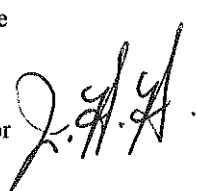


April 2, 1973

H.3.38

MEMORANDUM TO: A. R. Romine, Director
Division of Maintenance

ATTN: Bert Banks

FROM: Jas. H. Havens, Director 
Division of Research

SUBJECT: I 64, Boyd County; MP 188; SP-10-115-29-L; Repair of Unstable Embankment

REFERENCES:

- 1) B. H. Bank's memo, April 12, 1973; reporting inspection team's visit to MP 152.7, 188 and 190.
- 2) Research Report; May 1972; D. R. Houchin and J. H. Havens; "Proposed Remedies: Unstable Embankment at Milepost 188 and Channel Erosion at Milepost 190-191; I 64, Boyd County."
- 3) Research memo to A. R. Romine; June 30, 1972; re: Slide, MP 152.7, I 64, Carter County.
- 4) Research memo to A. R. Romine; August 21, 1972; re: Meeting with FHWA (August 2) and completion of research tasks.
- 5) FHWA letter; August 18, 1972; re: Referring to Department submissions, June 23 and July 25, requiring borings and sampling at MP 152.7 and 188.
- 6) J. T. Anderson's memo to W. B. Drake, September 7, 1972.
- 7) W. B. Drake's memo to Research Division, September 8, 1972.
- 8) Meeting; September 15, 1972 (unrecorded).

The principals involved in the development of plans to repair embankment failures at Mileposts 152.7 and 188.1 on I 64 -- especially MP 188.1 -- will recall that, in our report of May 1972, we proposed a conceptual, remedial plan based on limited borings but no sampling or triaxial testing. Later, from ground profiles, scarps (breaks) etc., we synthesized the effective shear strength parameters by use of our computer program. These analyses and sketches were carried to the August 2 meeting. It was our hope that the synthesis approach would be acceptable in this instance and in subsequent situations (such as MP 152.7 and some of the current sites on I 75 in Grant and Boone Counties). The arguments offered were somewhat persuasive to those who heard them but not to others who have higher authority. The counter-argument was to the effect that a minimum of boring, sampling and testing would suffice in a placative way. However, we elected to proceed with an in-depth investigation at MP 188. The option of doing the work with Research forces or having it done by a consultant was somewhat contingent upon a complete and thorough description of the work to be performed. In fact, after much delay, it seemed easier to do the work. A dozer-mounted drill rig enroute from Greenup to Manchester was detained about 10 working days at MP 188.1 and 152.7 during the first part of February. The report now submitted (herewith) presents boring logs, shear tests, and analyses in great detail. It was submitted in draft form, unedited and with pencil drawings in order to meet a promised deadline of April 1.

I recommend, dutifully, the 3:1 and 6:1 configuration shown in Figure 23 (mentioned there and elsewhere as Case 9). This is the same filling profile determined at the August 2, 1972, meeting (Note:

safety factor determined by synthesis was 1.59, adjusted to 1.65; ϕ by synthesis was 23.5°). My recommendation here gives some weight to the erodable soils and confluence of drainage which apparently caused the failure and which will likely persist after reconstruction although due care is taken to contain surface drainage in paved flumes. Recommendations enumerated 2 and 3 in my memo of August 2, 1972, remain unchanged.

I regret to say that the analysis of the slide at MP 152.7 has not been completed. It is not part of the subject project. Priority was given to MP 188. The other analysis is progressing, and a separate report will follow.

JHH:dw

Attachments: 1) Research Report 363; "Stability of a Side-Hill Embankment, I 64, Lexington - Catlettsburg Road"
 2) Research memo dated August 21, 1972; with attachments.
 3) Research Report, May 1972.

cc's: C. G. Grayson
 J. W. Spurrier
 G. F. Kemper
 J. T. Anderson
 J. E. McChord (Attn. Henry Mathis)
 C. S. Layson
 E. B. Gaither
 L. G. Sturgill (District 9)
 Marx Anderson (District 7)

Research Report
363

STABILITY OF A SIDE-HILL EMBANKMENT

I 64, LEXINGTON-CATLETTSBURG ROAD
Milepost 188, Stations 282+38 to 284+20
Project SP-10-115-29L

INTERIM REPORT
KYP-72-38; HPR-1(8), Part III

by

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Division of Research
Bureau of Highways
DEPARTMENT OF TRANSPORTATION
Commonwealth of Kentucky

The contents of this report reflect the views
of the authors who are responsible for the facts
and the accuracy of the data presented herein.
The contents do not necessarily reflect the official
views or policies of the Kentucky Bureau of Highways.
This report does not constitute a standard,
specification, or regulation.

April 1973

INTRODUCTION

The report submitted herein is a case history describing observations and in-depth analyses made at a highway site involving a massive, unstable side-hill embankment located on I-64 in Boyd County. The in-depth study was initiated in January 1973. The I-64 site was selected for study because it contained several design, construction and maintenance features as well as soil types that are typical of many highway embankments in Kentucky. Side-hill fill situations are common design problems and oftentimes have required extensive maintenance after construction. Major objectives of the study were to 1) determine the causes of instability of the I-64 embankment, 2) check a remedial solution previously reported (1, 2) for the I-64 site and present alternative solutions if necessary, 3) determine short-term (initial) and long-term safety factors of the embankment slopes, and 4) compare theoretical shear surfaces obtained from a slope stability program (3) based on Bishop's simplified method of slices (4) with actual failure points obtained from slope indicators and surface observations. All slope stability computations were carried out in terms of effective stress using shear strength parameters obtained from consolidated, isotropic, undrained triaxial tests with pore pressure measurements.

PROJECT DESCRIPTION

The project site is located on I 64 (Lexington-Catlettsburg Road, Project E I 64-8(9) 187) near Milepost 188 and lies in the northeastern portion of Kentucky. It is situated approximately 5 miles west of the Kentucky-West Virginia border in Boyd County. The unstable portion of the embankment is about 350 feet in length, extending from Station 282+00 to Station 285+50. The width of the slip is about 300 feet. I 64 in the site area is a 4-traffic lane, bituminous concrete facility, and the 2-lane roadway pavements are separated by a 36-foot median. Near Station 282+00, the roadway emerges from a cut section; it re-enters a cut section at Station 296+00. Between these two stations, the fill attains a maximum height of 110 feet at Station 288+50. The facility provides major arterial service between the central and northeastern portions of Kentucky. This portion of I 64 was completed in about 1965. A general view of the failure is shown in Figure 1.

An areal plan and typical sectional view of the site are shown in Figures 2 and 3, respectively. The unstable embankment (eastbound lanes) is a side-hill fill and is located on foundation soils some 15 to 25 feet in thickness. Slope of original groundline and rockline along a sectional view of the slip ranges from about -58 percent near centerline to -7 percent in the vicinity of the toe of the slip. From Station 284+00 to Station 288+20, along roadway centerline, slope of original groundline is about -18 percent. Grade of roadway in the slide area is +2.7 percent. The major portion of the westbound lanes of I 64 in the slide area is located in a cut section. Other features of the site include a median cross drain located at Station 282+00, a 24-inch pipe at Station 282+50 which drains the north side of the roadway, a median cross drain at Station 286+35, and a 4-foot by 4-foot reinforced culvert located at Station 288+50. Outlet end of the culvert is near Station 286+14. A small stream is located at the toe of the unstable embankment.

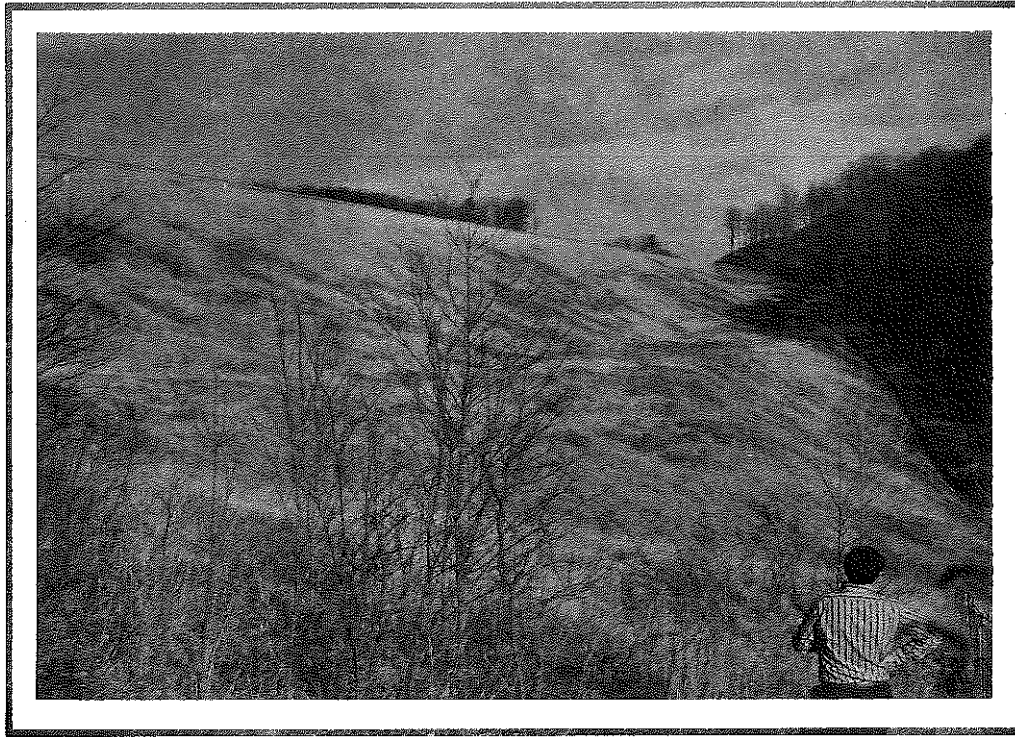


Figure 1. General View of the I-64 Embankment Slip Located Near Station 284+00.

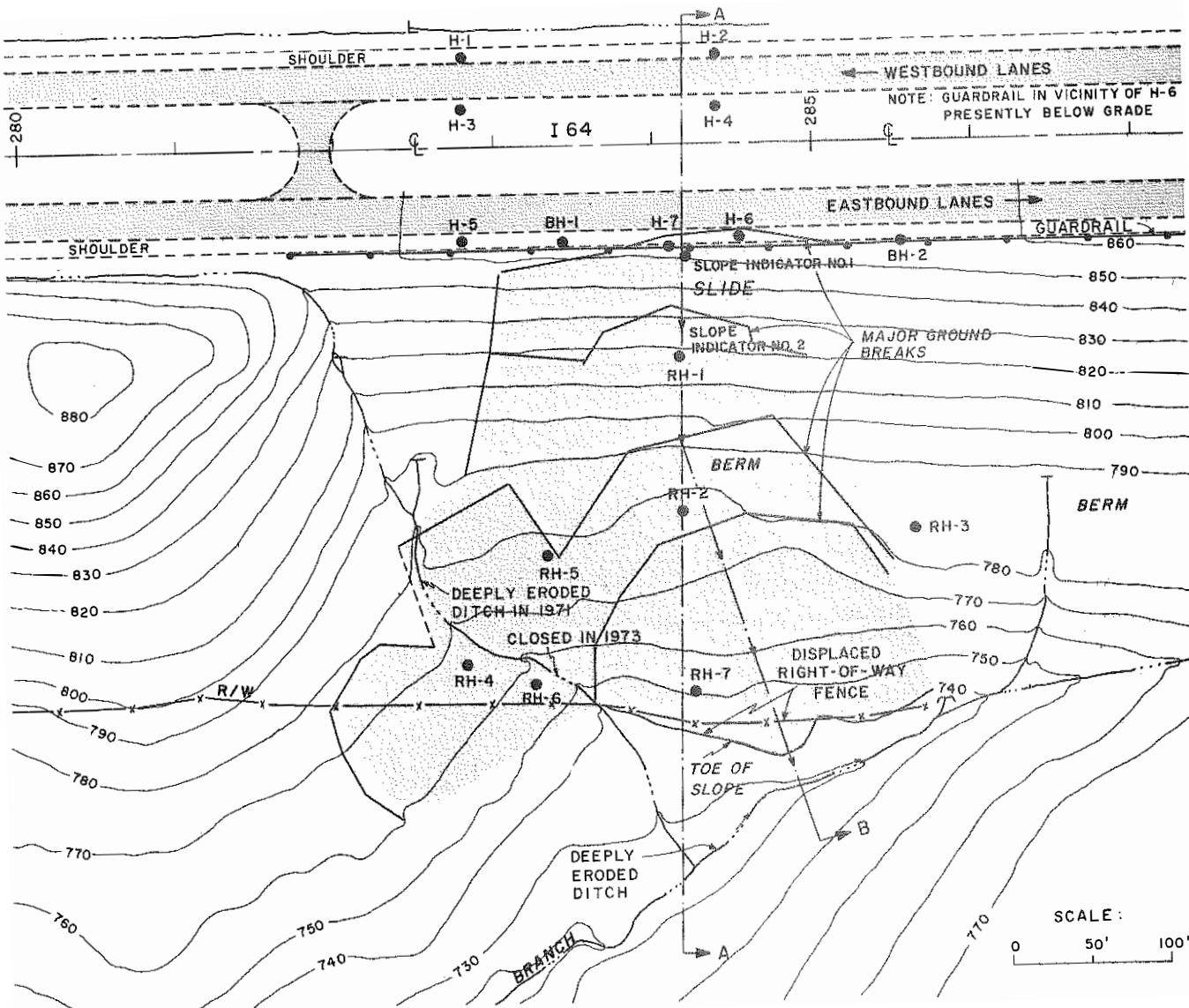


Figure 2. Areal Plan of the I-64 Embankment Slip.

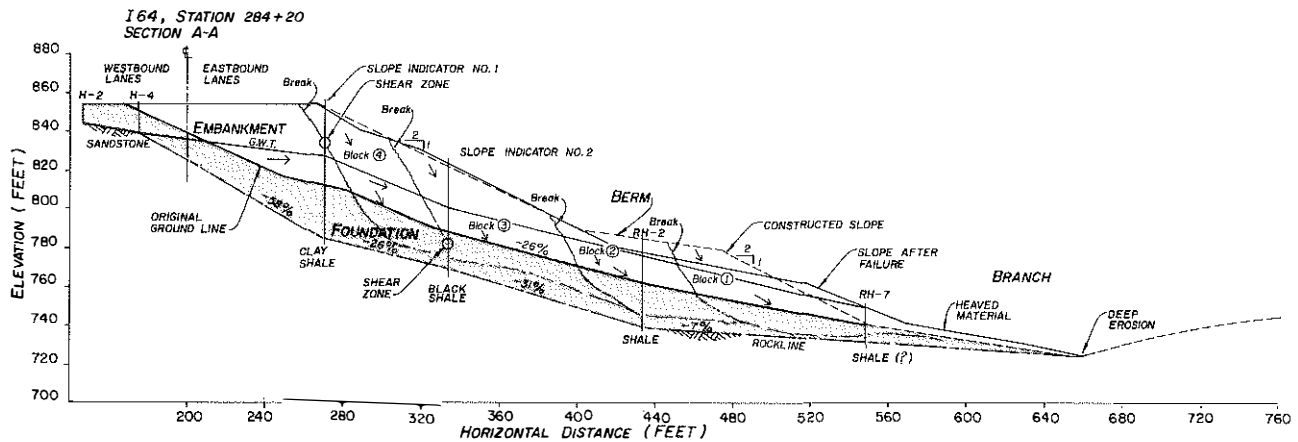


Figure 3. Typical Cross-Sectional View of the I-64 Slip, Station 284+20, Boyd County.

SURFACE INVESTIGATION

Topography. The site is located in the northeastern portion of the Eastern Kentucky Mountains, a major physiographic unit of Kentucky. This region is a part of the Cumberland (Allegheny) Plateau, a section of the Appalachian Plateau. The area is a maturely dissected plateau of varying altitude and relief. It is a region of dendritic drainage and contains irregularly winding, narrow-crested ridges and deep narrow valleys. The area is characterized by Pennsylvanian outcrops. Total variation in relief near the site is approximately 250 feet, ranging from an elevation of 700 feet in the bottom of the stream at the toe of the slip to an elevation of 950 feet at the crests of the adjoining hills.

Slide Surface Conditions. Surface conditions observed at the site in early 1973 are depicted in Figure 2. A portion of the surface study was devoted to mapping the limits of the slip. Stadia was used to determine the locations of the major ground breaks and all borings and slope indicator casings. These data were reduced using a computer program and transferred to an aerial topographic map obtained from the Division of Photogrammetry, Bureau of Highways. The mapped data show that there are four major surface breaks in the embankment slip area; a fifth break occurs at the western flank of the slide area.

The emergency lane of the eastbound lanes has been patched on numerous occasions (see Figure 4). The deeply eroded ditch running along the western flank of the slip and observed in March 1971 (1) has completely closed. This ditch, which ranged from 3 to 5 feet in depth and measured about 5 feet in width, was formed by large quantities of water draining from a median

drain at Station 282+00 and a 24-inch pipe at Station 282+50. As shown in Figure 3, the toe of the berm has moved outward (southward) some 20 feet; a deeply eroded ditch is located at the toe of the embankment failure. Other signs of incipient failure were observed in early 1973. The guardrail located on the shoulder of the eastbound lanes has moved outward and downward 1 or 2 feet. The right-of-way fence located at the toe of the berm has moved outward approximately 15 feet. The slide in the berm was observed in 1967 (1).



Figure 4. View of Patched Pavement.

