear zone found below Hole 3, near the water table, is probably associated with a wedge rather than a circular slide because this slope inclinometer data correlate better with the movements slightly above the water table at Hole 1. In addition, because Hole 1 is located 33 feet (10 m) to the left of centerline while facing east, movements near the top of Hole 1 may not be related to those

at Hole 3. Field inspections of the toe of the slope did not confirm existence of Circle 2. Expansion shoes on Pier 1 were not tilted, and evidence of soil "flow" around the base of the pier was not found.

A shallow failure surface similar to Circle 3 may exist along the centerline slope as shown in Figure 23. This circle would explain the displaced guardrail immediately behind both sides of the abutment and the depressions between the abutment and soil directly in front of the bridge. This circle may also explain some of the movement near the top of Hole 1.

Table 7 shows that Berms 1 and 2 improve the stability of Circle 3 from a factor of safety of 0.897 to 1.419. In contrast, the stability of Circles 1 and 2 improves only slightly. The Licking River limits the amount of berm material which can be placed to resist movements of Circles 1 and 2. However, these circles appear to be stable because of the retaining-wall effect of Pier 1.

The critical circles for the cross section taken skew to the centerline are shown in Figure 24 along with suggested berm configurations. This cross section approximately parallels the directions of slope inclinometer movement given in Figure 10. Of the critical circles found on this cross section, only the toe circle (Circle 1) had a factor of safety less than 1.0. This is because a substantial portion of Circle 1 lies below the water table. The location of the water table as it intersects the original

groundline near the toe was confirmed in a recent field inspection (April 1979). The scarp at the toe of this section (Figure 6) actually may be due to failure of several small sluff circles similar to Circle 1 instead of erosion, as previously suggested. Table 8 shows that small increases in stability will result with the addition of Berms 1 or 2. However, this escarpment area should be filled with rock as soon as possible to prevent further erosion.

Additional failure circles may be present as shown in Figure 24. Circle 2 may exist because Hole 4 is moving directly toward the river, as shown in Figure 10. Therefore, the increase in the factor of safety from 1.056 to 1.286 due to the addition of Berms 1 or 2 is very desirable.

Circles 3 and 4 are relatively shallow and have factors of safety greater than one. Table 8 shows that only the stability at Circle 4 would be improved with the addition of Berms 1 or 2. In contrast, the stability of Circle 3 is not affected. In any case, field inspections have revealed no evidence that these circles exist. However, while the addition of Berm 3 greatly improves the stability of Circle 3, Table 8 shows a reduction in the stability of all other critical circles, in particular, Circle 1. Unless definite field evidence is found for the existence of Circle 3, only Berms 1 or 2 should be used.

CORRECTIVE MEASURES FOR CIRCULAR MODES OF FAILURE

Figure 25 summarizes the measures proposed for correcting the possible circular modes of failure. The cross section left of centerline is the most critical. If right-of-way limitations permit, a berm with a 20-foot (6-m) bench and a 3:1 slope should be built from the original 635-foot (194-m) groundline elevation of the left cross section. In addition, a berm with a bench of approximately 40 feet (12 m) and having a 1.5:1 slope should be built from the 615-foot (187 m) elevation along the embankment of the Licking River. Efforts should be made to extend this berm as far past the river bank as possible.

WEDGE FAILURE MODES

Over 70 wedge configurations were analyzed searching for the critical configurations for the three cross sections shown in Figures 26, 27, and 28. The effective angles of friction along the failure surfaces were 23 and 16 degrees. The angle of friction within each wedge was assumed to be 23 degrees. Backed-in values of ϕ' were calculated from these results and are tabulated on Figures 26, 27, and 28. For simplicity, the embankment soils were assumed to have uniform unit weights of 125 pounds per cubic foot (2,002 kg/m^3). Each cross section was assumed to be infinitely wide. Because this last assumption is not valid, especially in the case of the skewed section shown in Figure 28, the critical wedge was not selected on the basis of the lowest factor of safety but according to the failure surface which

best correlated with slope inclinometer movements and observed guardrail displacement. This was best explained by Wedge 6 of Figure 28. Angles of 21.5 and 8.5 degrees with the horizontal for the two planes of sliding of Wedge 6 were used. A length of 184.7 feet (56.3 m) was used for the top sliding surface. The slope of the groundline was approximately 20.5 degrees. The pore pressure parameter, ru, was set equal to 0.05 to approximate the effect of the water table at the highest observed well elevations. A friction angle of 16 degrees was assumed; zero pounds per square foot (0 kg/m^3) was assumed for cohesion. The internal friction was obtained from consolidated-undrained direct shear tests and used as the residual value of $\emptyset'$. When plane-strain conditions are assumed, Wedge 6 has a factor of safety of 1.170. When ICES Lease[4] approximated Wedge 6 as a circle, a factor of safety of 1.15 was obtained.

The skewed cross section in Figure 28 lies on the intersecting edge of the cross sections in Figures 26 and 27. The section properties change significantly in the direction perpendicular to the plane of Figure 28. consequently, too much error results in the assumption of a plane-strain situation. Instead of assuming plane-strain, weights of the sliding block above each failure plane were used. In addition, because the use of berms to increase stability was limited by the proximity of the Licking River, a design using a lightweight fill was investigated.

The lightweight material is produced commercially by the Elastizell Lightweight Concrete Corporation of Dayton, Ohio. Two tests were made by Division of Research to determine the long-term wet density of this material (55 pounds per cubic foot (880 kg/m³) (Test 2 in Figure 29)). Elastizell Corporation has reported that the lightweight material having this wet density would have a compressive strength of between 260 and 400 pounds per square inch (1.79 and 2.76 MPa). A compressive strength of 200 pounds per square inch (1.38 MPa) was selected for design. Assuming a Ø-equal-zero analysis, half of this compressive strength would be equivalent to an undrained cohesion, c_u, of 100 pounds per square inch (0.69 MPa).

The wedge configuration shown in Figure 30 was used to analyze two conditions: 1) zero cohesion and 2) cohesion of 100 pounds per square inch (0.69 MPa). The driving wedge (upper block) was estimated to weigh 24,902 kips (110 MN) and act over a distance of 184.7 feet (56.3 m). The resisting wedge was estimated to weigh 79,260 kips (353 MN) and act over a distance of 265 feet (81 m). The value of internal friction along the failure surface was assumed to be 16 degrees. Any increase in the overall value of internal friction due to the addition of the lightweight fill was ignored. A uniform density of 125 pounds per cubic foot (2,002 kg/m³) was assumed for the embankment in the calculation of these block weights. Replacements of portions of the driving wedge with varying lifts of

lightweight fill reduced the driving force and increased the factor of safety. When the cohesion of the lightweight fill was ignored, the lightweight material was treated as a cohesionless material with a ϕ' of 16 degrees. When the compressive strength of lightweight fill was considered, the Elastizell Concrete was treated as a cohesive soil with a cohesion of 100 pounds per square inch (0.69 MPa) and a ϕ' of 16 degrees. The cohesion of the lightweight fill only acts along a portion of the total failure surface. Table 9 provides a summary of this procedure along with computed factors of safety. Figure 31 shows the variation of safety factor as a function of fill thickness for the cohesionless and cohesive lightweight fill materials described above.

CORRECTIVE MEASURES FOR WEDGE MODES OF FAILURE

Figure 31 summarizes the factors of safety obtained for different thicknesses of lightweight fill. Figure 31 shows that a cohesionless, lightweight fill configuration requires a thickness of at least 27 feet (8 m) to yield a factor of safety of 1.5. A lift of at least 30 feet (9 m) should be used in design. In contrast, a cohesive, lightweight (Elastizell concrete) fill configuration requires a thickness of only 10 feet (3 m) to give the same factor of safety. Figure 32 details the proposed lightweight fill configurations. The cross sections for the left and centerline wedges were estimated from observed guardrail and slope indicator movement.

RECOMMENDATIONS

To stabilize the approach embankment and increase the safety factor to an acceptable limit, it is proposed that the berm and lightweight fill configurations shown in Figures 25 and 32, respectively, be constructed. The berms should be constructed first to stabilize against the circular modes of failure shown in Figures 22, 23, and 24. Berms should be constructed of free-draining materials (sound limestone or a comparable material) having a minimum Ø' of 30 degrees. If right-of-way limitations permit, the berm left of centerline should be constructed with a 3:1 slope from a 20-foot (6-m) bench at 635-foot (194-m) elevation. If a free-draining material is used, the use of the drainage blanket is optional. If a drainage blanket is constructed, a 36-inch (0.9-m) blanket of No. 9 stone should be placed against existing slope below the 625-foot (190-m) elevation as shown in Figures 22 and 25. Provisions should be made for drainage of the upper flat reaches of each berm. A collector system should be used to funnel all excess water to drainage pipes. Water should be channeled into existing paved ditches. A perforated pipe should be placed in the trenches shown in Figures 22 and 25 at the toe of the left-of-centerline cross section and backfilled with rock or any other free-draining material. The face of the berm along the river should be protected with rip rap at least 2 feet (0.6 m) in thickness. This blanket should extend several feet upstream and downstream from the embankment area to minimize erosion.

Slope inclinometer wells at the site should be fully protected during installation of the berms. Data from these wells should continue to be taken to evaluate the short- and long-term effectiveness of the proposed berm configuration.

The need for a partial channel change to accomodate the proposed berm configuration along the west side of the Licking River during periods of high flow should be investigated. Debris piled in front of the old US-68 bridge should be cleared. This should increase the hydraulic efficiency of the river channel and reduce the elevation of the backwater curve.

Upon completion of the proposed berms, slope inclinometer data should be obtained to evaluate effectiveness of newly constructed berms. A decision should be made then on whether to proceed with one of the lightweight fill configurations proposed in Figure 32 to increase the stability of the embankment against the deep-seated wedge failure shown in Figure 30. Replacement of a portion of the upper reaches of the approach embankment with lightweight material will require closing the bridge and rerouting traffic over the old US-68 bridge. In addition, about 200 feet (60 m) of the present roadbed would have to be removed and rebuilt after placement of the lightweight fill. The lightweight fill configurations (cohesionless or cohesive) should have dimensions shown in Figure 32 to insure a safety factor of 1.5. In both cases, the fill material should not have a density greater than 55 pounds

per cubic foot (880 kg/m³). Provisions should be made to include temperature reinforcement to prevent excessive cracking in lightweight concrete fill (e.g. Elastizell). Finally, measures should be taken to insure that the lightweight fill is properly drained.

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TABLE 1. WATER-TABLE ELEVATIONS (IN FEET) DURING PERIOD OF MAY 1976 THROUGH FEBRUARY 1979

	HOLE 1	HOLE 2	HOLE 3	HOLE 4
MAXIMUM	589.3	584.6	577.3	576.2
AVERAGE	583.0	581.6	576.3	576.2
ASSUMED PEAK	589	588	580	576

NOTE: 1 foot = 0.3048 m

TABLE 2. SUMMARY OF SLOPE INCLINOMETER DATA AS OF FEBRUARY 1979

HOLE NUMBER	DEPTH (FEET)	MAXIMUM DISPLACEMENT (INCHES)	RATE (INCHES/MONTH)	TIME PERIOD FOR RATE
1	11	2.4	0.055	8/77 - 8/78
	12	2.8	0.055	
	59	1.0	0.015	
2	2	2.8	0.056	8/77 - 8/78
	18	1.9	0.030	
	38	1.3	0.020	
3	14	0.55	0.003	8/77 - 8/78
	28	0.30	0.020	
	34	0.28	0.015	
4	9	0.62	0.023	12/77 - 8/78
	21	0.48	0.010	
	35	0.39	0.006	

NOTE: 1 foot = 0.3048 m
1 inch = 25.4 mm

TABLE 3. SUMMARY OF INDEX PROPERTIES

HOLE NUMBER	STATION	SAMPLE DEPTH (FEET)	LIQUID LIMIT (PERCENT)	PLASTICITY INDEX (PERCENT)	MOISTURE CONTENT (PERCENT)	LIQUIDITY INDEX	DESCRIPTION	CLASSIFICATION	
								UNIFIED	AASHTO
1	221+48	15-17	39	19	21.4	0.07	Gray Shale	CL	A-6(19)
	33' Lt	30-32	40	19	19.8	-0.06	----	CL	A-6(19)
		60-62	37	15	17.6	0.37	----	CL	A-6(15)
2	221+43	35-37	40	20	----	----	----	CL	A-6(16)
	98' Lt	50-52	40	17	31.6	0.51	Dark Brown Silty Clay	CL	A-6(18)
4	222+27	33-37	37	14	----	----	Dark Brown Silty Clay	CL	A-6(15)
	108' Lt	40-42	34	11	----	----	----	CL	A-6(7)
	222+00 CENTER-LINE	FOUNDATION (1965, top of layer)	36	13.6	31.2	0.65	----	CL	A-6(9)

NOTE: 1 foot = 0.3048 m

TABLE 4. SUMMARY OF SHEAR STRENGTH PROPERTIES

HOLE NUMBER	STATION	SAMPLE DEPTH (FEET)	PLASTICITY INDEX (PERCENT)	ø' (DEGREES) [1]	CLAY FRACTION (%<0.002mm) [2]	RESIDUAL ø DEGREES [3]	RESIDUAL ø DEGREES [4]	TRIAXIAL c' (psf)	TRIAXIAL ø' (degrees)
1	221+43	15-17	19	33-26	29	24	13	---	---
	33' Lt	30-32	19	33-26	28.5	24.3	16	---	---
		60-62	15	36-31	34.0	21.9	---	---	---
2	221+43	35-37	20	33-25	23.8	26.6	16.7	---	---
	98' Lt	50-52	17	34-26	19.0	29.6	15.6	---	---
Foundation	221+60 Center-Line	80[5]	----	----	----	----	----	610	23
4	222+27	33-37	14	35-27	18	30.3	----	---	---
	108' Lt	40-42	11	36-29	19	29.6	----	---	---
Foundation	222+00	Profile (1965)	13.6	35-27	25.45[6]	25.7	----	---	---

[1] ø = 44.7 - 12 Log(PI) ± 3.67. (Ref 2)
[2] Extrapolated from clay (-0.002 mm).
[3] ø = 68.2 - 30.2 Log(CF, % < 0.002). (Ref 2)
[4] Direct Shear Test.
[5] Includes addition of 73' of embankment above original depth of 5-7' in 1967.
[6] Interpolated from clay between (-0.074 and -0.001 mm).

NOTE: 1 foot = 0.3048 m

TABLE 5. ESTIMATES OF SHEAR STRENGTH FROM STANDARD PENETRATION TESTS; HOLE 4, STATION 222 + 27, LEFT 108 FEET

DEPTH	SOIL DESCRIPTION	BLOW COUNT N	OVERBURDEN PRESSURE (TSF)	Ø' (DEGREES)[1]
33.5	Dark Brown Silty Clay	13	1.926	32
36.5	Dark Brown Silty Clay	13	2.099	32
41.5	Dark Gray Clay	15	2.386	32
46.5	Dark Gray Silty Sand	12	2.674	30

[1] Material treated as sand and De Mello's Method (2) used to estimate Ø' from SPT Data.