SUBSURFACE INVESTIGATION

Engineering Geology. Bedrock of the hills adjoining the site is composed of sedimentary rocks of Pennsylvanian age. A generalized geologic columnar section is presented in Figure 5. The slide is located stratigraphically in the lower portion of the Conemaugh Formation and possibly the extreme upper portion of the Breathitt Formation. Rocks of the area are a complex mixture of different types and consist of alternating layers of sandstones, shales, limestones, siltstones, and coal. The beds of rock or facies pinch in and out and are not consistent, even over short distances. Natural landslides occur throughout the area.

Rocks of the area are loosely consolidated and have little resistance to erosion; consequently, oversteepened slopes are common. Shales are the most predominate rock type occurring in the Conemaugh and Breathitt Formations. The shales are silty and sandy; when weathered, these shales usually become soft clays. Clay seams occur throughout both formations and are semiplastic to plastic materials. These soils are commonly referred to as underclays or "fireclays". Sandstone lenses, which are usually medium to coarse grained and loosely cemented, occur frequently and are a major cliff former in the area. Limestone layers do occur in the area but are of little significance.

The mountainous and geologically mature topography at the site was formed by weathering and erosion of the loosely consolidated rocks. The surface of sedimentary rock strata decomposed through weathering, forming a relatively thin zone of residual soils which measures 15 to 30 feet in thickness. These soils are plastic and are the most unstable regolith found in the area. The unstable embankment is located on the residual soils paralleling the underlying rock surface.

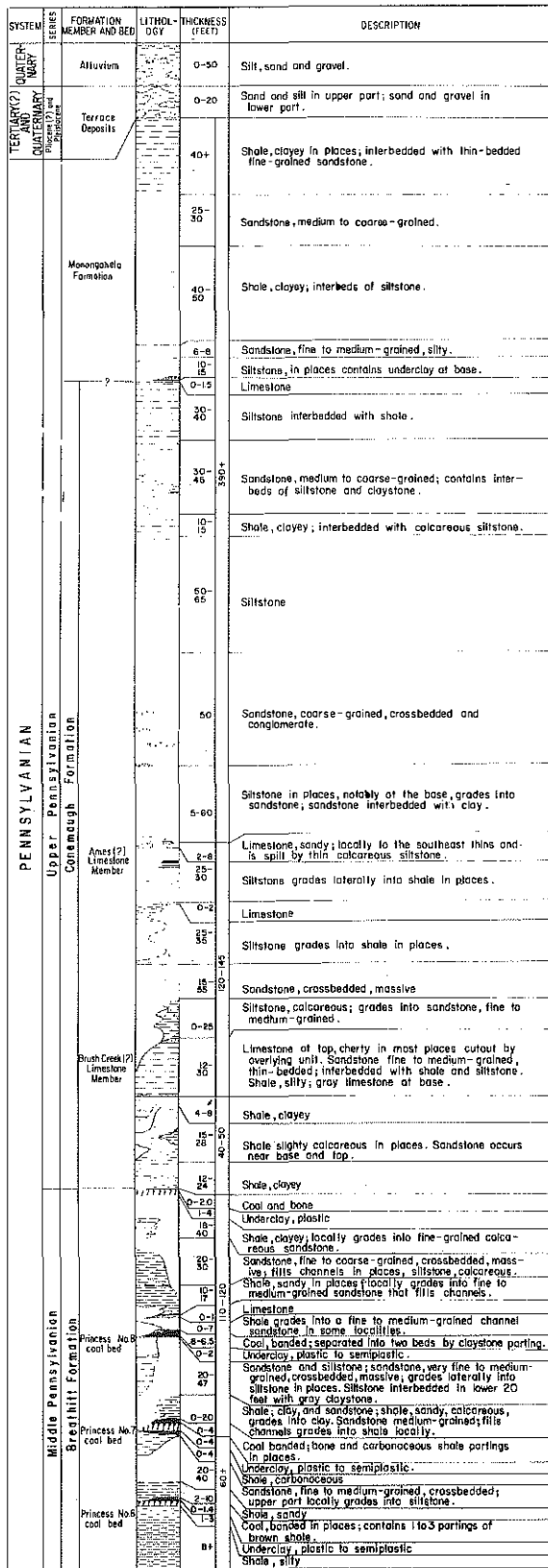
Based on past observations, most landslide problems in the area are associated with the impermeable underclays and plastic shale beds. More specifically, landslides frequently occur along the underclays of the Breathitt Formation and along the shales and siltstones of the Conemaugh Formation where hillsides are steep. Lateral ground-water seepage is one of the most significant factors leading to the development of slides in the area. The ground water percolates down through the permeable layers until it reaches an impermeable underclay. The water then migrates along the top of the clay until it reaches the surface.

Soil Exploration. Twenty-eight undisturbed Shelby tube soil samples were obtained from five borings. Locations of points from which the tube samples were extracted and of the borings are shown in Figures 6 through 8 and Figure 2, respectively. Soil exploration consisted of a total of 16 borings. Seven borings (H-1 through H-7) were drilled in 1971 (15) and were located at the top of the slide (grade elevation of roadway). The other nine borings (BH-1, BH-2, and RH-1 through RH-7) were obtained during the investigation made in March 1973. Borings RH-1 and RH-2 were obtained using a drill mounted on a dozer. Boring logs are presented in Figures 6 through 8. Five Dutch Cone penetration tests were performed at the top of the slide in an attempt to locate the failure zone and in an effort to develop a correlation between shear strength and cone values. Only two tests were successfully completed; the other three tests were abandoned because of rocky material in the embankment. One set of Dutch Cone data indicated that the failure zone was located in the top 20 feet of the upper portion of the slip. This correlated well with slope indicator data. However, a second set of cone data obtained near a slope indicator did not correlate with the slope indicator data.

Field Instrumentation. In 1971, slope indicator casing (No. 1 in Figure 3) was installed at Station 284+20, about 70 feet right of centerline (5). During the investigation conducted in February and March 1973, a second slope indicator was installed in Boring RH-1 (see Figure 2). These indicators were installed in an effort to determine the slip zones, rate and direction of movement, and mode of failure of the fill slip. Horizontal resultant movement as a function of depth and resultant movement-time curves for selected depths are shown in Figures 9 and 10. Data obtained from Slope Indicator 2 indicate a shear zone is located approximately 45 feet below ground surface. Slope indicator data obtained from Well No. 1 during the period March 1971 to April 1972 shows a distinct shear plane is located about 20 feet below ground surface. No slope indicator data was obtained from that well after the later date because the casing grooves closed at a depth of about 20 feet below ground surface.

Three Casagrande-type piezometers were installed in Borings RH-5, RH-6, and RH-7. Data from these instruments were not available at the end of this present investigation. The phreatic surface of the slide area was established mainly from the slope indicator wells and observed surface seepage areas located at the toe of the slide.

Figure 5. A Generalized Geologic Columnar Section of the I-64 Embankment Slip Area.



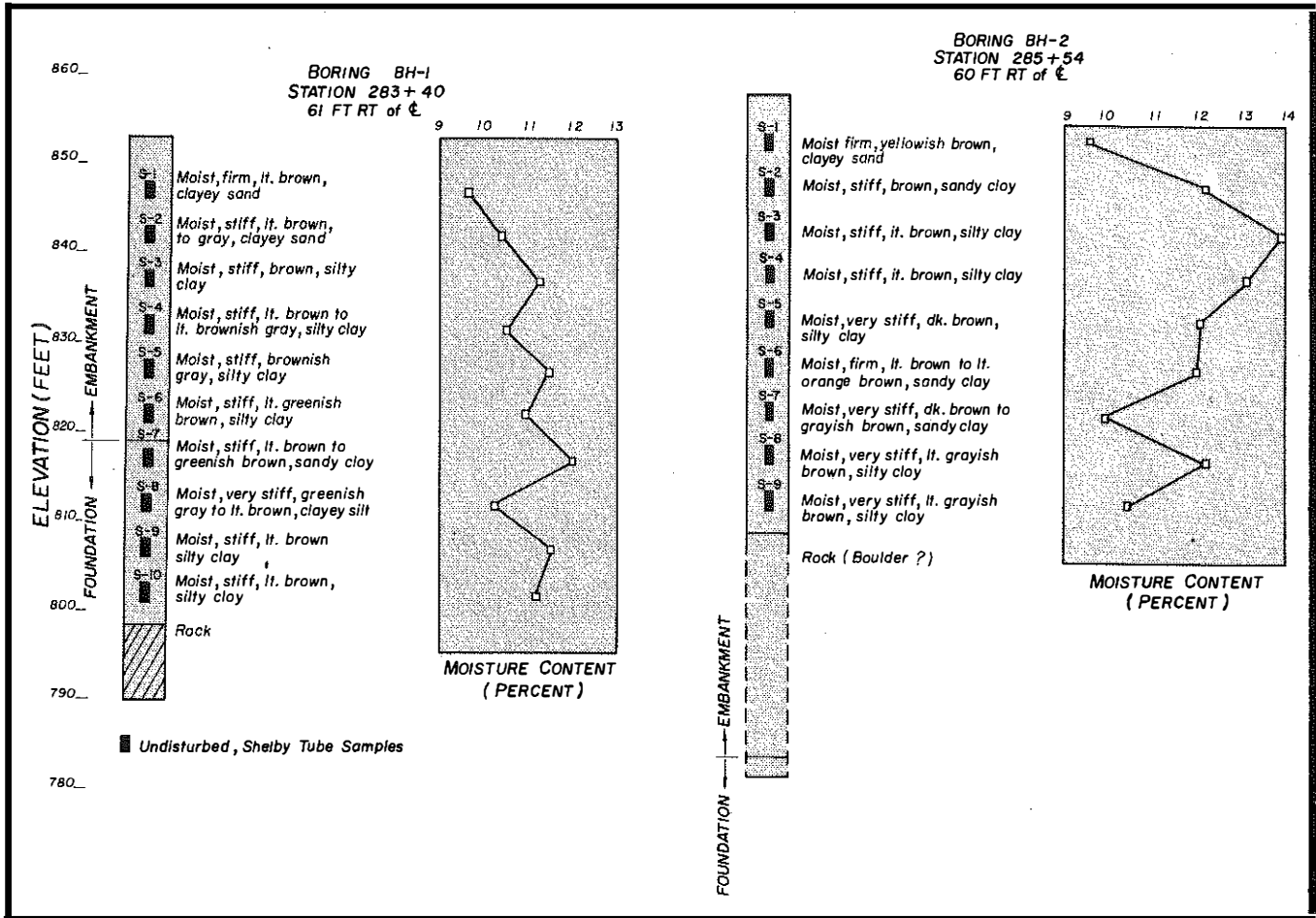


Figure 6. Boring Logs, BH-1 and BH-2.

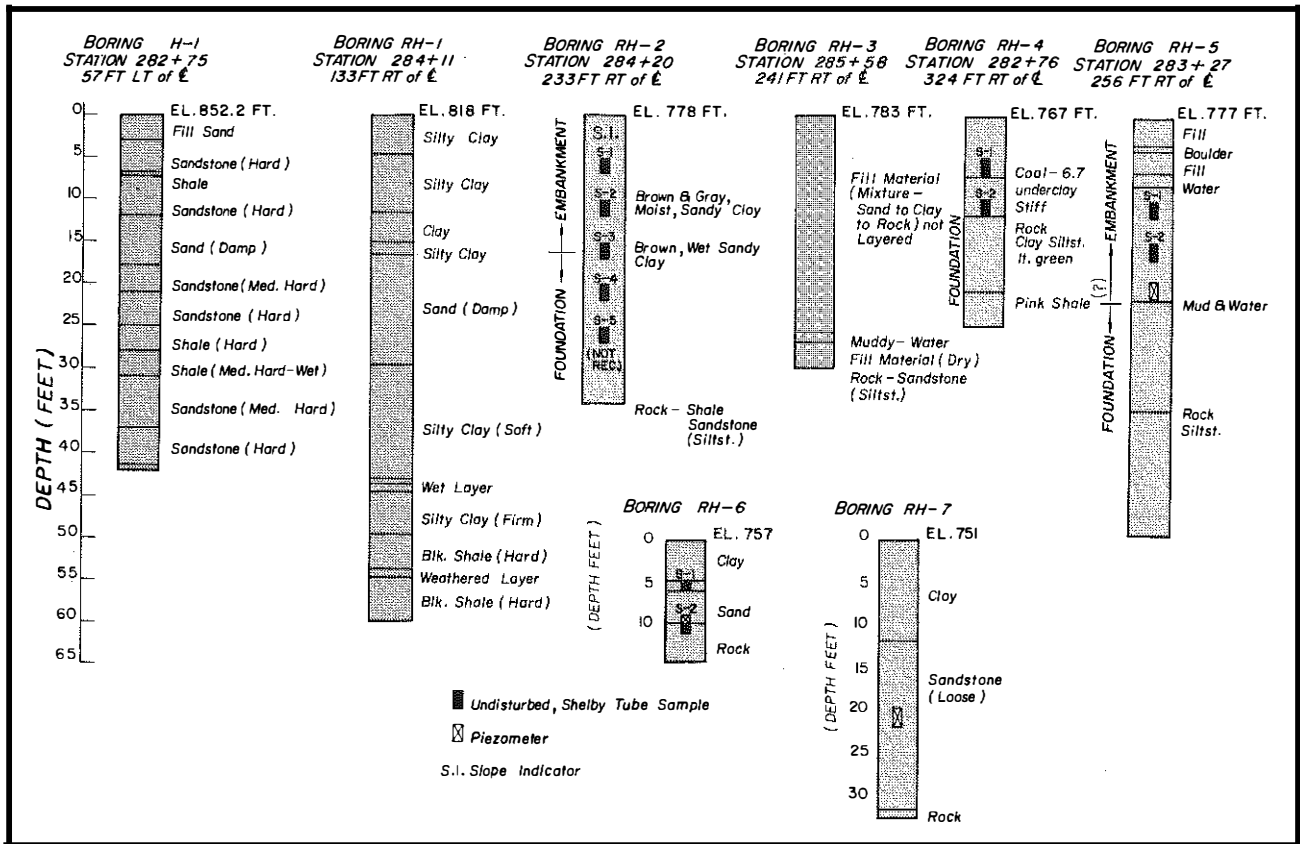


Figure 7. Boring Logs, H-1 and RH-1 through RH-7.

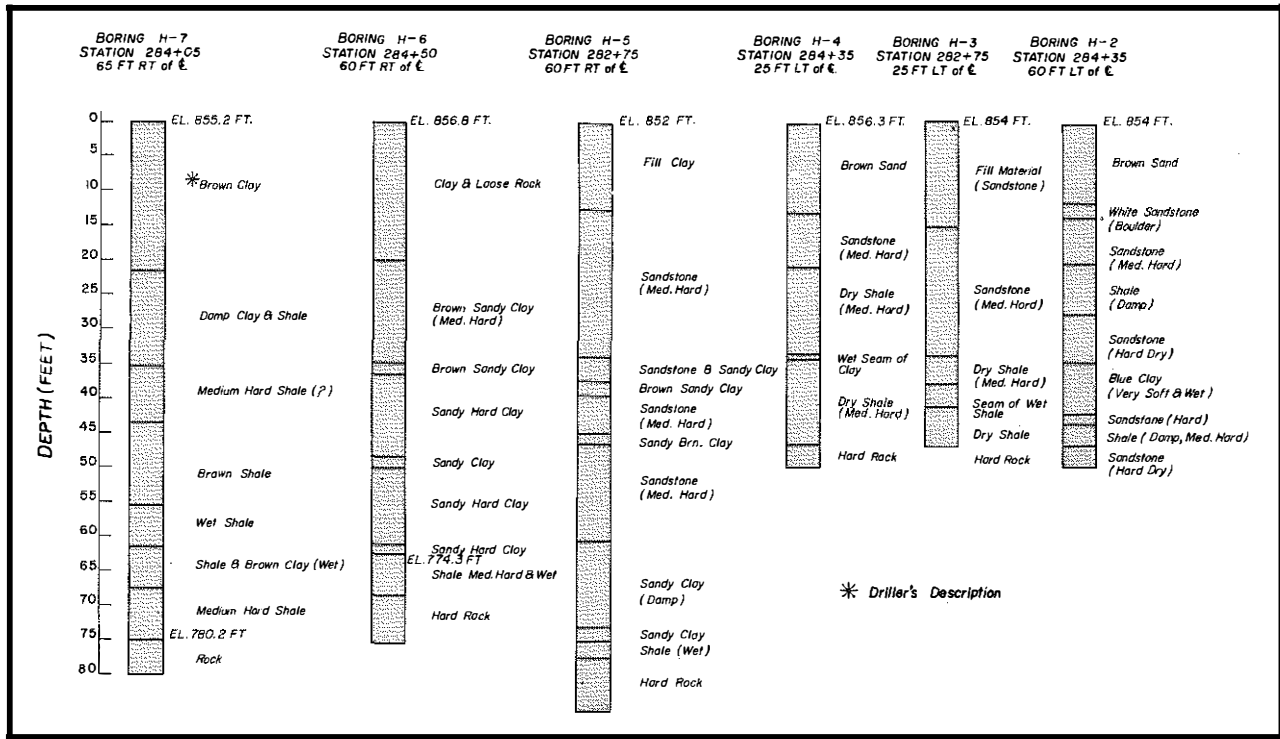


Figure 8. Boring Logs, H-2 through H-7.

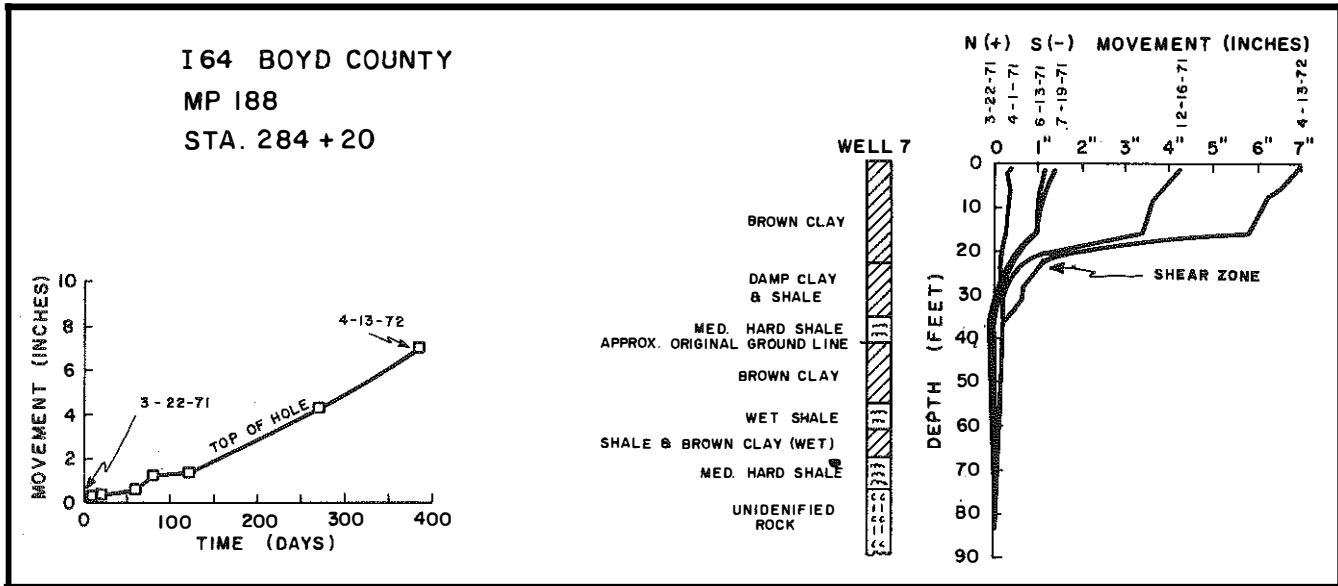


Figure 9. Slope Indicator Results, No. 1.

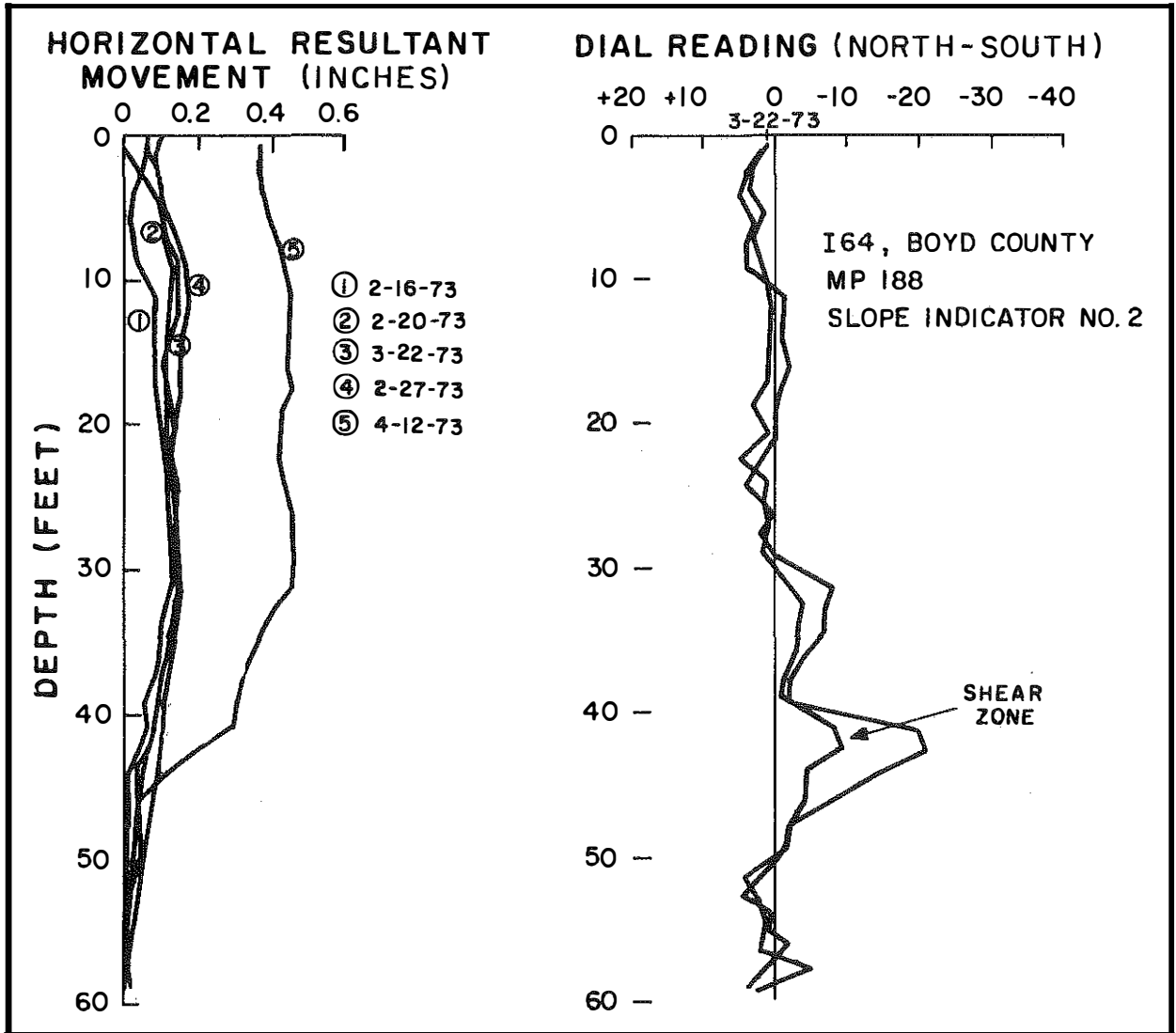


Figure 10. Slope Indicator Results, No. 2.

LABORATORY INVESTIGATION

Sample Preparation. Soil samples were extruded from the Shelby tubes, cut into 4-inch lengths, identified according to the visual-manual procedure (ASTM Designation: D 2488 T), waxed and stored until ready for testing. Water content determinations were performed on each of the extruded samples (see Figure 6 and Table 1).

Shear Strength Tests. Soil strength parameters of the embankment and foundation materials were established from consolidated, isotropic, undrained triaxial tests (CIU) with pore pressure measurements. The triaxial compression samples were subjected to a back pressure to completely saturate each test specimen and sheared undrained at a rate of 0.001 inches per minute. Pore pressures were obtained from pore pressure transducers. A total of 19 CIU tests were performed on the soils from the slide area. Three consolidated, drained, direct shear tests (CDS) were performed on specimens obtained from the upper portion of the embankment. The CDS testing procedure has been described elsewhere (6).

Shear strength parameters obtained from the CIU and CDS tests are tabulated in the left portion of Table 1. Results of the CIU, that is, effective stress paths and change in pore pressure and deviator stress as a function of strain, are presented in Figures 11 through 16. CDS test results are shown in Figure 17. All stress paths in Figures 11 through 16 generally are either vertical or curved to the right on the p-q diagram. This indicates the embankment and foundation soils of the slide are over-consolidated. Except for results shown in Figure 16, stress paths at failure generally follow the K_f -failure envelope. Results shown in Figure 16 indicate the foundation specimens obtained from Boring RH-4 were highly over-consolidated. As shown in Figure 7 (Boring RH-4, Sample 2), these test specimens were an underclay or clay shale. The unusual stress paths obtained for these samples were a result of the highly preconsolidated nature of the clay shale specimens; consequently, confining pressures used in testing the clay shales apparently did not influence their failure strengths.

The angle of shearing resistance, ϕ' , of the unstable embankment soils was generally about 29°; it ranged from 27° to 33° (see Table 1). The shear parameter, c' (cohesion), was zero as determined from the CIU tests. Based on the CDS tests, c' was 476 pounds per foot square. The shear strength parameter, ϕ' , of the foundation located in the vicinity of the centerline of roadway ranged from about 27° to 29°. In the lower region of the slide, the angle of shearing resistance, ϕ' , of the foundation soil was 23° or 24°; cohesion was assumed to be zero.

Soils. Based on boring and laboratory test results, the embankment soils consist of moist, stiff, brown to gray silty clay with some moist, stiff, brown to gray clayey sand in the upper zones. These soils classify as CL with liquid limit and plasticity index of about 36 and 18 percent, respectively. The foundation soils consist of moist, stiff, light brown sandy to silty clay underlain by shales. These soils classify as CL; at the toe of the slide, the soils classify as CL or MH. The liquid limit and plasticity index for the foundation soils are about 35 and 15 percent, respectively. Natural moisture contents ranged from 12 to 15 percent in the embankment and from 13 to 24 percent in the foundation.

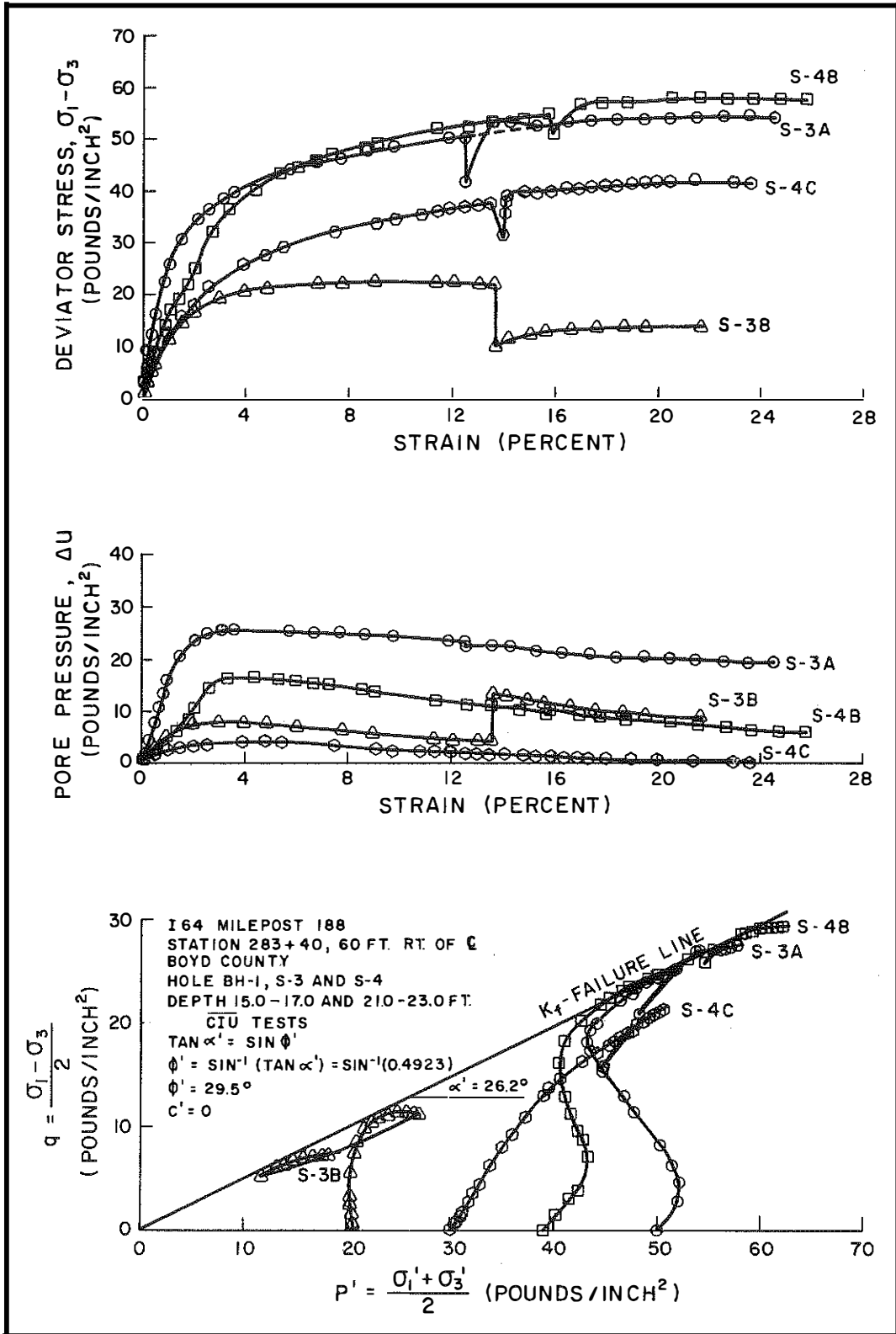


Figure 11. CIU Test Results, Boring BH-1, Samples 3 and 4.

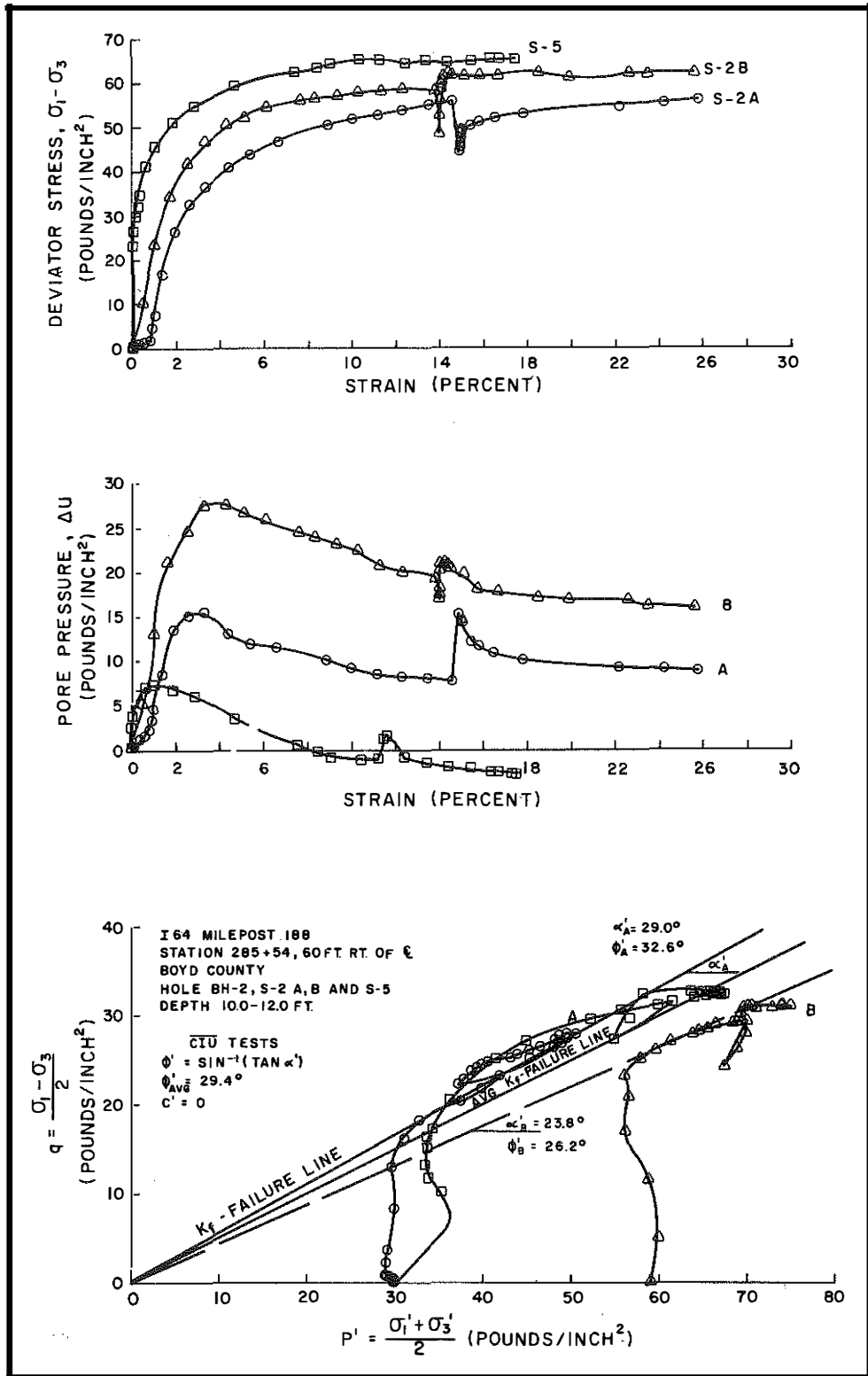


Figure 12. CIU Test Results, Boring BH-5, Samples 2A, 2B and 5.