NOTE: 1 TSF = 0.0479 Pa

TABLE 6. SUMMARY OF CIRCULAR STABILITY ANALYSES ON LEFT-OF-CENTERLINE CROSS SECTION (EFFECTIVE STRESS ANALYSES; Ø' = 23°, c' = 0)

	FACTORS OF SAFETY					
		WITH BERMS		WITH BERMS AND RAPID DRAWDOWN		
CIRCLE NUMBER	WITHOUT BERM	2:1	3:1	2:1	3:1	BACKED-IN Ø' (DEGREES)
1	0.897	1.381	1.524	1.257	1.398	26.3
2	0.859	1.436	1.520	1.357	1.445	27.0
AVERAGE						26.7

TABLE 7. SUMMARY OF CIRCULAR STABILITY ANALYSES ON CENTERLINE CROSS SECTION (EFFECTIVE STRESS ANALYSES; $\phi' = 23^{\circ}$, $c' = 0$)

	FACTORS OF SAFETY					
				RAPID-DRAWDOWN		
CIRCLE NUMBER	WITHOUT BERMS	BERM 1	BERM 2	BERM 1	BERM 2	BACKED-IN ϕ' (DEGREES)
1	0.861	0.956	0.947	0.806	0.821	26.0
2	0.917	1.030	1.064	0.912	0.952	24.8
3	0.897	1.061	1.419	1.061	1.419	24.0
AVERAGE						24.9

TABLE 8. SUMMARY OF CIRCULAR STABILITY ANALYSES ON SKEWED CROSS SECTION (EFFECTIVE STRESS ANALYSES; $\phi' = 23^{\circ}$, $c' = 0$)

	FACTORS OF SAFETY						
					RAPID-DRAWDOWN		
CIRCLE NUMBER	WITHOUT BERMS	BERM 1	BERM 2	BERM 3	BERM 1	BERM 2	BACKED-IN ϕ' (DEGREES)
1	0.853	1.030	0.902	0.816	0.868	0.789	26.3
2	1.056	1.242	1.286	1.182	1.015	1.071	22.0
3	1.142	1.142	1.142	1.479	1.142	1.142	22.0
4	1.034	1.174	1.191	1.133	0.974	1.015	22.3
5	1.158	1.267	1.303	1.279	1.087	1.130	22.0
AVERAGE							23.0

TABLE 9. WEDGE ANALYSIS SUMMARY FOR COHESIVE, LOW-DENSITY FILL (ELASTIZELL CONCRETE)

FILL THICKNESS (FEET)	Ø FAILURE SURFACE (DEGREES)	WEDGE LENGTH W/COHESION (FEET)	ADJUSTED COHESION[1] (PCF)	FACTOR OF SAFETY[2]
5	16	118	3800	1.450
10	16	118	3800	1.494
15	16	137	4400	1.557
20	16	137	4400	1.614
30	16	154	4900	1.758
40	16	184.7	5900	1.910
50	16	184.7	5900	2.032

[1]Given: Total length of Failure Surface = 449.7 feet
Cohesion of 100 psi = 1.44×10^4 psf
Adjusted Cohesion = (Length with cohesion/total length) X 1.44×10^4 psf

[2]SWASE Program

NOTE: 1 foot = 0.3048 m
1 pound per cubic foot = 16.0 kg/m^3

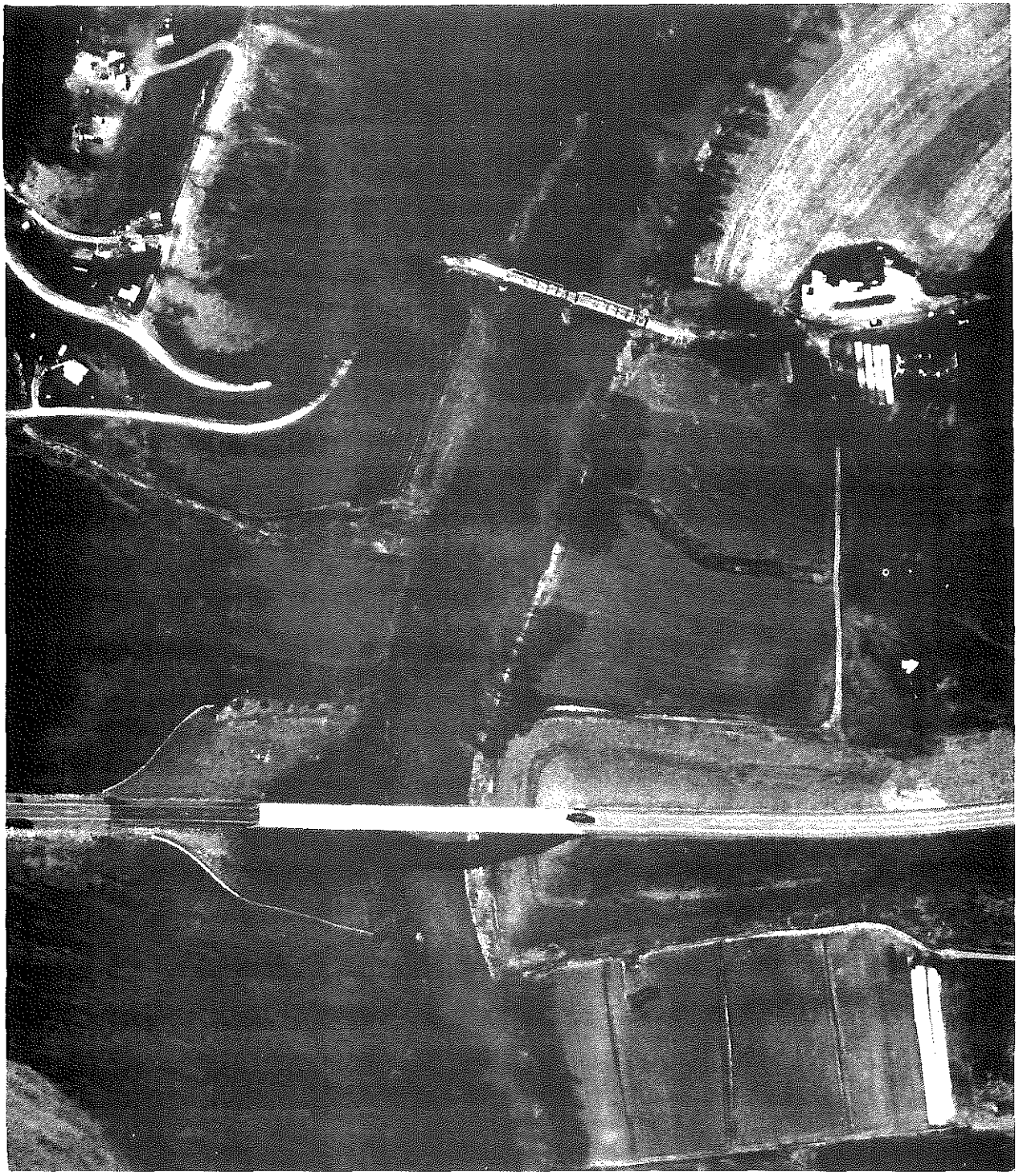
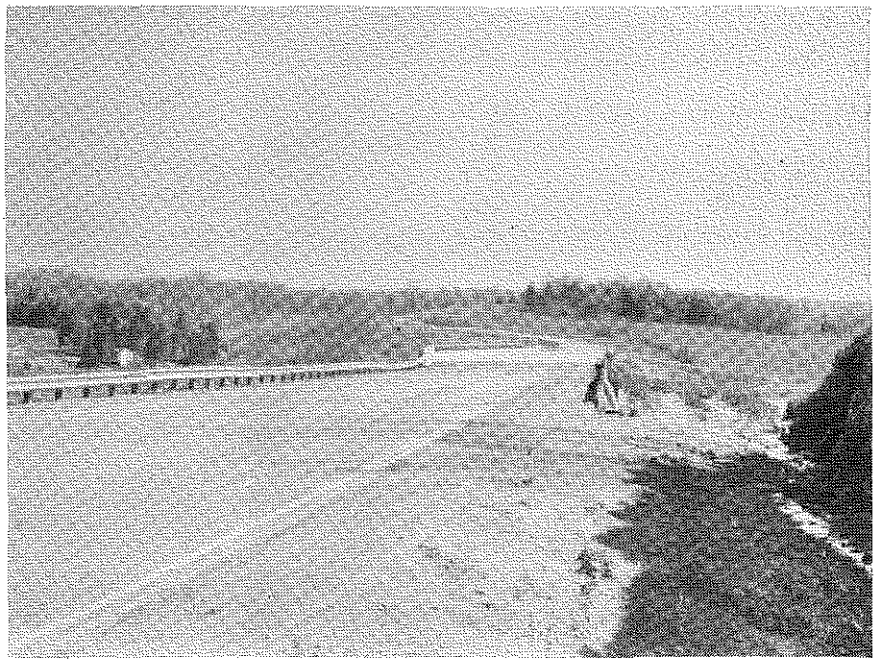
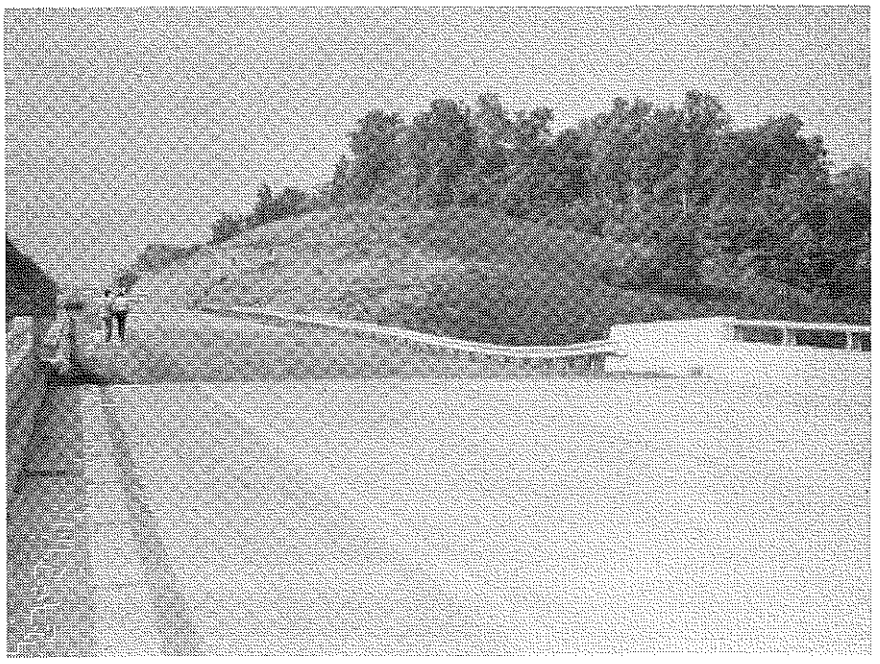


Figure 1. Aerial Photograph of US-68 Bridge over the Licking River, Blue Licks (East is to the Right).



a



b

Figure 2. View of Displaced Guardrail, West End of Bridge over the Licking River on US 68 (a) Looking East and(b) Looking West.

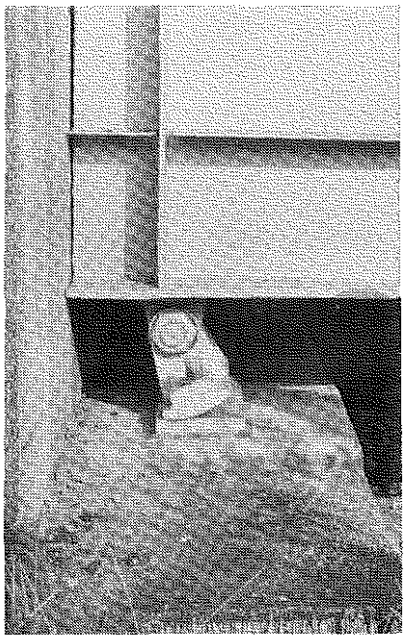


Figure 3. Extreme Tilt of Rocker Arms and Jamming of Bridge Girders against Concrete Abutment.

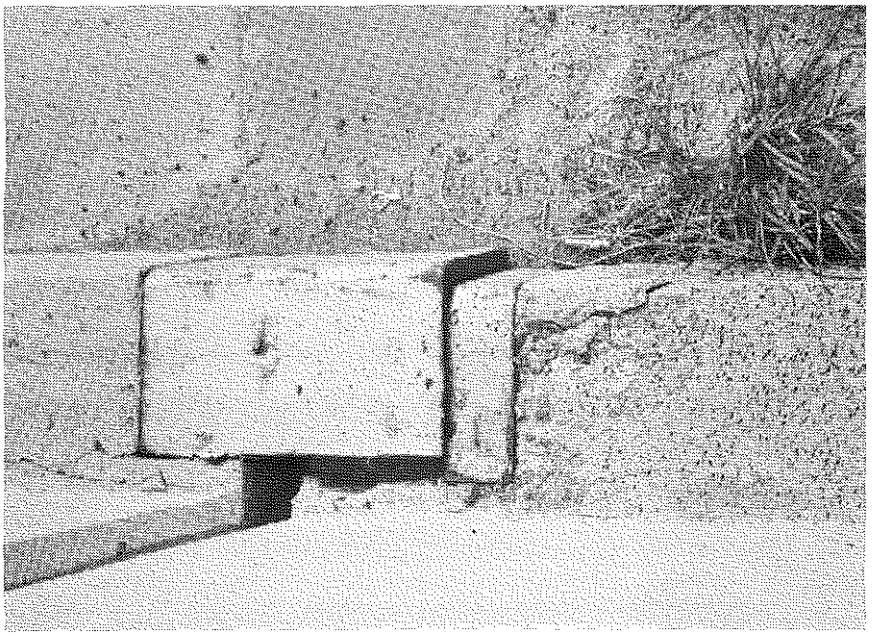


Figure 4. Breakup of Concrete Expansion Dams on West Abutment of US-68 Bridge over the Licking River.

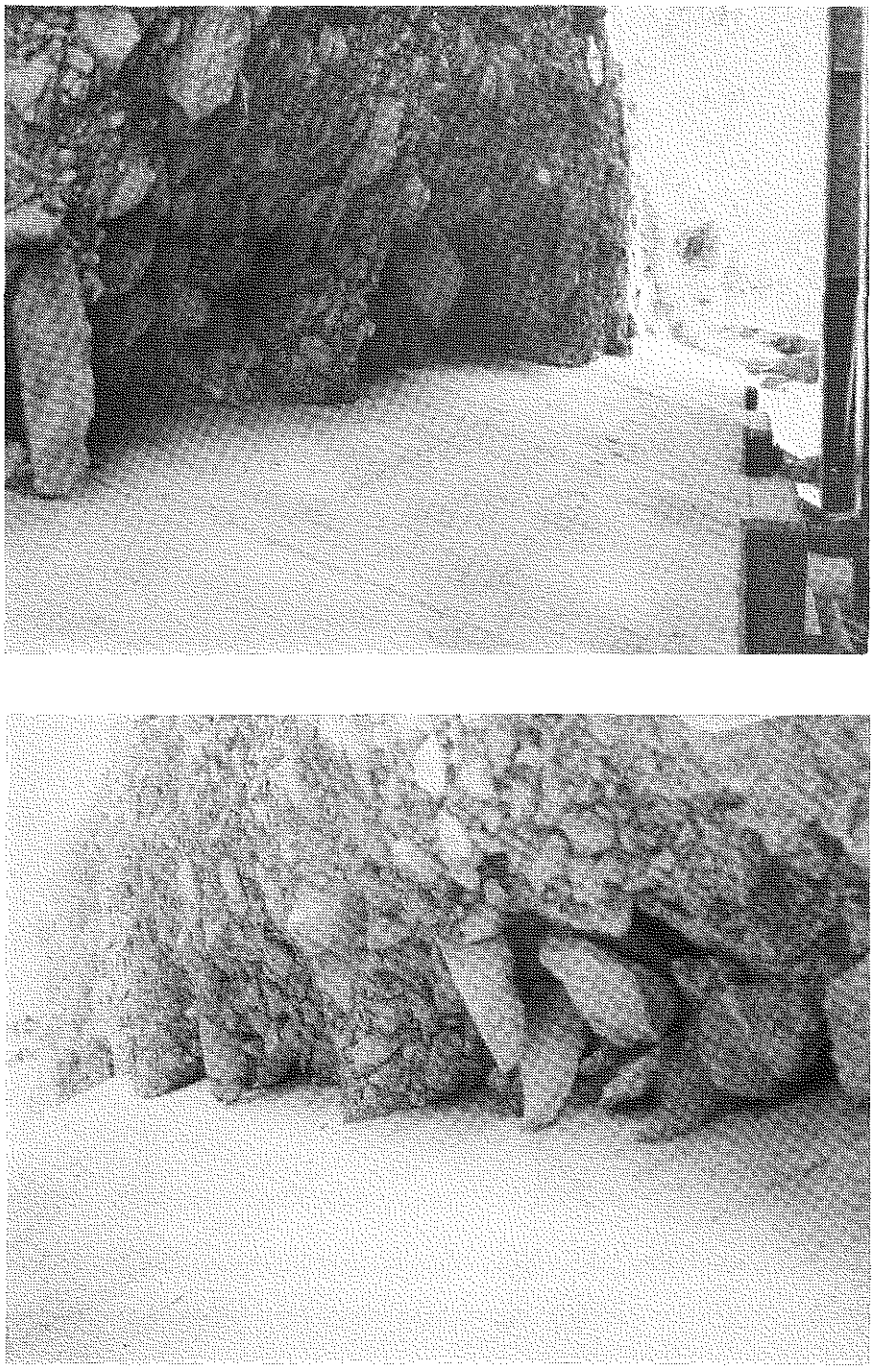


Figure 5. Voids and Cracks in Soil along Face of West Abutment of US-68 Bridge over the Licking River.

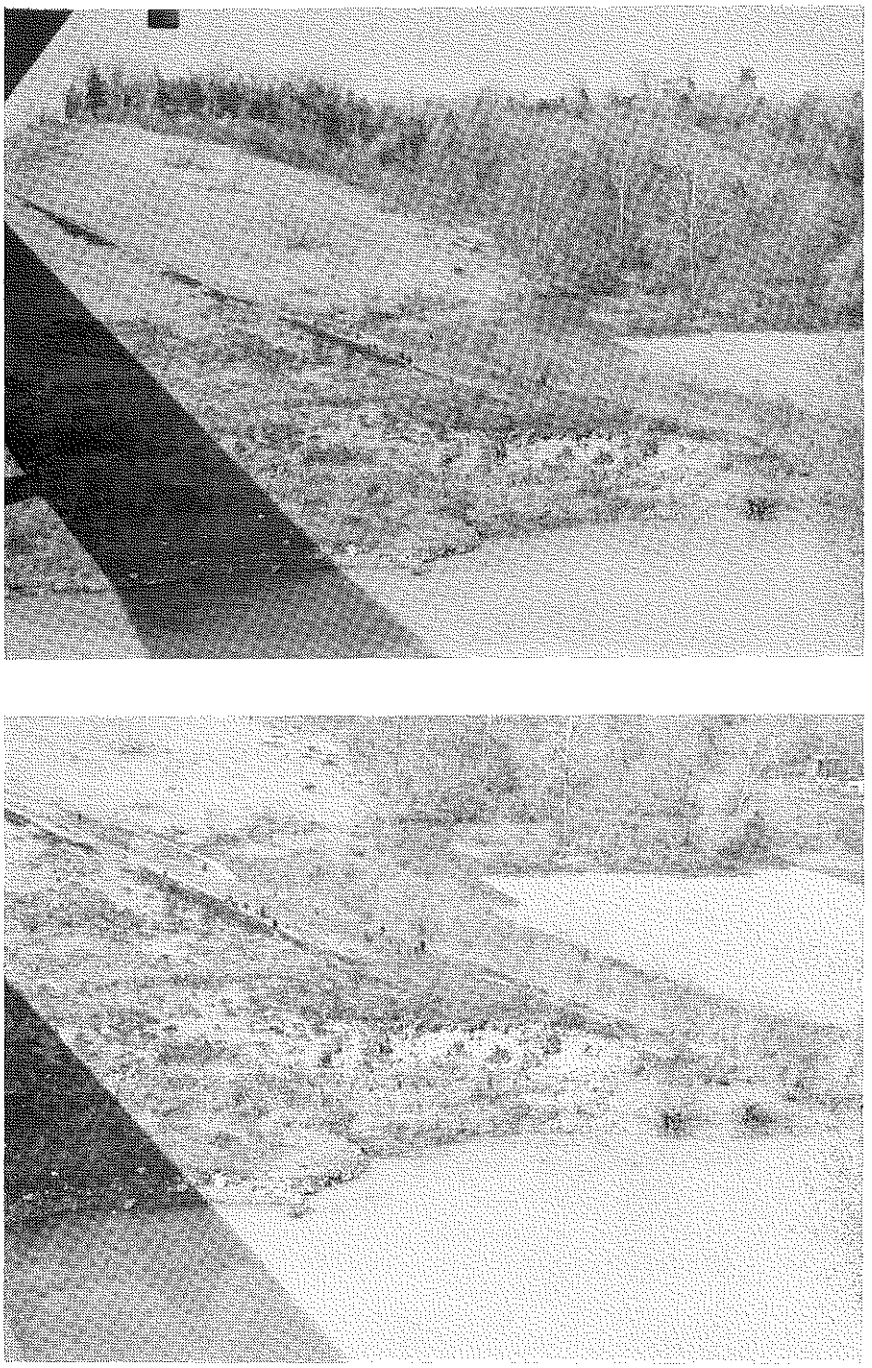


Figure 6. Erosion of Toe along Downstream Side of West Embankment, US-68 Bridge over the Licking River.

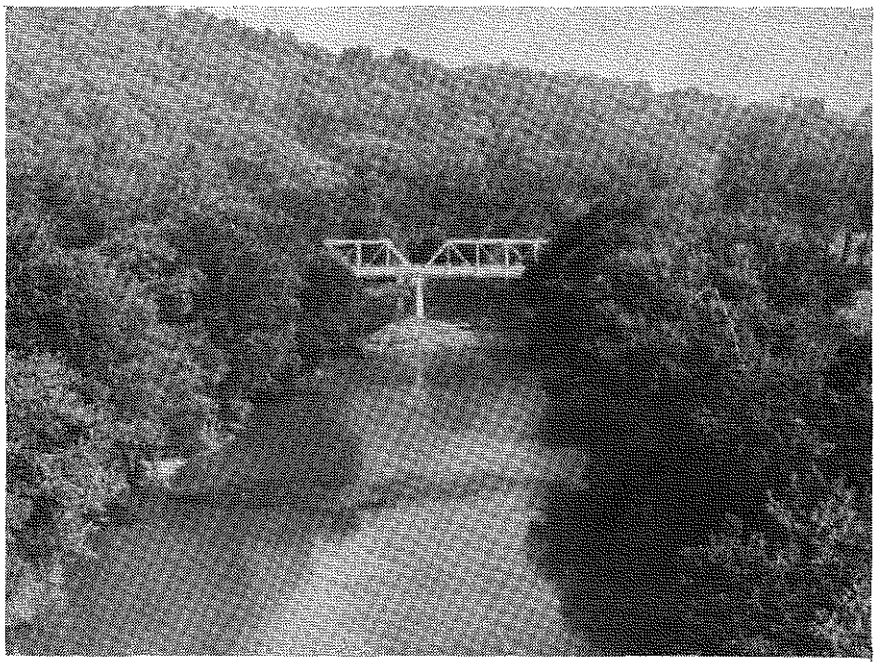


Figure 7. Debris in Front of the Piers of the Old US-68 Bridge Downstream from Present Bridge.

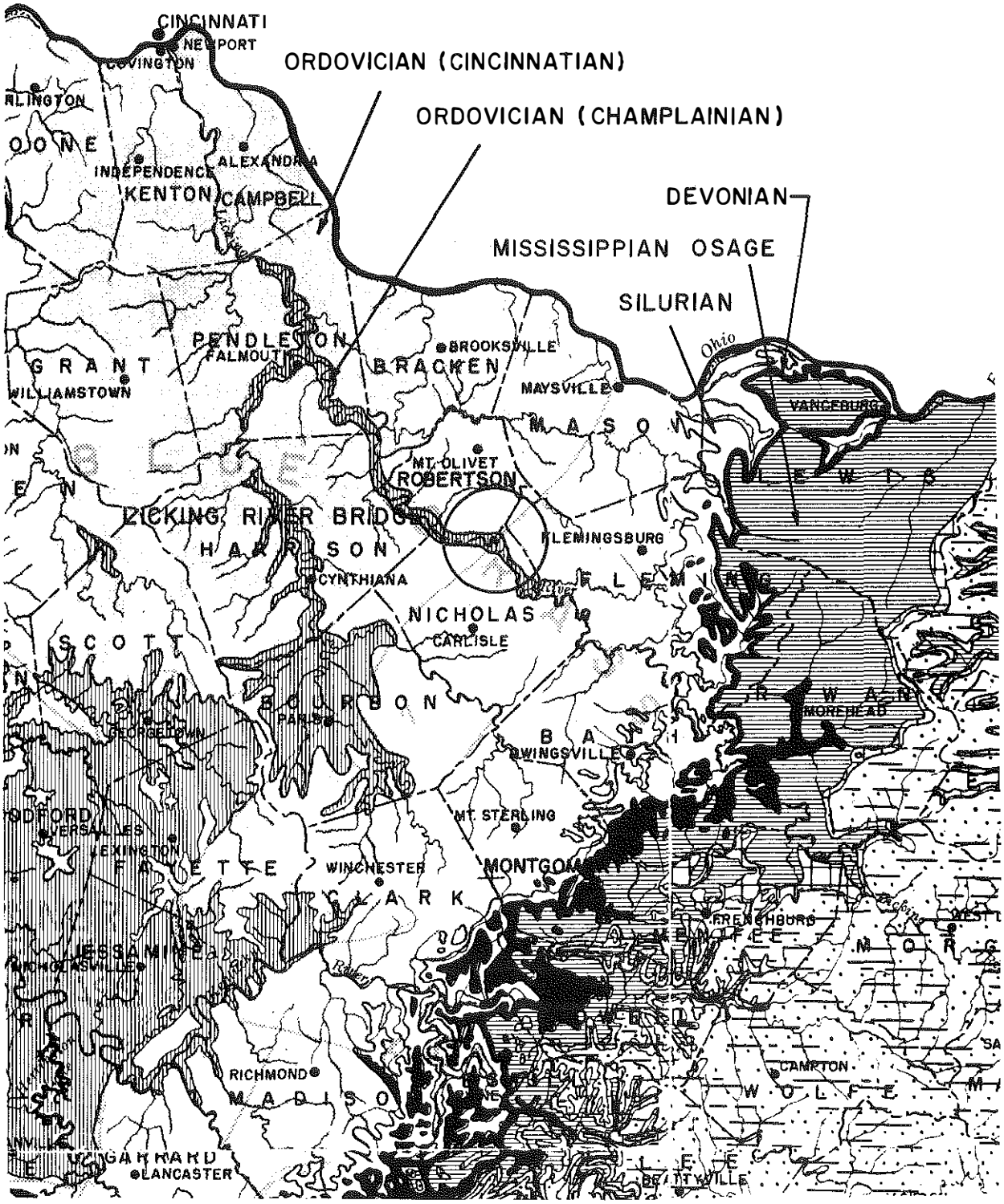
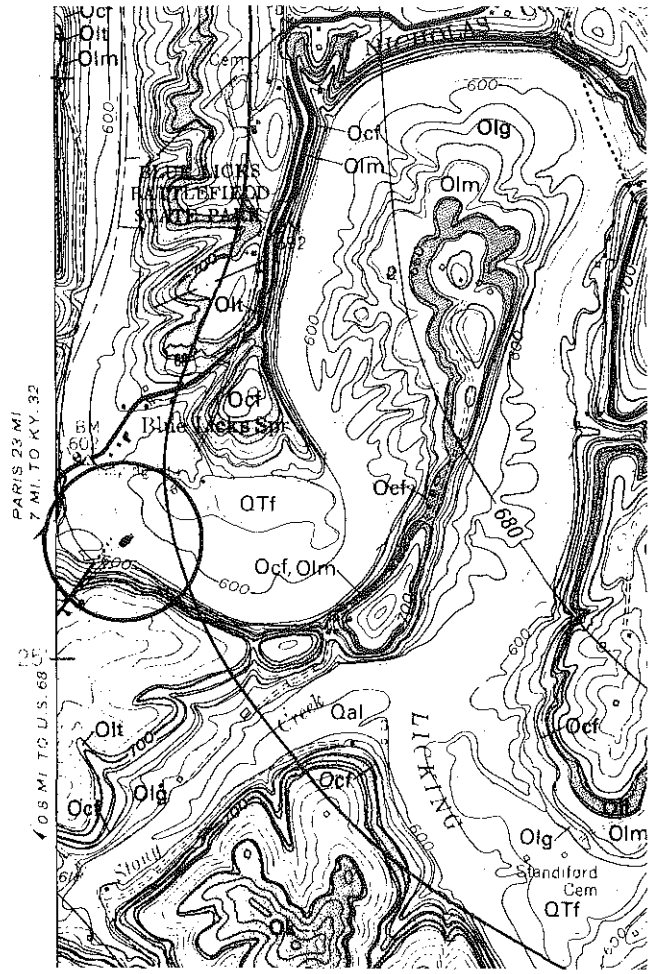
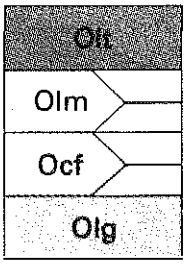


Figure 8. Geologic Map Showing Location of Site in the Outer Bluegrass, Northern Kentucky.





Clays Ferry Formation and Lexington Limestone

Lexington Limestone

*Ocf, *tongue of Clays Ferry Formation*
Olt, *Tanglewood Limestone Member*
*Olm, *Millersburg Member*
Olg, *Grier Limestone Member*

*Shown by line where too thin to show color

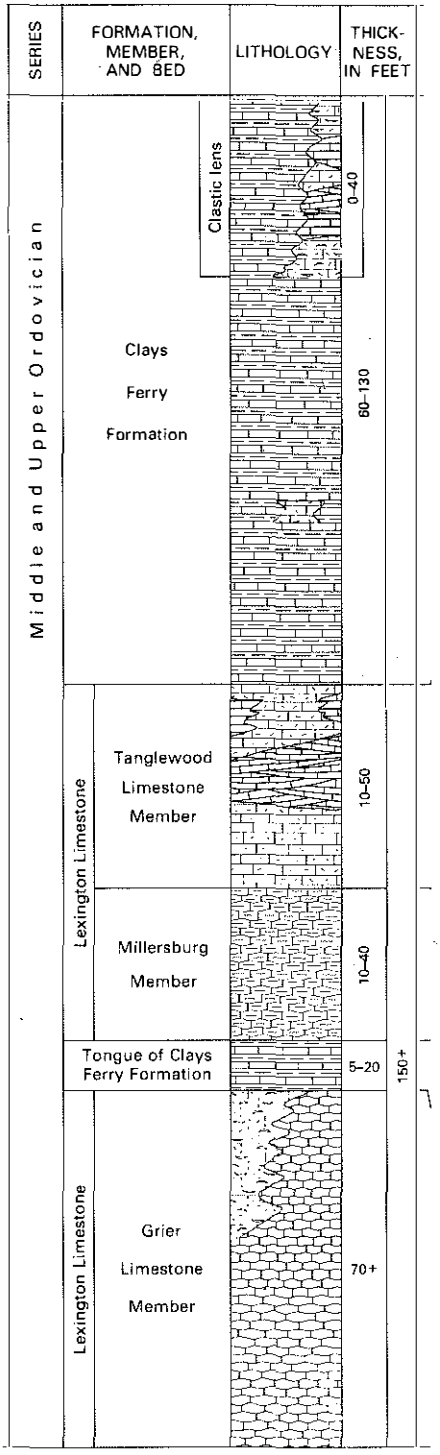


Figure 9. Stratigraphic Section of Geology in Vicinity of Site.