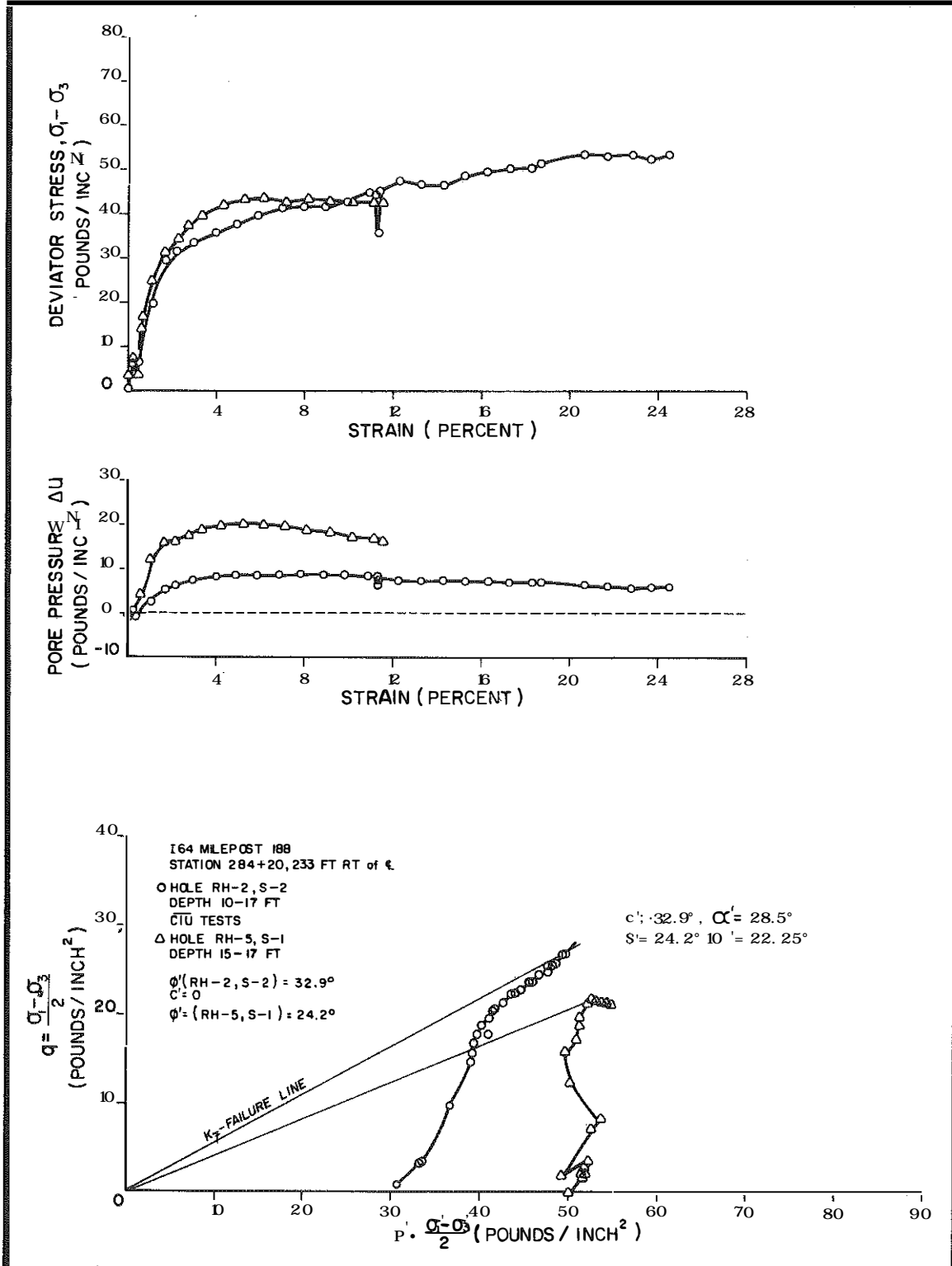


Figure 13. CIU Test Results, Boring RH-2, Sample 2, and Boring RH-5, Sample 1.

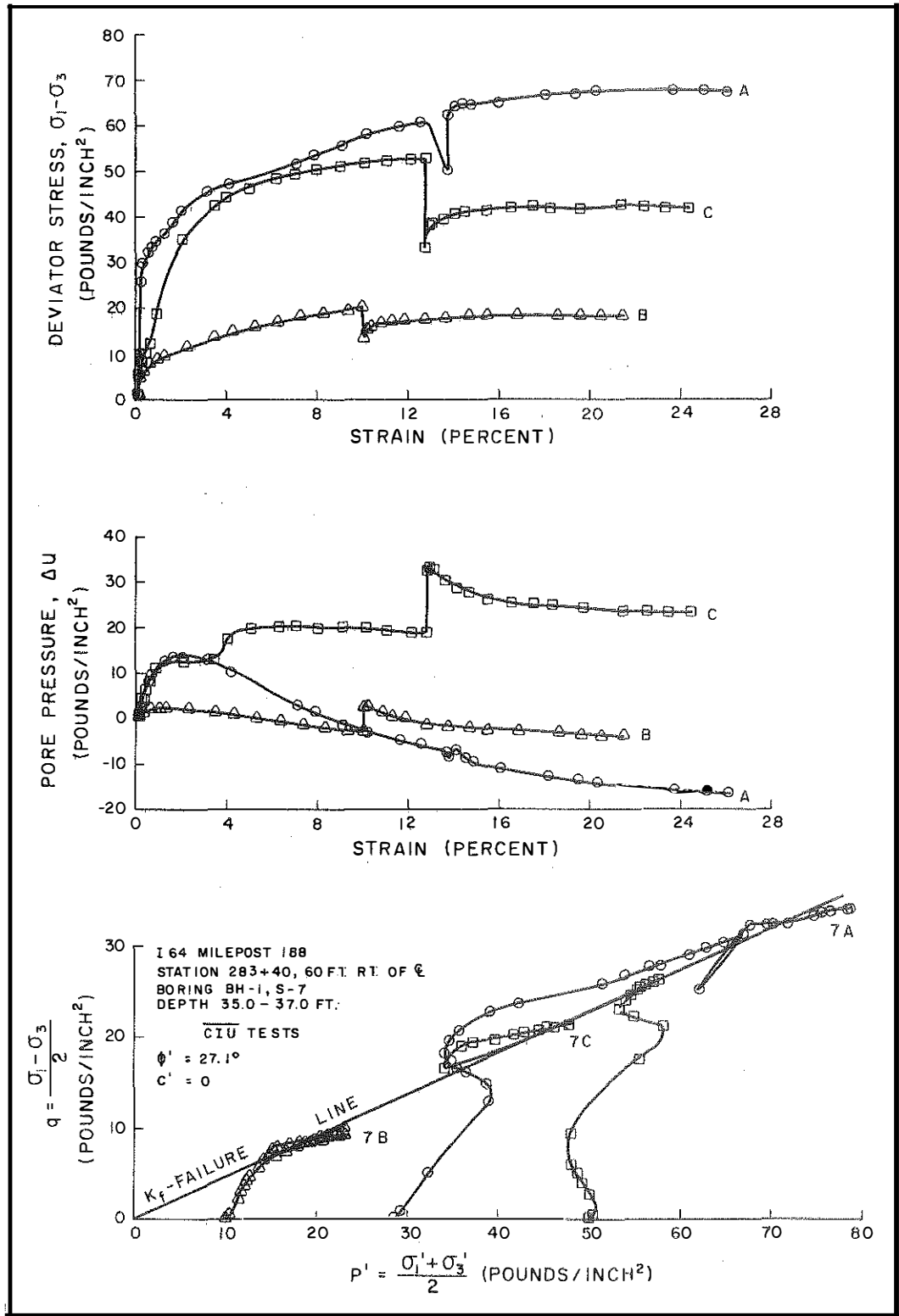


Figure 14. CIU Test Results, Boring BH-1, Sample 7.

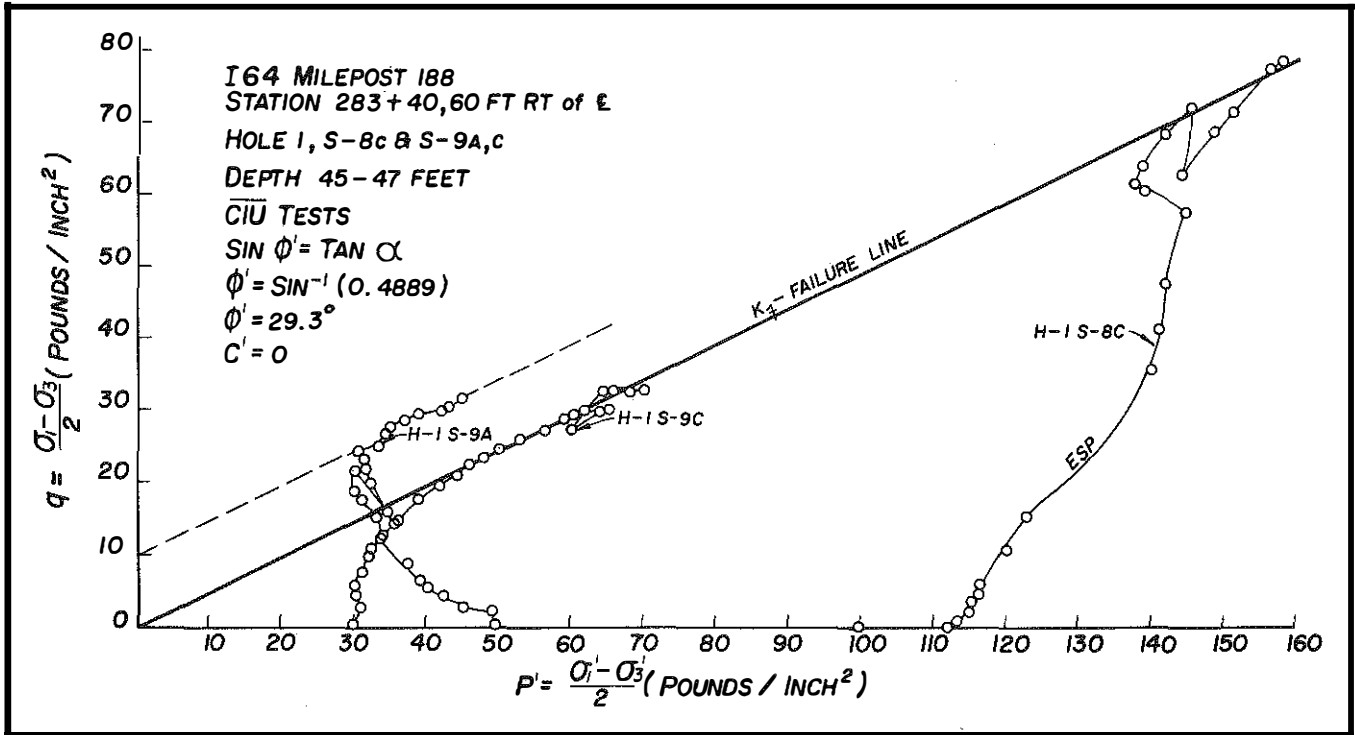


Figure 15. CIU Test Results, Boring BH-1, Samples 8C, 9A and 9C.

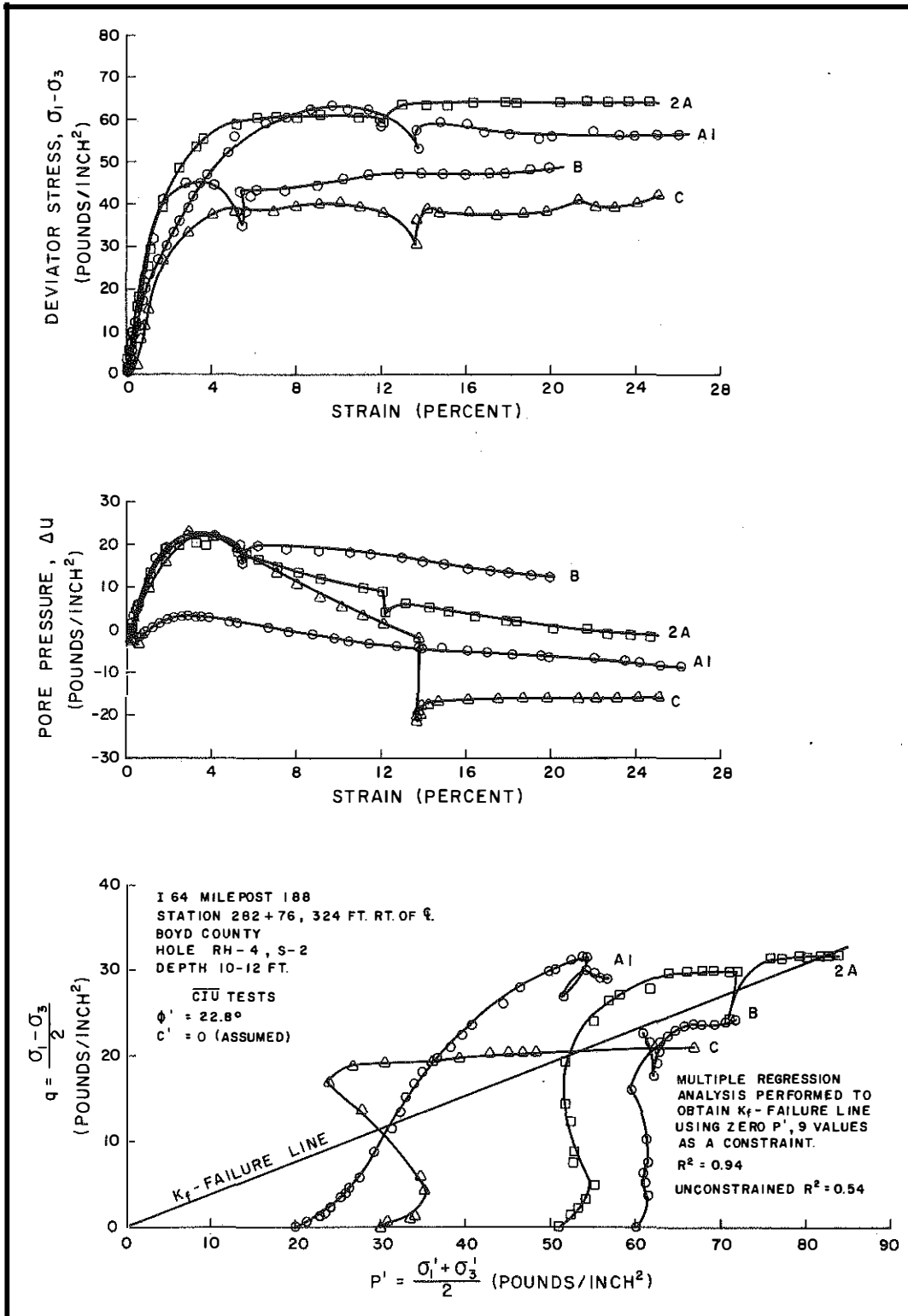


Figure 16. CIU Test Results, Boring RH-4, Sample 2.

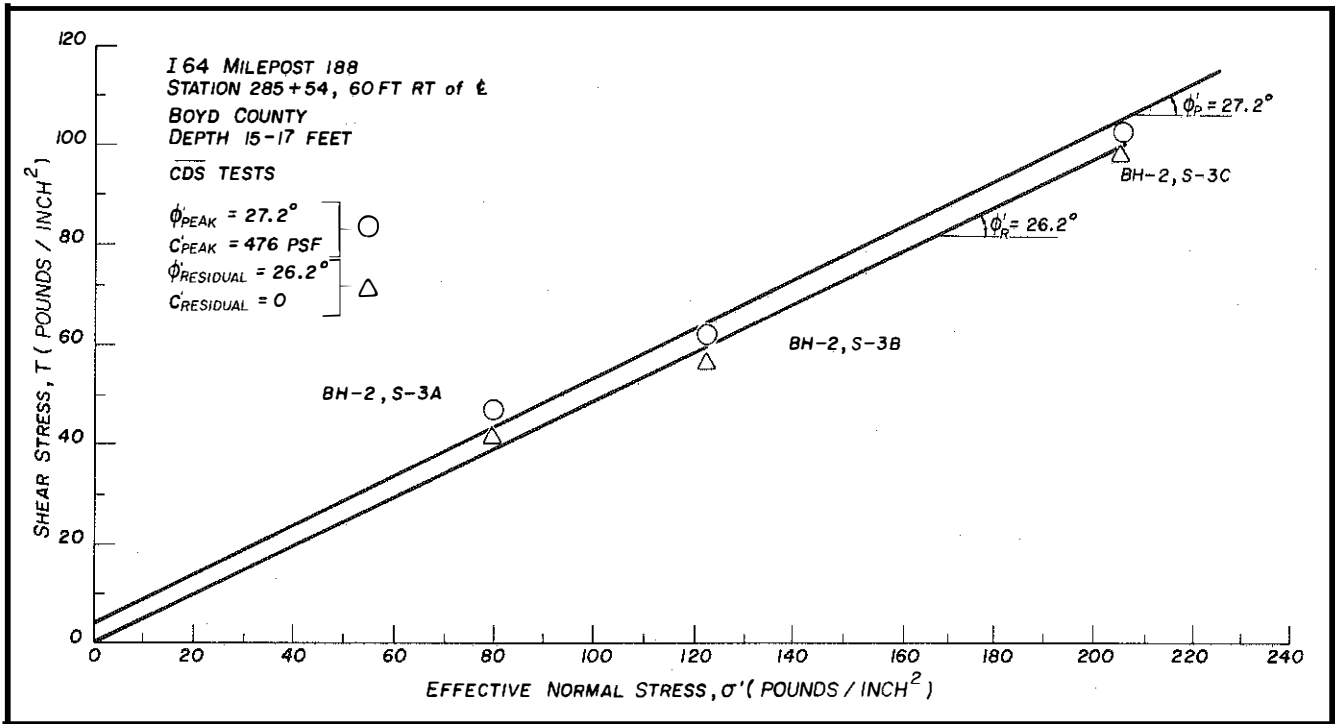


Figure 17. Consolidated Drained, Direct Shear Test Results, Boring BH-2, Samples 3A, 3B, and 3C.