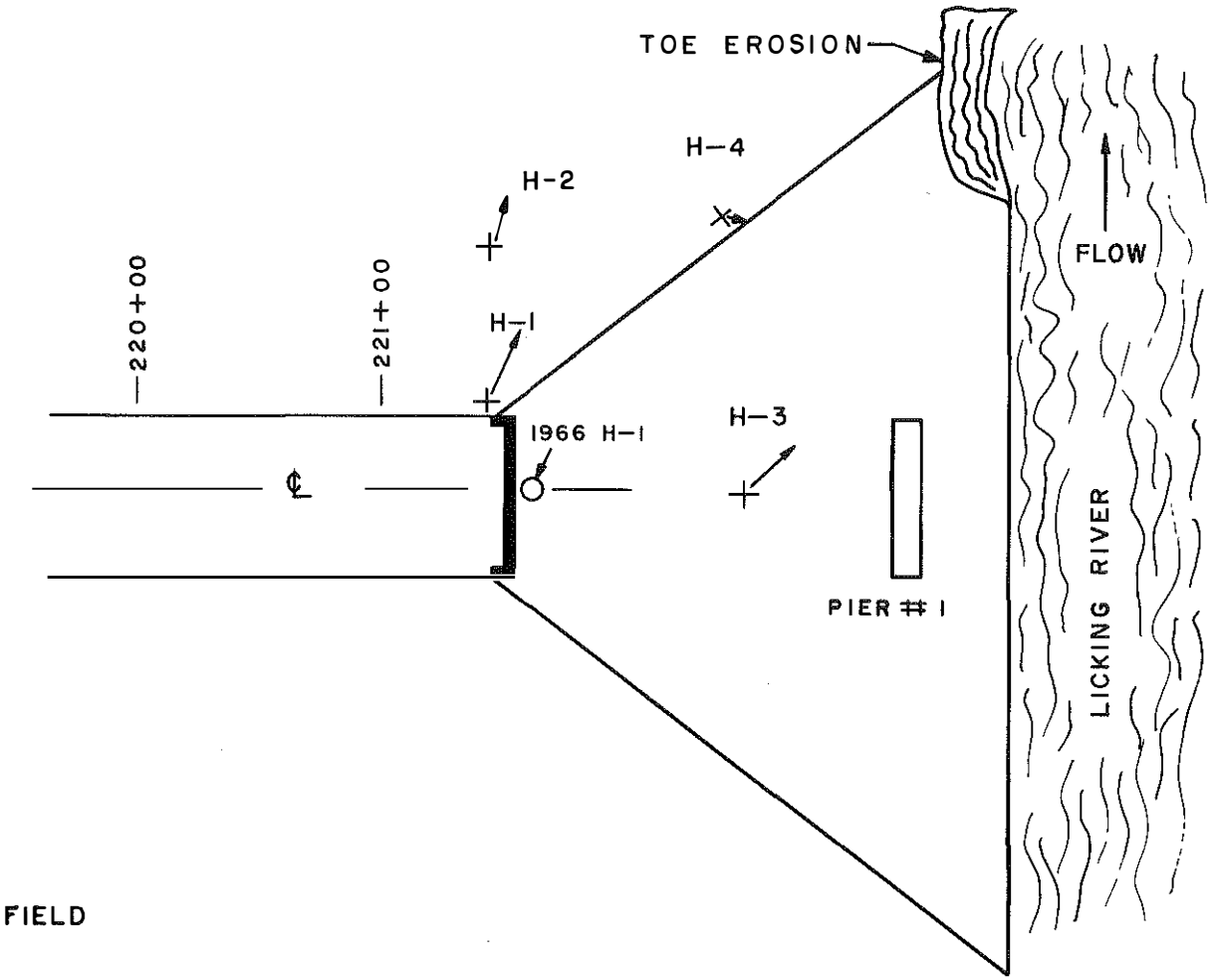


Figure 10. Direction and Relative Magnitudes of Slope Inclinometer Movements, Holes 1, 2, 3, and 4.

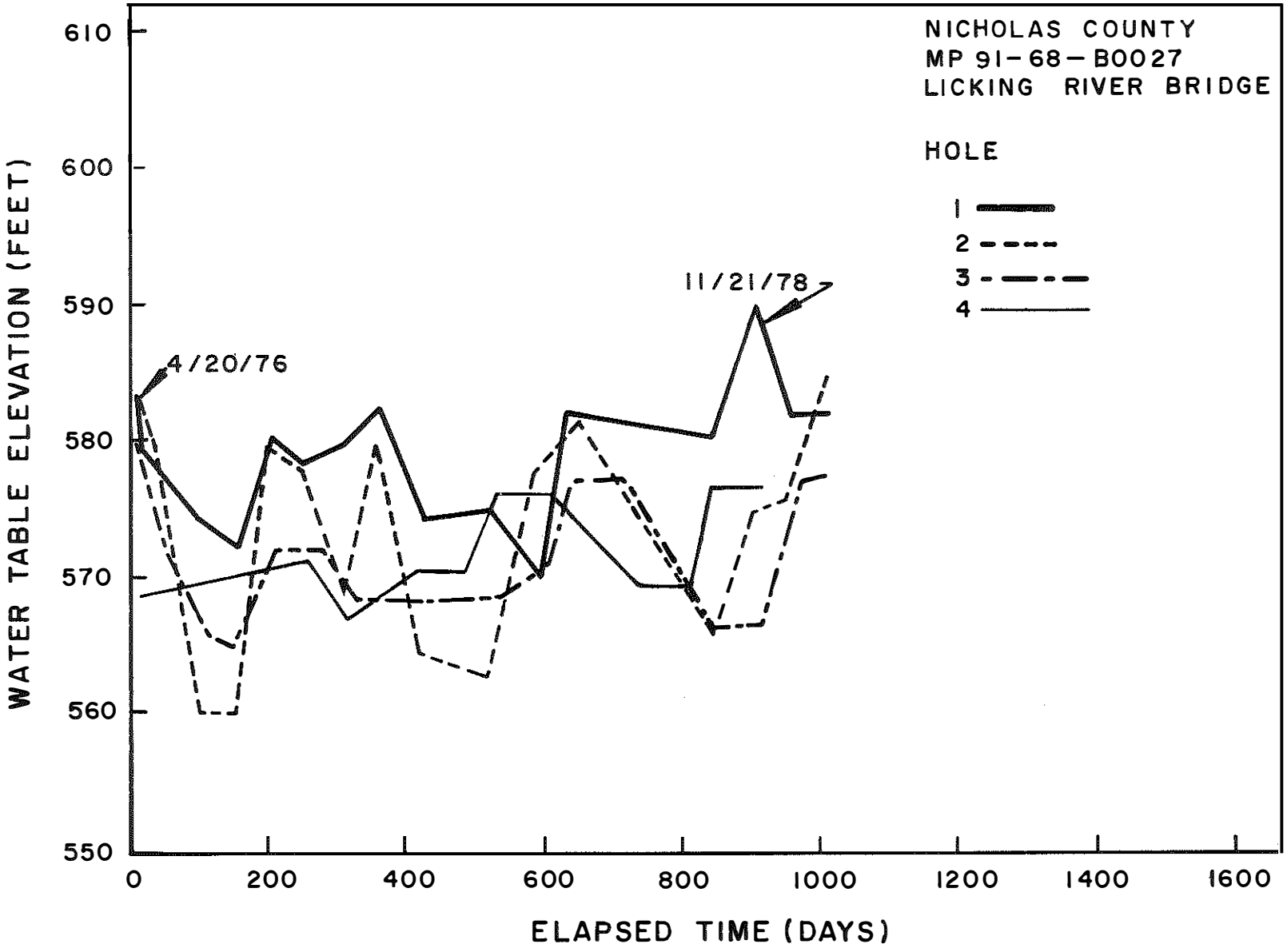


Figure 11. Observed Elevation of Water Table versus Elapsed Time in Days, Holes 1, 2, 3, and 4.

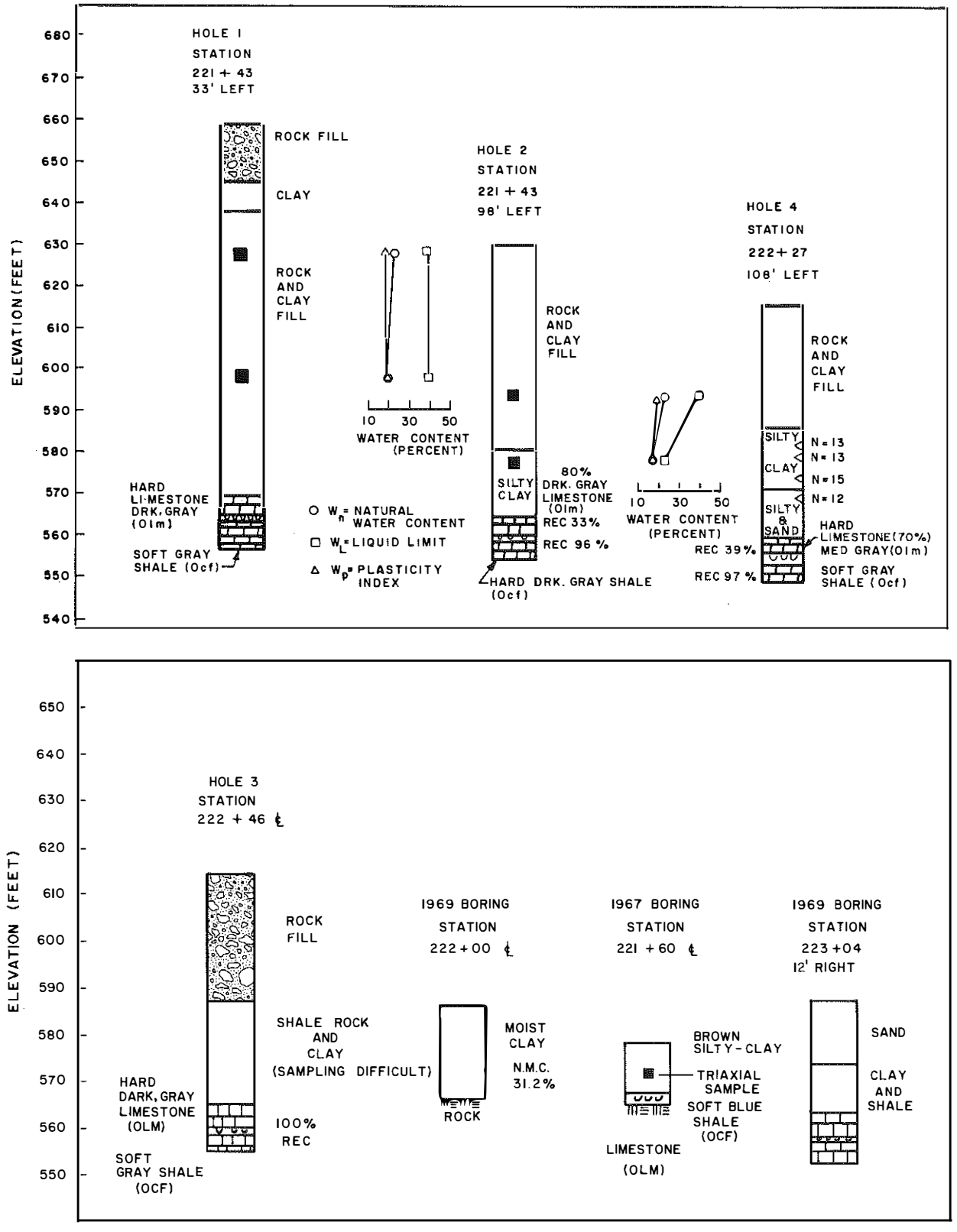


Figure 12. Boring Results, Holes 1, 2, 3, and 4 for Period 1967 through 1969.

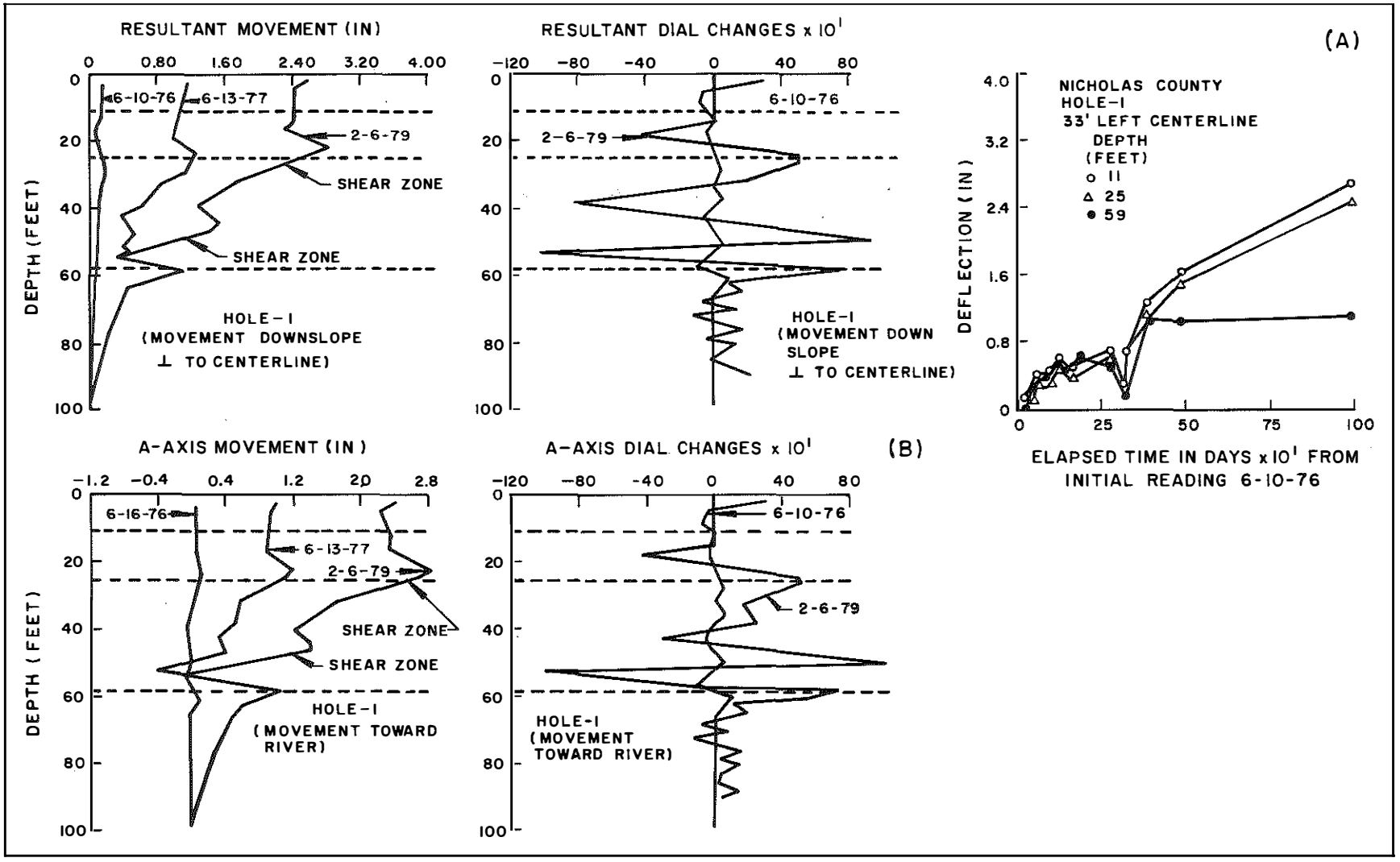


Figure 13. Slope Inclinometer Results: Hole 1; a) Resultant Movement and b) A-Axis.

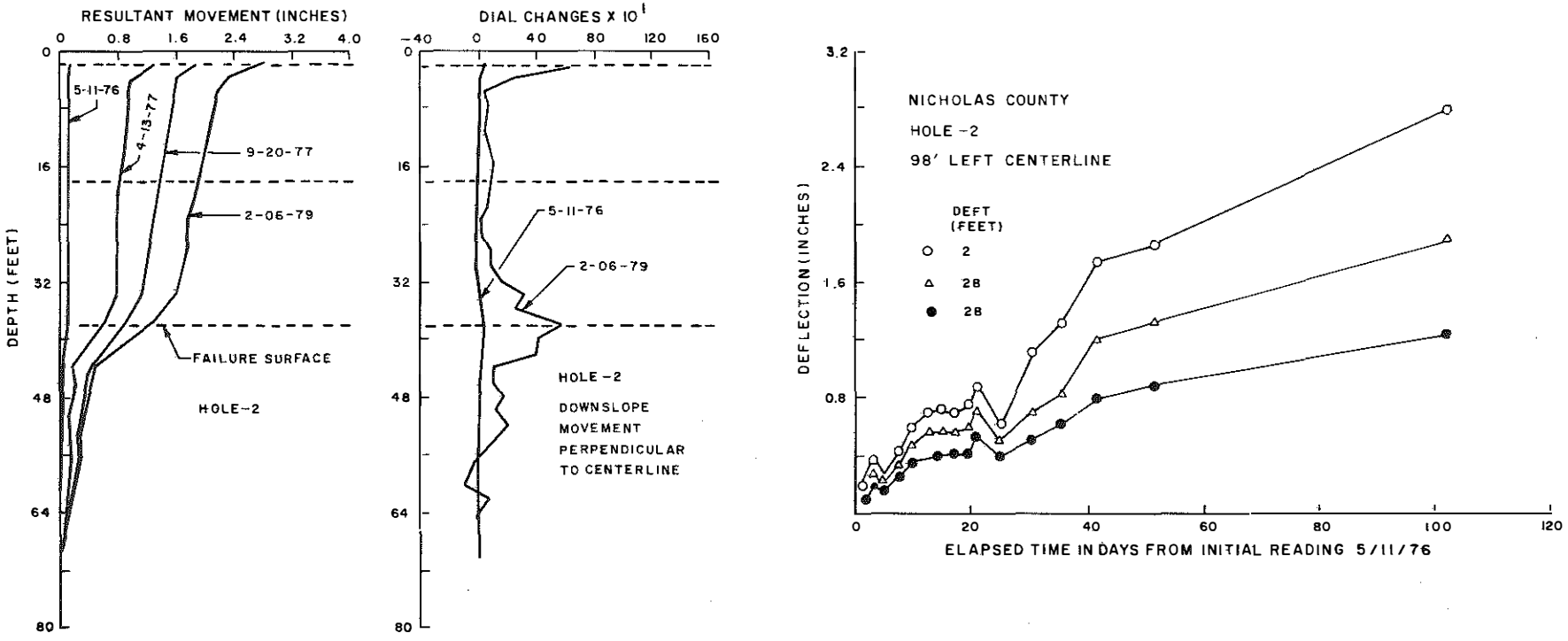
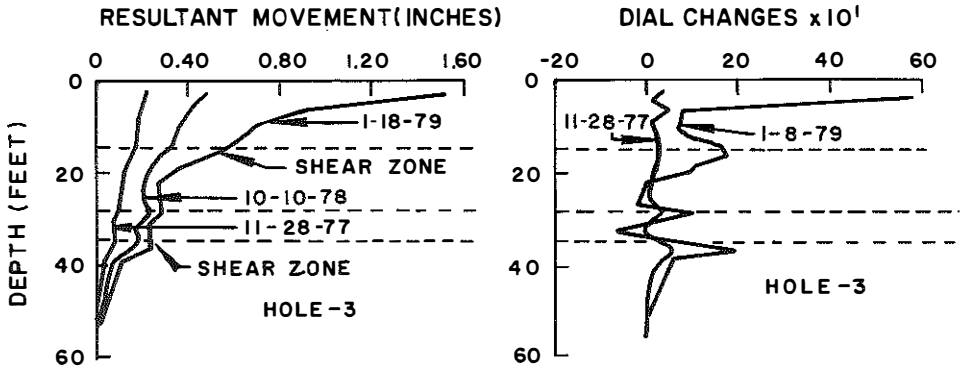


Figure 14. Slope Inclinometer Results, Hole 2.



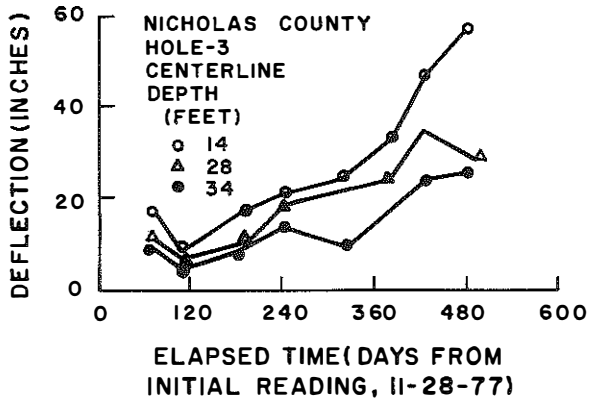
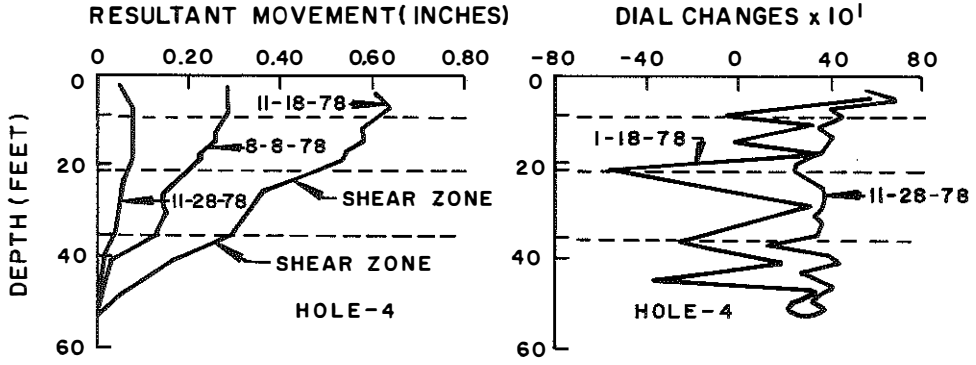


Figure 15. Slope Inclinometer Results, Hole 3.



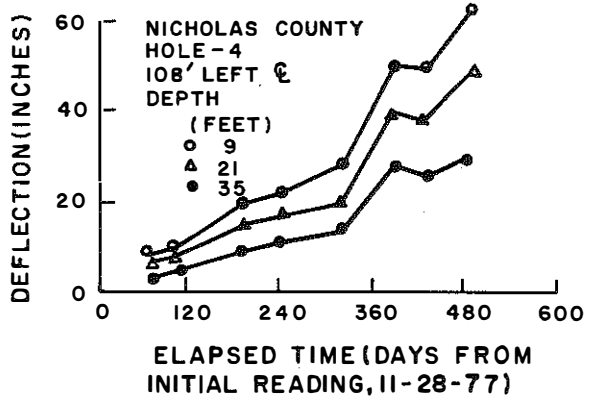


Figure 16. Slope Inclinometer Results, Hole 4.

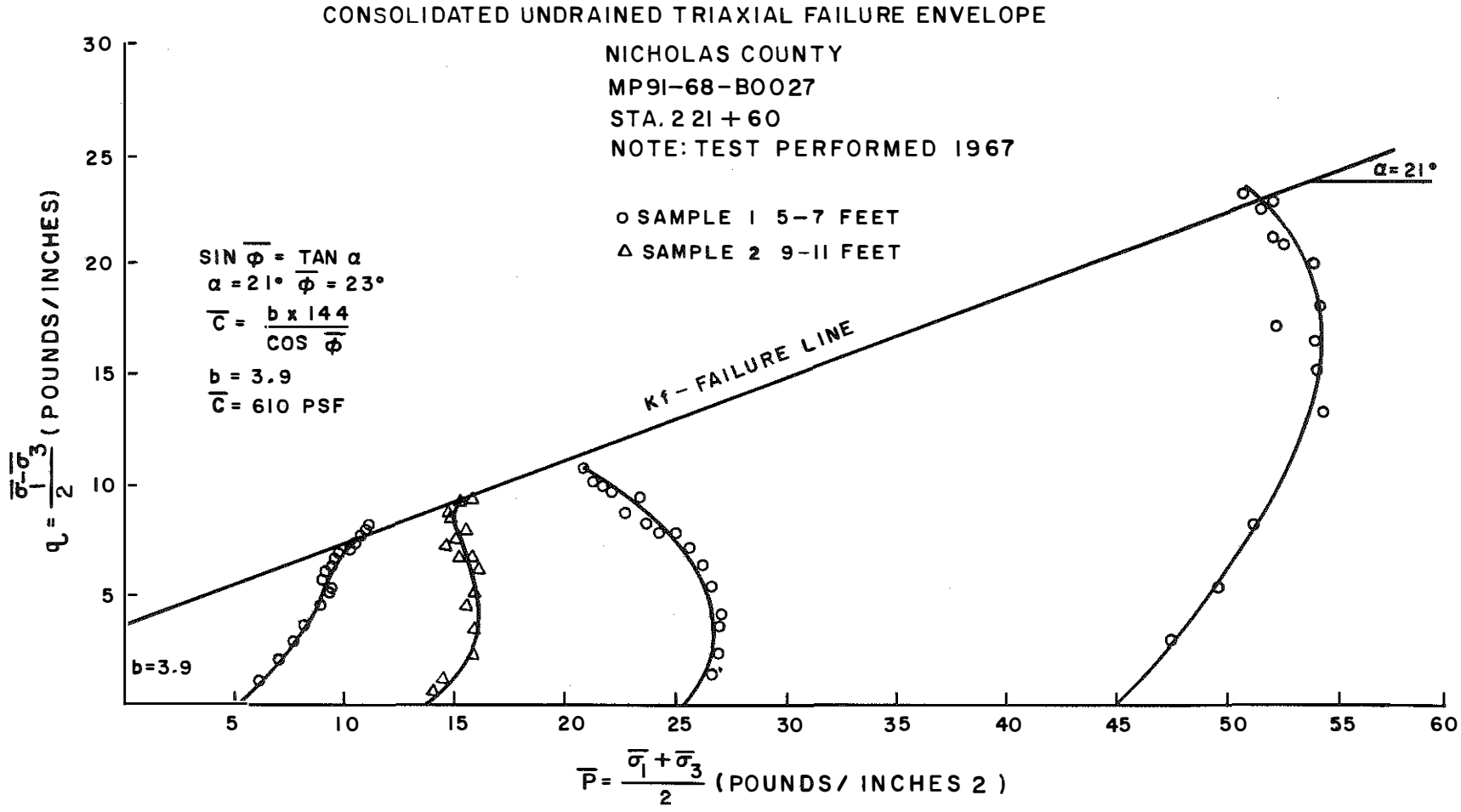
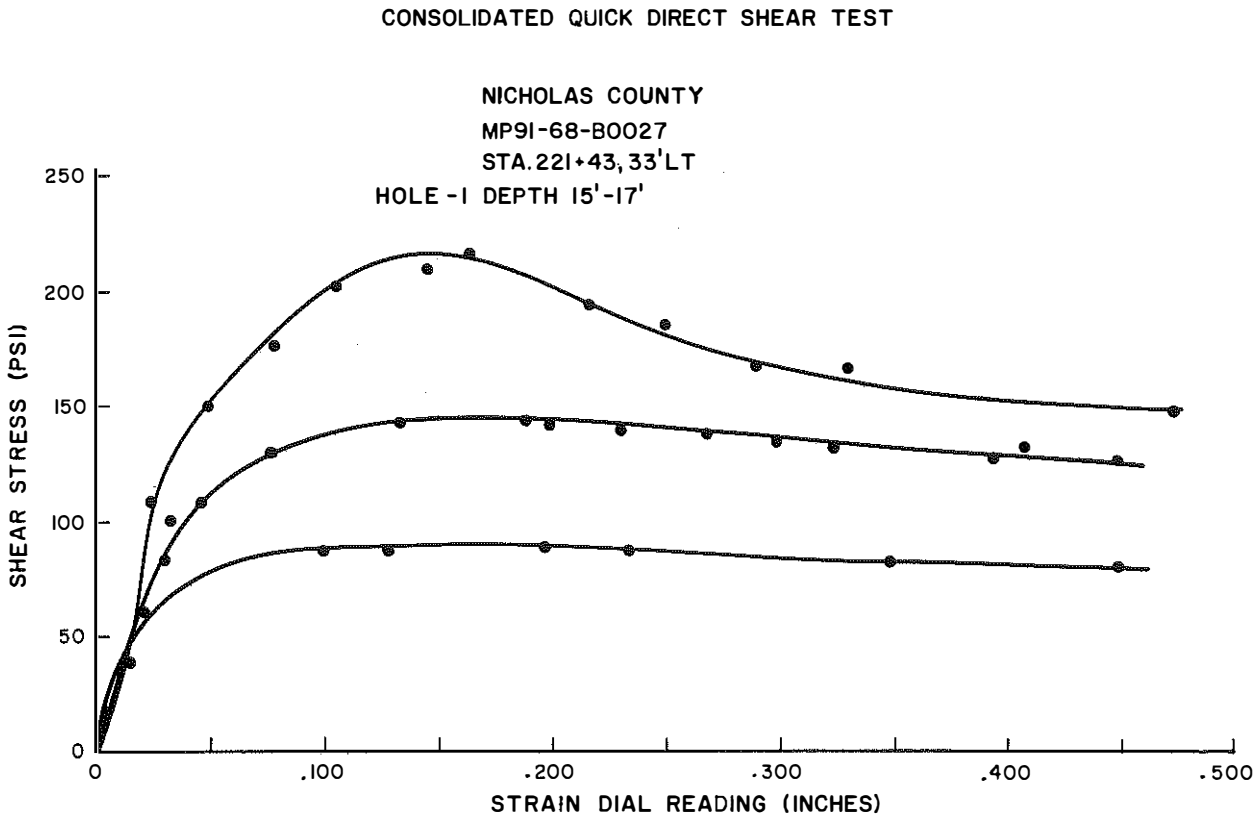


Figure 17. Results of Consolidated-Undrained Triaxial Test with Pore-Pressure Measurements, 1967.



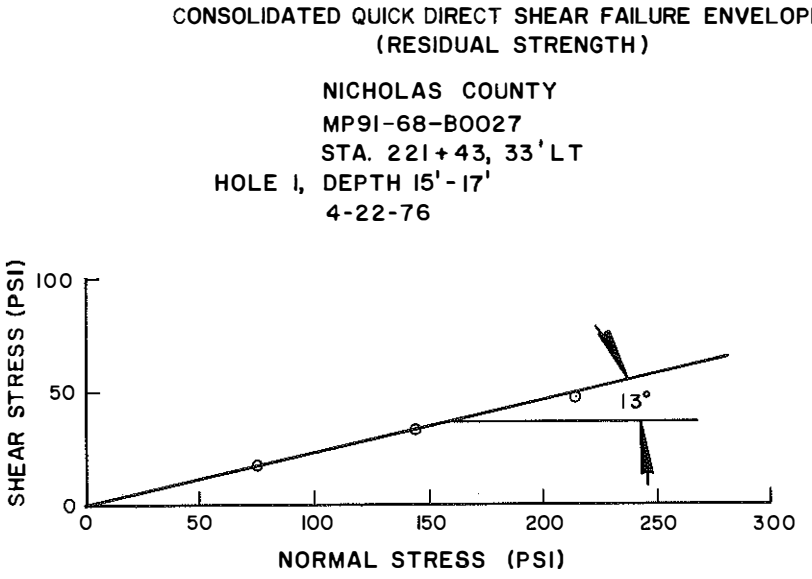


Figure 18. Results of Consolidated-Undrained Direct Shear Test: Hole 1, Depth 15 - 17 Feet (4.6 - 5.2 m).