TABLE 1
SUMMARY OF SOIL TEST DATA

BORING NUMBER	SAMPLE NUMBER	GENERAL LOCATION OF SPECIMEN	SHEAR STRENGTH PARAMETERS		ATTERBERG LIMITS (PERCENT)		NATURAL WATER CONTENT (PERCENT)	CLAY FRACTION <2 μ (PERCENT)	UNIFIED CLASSIFICATION
			ϕ^{\dagger} peak (DEGREES)	c^{\dagger} peak (POUNDS/FOOT ²)	LIQUID LIMIT	PLASTICITY INDEX			
BH-1	S-3A	Embankment, Upper Portion of Slide	29.5	0			13.5		
	S-3B						13.6		
	S-4B						12.2		
	S-4C						12.1		
BH-2	S-2A	Embankment, Upper Portion of Slide	29.4	0			12.9		
	S-2B						14.7		
	S-5								
BH-2*	S-3A	Embankment, Upper Portion of Slide	27.2	476	35.0	13.8			CL
	S-3B		26.2 ⁺	0 ⁺					
	S-3								
RH-2	S-2	Embankment, Berm Area of Slide	32.9	0	37.8	22.4	13.9	22	CL
BH-1	S-7A	Foundation, Upper Portion of Slide	27.1	0			14.4		
	S-7B						15.0		
	S-7C						14.5		
BH-1	S-8C	Foundation, Upper Portion of Slide	29.3	0	27.3	10.4	13.1		CL
	S-9A						15.8		
	S-9C						12.8		
RH-5	S-1	Foundation, Berm Area of Slide	24.2	0	33.8	14.2	23.8	34	CL
RH-4	S-2A1	Foundation, Berm and Toe Area of Slide	22.8	0	47.4	21.1	19.4	44	CL or MH
	S-2A2						16.9		
	S-2B						20.3		
	S-2C						23.3		

*Consolidated, Drained, Direct Shear Tests,
+Residual Values.

ANALYSIS

Nature of Embankment Failure. As shown in Figure 2, the upper portion of the embankment is moving southward and in a direction approximately perpendicular to roadway centerline. The lower half of the unstable mass is moving in a southeasterly direction and about perpendicular to the flowline of the branch located at the toe of the failure mass. The direction of movement is indicated by Section A-B in Figure 2. Surface cracking indicates there are four major failure blocks (numbered 1 through 4 in Figure 3). Three factors contributed to the embankment slip: 1) erosion at the toe of the failure and in a ditch running along the western flank of the slip, 2) intrusion of ground water into the embankment lowering the shear strength of the soils in the slip area, and 3) marginal stability of the original embankment and berm configuration, that is, oversteepened slopes.

The slow, progressive failure of the I-64 embankment developed when a slump (Block 1 in Figure 3) occurred in the berm located in the bottom portion of the embankment. This slip occurred in or prior to 1967 (5) and was triggered by erosion in the stream. With downward and outward movement of Block 1, the shear strength of soils in Block 2 were mobilized. Gradually, the failure continued to spread up the embankment slope until the eastbound lanes were affected. Large pavement settlements observed in 1972 were probably due to movement of Blocks 3 and 4 toward the deeply eroded ditch located at the western flank of the slide.

Stability Analysis. Slope stability of the I-64 embankment was analyzed using a computerized solution (3) of Bishop's simplified method of slices (4). The slope stability analyses were carried out in terms of effective stress using shear strength parameters obtained from consolidated, isotropic, undrained triaxial tests. In performing the analyses, the critical shear surfaces (a surface having a minimum value of safety factor) and potential shear surfaces (a surface having a safety factor of unity or less) were located by the computer program using a grid type, search operation. Based on data shown in Table 1, the adopted shear parameters, ϕ' and c' , for the embankment were 29.4° and 0, respectively. Corresponding values for the foundation were 29.4° and 0, respectively. In all analyses, the cohesive parameter, c' , was assumed to be zero. Such assumption was based on the condition that once movement occurred (as in this case) the cohesion of soils in the failure surface is destroyed. The phreatic surface observed in March 1973 was used in the analyses; this surface was assumed to be in an equilibrium condition.

Results of the stability analyses are summarized in Table 2. Two slope configurations, A-A and A-B (see Figure 2), were analyzed. Results in the top portion of the table represent analyses of the constructed slopes in combination with various water table conditions and the failed slopes (Cases 1 through 3 and Case 4, respectively). Results shown in the lower portion of the table pertain to remedial stability analyses. For Section A-A, the potential slip surfaces and critical shear surfaces for the various cases, except Cases 2 and 3, are presented in Figures 18 through 23.

Case 1 considers the long-term stability of the constructed slopes using the observed phreatic surface obtained in March 1973. As shown in Figure 18, two potential slip surfaces were obtained by the computer program's search operation. Both slip circles were located very close to the observed failure points. Safety factors of the smaller and larger slip circles were 0.93 and 0.99, respectively. In the analyses represented by Case 2, both the embankment and foundation soils were assumed to have a ϕ' -value of 29.4° and a c' equal to zero. The water table was assumed to be 5 to 15 feet lower than that observed in March 1973. The computed safety factor was 1.14. Case 3 represents the probable short-term stability of the slopes, assuming excess pore pressures due to consolidation of the foundation were equal to zero. The safety factor obtained for this case (1.19) indicates that the embankment slopes were initially stable, although the stability was relatively low. Case 4 considers the long-term stability of the failed slopes. The safety factor (0.96) obtained was about the same as for Case 1, indicating the present slope configuration of the embankment is unstable. Additionally, the potential shear surface (see Figure 19) associated with Case 4 is shifted deeper into the upper portion of the embankment. This analysis indicates that all traffic lanes in the slide area may eventually be affected.

In the remedial analyses, several berm and slope configurations were investigated. For a 3 horizontal to 1 vertical slope, the computed safety factor was 1.21 (Case 5). For the sloping berm configuration shown in Figure 20 (Case 6), the minimum safety factor was 1.31. Both of these remedial designs are considered inadequate. For the slope configurations shown in Figures 21, 22 and 23, the minimum safety factors were 1.43, 1.43 and 1.46. The configuration shown in Figure 23 had been proposed previously (2) as a remedial solution. The safety factor for this configuration is near the value (1.50) normally accepted for design of permanent structures.

TABLE 2
SUMMARY OF STABILITY ANALYSES

CASE NUMBER	SAFETY FACTORS		ANGLE OF SHEARING RESISTANCE*, ϕ' (DEGREES)			WATER TABLE LOCATION	REMARKS
	SECTION		UNSTABLE SOIL MASS				
	A-A	A-B	EMBANKMENT	FOUNDATION	BERM		
1	0.99	1.04	29.4	24.0		Observed March 1973	As-Constructed Slopes (Figure 18)
2	1.14		29.4	29.4		About 5 to 15 Feet Below Observed W. T. of March 1973	As-Constructed Slopes
3 [†]	1.19		29.4	24.0		Midpoint of Foundation	As-Constructed Slopes
4	0.96	0.96	29.4	24.0		Observed March 1973	Failed Slope Configuration (Figure 19)
5	1.21	1.44	^b 29.4	24.0	29.4	Observed	3:1 Slope Extending from Shoulder of Roadway to Toe of Failed Embankment (Figure 20)
6	1.31		29.4	24.0	29.4	Observed	3:1 Embankment Slope with Berm at Mid-Height Having Width of 30' and 2.5:1 Slope (Figure 20)
7	1.43		29.4	24.0	29.4	Observed	3:1 Slope with 6:1 Sloped Berm at Mid-Height (Figure 21)
8	1.43		29.4	24.0	29.4	Observed	2:1 Slope with 4:1 Sloped Berm Located About Two-Thirds from Top of Roadway (Figure 22)
9	1.46	1.71	29.4	24.0	29.4	Observed	Proposed Remedial Solution: 3:1 Slope and 6:1 Sloped Berm at Bottom of Failure (Figure 23)

*c' Assumed Equal to Zero.

[†]Short-Term Stability.

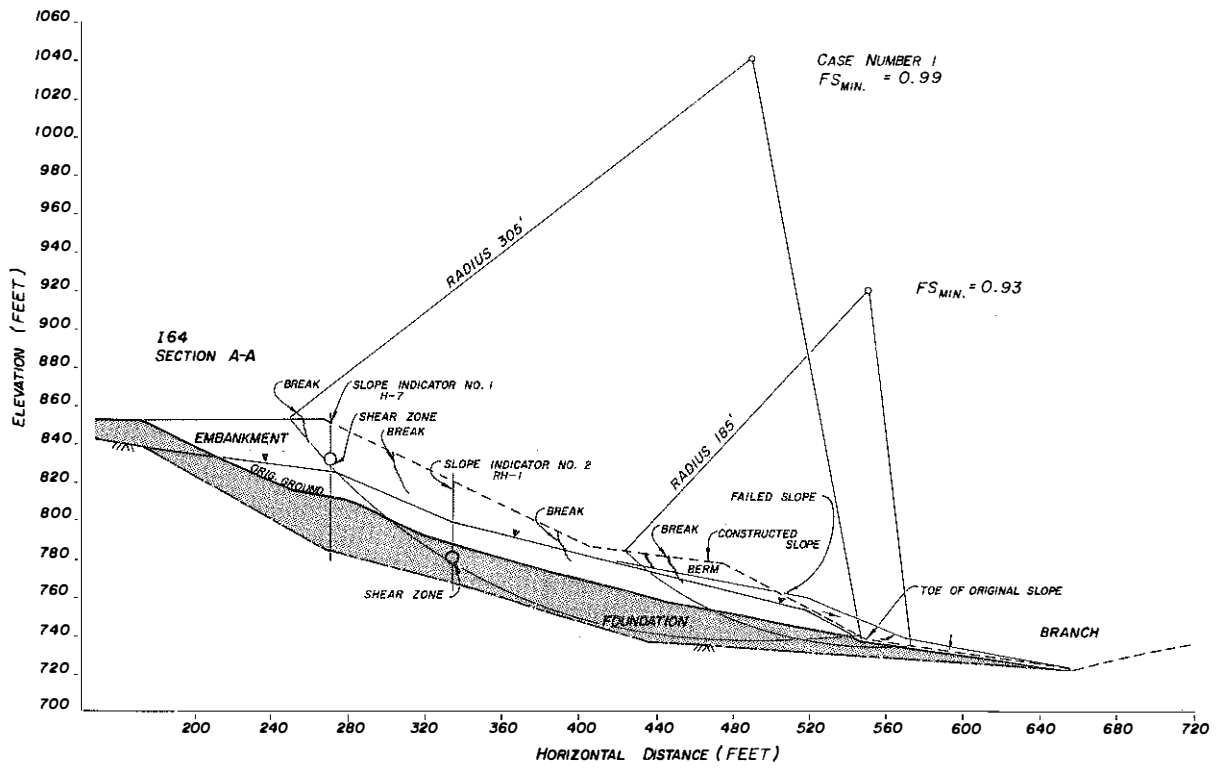


Figure 18. Potential Slip Surfaces of As-Constructed Slopes, Case 1.

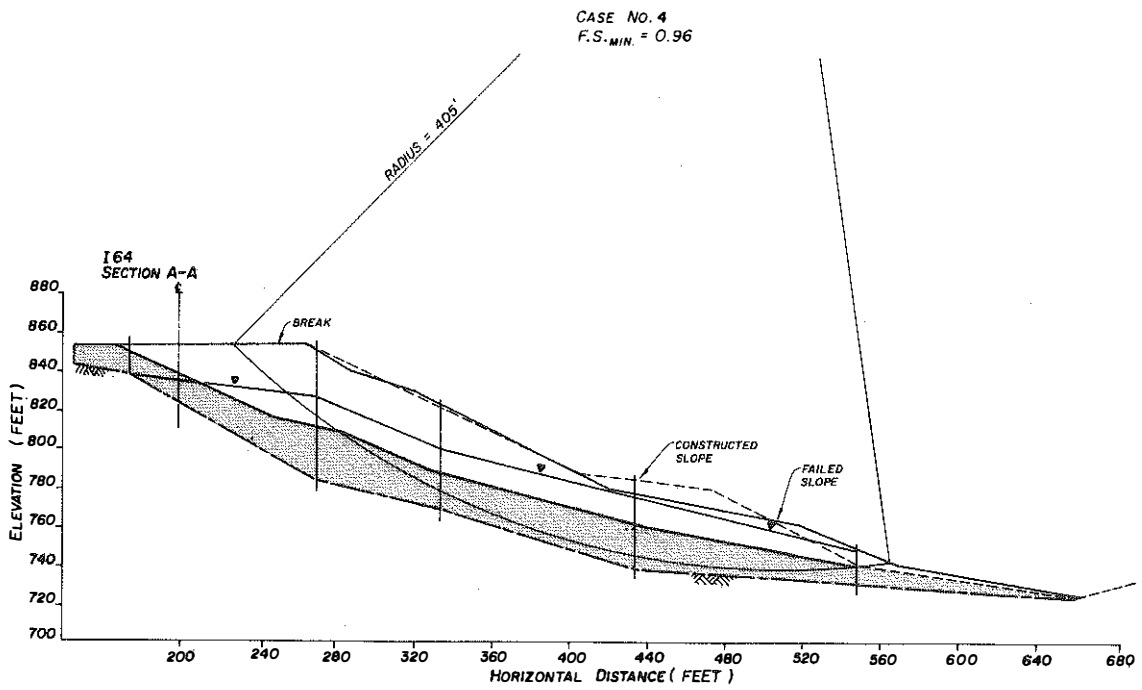


Figure 19. Potential Slip Surface of Failed Slope Section, Case 4.

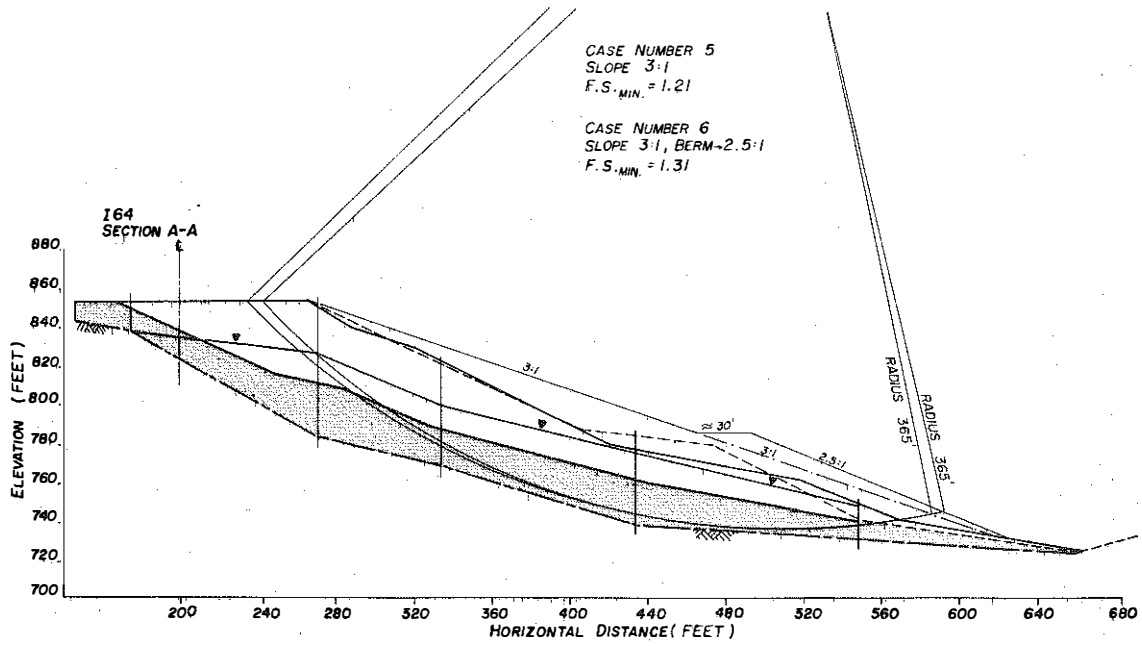


Figure 20. Critical Shear Surfaces, Cases 5 and 6.

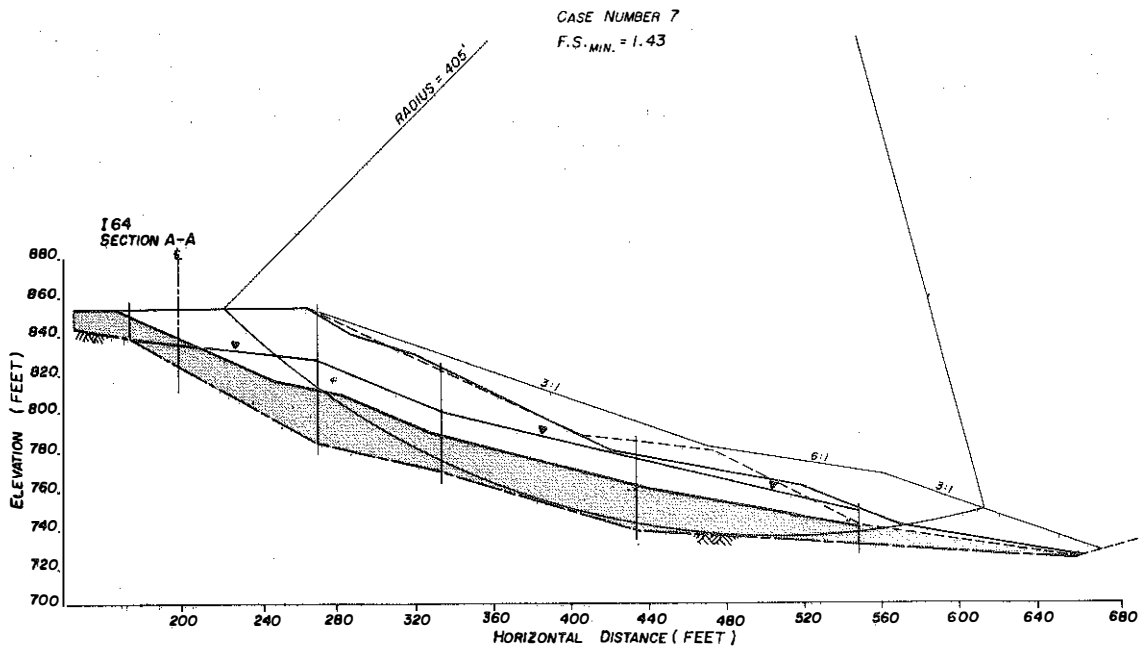


Figure 21. Critical Shear Surface, Case 7.