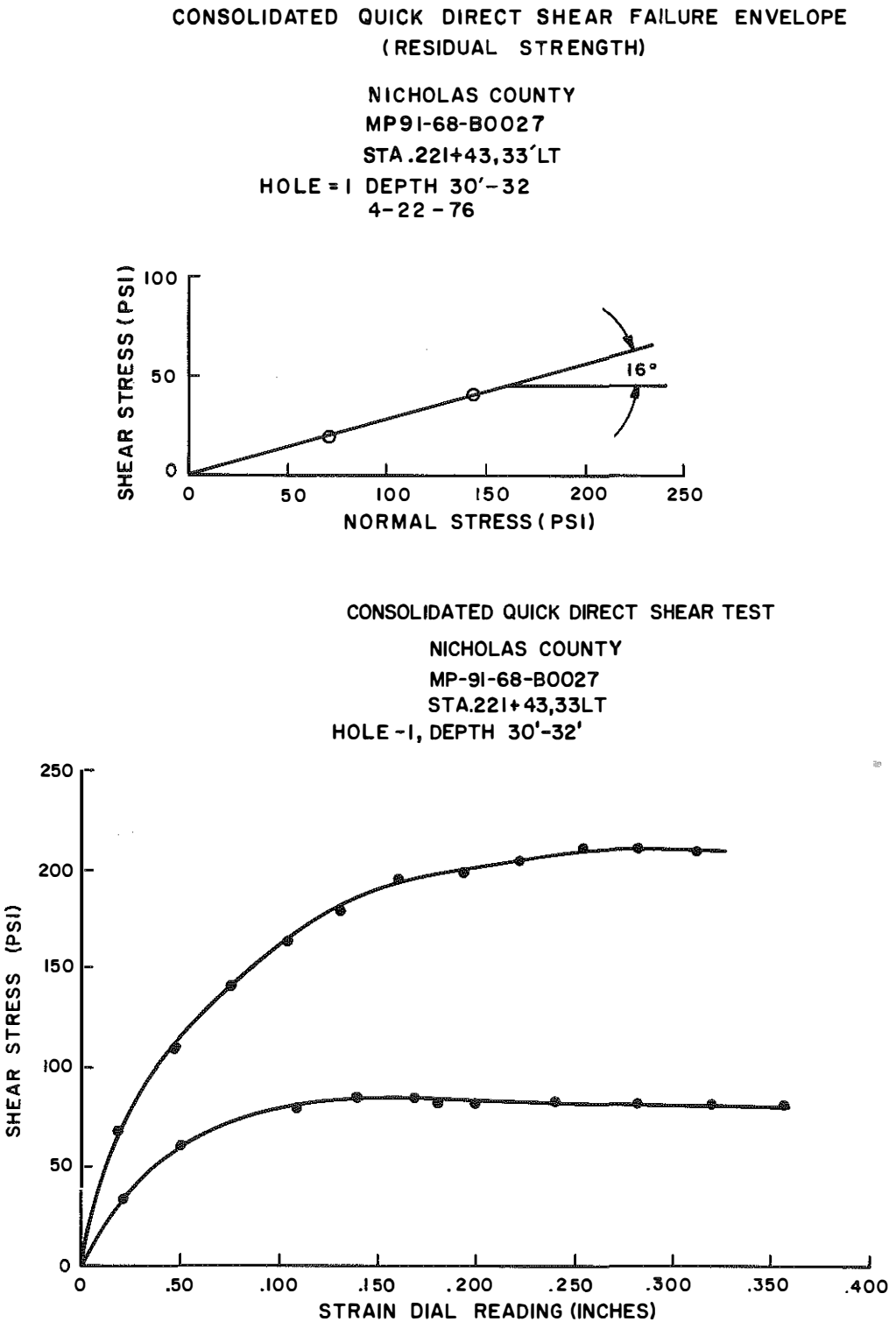
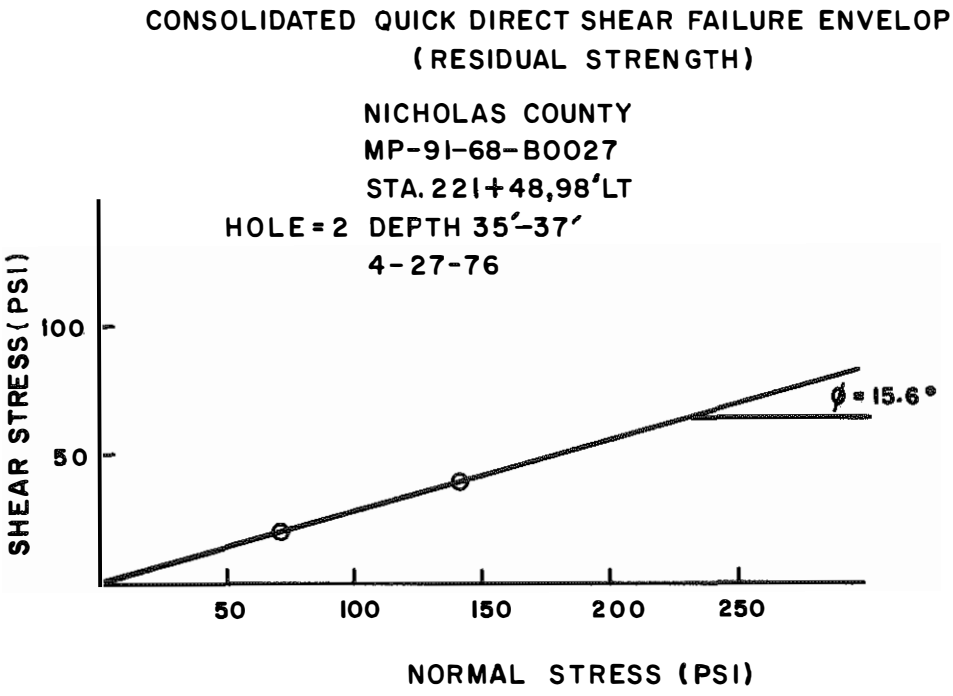


Figure 19. Results of Consolidated-Undrained Direct Shear Test: Hole 1, Depth 30 - 32 Feet (9.1 - 9.8 m).



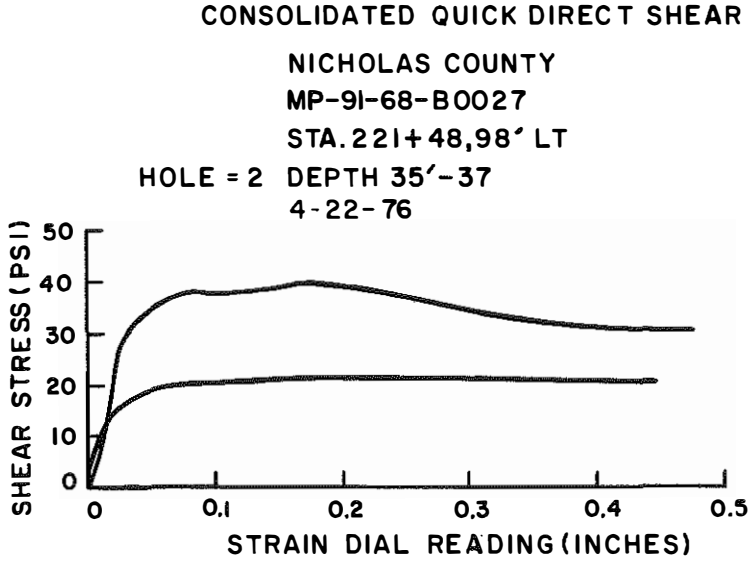
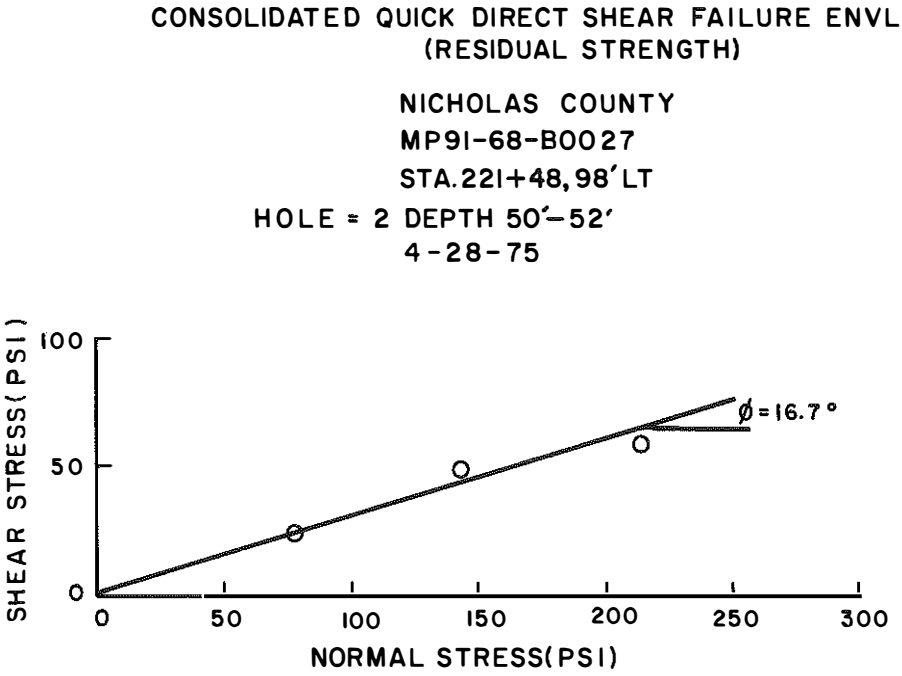


Figure 20. Results of Consolidated-Undrained Direct Shear Test: Hole 2, Depth 35 - 37 Feet (10.7 - 11.3 m).



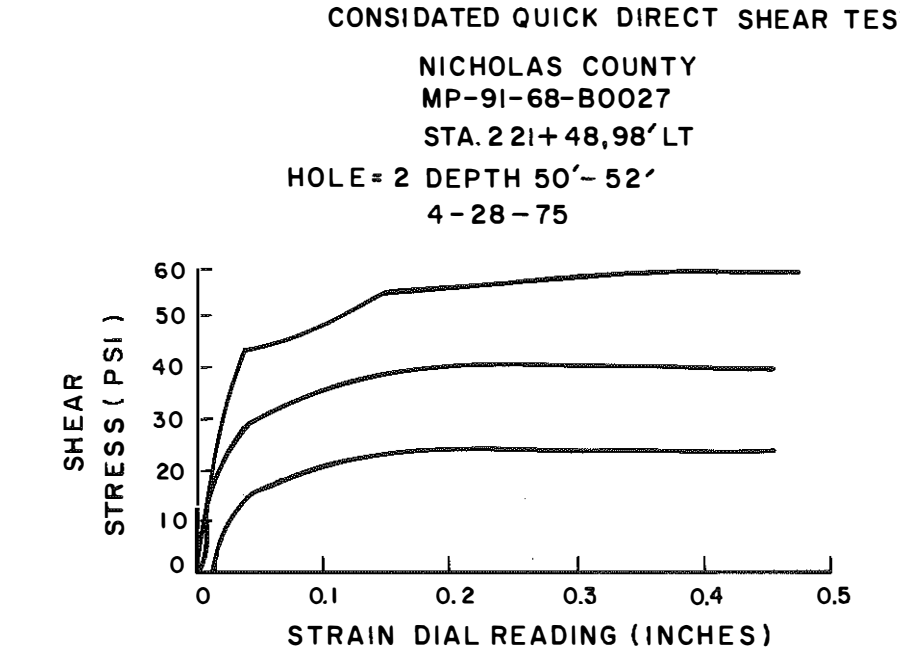


Figure 21. Results of Consolidated-Undrained Direct Shear Test: Hole 2, Depth 50 - 52 Feet (15.2 - 15.8 m).

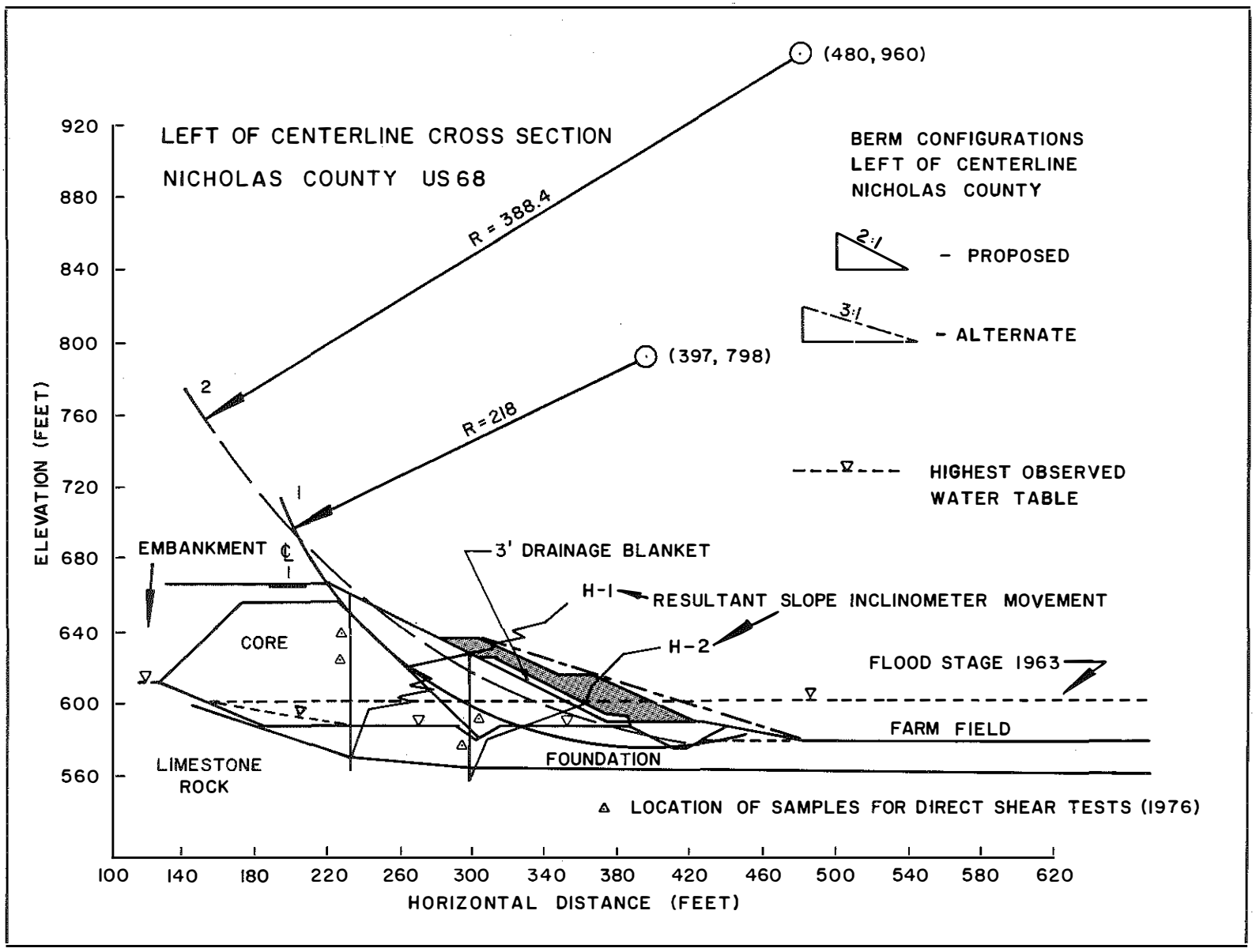


Figure 22. Left-of-Centerline Cross Section, Critical Circles.

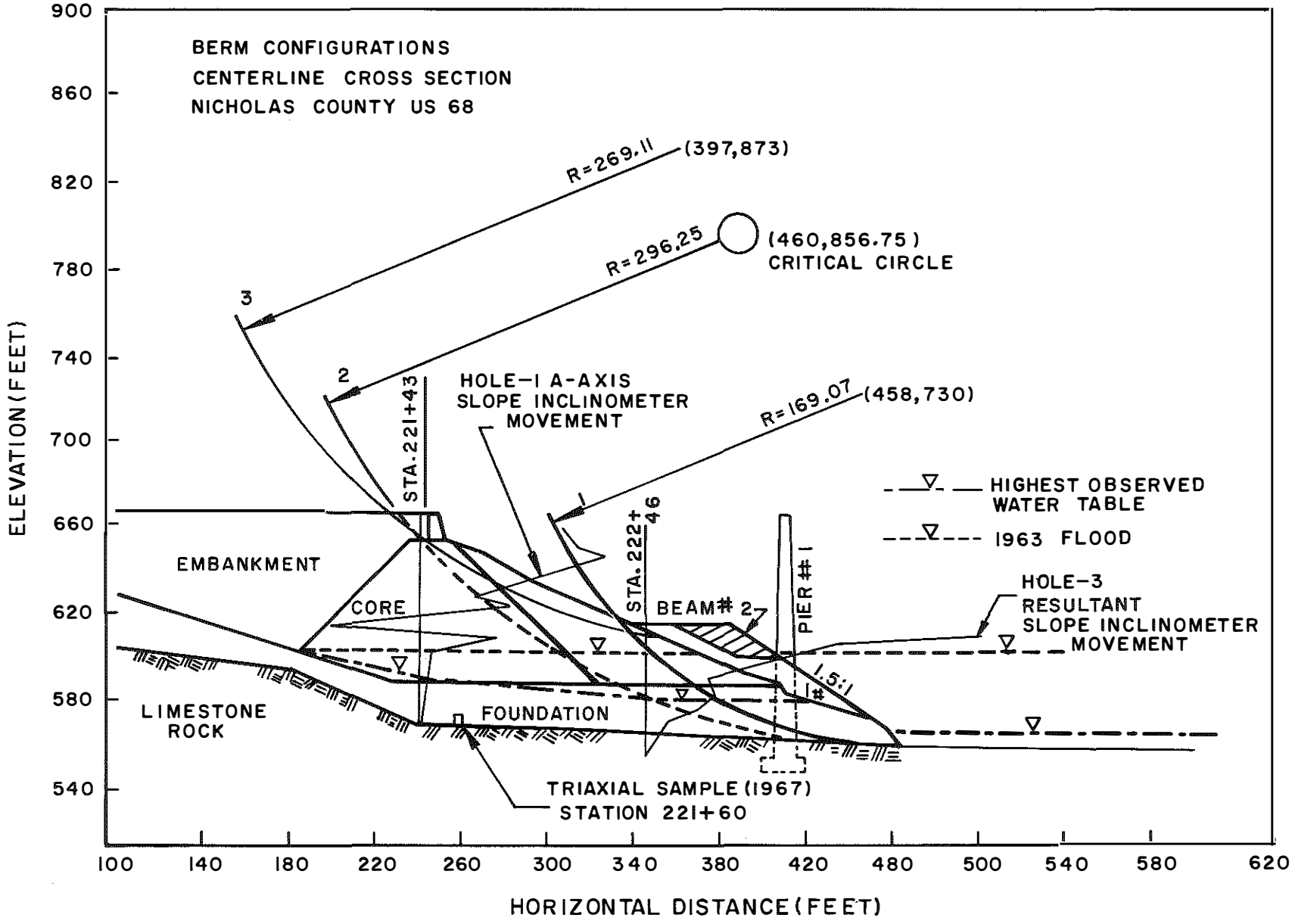


Figure 23. Centerline Cross Section, Critical Circles.

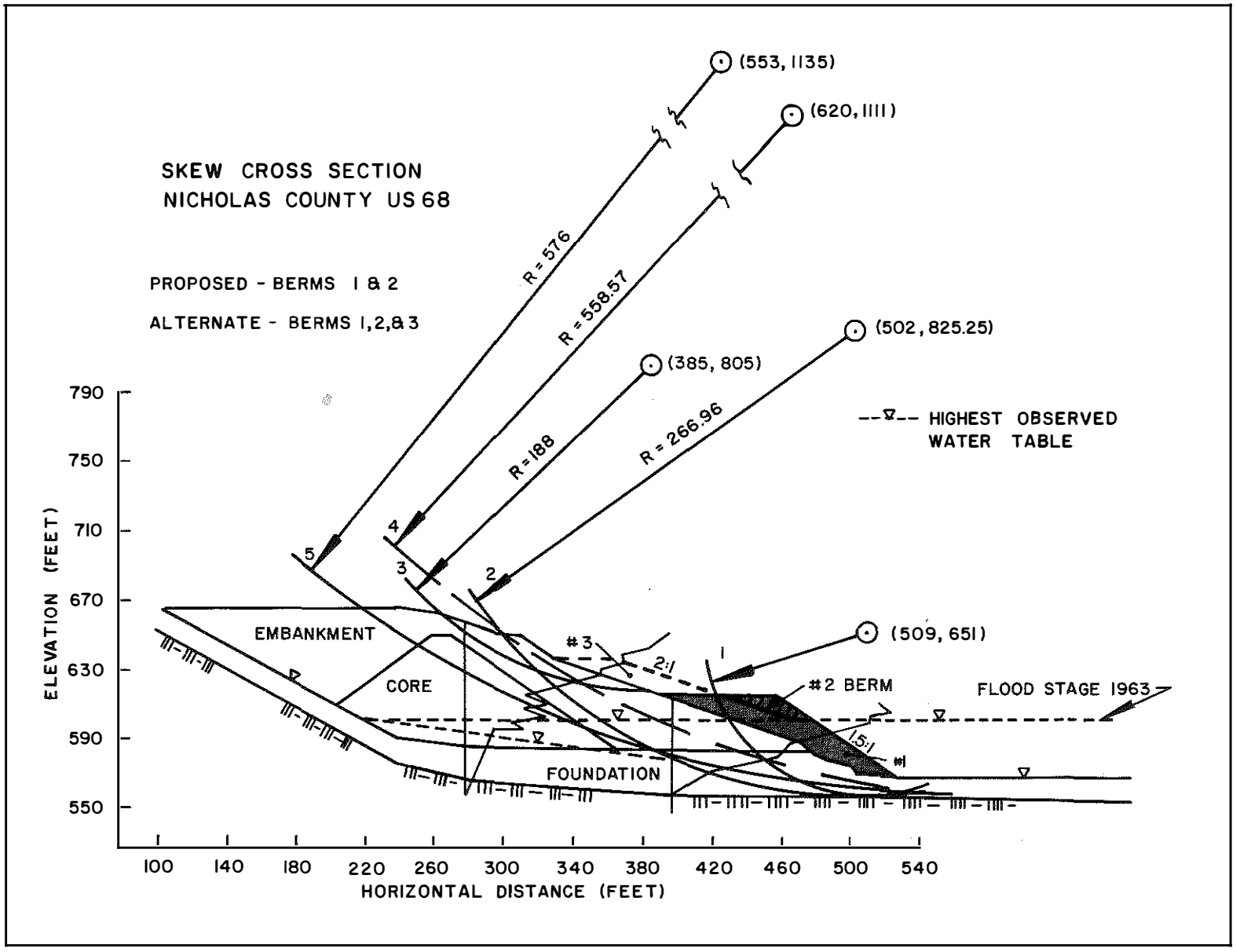


Figure 24. Skewed Cross Section, Critical Circles.

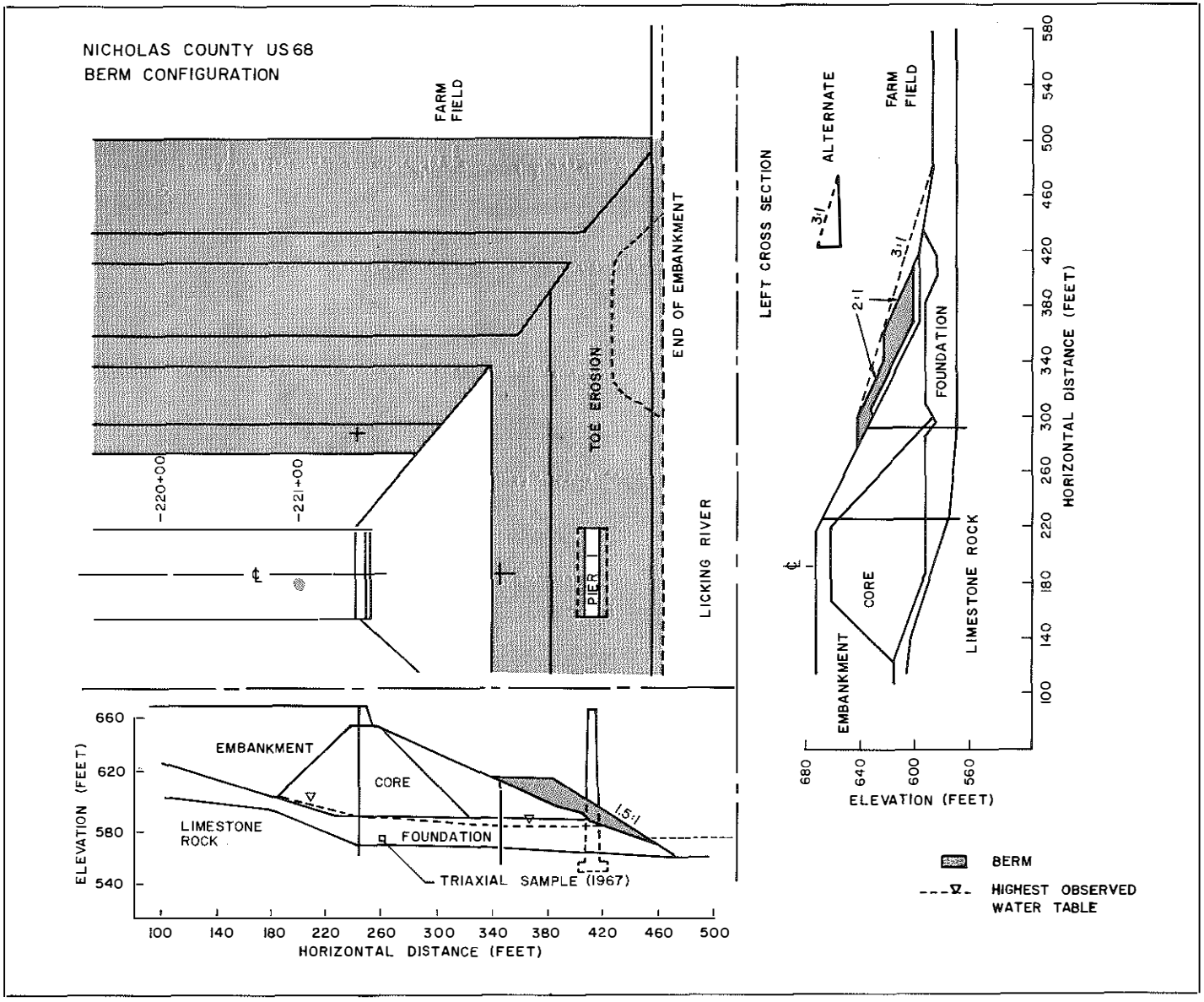


Figure 25. Proposed Berm Configuration for West Embankment of US 68 over the Licking River.

LEFT OF CENTERLINE CROSS SECTION
US 68 NICHOLAS COUNTY

▽ HIGHEST OBSERVED WATER TABLE

WEDGE NUMBER	FACTOR OF SAFETY Ø=23°	Ø=16°	BACKED-IN Ø°
1	0.921	0.635	24.7°
2	1.019	0.713	22.7°
3	1.044	0.716	22.7°
4	1.244	0.797	19.3°
5	1.163	0.804	19.9°

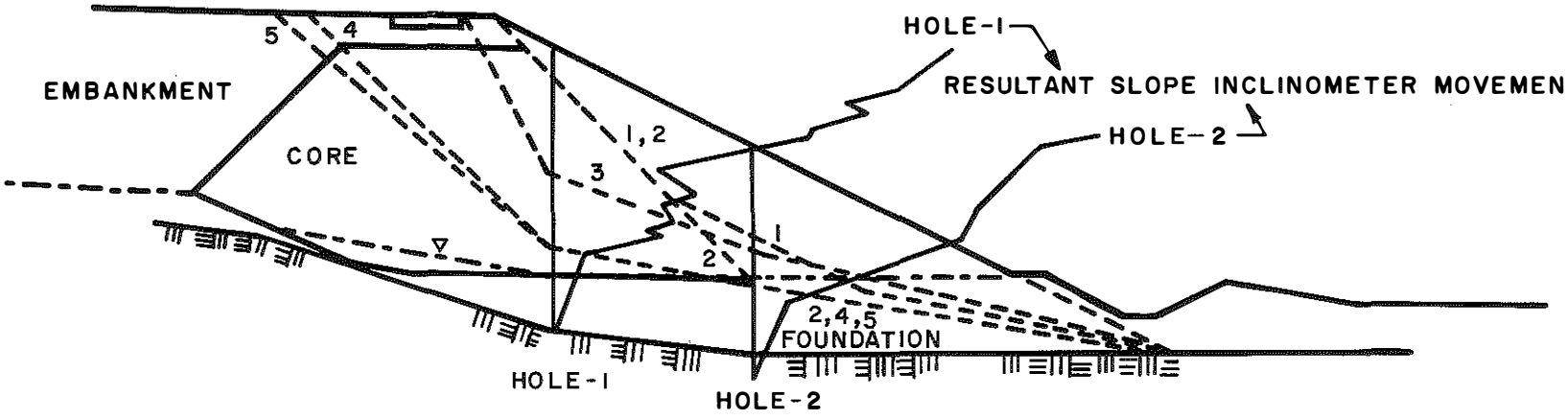


Figure 26. Left-of-Centerline Cross Section, Critical Wedges.

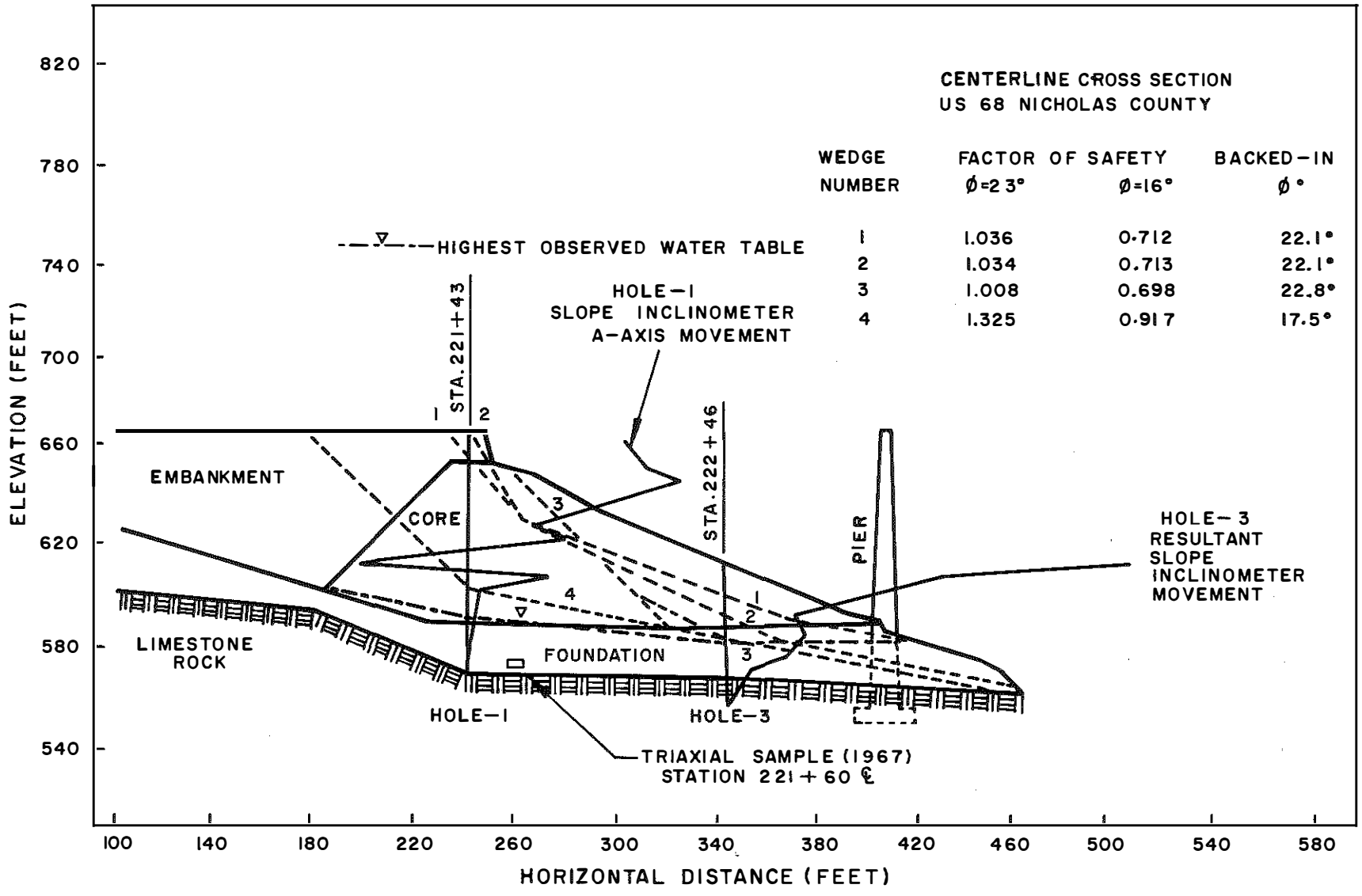


Figure 27. Centerline Cross Section, Critical Wedges.

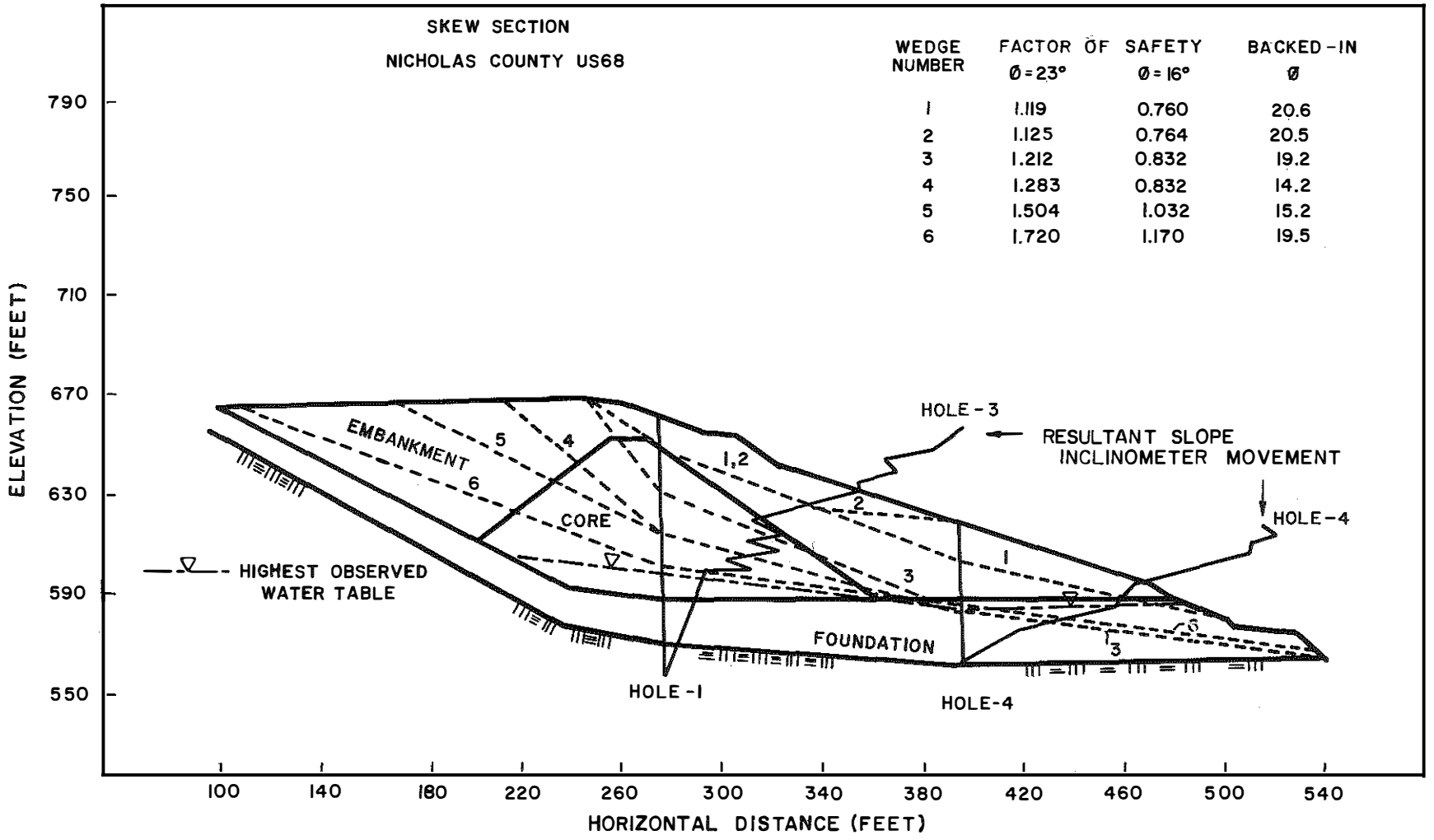


Figure 28. Skewed Cross Section, Critical Wedges.