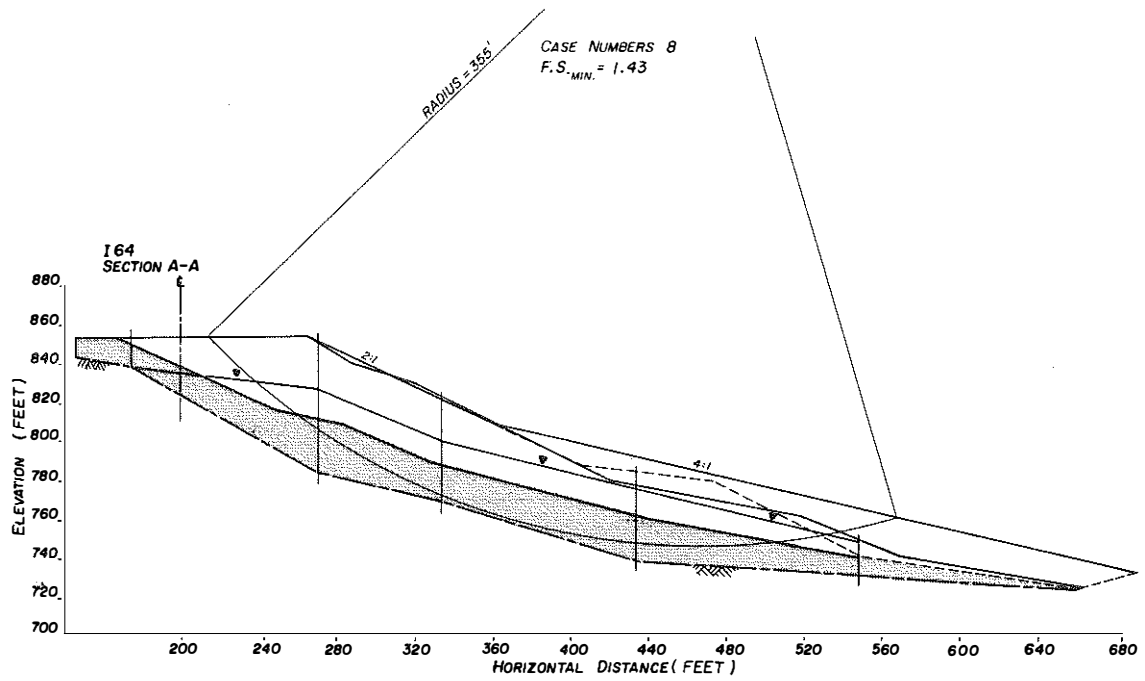


Figure 22. Critical Shear Surface, Case 8.

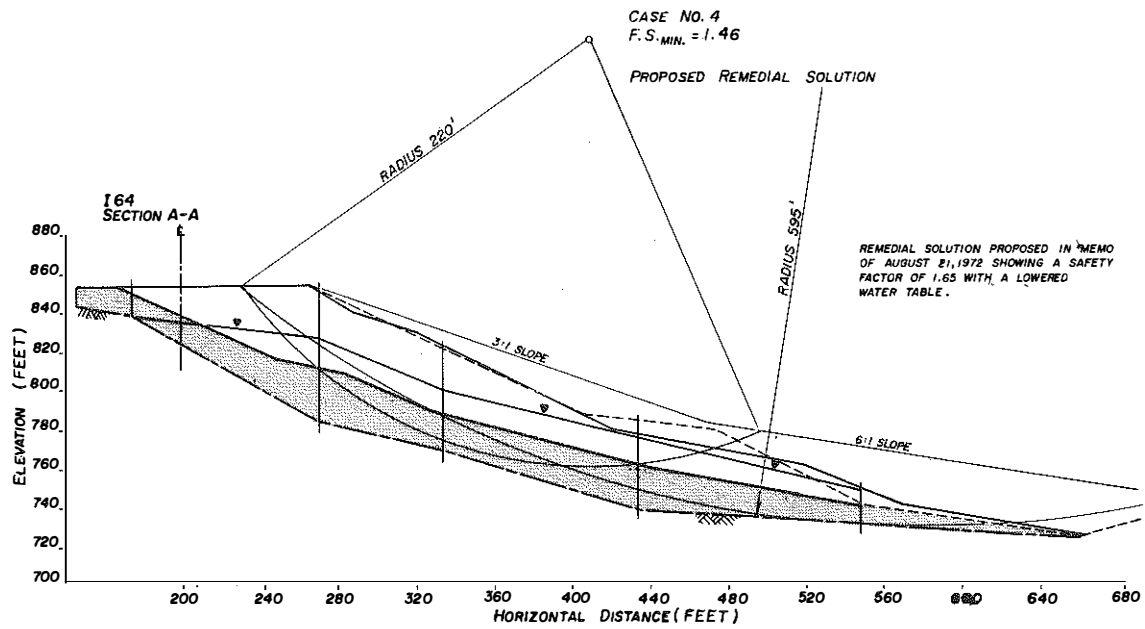


Figure 23. Critical Shear Surface of Proposed Remedial Solution, Case 9.

CONCLUSIONS

The short-term or initial safety factor of the I-64 side-hill embankment was relatively low (1.19). Normally, a safety factor of 1.25 or 1.30 is desirable during or immediately after construction. The computed safety factor of 1.19 was a conservative estimate of the initial stability of the I-64 slopes since no consideration was given in the analyses to excess pore pressures. Had such pore pressures been present during or immediately after construction, the short-term safety factor would have been critical.

The long-term safety factor (0.99) based on ground water observations shows that the entire embankment is in an unstable condition and that total failure will eventually occur. Movement of the embankment started when a slide occurred in the berm (1967) located at the bottom portion of the embankment. The long-term safety factor of the berm was 0.93. This small slide was triggered by deep erosion in a branch located at the toe of the berm. Consequently, with failure of the berm, progressive failure spread throughout the embankment and eventually the eastbound lanes of I 64 were endangered. Deep erosion in a ditch located in the left flank of the slip triggered additional movement of the upper portion of the embankment. There was excellent agreement between the theoretical critical or potential shear surfaces obtained from the computer program and the actual failure surface determined from slope indicators and surface observations (see Figure 18).

Slope stability analyses indicated the remedial slope configuration previously proposed (2) and shown in Figure 23 (Case 9) is sufficient to increase the stability of the unstable embankment to an acceptable level. Two alternate solutions shown in Figure 21 (Case 7) and Figure 22 (Case 8) might be considered on the basis of economy. Both indicate comparable factors of safety to Case 9 but would reduce the amount of earthwork required.*

REFERENCES

1. Houchin, D. R. and Havens, J. H., *Proposed Remedies: Unstable Embankment at Milepost 188 and Channel Erosion at Milepost 190-191; I 64, Boyd County*, Division of Research, Bureau of Highways, Kentucky Department of Transportation, May 1972.
2. Memo from Jas. H. Havens, Director of Division of Research, to A. R. Romine, Director of Division of Maintenance, *Repair of Unstable Embankment near Milepost 188.1; I 64, Boyd County*, August 21, 1972.
3. Yoder, S. M. and Hopkins, T. C., *Slope Stability Analysis: A Computerized Solution of Bishop's Simplified Method of Slices*, Division of Research, Bureau of Highways, Kentucky Department of Transportation, February 1973.
4. Bishop, A. W., *The Use of the Slip Circle in the Stability Analysis of Slopes*, Geotechnique, 1954.
5. Memo from T. C. Hopkins, Research Engineer, to C. J. Henry, Chief Maintenance Engineer, *Landslide on I 64, Boyd County, MP 10-115-D*, March 26, 1971 (contained interim repair recommendations).
6. Hopkins, T. C., *Settlement of Highway Bridge Approaches and Embankment Foundations, Bluegrass Parkway Bridges over Chaplin River*, Division of Research, Bureau of Highways, Kentucky Department of Transportation, February 1973.

August 21, 1972

H.3.38

MEMORANDUM TO: A. R. Romine, Director
Division of Maintenance

ATTENTION: Bert Banks

FROM: James H. Havens, Director
Division of Research

SUBJECT: Repair of Unstable Embankment near Milepost 188.1; I 64, Boyd County

A meeting was held August 2 to discuss repair of the slide near Milepost 188 on I 64 in Boyd County. Those in attendance were Marx Anderson and Carroll Bartley, District 7 Design; H. L. Mathis, Division of Materials; David R. Houchin and James H. Havens, Division of Research; and Roger D. Goughnour and Toni Horner, FHWA.

A conceptual design had been submitted in a research report dated May 1972. Mr. Anderson had preliminary drawings taken from this report ready for inspection. A more thorough stability analysis had been run on this site and was presented for review (see attachment). Based upon this review, department personnel present made recommendations to:

1. Change the berm to 6:1 slope,
2. Recommend the 2-foot rock blanket extend to the top of the 3:1 slope change, and
3. Change specifications on the drainage blanket material to read:

The materials used may be crushed limestone, crushed slag, or crushed or uncrushed gravel, and must meet the following requirements:

- A. Be well graded and meeting the following specific gradations:

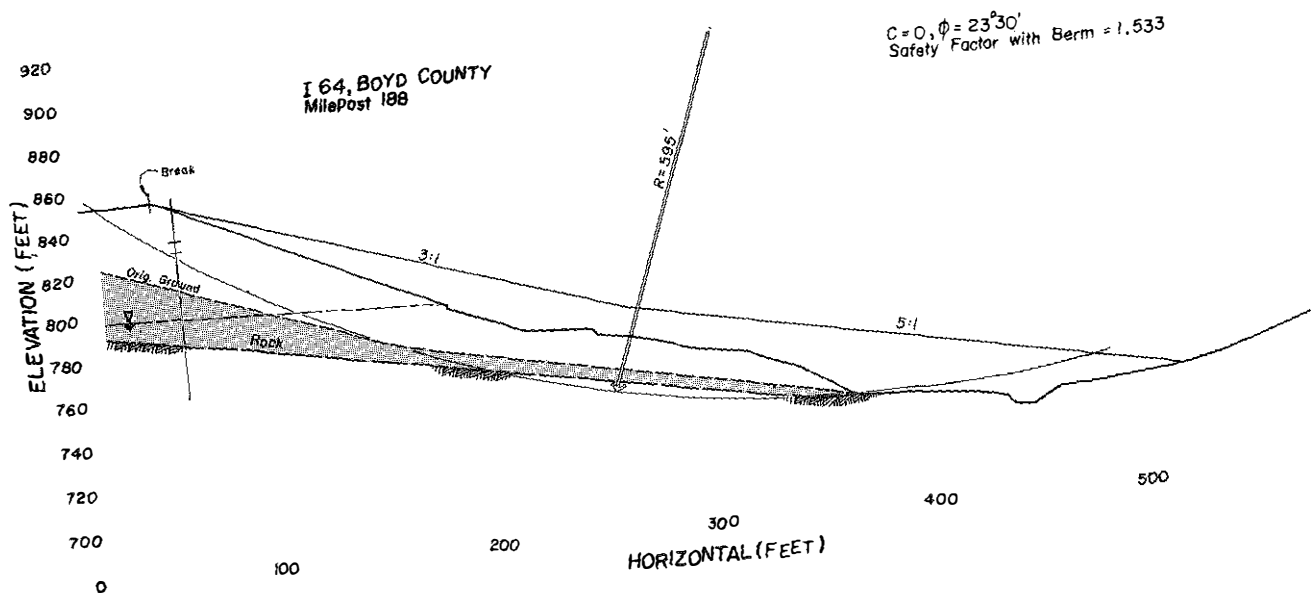
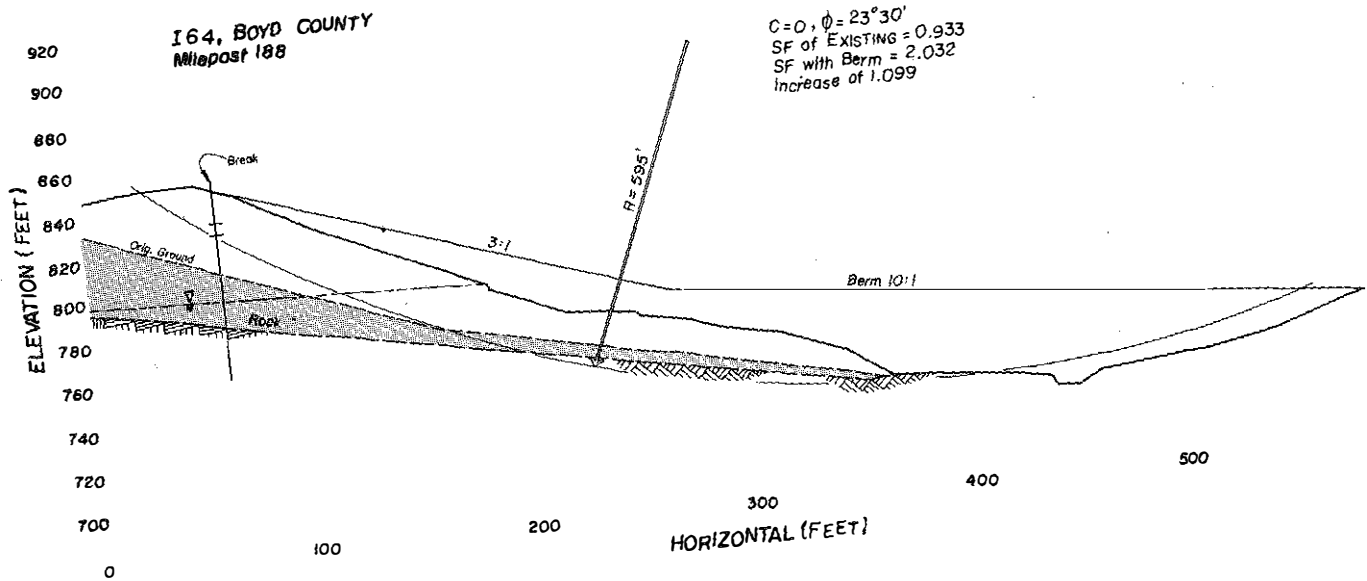
SIZE	PERCENT BY WEIGHT
Passing 1 1/2 inch sieve	100
Passing No. 100 sieve	0-5

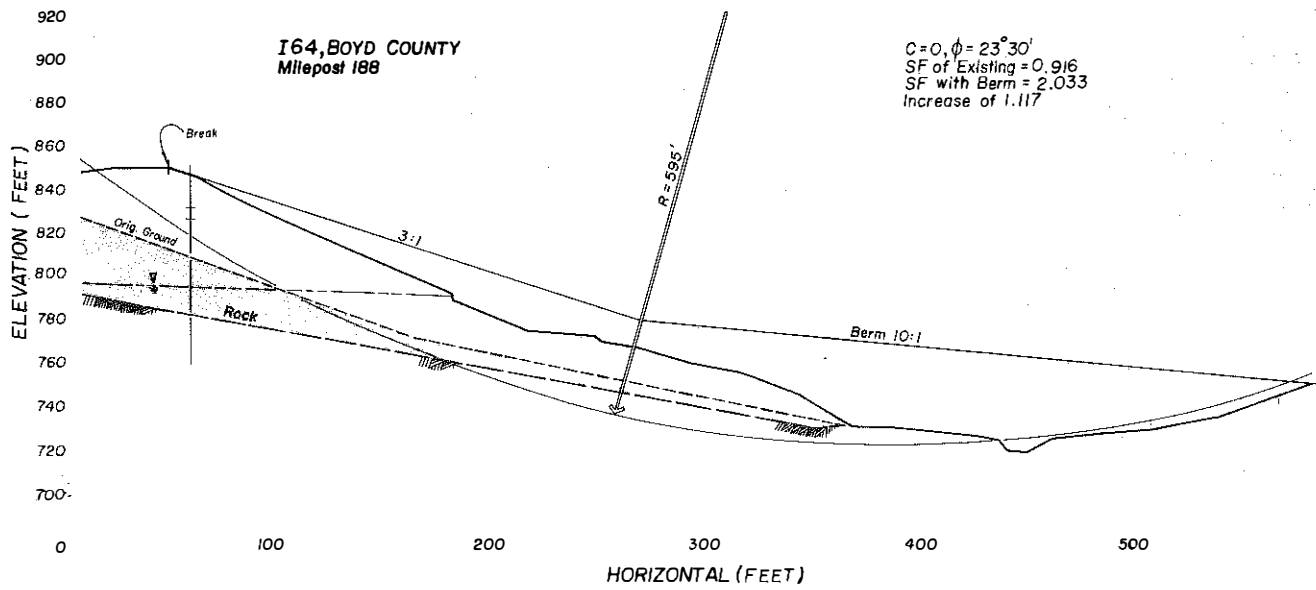
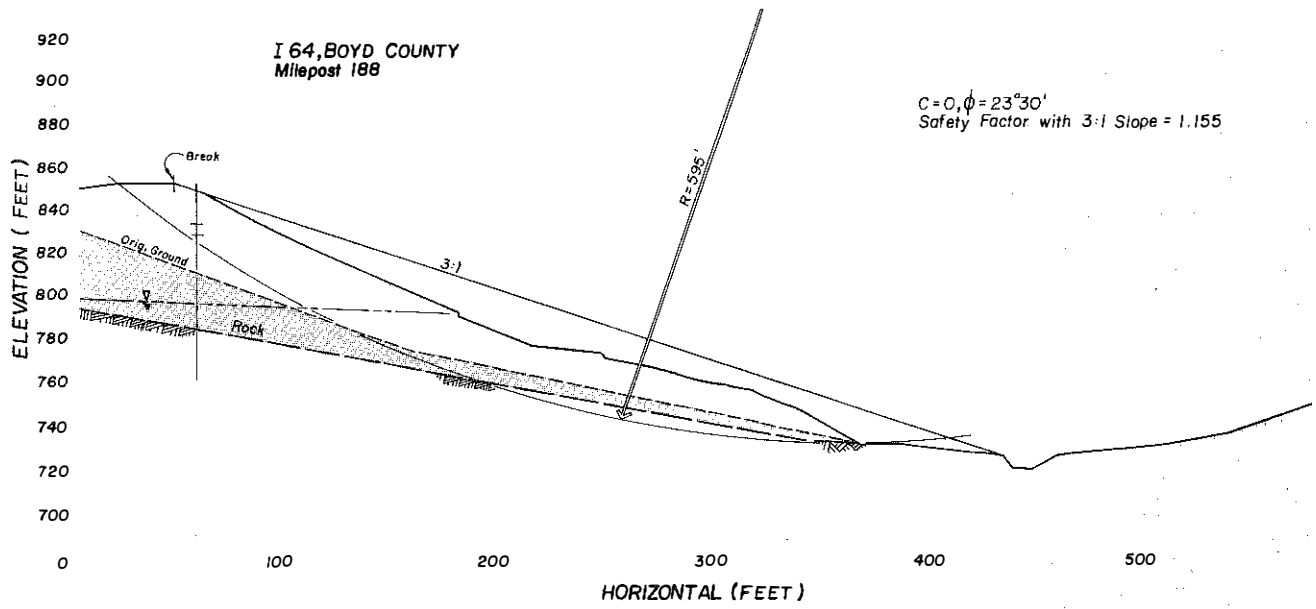
- B. Contain not more than 2 percent shale.
- C. Percent of wear not more than 40.
- D. Free of organic material, clay balls, or other deleterious substances.