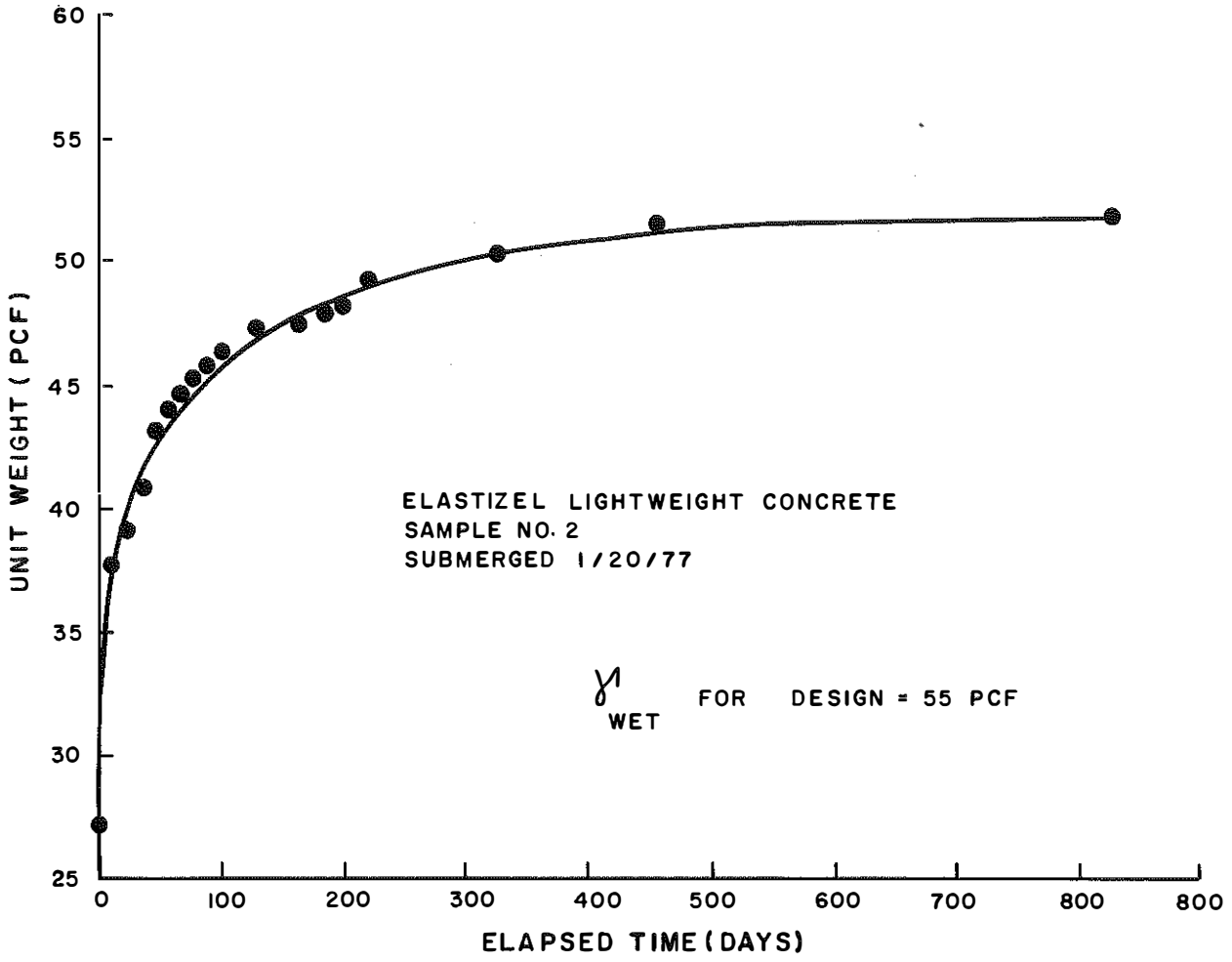


Figure 29. Effect of Submergence on Density of Elastizell Lightweight Concrete, Sample No. 2.

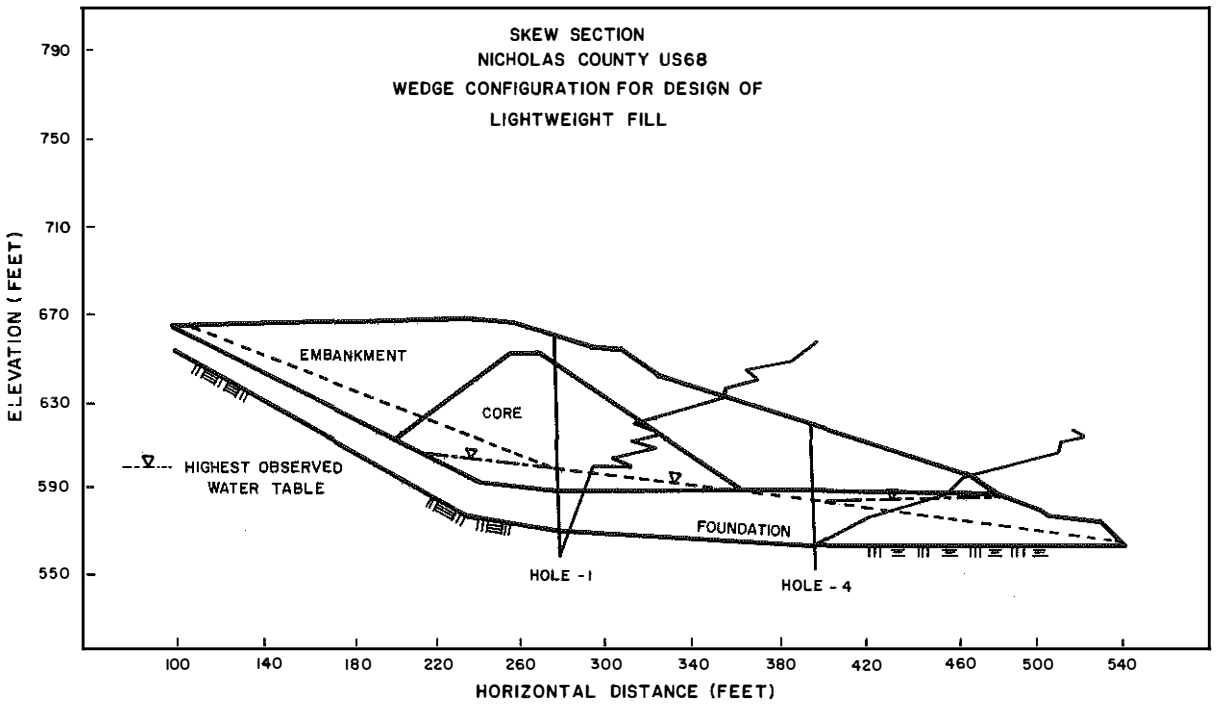


Figure 30. Wedge Configuration Used in Design of Lightweight Fill.

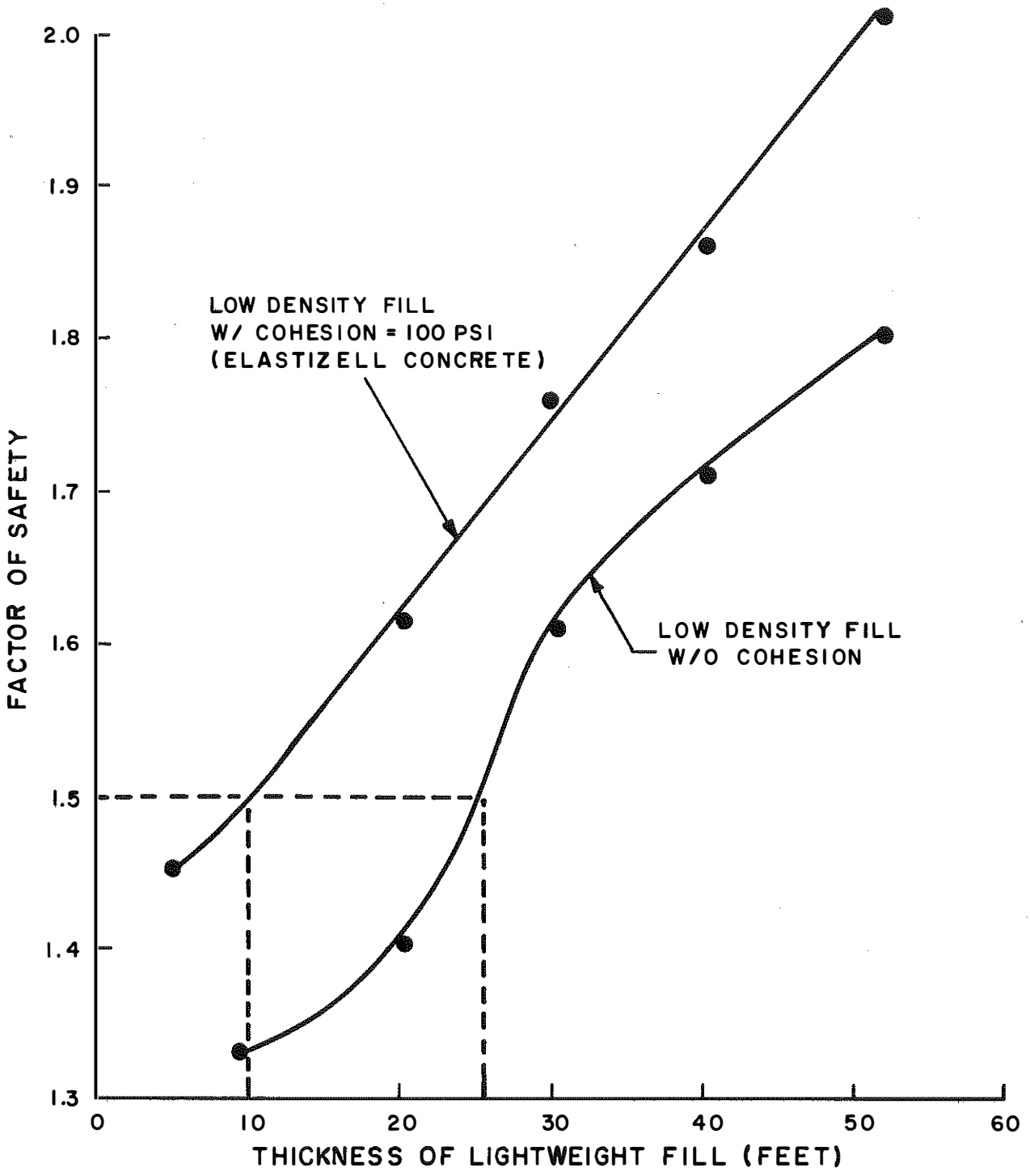


Figure 31. Summary of Factors of Safety as a Function of Thickness of Lightweight Fill Material (Density of 55 Pounds per Cubic Foot (880 kg/m^3)).

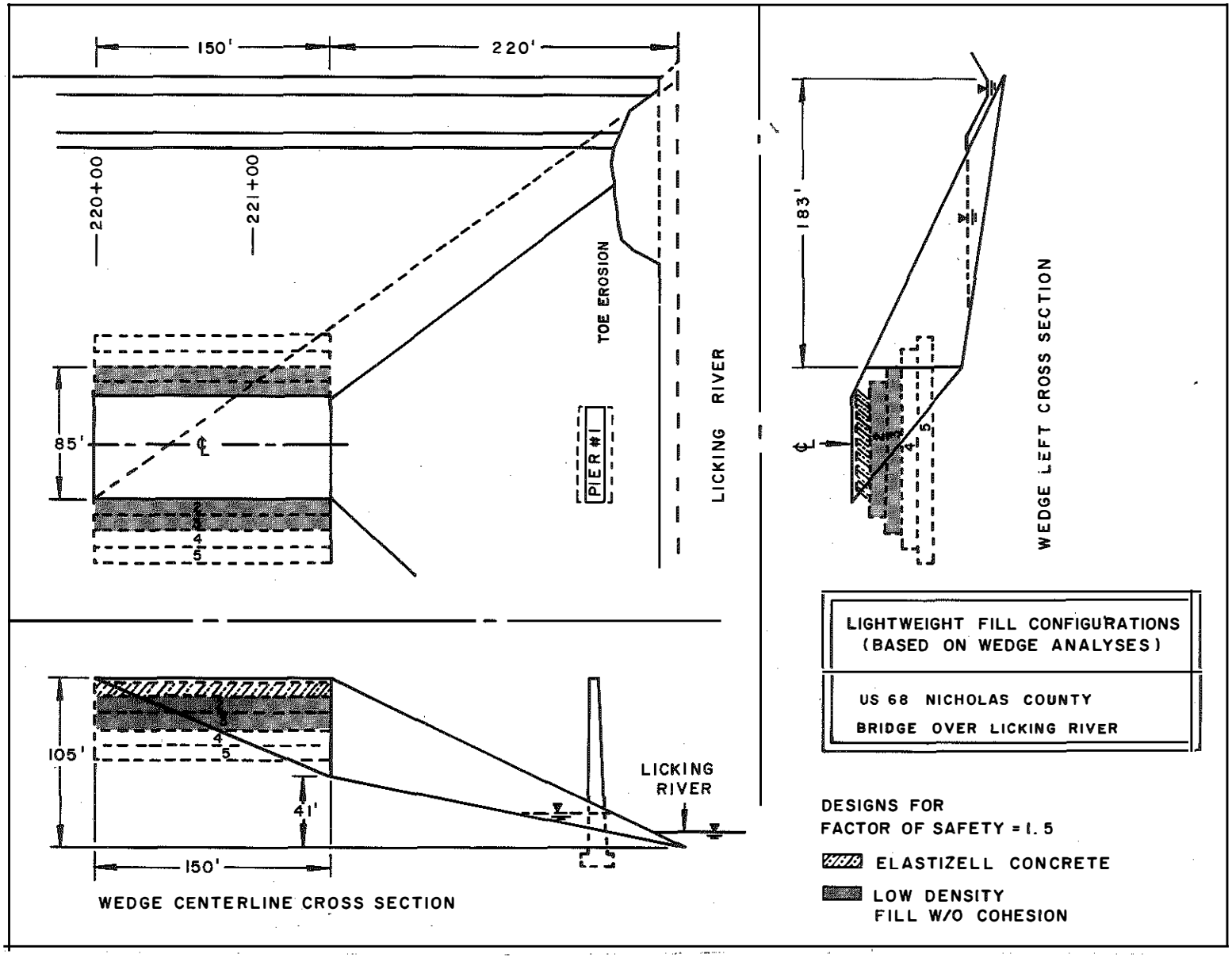


Figure 32. Proposed Lightweight Fill Configurations.