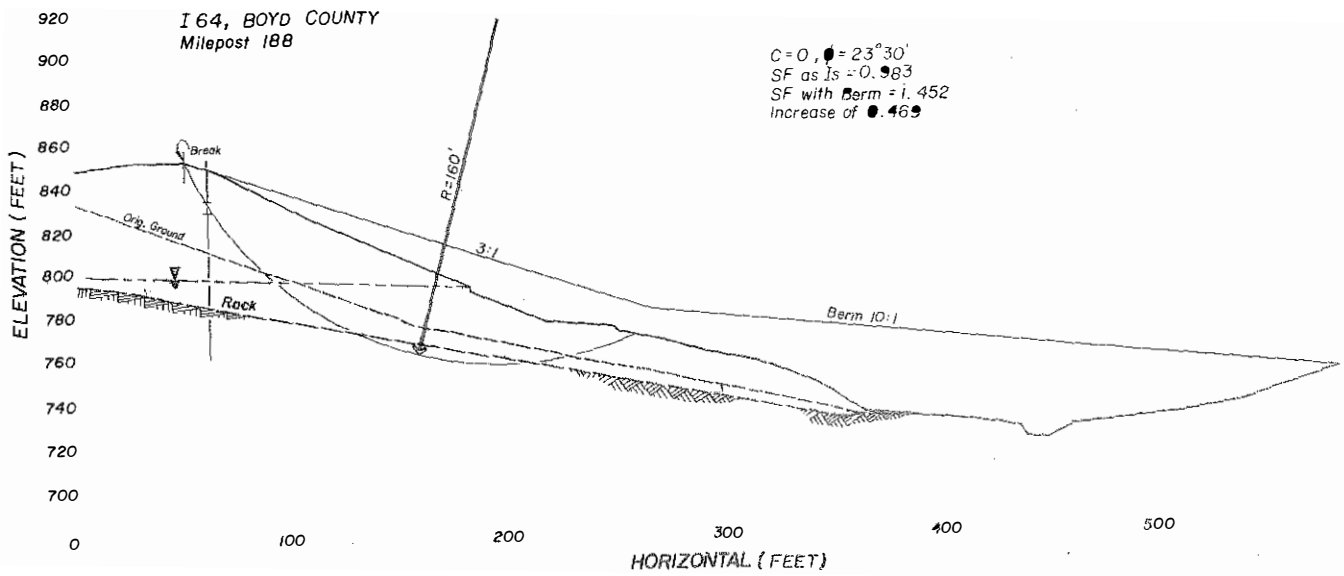
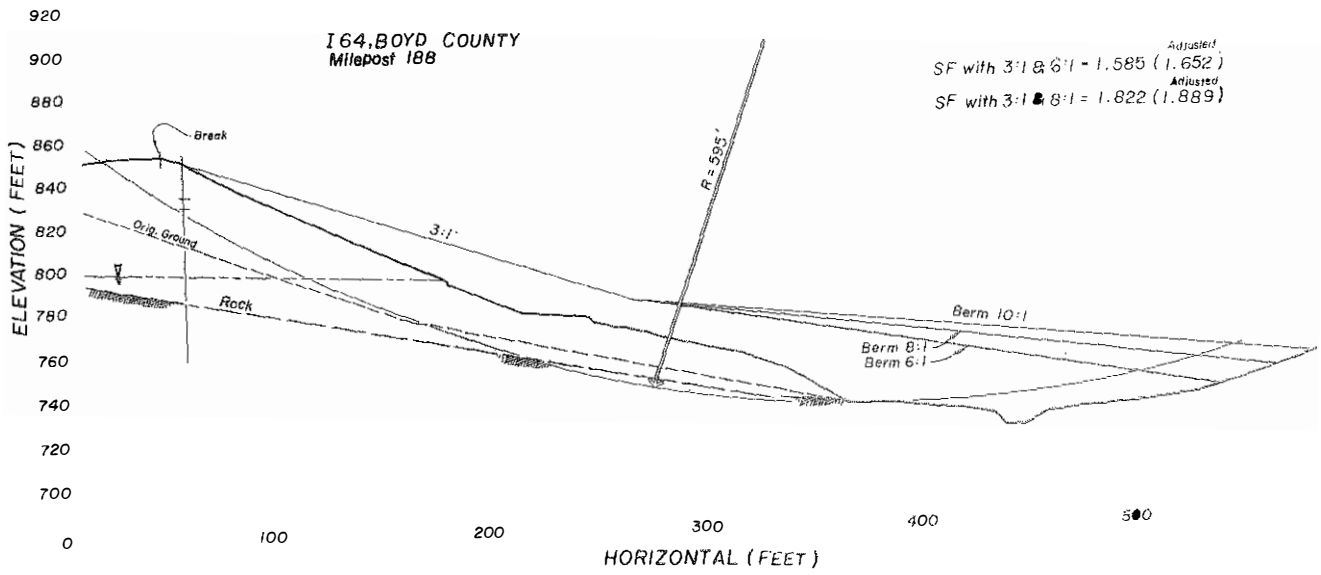
A suggestion was made by Mr. Goughnour to incorporate the above changes into the earlier report and submit it for federal approval.

JHH:dw

cc's: J. E. McChord (Attn. Henry Mathis)
C. S. Layson (Attn. John S. Riley)
L. G. Sturgill (District 9)
J. W. Spurrier
E. B. Gaither
W. B. Drake
Frank Kemper
J. T. Anderson
Marx Anderson (District 7)







RESEARCH REPORT

**PROPOSED REMEDIES: UNSTABLE EMBANKMENT AT
MILEPOST 188 AND CHANNEL EROSION AT MILEPOST
190-191; I 64, BOYD COUNTY**

by

David R. Houchin
Research Engineer

and

Jas. H. Havens
Director

Division of Research
DEPARTMENT OF HIGHWAYS
Commonwealth of Kentucky

May 1972

MILEPOST 188

The Department became aware of fill slippage on I 64 some 300 feet east of Milepost 188 during the summer of 1967. In September of that year, Mr. L. E. Richardson, Division of Maintenance, and Mr. Gordon D. Scott, Division of Research, made an inspection of the area. Mr. Scott reported cracking of the shoulder, movement of the guardrail, and a failure in the berm along the toe. However, subsequent inspections showed the slip to be stabilizing by itself until late in 1970 when some additional movement was observed. In January of 1971, the most conspicuous pavement failure was located almost directly above the berm failure. It appeared that the berm failure may have affected the roadway failure; one crack was observed in the embankment surface about midway between the top of the eastern roadway shoulder and the top of the berm. The embankment slip appeared to have been triggered by deep erosion along the western margin of the fill and toe of the berm and extending from Station 282+00 to Station 285+50. The deeply eroded ditch was approximately 3 to 5 feet in depth and carried water from a median drain at Station 282+00 and a 24-inch cross drain at Station 282+50.

Seven borings were made at the site; logs are attached hereto. Depth from roadway elevations varied from 42 to 90 feet. Results show that the foundation is composed of alternating layers of shale and sandstone. In the main failure area, the embankment is resting on damp clay and shale (Figure 1).

To obtain water-table measurements and to prevent caving, Holes 2, 3, and 6 were cased with downspout. The lower portion of Hole 4 caved in before casing could be extended to the bottom. Piezometers were installed in the bottom of Holes 1 and 5 to insure accurate water-table measurements. The purpose of these observation wells was to determine if water was seeping into the embankment. Seepage of water into the embankment can induce movement by saturating the fill, thereby lowering the shear strength of the fill material.

Water-table measurements obtained March 3, 1971, indicate the phreatic surface (water table) exits along the original groundline slope, rightward of the centerline. But readings obtained March 18, 1971 (two weeks later), positioned the water table about 10 feet below original groundline. Measurements obtained to date show that water in the sandstone layers is not under pressure. Water-table levels are plotted in Figure 1. They show fluctuations just below original groundline in the slope indicator area. Observations of water levels and seepage suggest the bottom half (including berm) of the fill is saturated more or less constantly. A median drain

constructed in early 1971 relieves much of the ground water there, but it is too shallow to effectively prevent infiltration into the fill.

To chart the magnitude and rate of subsurface horizontal movements of the fill slip and to determine the depth(s) at which slippage is occurring, a special boring (Hole 7, Figure 1) was drilled and cased with 2 7/8-inch (ID), slotted (four grooves), aluminum slope-indicator casing. Subsurface movements were obtained by lowering a pendulum-type instrument into the casing and determining the inclination of the casing at various depths; changes in the position of the tubing are computed from changes in dial readings between successive sets of data.

The latest reading of the slope indicator was April 13, 1972. A period of 399 days had elapsed since initial readings (Figure 1). During this time, the maximum movement was approximately 7 inches. This displacement extended from the top of the casing to some 16 feet below pavement elevation. A definite zone of slippage was found in the embankment between 16 and 21 feet below roadway elevation. Figure 1 also shows the rate of movement. Acceleration of movement is apparent. Failure seems inevitable. Only the eastbound lanes appear to be endangered at this time.

Immediate action is recommended. Two remedial solutions have been considered. Both have been successfully used elsewhere. One is to remove the slide completely, install drainage systems and rebuild the embankment with normal or lighter weight material. It is felt this scheme would imperil the remaining embankment and would not correct the problem in the berm at the toe of the fill. The other solution, which is preferred, is to: 1) place a pipe along the southeastern margin of the slope to carry runoff now carried by a paved ditch, 2) extend existing culvert downstream, 3) construct granular drainage blankets by terracing or benching existing slopes, 5) construct the berm on 10:1 and additional fill to provide embankment slope of 3:1 (Figures 2 and 3), and 5) construct paved ditches. The berm is to be merged or buttressed into both hill sides opposite the present embankment. On-site surveys should be made to establish complete plans. No. 9 stone is suggested for the drainage blankets. Perforated pipe is recommended in conjunction therewith. Two previous slides on I 64 have been corrected by a similar method (Stations 3030 to 3033 and 3167 to 3170). This method not only seems to be giving immediate results in this area but also seems to be better for long-range stability.

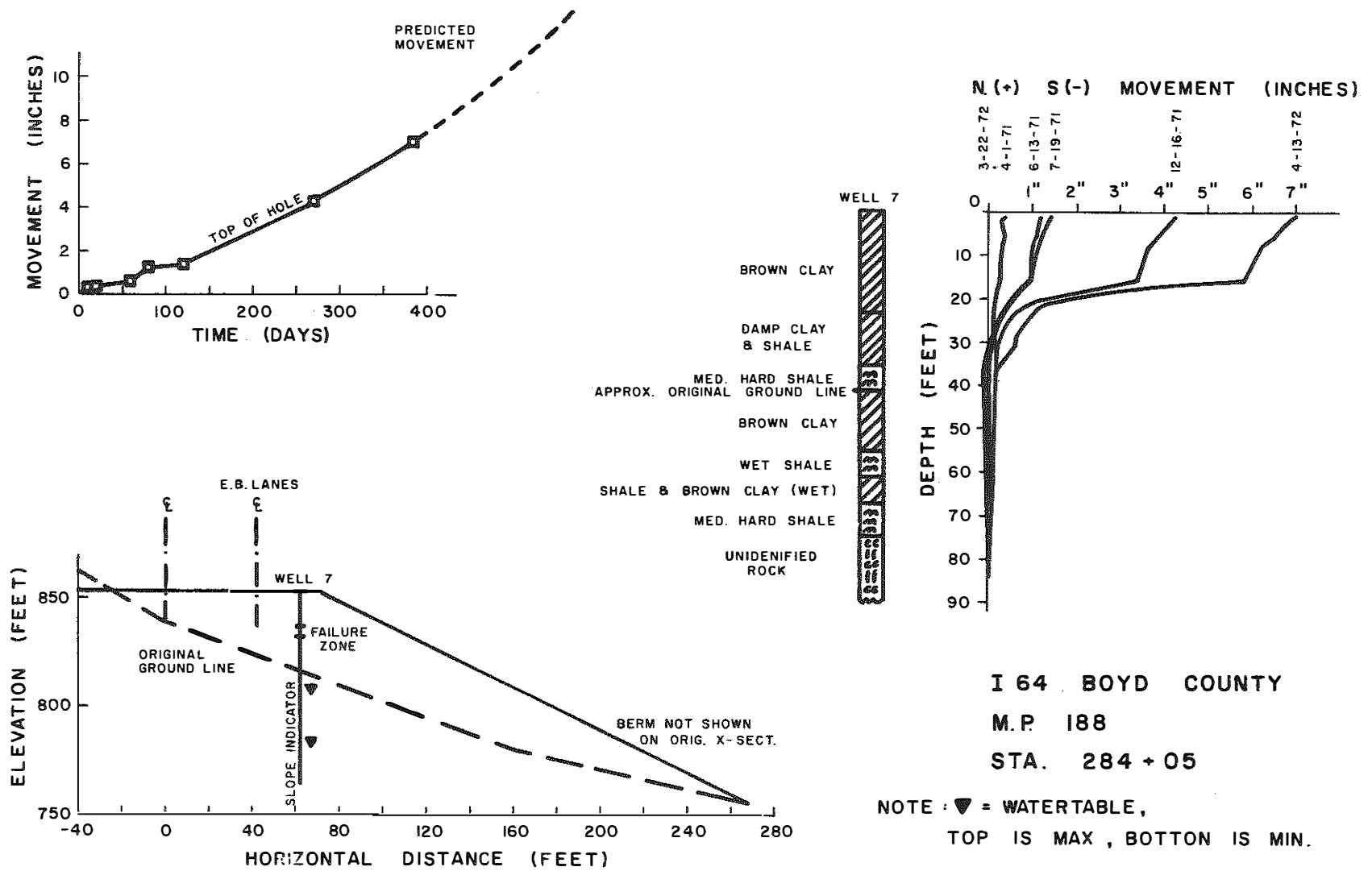


Figure 1.

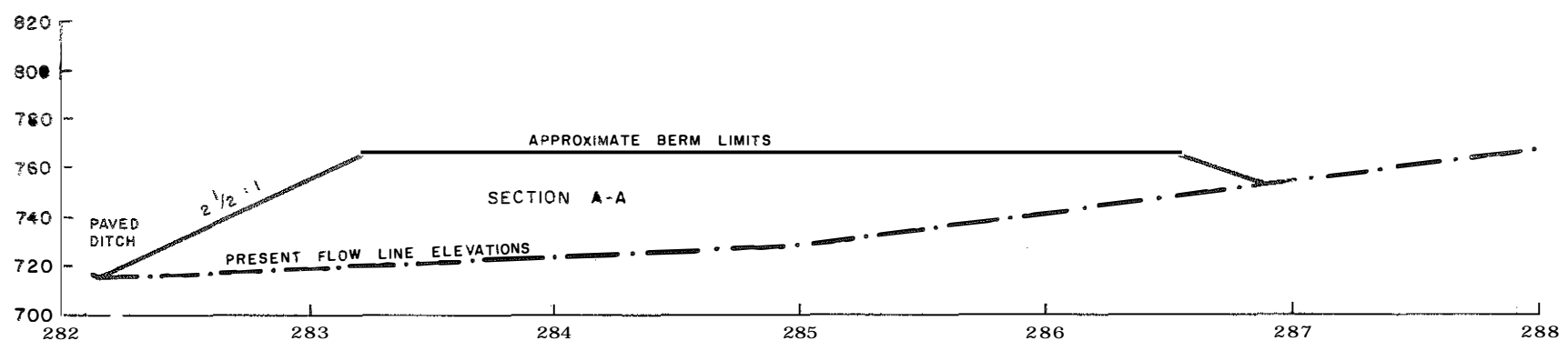
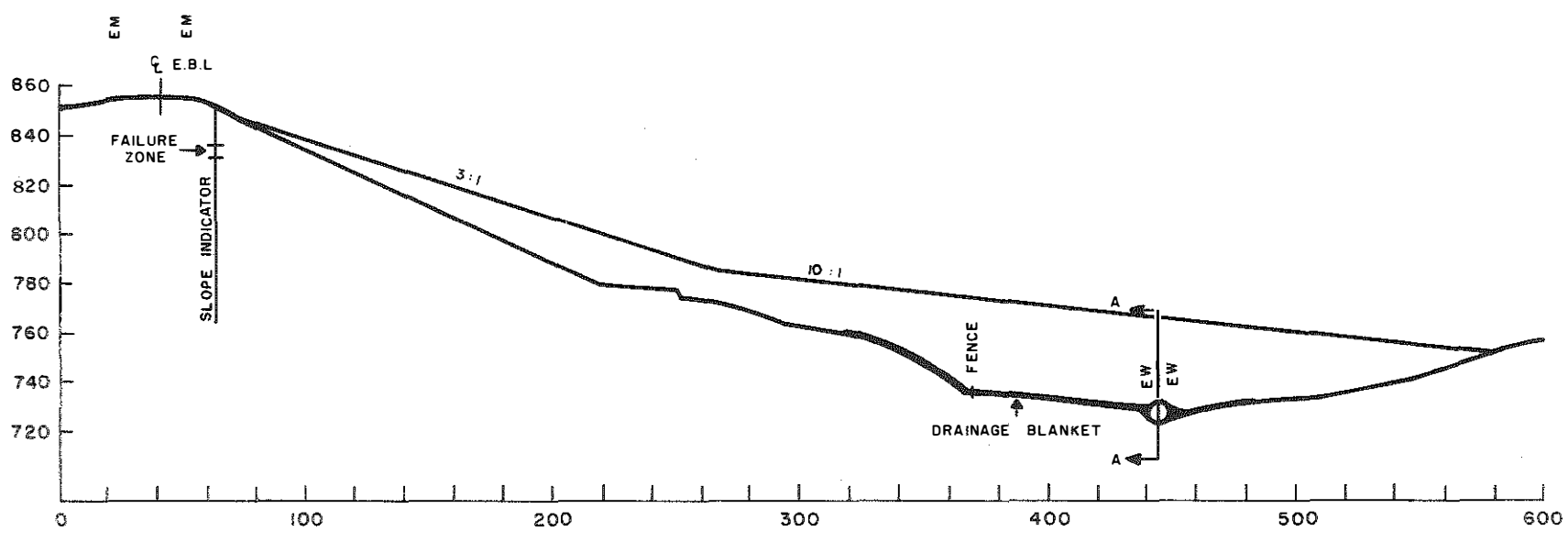
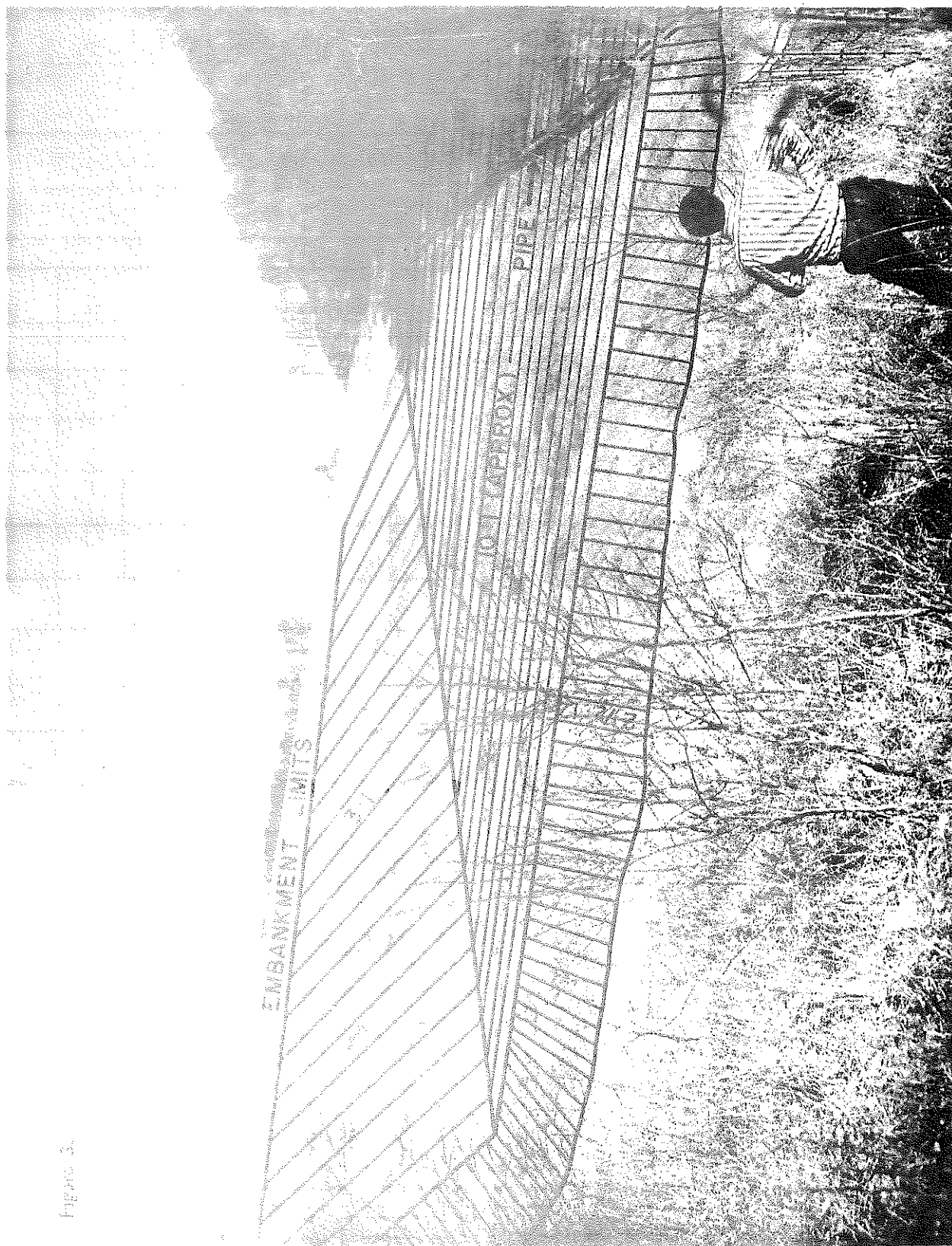


Figure 2.

Figure 3.



Hole # 1

DEPARTMENT OF HIGHWAYS
FRANKFORT, KENTUCKY

Lane W.B.L

Date 1-21-71

County Boyd

Location 1.5' Lt E of W.B.L ON I-64 @ M.P 188
282+75

Project # MP-10-115-0 Sta # _____ Gr. Elev. 852.2

Boring Log

From	To	Visual Classification	Type of Sample	No.	%	Blows
0.0	3.6	Fill sand				
3.6	7.4	sandstone hard				
7.4	8.2	shale				
8.2	12.6	sandstone hard				
12.6	18.3	sand - damp				
18.3	21.7	sandstone med. hard				
21.7	25.4	sandstone - hard				
25.4	28.6	Hard shale				
28.6	31.3	shale - med HD (wet)				
31.3	37.4	sandstone med HD				
37.4	42.0	sandstone - hard				
		~ installed Piezometer ~				
		water at 28.7 24 HOURS				
		AFTER DRILLING				
		25.6 2-16-71				

Date Completed _____

Hole # 2

DEPARTMENT OF HIGHWAYS
FRANKFORT, KENTUCKY

Lane W.B.L

Date 1-22-71

County Boyd Co

Location 15' Lt. E of W.B.L on E-64 @ MP 188
284+3.5

Project # MP 18-115-0 Sta # _____ Gr. Elev. 850.3
850.0

Boring Log

From	To	Visual Classification	Type of Sample	No.	%	Blows
0.0	11.7	BROWN SAND				
11.7	13.6	white sandstone (Boulder)				
13.6	20.2	SANDSTONE (med Hard)				
20.2	27.7	shale (Damp)				
27.7	34.5	SANDSTONE (HARD) (Dry)				
34.5	42.1	Blue Clay (very soft & wet)				
42.1	43.4	SANDSTONE (Hard)				
43.4	46.7	shale (Damp) (med Hard)				
46.7	50.0	Hard dry sandstone				
		Put-down spout				
		water came in this				
		hole immediately after				
		pulling out augers				
		water @ 284- 2-13-71				

Date Completed _____

Hole # 5

DEPARTMENT OF HIGHWAYS
FRANKFORT, KENTUCKY

Lane E-B-L

Date 2-18-71

County Boyd

Location 17 RT 4 ON E.B.L of I-64 S

Project # MP-10-115-10

Sta # 282475

Gr. Elev. 852.0

Boring Log

From	To	Visual Classification	Type of Sample	No.	%	Blows
0.0	12.5	FILL CLAY				
12.5	34.0	SANDSTONE (med. Hard)				
34.0	37.6	SANDSTONE & SANDY CLAY				
37.6	38.5	BROWN SANDY CLAY	S.T.	1	90	
38.5	45.0	SANDSTONE (med. Hard)				
45.0	46.5	SANDY BROWN CLAY	ST	2	90	
46.5	60.5	med. Hard SANDSTONE				
60.5	73.5	SANDY CLAY (Damp)				
73.5	75.0	SANDY CLAY	ST	3	90	
75.0	77.4	SHALE (wet)				
77.4	85.0	HARD ROCK				
		installed Piezometer				
		WATER AREA 45.5 2-18-71				

Date Completed _____

Hole # 6

DEPARTMENT OF HIGHWAYS
FRANKFORT, KENTUCKY

Lane E-B-L

Date 2-18-71

County Boyd

Location 17' rt of E EBL on E:BL of I-64 sta 284150

Project # MP-10-115-0 Sta # 284150 Gr. Elev. 856.8

Boring Log

From	To	Visual Classification	Type of Sample	No.	%	Blows
356.8	356.8					
0.0	20.0	clay & Loose Rock				
356.8	321.8					
20.0	35.0	BROWN SANDY clay (med. Hard)				
321.8	320.3					
35.0	36.5	BROWN SANDY clay	ST	1	90	
320.3	308.3					
36.5	48.5	SANDY HARD clay				
308.3	306.8					
48.5	50.0	" clay	ST	2	90	
306.8	295.8					
50.0	61.0	" HARD clay				
295.8	294.3					
61.0	62.5	" " "	ST	3	90	
294.3	288.4					
62.5	68.4	shale med. Hard & wet				
288.4	281.8					
68.4	75.0	Hard. Rock				
		Put in down Spout				
		water @ 57.9 2-19-71		798.9		

Date Completed _____

Hole # 7

DEPARTMENT OF HIGHWAYS
FRANKFORT, KENTUCKY

Lane E.B.L

Date 3-18-71

County Boyd

Location 22' Ret E of E.B.L (Slope Indicator)

Project # MP-10-115-D

Sta # 284105

Gr. Elev. 855.2

Boring Log

From	To	Visual Classification	Type of Sample	No.	%	Blows
0.0	21.7	Brown clay				
21.7	35.5	Damp clay & shale				
35.5	43.7	med hard shale				
43.7	55.6	Brown clay				
55.6	61.4	wet shale				
61.4	67.5	shale & Brown clay (wet)				
67.5	75.0	med hard shale				
75.0	80.0 90.0	Rock				
		Pumped out water @ 70.0				

Date Completed _____

EROSION OF CHANNEL CHANGE NEAR MILEPOST 190 ON I 64; PROPOSED REMEDY

A channel change constructed on the north side of I 64 from Station 390+00 to 420+00 has been eroding since the time of construction. Soil conditions in this area are such that velocities of 2-3 fps cause constant erosion. Down-cutting of the stream bed (Figure 4) has caused loss of sod and severe sluffing of the banks. This erosion has now lowered the flow line of the stream some 6 feet (according to District 9 officials). The channel is typically V-shaped. An inspection of the site on April 11, 1972, indicated that erosion had progressed to the point of endangering both private property and the road. An earlier attempt to control the situation by dumping large stone (quarry run) in the channel at some locations was not successful; high velocities during peak discharge dilapidated those fill-ups.

Original plans show a drop of 25.3 feet between Stations 397+53 and 421+37. They also show slopes of -0.90 percent and -1.24 percent through this area, but apparently the valley-fill material (silty alluvium) continues to erode steadily. Any remedy other than continuous paving would have to slow the low-flow velocity significantly. One device which seems to offer remedy at a relatively low cost is some type of ditch-check. These devices have apparently fallen into disuse in roadside ditches. However, it is believed they have not been designed to provide continuous (stair-step) ponding. For instance, Standard Drawings 11.19b and 11.20a would not necessarily provide this feature. These should be used at low heights (2 to 3 feet) and frequent intervals (150 to 350 feet -- depending on stream profile) to obtain the desired effect (Figure 5). This structure system has been used for similar purposes in the past and seems to be the most logical solution for this problem. Each check should have a rock blanket extending 20 to 30 feet downstream and 3 to 4 feet up each stream wall to dissipate the energy from the weir-type checks. A spillway flume does not seem necessary. Soil-saving dams (Cf. **Handbook of Culvert and Drainage Practices**, Armco, 1937) or small dams, such as those used for settling basins, might be considered as alternatives.

Figure 4.

I 64 - 8(6) 180, M.P. 190-191
May 17, 1972

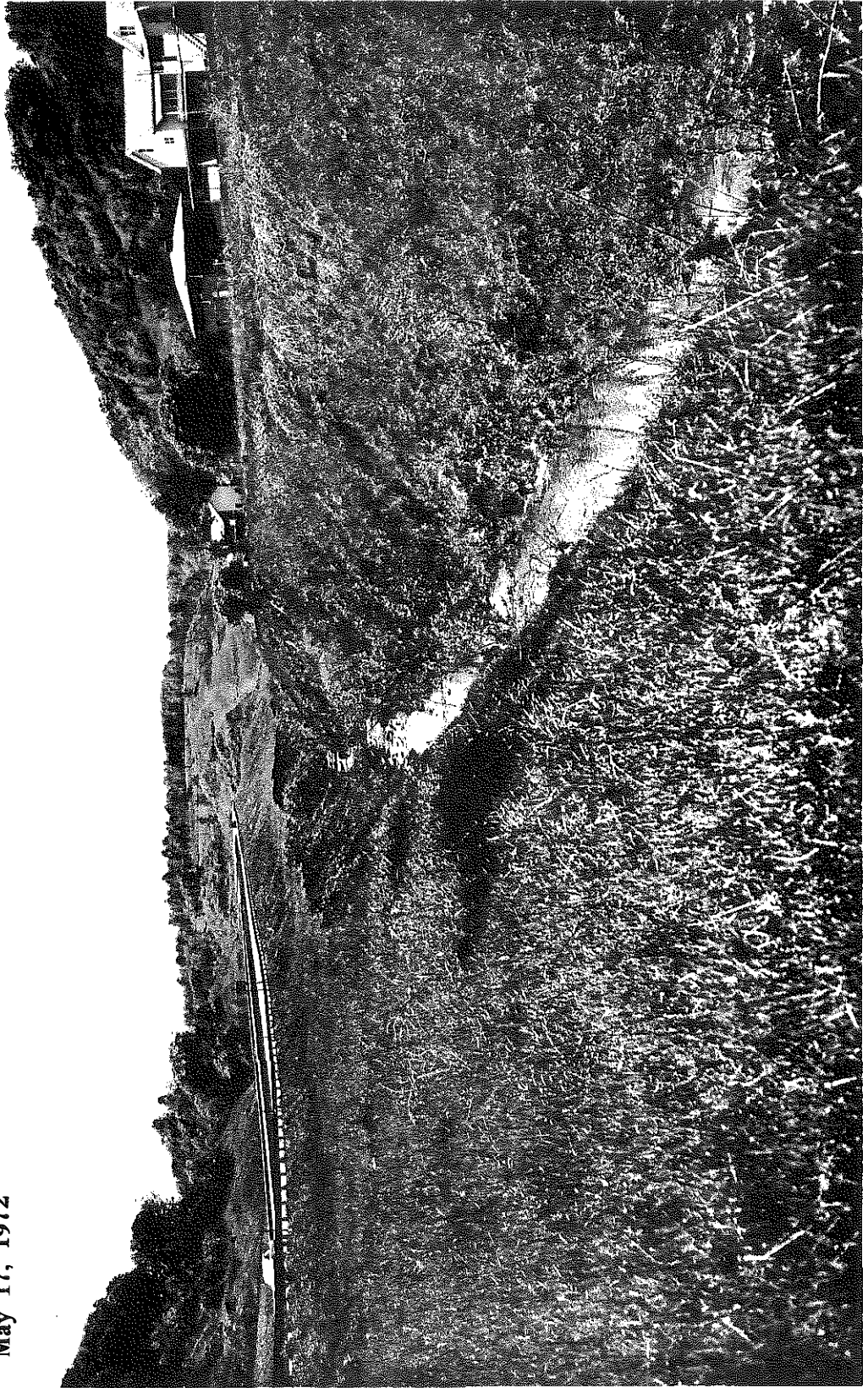


Figure 5.

I 64 - 8(6) 180, M.P. 190-191
CONCEPTIONAL REMEDY FOR CHANNEL EROSION
Ditch - Checks, 2 to 3 Ft. High
23 - Ft. Drop in 1900 Ft.
May 17, 1972



